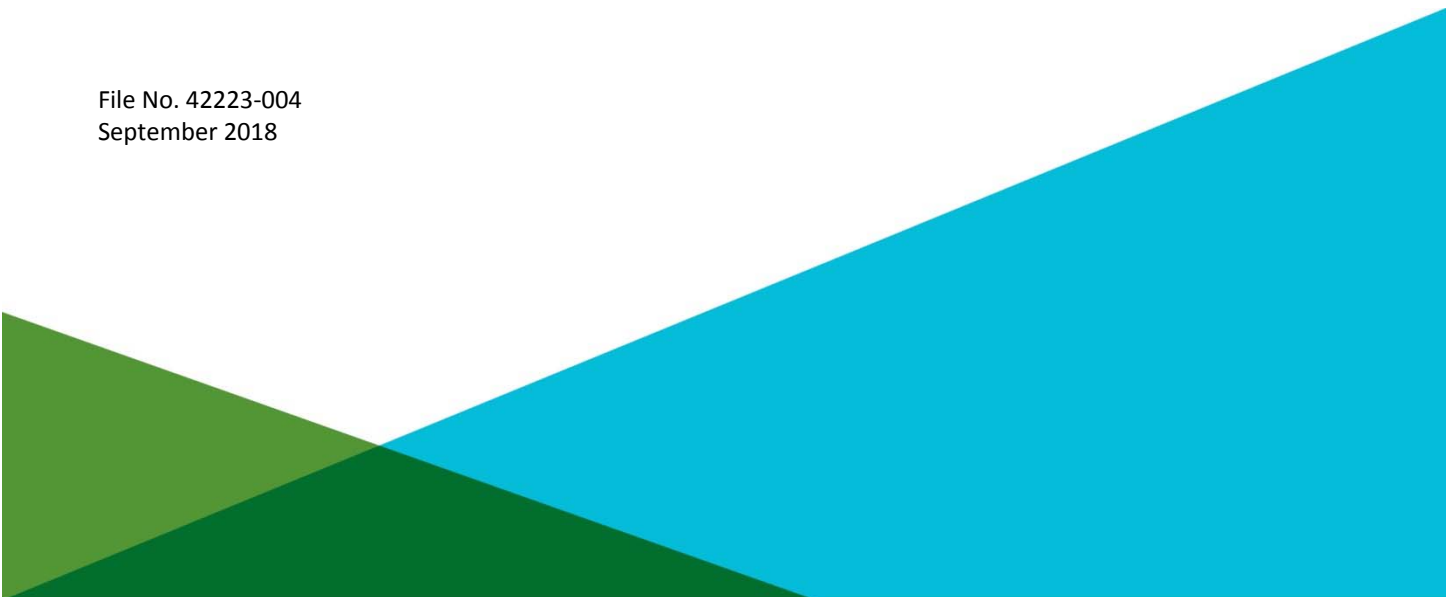


GEOTECHNICAL DESIGN REPORT – 100% SUBMISSION
BRIDGE STREET BRIDGE
ROUTES U.S. ROUTE 201 AND 9 OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18735.00, BRIDGE NO. 2101

by Haley & Aldrich, Inc.
Portland, Maine

for Stantec
Scarborough, Maine

File No. 42223-004
September 2018





HALEY & ALDRICH, INC.
75 Washington Avenue
Suite 1A
Portland, ME 04101
207.482.4600

26 September 2018
File No. 42223-004

Stantec
482 Payne Road Scarborough Court
Scarborough ME 04074-8929

Attention: Tim Merritt, P.E.
Senior Project Manager

Subject: Geotechnical Design Report – 100% Submission
Bridge Street Bridge
Routes U.S. 201 and 9 over Cobbosseecontee Stream
Gardiner, Maine
MaineDOT WIN 18735.00, Bridge No. 2101

Ladies and Gentlemen:

This geotechnical design report (GDR) presents the results of subsurface investigation and geotechnical laboratory testing programs, and also provides geotechnical design recommendations and construction considerations for the proposed bridge replacement. This GDR has been prepared in support of Stantec's 100 percent submission to the Maine Department of Transportation (MaineDOT).

This work has been completed in accordance with our proposal dated 27 June 2017, and your subsequent authorization.

Project Understanding

The existing bridge carries Bridge Street/Routes U.S. 201 and 9/126 over the Cobbosseecontee Stream and Arcade, as shown on Figure 1, Project Locus. Based on our review of the historic bridge plans dated 1917 (Appendix D), we understand that the existing bridge consists of an approximately 223-ft long, approximately 45-ft wide, four-span structure. The existing bridge foundations consist of concrete spread footings bearing between approximately El. -3 and El. 3.

The proposed replacement bridge will be located approximately along the same alignment as the existing bridge. The replacement bridge is proposed to be a 135-ft single span structure with semi-integral abutments. The south abutment will be located immediately north of existing Pier 1. The north abutment will be located between the existing Pier 3 and north abutment.

Horizontal Coordinate System and Elevation Datum

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone (refer to Table I and boring logs in Appendix A). Elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

Elevation datum information is not provided on the historic bridge plans. Based on several comparison points between the historic plans and current survey points (abutment seat and finished grade elevations), the elevations on the historic bridge plans can be approximately converted to NAVD 88 by subtracting 6.9 ft (i.e., El. 6.9 historic plans = El. 0 NAVD 88).

Geologic Setting

According to Maine Geological Survey's Gardiner Quadrangle, Maine (2009), surficial geologic units mapped within the site vicinity consist largely of stream alluvium from both the Kennebec River and the Cobbosseecontee Stream. The recent test borings drilled at the site encountered man-placed fill overlying these alluvial deposits. Underlying the alluvial deposits is a glaciomarine deposit consisting of sand, silt and clay typical of the Presumpscot Formation. This deposit was only encountered in two of the recent test borings. Deposition of this formation occurred as a result of melting glacial ice in conjunction with sea level rise during deglaciation in Late Wisconsinan time. Below the marine deposit is a very dense glacial till deposit consisting of a heterogeneous mix of sand, silt, gravel, cobbles and boulders which directly overlies bedrock.

According to Maine Geological Survey's Bedrock Geology of the Gardiner Quadrangle, Maine (1982), bedrock at the site is mapped as Silurian-Ordovician age rocks of the Vassalboro Formation. Bedrock core samples collected for this site investigation consisted of metamorphic rocks composed of schist, quartzite and granofels. The above referenced map indicates a fault zone located approximately 2,000 ft south of the Bridge Street bridge site. The presence of the fault zone has been evaluated and, in our opinion, will not impact the design of the new bridge foundations or bridge superstructure.

Subsurface Explorations

HISTORIC EXPLORATIONS BY OTHERS

No subsurface information at the existing bridge is shown on the historic drawings included in Appendix D.

PRELIMINARY DESIGN PHASE EXPLORATIONS

Haley & Aldrich conducted a preliminary geotechnical exploration program at the site, consisting of three phases of drilling:

Preliminary Design Phase Program: The first phase consisted of drilling four test borings (BB-GCS-101, BB-GCS-102, BB-GCS-102A and BB-GCS-103) to identify general subsurface conditions along the proposed bridge alignment. During the same phase, three additional test borings (BB-GCS-101A, BB-GCS-201 and BB-GCS-202) were performed for environmental purposes and those test boring logs are also included in this report. For more information on the environmental exploratory borings performed by Haley & Aldrich, refer to the environmental report entitled “Environmental Investigation Report, Bridge Street Bridge, Routes U.S. 201 and 9 over Cobbosseecontee Stream, Gardiner, Maine, MaineDOT WIN 18735.00, Bridge No. 2101,” dated 8 June 2017 (Environmental Report). This work was conducted between 9 November and 1 December 2015.

Supplemental Program, Mast Arm: The second phase of drilling included one boring (BB-GCS-301) for a traffic signal foundation at the northeast corner of the intersection of Bridge Street and Water Street. This work was conducted between 29 and 30 November 2016.

Supplemental Program, Proposed Abutment Locations: The third phase of drilling included five borings (BB-GCS-401 through BB-GCS-403, BB-GCS-401A and BB-GCS-402A) drilled at the proposed north and south abutment locations, to obtain additional information at specific foundation locations. This work was conducted between 10 and 18 April 2017.

Boring locations were laid out in the field by Haley & Aldrich by taping from existing site features. “As-drilled” locations and ground surface elevation at boring locations were surveyed by Maine DOT and the locations of the borings are shown on Figure 2. Coordinate location data and ground surface elevations at exploration locations are provided on individual test boring logs provided in Appendix A and are summarized in Table I.

All test borings were drilled by New England Boring Contractors of Hermon, Maine using a truck-mounted Mobile B53 drill rig or track-mounted Mobile B53 drill rig. Test borings were drilled to depths ranging from 6.9 ft to 80.7 ft below ground surface (BGS).

Geotechnical test borings were typically performed by advancing a 4.0-in. (HW-size) inside diameter (ID) steel casing to a certain depth and then telescoping a 3.0-in. (NW-size) ID steel casing within the HW casing. Borings were advanced using cased-wash methods by either driving or spinning the casing. Soil samples were generally collected continuously through the man-placed fill and then typically at 5-ft intervals once native soils were encountered. The first and third phase of test borings were extended to bedrock and sampled 10 ft of rock core. The second phase of test borings was extended to a depth of 22 ft, for traffic signal foundation design. Several borings encountered problems during drilling (difficulty seating casing, or broken casing). In these cases, a second boring was drilled immediately adjacent to the first to extend the boring to the required depths.

Environmental explorations were typically performed using 2.0-in. ID hollow-stem augers (HSA). Soil samples were generally collected continuously through the man-placed fill. Environmental test borings were terminated in natural soil.

Soils samples were collected by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. Drilling and sampling were performed in accordance with MaineDOT specifications. The drill rig was equipped with either a standard rope and cathead and safety hammer or a winch driven automatic hammer calibrated annually per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A theoretical hammer efficiency factor of 0.6 was assumed for the rope and cathead/safety hammer system and a calculated hammer efficiency of 0.569 (BB-GCS-301) and 0.677 (BB-GCS-400 series) for the calibrated winch and automatic hammer system.

The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N₆₀) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

Soil samples obtained during both the geotechnical and environmental explorations were screened with a photoionization detector (PID) to assess the presence of volatile organic compounds (VOCs). There were several instances of elevated PID readings for this subsurface exploration program. Refer to individual boring logs provided in Appendix A for PID readings. PID readings are discussed further in the Environmental Report. The potential for corrosive soil conditions, specifically in the in-situ fill soils will be evaluated further during final design.

The “-100” and “-400” series geotechnical borings sampled 10 to 15 ft of bedrock each using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

All soil and bedrock samples were classified in accordance with the MaineDOT classification system and were preserved in glass sample jars and wooden core boxes. The samples that were not submitted for laboratory testing are available for review upon request. Soil and bedrock samples will be stored at the Haley & Aldrich laboratory facility in Portland, Maine until the construction phase of the project is completed.

FINAL DESIGN PHASE EXPLORATIONS

Based on the extent and quality of the subsurface data collected during the preliminary design phase and because the proposed vertical and horizontal alignment did not change significantly after the preliminary design phase, Haley & Aldrich determined that additional, design phase field work was not needed to finalize geotechnical and foundation support requirements for the proposed bridge.

Generalized Subsurface Conditions

The subsurface conditions encountered at the site consist of the following geologic units presented in order of increasing depth below ground surface: man-placed fill, alluvial deposits, marine deposits, glacial till, weathered bedrock and bedrock. Refer to Table I for a summary of the soil units and

encountered thicknesses at each test boring location. Refer to Figure 3 for a graphical representation of the subsurface conditions along the bridge alignment. A description of each soil unit is provided separately, below. Detailed soil descriptions are provided on the test boring logs in Appendix A.

Please note that the soil descriptions provided on the test boring logs and summarized below do not represent actual field conditions other than at the specific test boring locations. The actual conditions will vary from those described and shown herein.

SOIL

Unit	Approximate Range in Encountered Thickness (ft)	Generalized Description
Fill	3.8 to 26.2	Very loose to very dense SAND, silty SAND and gravelly SAND with varying amounts of silt and gravel (SP, SM, SW-SM); GRAVEL, sandy GRAVEL and silty GRAVEL with varying amounts of silt and sand (GP, GP-GM, GM, GW). Contained zones of soft to hard CLAY, silty CLAY, clayey SILT and sandy SILT with varying amounts of sand and gravel (CL, ML). <u>The fill contained frequent cobbles, boulders and wood</u>; occasionally contained brick fragments, concrete or concrete fragments, wood chips, roots, glass, ceramics, ash and paint chips. <i>(Encountered in all test borings.)</i>
Alluvial Deposit	2.0 to 5.7	Medium dense to very dense, SAND and gravelly SAND with varying amounts of silt and gravel (SM, SW); GRAVEL and sandy GRAVEL with varying amounts of silt and sand (GM, GW, GW-GM). The alluvial deposit contained occasional organics and wood fragments. <i>(Encountered in borings BB-GCS-102A, BB-GCS-402, BB-GCS-402A and BB-GCS-403. These borings were located immediately north and south of the stream.)</i>
Marine Deposit	Not fully penetrated	Medium dense, silty fine SAND (SM). The full thickness of deposit was not penetrated in any of the borings. <i>(Encountered in borings BB-GCS-201 and BB-GCS-202 only. These borings were located south of the bridge, adjacent to Chapman Fuel on Bridge Street.)</i>

Unit	Approximate Range in Encountered Thickness (ft)	Generalized Description
Glacial Till	43.7 to 54.3	Hard SILT, sandy SILT, clayey SILT, and CLAY, with varying amounts of sand and gravel (ML, CL). Contained zones of very dense SAND or GRAVEL, with varying amounts of sand and silt (SM, GW). The glacial till was well bonded and contained frequent cobbles and boulders. <i>(Encountered in all of the borings that fully penetrated the fill and alluvial deposits.)</i>
Weathered Bedrock	1.1 to 2.4	Weathered bedrock was encountered in the three deep borings located on the south side of the stream. <i>(Encountered in BB-GCS-101, BB-GCS-102A and BB-GCS-401A.)</i>

BEDROCK

Bedrock was cored in borings BB-GCS-101, BB-GCS-102A, BB-GCS-401A, BB-GCS-402A and BB-GCS-403, and probable bedrock was encountered in BB-GCS-103*. The top of the bedrock surface ranged from approximately 57.0 to 76.0 ft BGS (El. -42.3 to El. -55). A summary of top of bedrock elevations is provided below.

Substructure	Test Boring	Approx. Elev. Top of Bedrock (ft, NAVD 88)	Rock Surface Slope		
			Test Boring Pair	Horizontal Distance	Approx. Slope of Rock Surface, Direction Relative to Bridge
Abutment 1	BB-GCS-101	-42.3			
	BB-GCS-102A	-54.9	BB-GCS-101 to BB-GCS-102A	74.8 ft	1V:6H Longitudinal
	BB-GCS-401A	-55.0	BB-GCS-101 to BB-GCS-401A	68.9 ft	1V:5H Transverse
Abutment 2	BB-GCS-103	-45.7*	B-GCS-103 to BB-GCS-403	46.6 ft	1V:12H Longitudinal
	BB-GCS-402A	-50.5	BB-GCS-402A to BB-GCS-403	25.3 ft	1V:25H Transverse
	BB-GCS-403	-49.5			

* Probable bedrock based on rollerbit behavior and observation of rollerbit spoils/cuttings.

In general, the bedrock surface slopes down from outside the abutment footprints, towards the Cobbosseecontee Stream.

Bedrock encountered at the site consisted of moderately hard to very hard SCHIST, GRANOFELS and QUARTZITE. Highly fractured zones were encountered within the sampled bedrock.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the core run. RQD values for bedrock encountered at the site are summarized below.

Substructure	Test Boring	Length of Rock Core Sampled (ft)	Range in RQD (%)	Average RQD (%) ¹
Abutment 1	BB-GCS-101	10.0	52 to 53	53
	BB-GCS-102A	10.0	0	0
	BB-GCS-401A	15.5	32 to 80	64
Abutment 2	BB-GCS-103*	Bedrock not sampled	Bedrock not sampled	Bedrock not sampled
	BB-GCS-402A	15.4	0 to 75	41
	BB-GCS-403	12.0	78 to 100	89

¹ Average RQD is the mean of RQDs recorded for each test boring.

Photographs showing the visual condition of the sampled bedrock core are included in Appendix B.

WATER ELEVATIONS

The following tidal data was provided by Stantec for the Cobbosseecontee Stream at the project site in their 40 percent PIC submission plans dated 1 September 2017:

Q1.1: El. 7.0
Q50: El. 11.1
Q100: El. 11.6

Laboratory Testing Program

PRELIMINARY DESIGN PHASE TESTING PROGRAM

A preliminary laboratory testing program was undertaken on soil samples collected during the field investigations to assist in soil classification/identification and scour evaluations. In general, laboratory testing was performed on disturbed soil samples collected during SPT sampling. All laboratory soil testing was performed by GeoTesting Express of Acton, Massachusetts. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures.

The preliminary phase testing program included fifteen grain size analyses (sieve only, no hydrometer) and two Atterberg limits. In addition, one compressive strength rock test was conducted with elastic moduli measurements. A summary of laboratory test results is provided below.

Grain Size Analyses

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Strata	Percent Finer than #200 Sieve (%)	USCS Classification
BB-GCS-101	1D	0.0 to 2.0	Fill	16.4	SW-SM
BB-GCS-101	3D	4.0 to 6.0	Fill	13.5	SW-SM
BB-GCS-101	8D	20.0 to 22.0	Glacial Till	54.0	CL
BB-GCS-102	1D	1.0 to 3.0	Fill	8.9	SP
BB-GCS-102	3D	4.2 to 6.2	Fill	7.1	GW
BB-GCS-102A	4D	13.5 to 15.0	Glacial Till	82.7	CL
BB-GCS-103	2D	2.5 to 4.5	Fill	14.9	SM
BB-GCS-103	3D	4.5 to 6.5	Fill	48.5	ML
BB-GCS-103	6D	12.0 to 14.0	Fill	48.7	ML
BB-GCS-401	2D	4.0 to 6.0	Fill	8.0	SW-SM
BB-GCS-401	6D	14.0 to 16.0	Glacial Till	50.6	ML
BB-GCS-402	1D	0.0 to 2.0	Fill	27.8	SM
BB-GCS-402	3D	4.0 to 6.0	Alluvial Deposit	18.2	SM
BB-GCS-403	3D	4.0 to 6.0	Alluvial Deposit	6.0	GW-GM
BB-GCS-403	6D	15.0 to 17.0	Glacial Till	52.0	ML

Atterberg Limits

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Strata	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification
BB-GCS-101	8D	20.0 to 22.0	Glacial Till	9	21	13	8	CL
BB-GCS-102A	4D	13.0 to 15.0	Glacial Till	20	30	15	15	CL

Rock Core Testing

Test Boring No.	Rock Core No.	Sample Depth (ft)	Peak Compressive Stress (psi)	Young's Modulus (ksi)
BB-GCS-101	R2	65.5 to 65.9	6,343	885 to 1,000

FINAL DESIGN PHASE TESTING PROGRAM

A supplemental laboratory testing program was undertaken during the final design phase on soil samples and rock samples collected during the preliminary field investigations to assist in micropile design.

The final design phase testing program included six grain size analyses (sieve only, no hydrometer) and one series of tests to determine the corrosive potential of the fill soils that will be present adjacent to the new micropiles at the south abutment. In addition, three compressive strength rock tests were conducted with elastic moduli measurements. A summary of laboratory test results is provided below.

Grain Size Analyses

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Strata	Percent Finer than #200 Sieve (%)	USCS Classification
BB-GCS-401	5D	9.0 to 11.0	Glacial Till	9.0	GW
BB-GCS-401	10D	34.0 to 36.0	Glacial Till	51.9	ML
BB-GCS-401A	15D	59.0 to 61.0	Glacial Till	53.3	ML
BB-GCS-402	6D	13.0 to 15.0	Glacial Till	78.3	ML
BB-GCS-403	10D	35.0 to 36.8	Glacial Till	53.9	ML
BB-GCS-403	14D	55.0 to 57.0	Glacial Till	58.7	ML

Corrosivity Testing

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Strata	Organic Content, %	pH of soil in water/CaCl	Electrical Resistivity, ohm-cm	Sulfates, Soluble, mg/kg	Chloride, Soluble, mg/kg
BB-GCS-401	1D, 3D and 4D	0.0 to 8.2	Fill	1.5	6.1 / 5.9	992	1,616	320

Rock Core Testing

Test Boring No.	Rock Core No.	Sample Depth (ft)	Peak Compressive Stress (psi)	Young's Modulus (ksi)
BB-GCS-401A	R4	78.9 to 79.3	3,043	1,340 to 1,950
BB-GCS-402A	R4	76.1 to 76.7	20,003	6,700 to 7,720
BB-GCS-403	R1	63.3 to 63.9	10,719	6,590 to 7,060

Complete laboratory test results are provided in Appendix C.

Ground Penetrating Radar Survey

As part of the third phase of explorations during preliminary design, a ground penetrating radar (GPR) survey was performed. The GPR survey was conducted in the area of the A1 Diner, to the north of existing Pier 1 in Arcade (proposed south abutment area) and between existing Pier 3 and the north abutment (proposed north abutment area). The purpose of the GPR survey was to attempt to define the limits of existing foundations, and identify below-grade structures that could be encountered during foundation construction.

Hager-Richter Geoscience, Inc. of Salem, New Hampshire performed the GPR survey on 18 April 2017, under contract with MaineDOT and under the direction of Haley & Aldrich. Their report outlining the details of the procedures, equipment used to complete the study, and results of the survey is included in Appendix E.

As described in the report in Appendix E, the GPR signal penetration depth was generally good, and is estimated to have been about 3 to 6.5 ft. There were some areas along the north side of Arcade with

limited signal penetration of about 2 ft. As shown in the figures of the report in Appendix E, the GPR survey identified the following:

- A1 Diner area: shallow buried objects ranging from about 2 ft by 2 ft to about 5 ft by 15 ft; the tops of the objects are approximately 1 to 2 ft below grade.
- Proposed south abutment area: several subsurface utilities in Arcade; the tops of the utilities are approximately 1 to 2 ft below grade.
- Proposed north abutment area: two possible buried structures (30 to 40 ft long by 3 ft wide), possible buried object, possible buried metal plate, and possible utility.

Exploratory Test Pits

Exploratory test pits were excavated on 6 July 2017 adjacent to the four western supports for the elevated A1 Diner and adjacent to the existing bridge Pier 1 in an effort to determine the foundation type (i.e., concrete or stacked blocks) and foundation bearing elevation, and to observe and document the existing foundation condition. The report outlining the details of the procedures, equipment used to complete the study, and results of the survey is included in Appendix E, A1 Diner Memorandum.

In general, the test pits were excavated to depths ranging from approximately 2 to 6 ft BGS by McGee Construction, LLC (McGee) of West Gardiner, Maine using a Yanmar B22 mini-excavator. Each test pit was backfilled using excavated material upon completion. The excavated material was placed in controlled lifts and tamped into place with the excavator bucket.

A Haley & Aldrich geotechnical engineer was on-site to monitor the excavations and record the soil, groundwater and foundation conditions encountered. Test pit locations, plan and profile sketches, and photographs of the test pit excavations are provided in Appendix E.

Where visual and olfactory evidence of petroleum-impacted soil were observed, soil samples were screened with a photoionization detector (PID) to assess the presence of volatile organic compounds (VOCs). There were several instances of elevated PID readings in test pits TP4 and TP5, with PID readings ranging from 455 to 827 ppm. Refer to individual test pit logs provided in Appendix E for PID readings.

Geotechnical Design Recommendations

Geotechnical design recommendations for the subject project, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design Bridge Design Specifications, Seventh Edition, 2014 with Interim Revisions through 2016, referred to herein as AASHTO LRFD,
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011, referred to herein as Guide Specification; and
- MaineDOT Bridge Design Guide, August 2003, with Interim Revisions through March 2014, referred to herein as the BDG.

We coordinated our technical evaluations with Stantec, the bridge designer, throughout the preliminary and final design phases of the project. Engineering calculations that support the recommendations outlined in this section are provided for reference in Appendix F.

We understand that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans, standard specifications and special provisions) and that the information contained in the CDs takes precedence over the information provided in this report.

SCOUR

Scour effects at the abutments were not considered due to the presence of heavy riprap that will be placed along the edges of the river as part of the proposed project improvements. The lateral and vertical extent of heavy riprap needed to protect the bridge foundations were determined by Stantec.

FROST PROTECTION

The minimum depth of embedment/cover for footings or other below grade structures was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG. Based on a design freezing index equal to 1,500 freezing degree-days, we recommend that the footings and walls bear a minimum of 5.5 ft below the lowest adjacent ground surface exposed to freezing. Refer to Appendix F, Calculations, for supporting documentation.

APPROACH EMBANKMENTS

Settlement

Up to 13 ft of raise-in-grade is planned at the south approach embankment and up to 16 ft of raise-in-grade is planned at the north approach embankment (over a very limited distance between the existing north abutment and proposed north abutment). Given the proposed raises in grade, we estimate that the total elastic settlement will be on the order of 1 to 2 in. Refer to Appendix F, Calculations, for supporting documentation.

Given the nature and consistency of the fill and naturally-deposited soils that are present at the site, we anticipate that the settlement will likely occur rapidly during construction, soon after new site fills are placed and prior to final paving. We anticipate post-construction settlement to be negligible (i.e., < ½ in.). Based on subsurface conditions at the site, we do not anticipate that long-term consolidation settlements of the in-situ soils will occur as a result of the planned raises in grade.

Global Stability

Computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 6.0 by Rocscience Inc. to evaluate global stability of the bridge approach embankments. Evaluations were performed perpendicular to the face of each abutment.

Soil and rock material and strength properties used in the global stability evaluations are summarized below.

	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
RipRap	140	45	0
Granular Borrow	125	32	0
Common Borrow	120	30	0
Existing Fill	115	30	0
Alluvial Deposits	115	30	0
Glacial Till/ Weathered Bedrock	130	36	0

The minimum calculated static factor of safety from our evaluations is 2.0. The minimum factor of safety required for static stability evaluations is 1.5 where the slope does support or contain a structural element, based on the requirements of LRFD Article 11.6.2.3. The calculated factor of safety under pseudo-static earthquake loading from our evaluations is 1.6, using a horizontal coefficient of 0.062 (i.e., one half of the peak ground acceleration coefficient, A_s). Values ranging from $A_s/3$ to $A_s/2$ are recommended in literature (Melo and Sharma, 2004); the reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a very short period of time. The minimum factor of safety required for pseudo-static stability evaluations is 1.0 based on the requirements of LRFD. The factor of safety for both the static case and under pseudo-static earthquake loading exceed the minimum required.

SEISMIC DESIGN

Due to the nature and thickness of the overburden soils encountered in the test borings, we recommend the site be considered "Site Class D." Based on the site location and the assignment of "Site Class D," the United States Geological Survey (USGS) software application Seismic Design Parameters v. 2.0 provided the recommended AASHTO response spectra for a 7 percent probability of exceedance in 75 years. The recommended values are summarized below.

Parameter	Design Value
Mapped Peak Ground Acceleration, PGA =	0.077
Mapped Short (0.2-second) Period, S_s =	0.160
Mapped Long (1.0-second) Period, S_1 =	0.045
Site Factor, F_{pga} =	1.6
Site Factor, F_a =	1.6
Site Factor, F_v =	2.4
Acceleration Coefficient, A_s (0 second period) =	0.124
Horizontal Response Spectral Acceleration, S_{DS} (0.2 second period) =	0.256
Horizontal Response Spectral Acceleration, S_{D1} (1 second period) =	0.108
Seismic Performance Zone =	1
Seismic Performance Category =	A

LIQUEFACTION CONSIDERATIONS

The liquefaction susceptibility of the granular soils at the site was evaluated by calculating the ratio between the cyclic resistance ratio (CRR) of the in-situ soils at each sample location and equivalent uniform cyclic stress ratio (CSR) imposed by the design earthquake. This ratio was modified using a magnitude scaling factor (MSF) to account for earthquake magnitudes different from M 7.5 to obtain the factor of safety against liquefaction. Liquefaction of the in-situ granular soils will likely occur when the factor of safety is less than 1.0.

The CRR is a function of clean sand-corrected SPT N-value, $(N_1)_{60-CS}$ following the simplified methodology outlined by Youd and Idriss (2001). The CRR was estimated using raw SPT N-values that were corrected for various effects including overburden pressure, hammer energy, borehole diameter, drill rod length, sampling method and fines content (estimated from field samples or obtained from the laboratory test results, as available). In cases where laboratory grain size analyses were available, that data was used in lieu of field observations/soil classifications made during drilling. Since the CRR calculated from $(N_1)_{60-CS}$ using the simplified methodology is based on a M 7.5 earthquake, the CRR was corrected to the design earthquake magnitude (M 5.1; see below) using the MSF (equal to 2.68, based on Youd & Idriss, 2001).

The magnitude (M) for the design earthquake used to assess liquefaction susceptibility at the site was determined from earthquake deaggregation data obtained from the USGS in accordance with AASHTO Guide Specification Section 6.8. This data is based on a seismic event (earthquake) having a seven percent probability of exceedance in 75 years (approximate 1,000-year return period). Based on our review of the USGS deaggregation data, we judged that the modal magnitude earthquake (M 5.1) is appropriate for use in assessing liquefaction susceptibility for this site based on the large number of earthquakes that are significant to the deaggregation (Dickenson, 2005).

In accordance with AASHTO Guide Specifications Section 6.8, the “seasonally averaged groundwater elevation” should be used when determining the extent of soils that are adequately saturated for liquefaction to occur. The liquefaction analyses summarized herein are based on a water level equal to the level encountered during drilling. These were all above the Q1.1 elevation (El. 7), which we judged to be representative of the “seasonally averaged groundwater elevation.”

Since liquefaction is considered at the Extreme Event Limit State and AASHTO LRFD specifies that the overall goal in designing for any extreme event is to prevent structure collapse and protect life safety, calculated factors of safety greater than 1.0 are considered acceptable.

The factor of safety in the samples from all the borings ranged from 4 to 12. Based on the results of the simplified liquefaction evaluation using the data from the recent borings, it is our opinion that the site soils are not liquefaction susceptible. Refer to calculations in Appendix F for plots of factor of safety.

FOUNDATION SUPPORT ALTERNATIVES

As shown on the interpretive subsurface profile (Figure 3), the subsurface conditions primarily consist of, in order of increasing depth below ground surface, man-placed fill, alluvial deposits, glacial till, weathered bedrock, and bedrock. Based on the loading information provided by Stantec, it is our opinion that the glacial till and bedrock are considered suitable for support of the bridge superstructure. Based on the depth to the suitable foundation bearing strata and the subsurface data available at this time, we consider the following foundation alternatives to be technically feasible for support of the new bridge:

- Micropiles bearing in glacial till and/or bedrock (friction pile): A wide range of axial micropile capacities are achievable, depending on the micropile diameter (7 to 9 in. is typical) and micropile rock socket length. A permanent steel casing is typically extended through the existing fill and alluvial deposits to glacial till (El. 0 to El. 3 in the vicinity of the proposed abutments) and the socket length selected as needed to provide the necessary axial and lateral resistance. Lateral resistance of plumb/vertical micropiles is typically relatively small. If additional lateral resistance is required, battered micropiles may be used to resist lateral loads.
- Piles driven to refusal in glacial till (end bearing pile): Piles would be driven through approximately 8 to 12 ft of existing fill and alluvial deposits, and into the glacial till. Due to the thickness of the glacial till (up to 54 ft), it is likely the piles would achieve the required resistance in the glacial till and not reach bedrock. Battered piles to resist lateral loads may be needed.

Driven piles would need to be advanced through cobbles, boulders, wood and other structures in the existing fill. Because of this it is our opinion that it would be difficult to drive piles through this material and therefore pre-augering, pre-excavation, or other methods would be required to advance through the existing fill, increasing the cost of pile installation. In addition, there would be an increased chance of damage to the piles during installation. Also, impact driving is a vibration-inducing activity and could adversely impact the adjacent A1 Diner.

Due to issues with low headroom below the existing bridge, driven piles could not be constructed until the existing bridge is closed and removed. Although technically feasible, driven piles do not meet the construction milestone requirements for the project and therefore were not selected.

- Spread footings bearing on glacial till: The top of the glacial till ranged from El. 0 to El. 3, or approximately 8 to 12 ft below ground surface, at the proposed abutment locations. Excavations to construct spread footings would need to extend to glacial till. This would require either large sloped excavations, or temporary excavation support (e.g., sheetpile cofferdams). Due to the limited headroom, we do not anticipate that sheeting could be installed beneath the existing bridge, prior to closing the bridge. In addition, groundwater would likely be encountered in the excavations. While drilling the recent test borings, the water level in the stream was observed to almost overtop Arcade at Dennis' Pizza (located between the proposed south abutment and the stream), and the Q1.1 water level is El. 7.0. Although technically feasible, spread footings are not preferred to support the proposed bridge due to adverse constructability issues.

Based on several discussions with Stantec and MaineDOT during the preliminary design phase, micropiles were selected to support the new bridge abutments and southeast retaining wall. Geotechnical engineering evaluations and design recommendations for micropiles are summarized in the following sections of this report. Additional evaluations for driven piles were not conducted during final design phase. Geotechnical evaluations and design recommendations for spread footings to support the A1 Diner and small site retaining wall are summarized below. Additional evaluations for spread footings to support the bridge were not conducted during final design.

MICROPILES – BRIDGE ABUTMENTS AND SOUTHEAST RETAINING WALL

Corrosion and Deterioration

The design of micropiles should include consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. As discussed in previous sections of this report, a chemical laboratory testing program (i.e., pH, electrical resistivity, sulfates, chlorides) was undertaken to assess the corrosion potential of in-situ soils that will be present directly adjacent to the new micropiles.

Criteria provided by the Federal Highway Administration (FHWA) National Highway Institute (publication no. FHWA HI 97-013) suggests that an “aggressive” corrosive environment for steel piles can generally be identified as follows:

- $\text{pH} \leq 4.5$
- electrical resistivity $< 2,000$ ohm-cm
- $2,000 \text{ ohm-cm} < \text{electrical resistivity} < 5,000 \text{ ohm-cm}$ and chloride ion content > 100 ppm and/or sulfate ion content > 200 ppm

In addition, electrical resistivity values greater than 5,000 ohm-cm are considered “non-aggressive.”

The results of the laboratory chemical testing that are summarized above were compared to this criteria. Based on this comparison, it is our opinion that the existing fill should be considered “aggressive.” Although chemical testing was not completed on samples of the naturally deposited soils, it is our opinion that these soils should be considered “non-aggressive.”

Based on the results of laboratory chemical testing, it is our opinion that the abutment micropiles at the south abutment should be designed to account for section loss due to corrosion. We recommend that the design axial compressive pile resistance and lateral capacity of micropiles consider a 1/16-in. reduction in cross-sectional area of the permanent casing over the planned design life of the bridge (75 years) to account for corrosion.

Minimal existing fill (4 ft or less) is present at the north abutment, and the bottom of the new pile cap will be located below the bottom of the existing fill. Since the piles will be embedded within undisturbed naturally-deposited soil and will only be exposed to fresh water, it is our opinion that the subsurface environment at the north abutment has a low corrosive potential. Therefore, we recommend that the micropiles at the north abutment do not need to be designed to account for section loss due to corrosion.

Bridge Loads

Stantec provided loads and moments for the Service I, Strength I, and Extreme I Limit States on 5 September 2018, which are included in Appendix F.

Lateral Micropile Evaluations

Using the loads and moments provided by Stantec, Haley & Aldrich conducted lateral pile evaluations at Abutment 1 and Abutment 2 using the computer program FB-Multiplier by Bridge Software Institute. FB-Multiplier is a nonlinear finite element analysis program that is capable of analyzing multiple bridge pier structures interconnected by bridge spans. The program couples nonlinear structural finite element analysis with nonlinear static soil models for axial, lateral, and torsional soil behavior to provide a system of analysis for couple bridge pier structures and foundation systems.

Pinned head conditions were modeled. Axial and bending stiffness were calculated using the gross concrete properties of the micropile. The permanent steel casing was considered in the stiffness calculation but was reduced by 1/16 in. at Abutment 1 for corrosion considerations. Lateral group effects (shaft-soil-shaft interaction) were considered using P-multipliers (i.e., p-y curve reduction factors) prescribed in AASHTO LRFD. Scour effects were not considered due to the presence of heavy riprap placed to protect the piles along the edges of the river.

Maximum micropile vertical loads, internal shear forces, bending moments, and lateral deflections were summarized for each limit state (see Appendix F).

Axial Compression and Uplift Micropile Resistance

For design, we recommend that a strength limit factored unit frictional resistance of 13.9 ksf in bedrock be used to resist axial loads. This design resistance value is based on a resistance factor of 0.7, assuming tension load tests will be performed. Axial resistance in glacial till was also considered during preliminary design. However, based on discussions with Stantec and MaineDOT during final design, it was decided to design micropiles to resist axial loads using the rock socket frictional resistance only.

Micropile size may vary between the abutments and retaining wall, due to the variability in the load demands and headroom conditions at each structure.

A summary of recommended strength limit factored axial geotechnical resistances for several micropile diameters and lengths are provided in the table below.

Embedment in Bedrock (ft)	Strength Limit Factored Axial Geotechnical Resistance (kips) ¹	
	7-in. uncased dia. ²	9-in. uncased dia. ³
5	125	165
10	255	325
15	380	490
20	510	655

Notes:

1. Resistance factor of 0.7 used for the strength limit state.
2. 7.625-in. O.D. casing to top of rock, nominal 7-in. diameter uncased pile diameter in bedrock.
3. 9.625-in. O.D. casing to top of rock, nominal 9-in. diameter uncased pile diameter in bedrock.

Micropile Tip Elevations

The micropiles will develop their axial compressive resistance through side friction in the bedrock. Therefore, the recommended tip elevations for estimating micropile lengths are based on the depths to rock observed in test borings and the maximum axial loads. For estimating bid quantities, we recommend the following estimated bedrock elevations be used at each substructure. Final socket lengths will be selected by Stantec based on the unit resistance provided above and the final design loads.

Substructure	Representative Test Boring	Estimated Bedrock Elevation (ft, NAVD 88)
Abutment 1	BB-GCS-401/401A	El. -55
Abutment 2	BB-GCS-402/402A, BB-GCS-403	El. -50
Southeast Retaining Wall	BB-GCS-401/401A	El. -55

Micropile Settlement and Elastic Compression

Micropile settlement due to elastic shortening of the micropiles as well as pile tip settlement was evaluated based on the maximum factored Service Limit State loads provided by Stantec. Estimates of elastic pile compression for individual piles at each substructure are summarized below.

Substructure	Maximum Factored Service Limit State Load (kips) ¹	Approximate Elastic Pile Compression (in.)
Abutment 1	241	< ½ in.
Abutment 2	232	< ½ in.

¹ Maximum factored service limit state compressive loads from Stantec.

The values summarized above do not include micropile tip settlement, which is considered to be negligible for two primary reasons: 1) the relatively small load transmitted to the micropile tip during service limit state loading and 2) the micropiles will be installed into rock. The elastic shortening of the micropiles is anticipated to occur primarily during construction, soon after the superstructure loads are applied.

Pile Embedment, Spacing and Clearance

We recommend that minimum micropile spacing, clearance and embedment (into the pile cap) meet the requirements of AASHTO LRFD Section 10.9.1.2.

ABUTMENT AND WINGWALL LATERAL EARTH PRESSURE RECOMMENDATIONS

Based on discussions with Stantec and their evaluation of anticipated magnitude of lateral movement of the abutment, we understand the abutments may be designed using active earth pressures. We recommend that abutments and wingwalls be designed for static lateral earth pressures using an equivalent fluid unit weight of 39 pounds per cubic foot (pcf) which assumes an active earth pressure coefficient of 0.31 and a soil unit weight of 125 pcf. This recommendation assumes that the granular soil behind abutments and wingwalls will be drained and no unbalanced hydrostatic pressures will develop behind the abutments and wingwalls.

For structural design of the superstructure backwall (end diaphragm), we recommend use of passive earth pressures, using a Coulomb passive earth pressure coefficient, K_p , of 6.6. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005 (where H is measured from bottom of pile cap to top of abutment). If the calculated displacements are significantly less than that required to develop full passive pressure (i.e., y/H less than 0.005), the Rankine passive earth pressure coefficient, 3.25, should be used.

In addition, walls should be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should be applied to abutments and walls to account for the live load surcharge.

If determined applicable by Stantec, the walls and abutments should be designed for a uniform lateral load to account for seismic soil loading in accordance with LRFD Article A.11.3.1 (Mononobe-Okabe Method). Based on the seismic site class (Site Class "D"), we recommend a seismic active earth pressure

coefficient, K_{AE} , of 0.360 be used for design. Note that this soil pressure includes both the static and seismic lateral earth loads.

SPREAD FOOTINGS – A1 DINER AND SITE RETAINING WALL

As described in the A1 Diner memorandum in Appendix F, two of the existing A1 Diner footings interfere with the proposed southwest retaining wall alignment. Therefore the A1 Diner will need to be temporarily supported by a Contractor-designed support system, and the two footings will need to be removed and replaced with new cast-in-place concrete footings.

In addition, there is a small site retaining wall located to the northwest of the north bridge abutment in the Skehan's Parcel. The maximum wall height is less than 5 ft.

The A1 Diner and site retaining wall may be supported on footings bearing on existing fill. Specific foundation design recommendations follow.

- A1 Diner Bearing Resistance
 - For the service limit state, footings should be designed such that footing contact pressures do not exceed 2.0 kips per square foot (ksf). At this pressure, it is estimated that settlement of footings bearing on existing fill and glacial till will be less than ½ in.
 - For the strength limit state, footings should be designed for a factored bearing resistance of 3.5 ksf, which is based on a resistance factor of 0.45 and assuming minimum footing width of 3 ft. We have assumed that footings will bear approximately 5.5 ft (or more) below finished grade, based on the PIC plans.
 - For the extreme event limit state, footings should be designed for a factored bearing resistance of 6.0 ksf, which is based on a resistance factor of 0.8 and assuming minimum footing width of 3 ft.
- Site Retaining Wall Bearing Resistance
 - For the service limit state, retaining wall footings should be designed such that footing contact pressures do not exceed 1.0 kips per square foot (ksf). At this pressure, it is estimated that settlement of the footings bearing on existing fill and glacial till will be less than 1 in.
 - For the strength limit state, footings should be designed for a factored bearing resistance of 1.2 ksf, which is based on a resistance factor of 0.45 and assuming minimum footing width of 2 ft. We have assumed that the retaining wall will bear 2 ft (or more) below finished grade.
 - For the extreme event limit state, footings should be designed for a factored bearing resistance of 2.1 ksf, which is based on a resistance factor of 0.8.

- Nominal sliding resistance between the base of cast-in-place concrete footing and soil subgrade should be calculated using a coefficient of sliding friction of 0.45 (interface friction angle, δ , of 24°).
 - Resistance factor for the strength limit state should be 0.8 for cast-in-place concrete
 - Resistance factor for the extreme event limit state should be 1.0
 - Values provided in accordance with LRFD Table 10.5.5.2.2-1
- Lateral passive soil resistance for retaining wall design should be ignored.
- Debris, topsoil and organic material (if encountered) should be removed from beneath the proposed footings and replaced with granular borrow.

STAIR FOUNDATION

A metal stair is proposed immediately north of the A1 Diner, to allow pedestrian access between the proposed bridge and Arcade. Based on information provided by Stantec on 14 March 2018, we understand the pad at the bottom of the stairs will be supported by four, 24-in. diameter foundations, and the columns at the intermediate landing will be supported by two, 24-in. diameter foundations, similar to those used by MaineDOT for traffic signals, highway signing and lighting. The maximum service limit state load provided by Stantec is approximately 8 kips per foundation.

- For the service limit state, under an applied load of 8 kips, it is estimated that settlement of the foundations will be less than ½ in.
- For the strength limit state, foundations should be designed for a factored bearing resistance of 8.5 kips, which is based on a resistance factor of 0.45.
- We recommend a minimum foundation length of 6 ft so that foundations extend through loose fill materials encountered within approximately 5 ft of ground surface.

TRAFFIC SIGNAL FOUNDATION

A proposed traffic signal is located at the northeast corner of the intersection of Water Street and Bridge Street. Test boring BB-GCS-301 was performed as close as possible (given access and utility locations) to the traffic signal location. The test boring encountered the following subsurface conditions:

Unit	Thickness (ft)	Elevation
Fill	13.2	El. 25.3 to El. 12.1
Glacial Till	>8.8	El. 12.1 to <El. 3.3

Based on MaineDOT design charts for foundations for traffic signals, highway signing and lighting, we anticipate foundation depth will range from 10 to 20 ft. Since the foundation will be located primarily in the existing fill, we recommend using a friction angle of 30 degrees for design of the foundations. We understand that TMSI Engineers will perform design of the traffic signal.

Construction Considerations

Based on the subsurface conditions encountered and our understanding of the proposed bridge replacement, we offer the following general geotechnical observations regarding the planned bridge construction:

TEMPORARY EARTH SUPPORT

Based on the proposed elevation of the bottom of the foundations, temporary earth support systems will likely be needed to construct the abutment substructures. Based on the subsurface soil, rock and groundwater conditions at the site, we anticipate that the most cost-effective excavation support systems for construction of the substructures will consist of steel sheeting.

Design of temporary works, including temporary earth support system(s) are typically the responsibility of the Contractor and should be conducted by a Licensed Professional Engineer in the State of Maine hired by the Contractor. We recommend that temporary earth support system(s) be designed to support all appropriate combinations of earth, water and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The Contractor is responsible for choosing an applicable factor of safety for the earth support system(s). The Contractor's design should also consider the means and methods and construction sequencing proposed. We recommend that design calculations and shop drawings be prepared by the Contractor, stamped by a Licensed Professional Engineer in the State of Maine and submitted to MaineDOT for review prior to construction.

MICROPILES

Micropiles will need to be advanced through cobbles, boulders, wood and other structures in the existing fill and glacial till. Micropile installation is well suited to penetrate through oversized rock fragments. Obstruction allowances should be considered in the contract.

Micropiles can be constructed with limited headroom, and therefore can be constructed beneath the existing bridge, prior to closing the bridge, which will minimize the duration of time the bridge will need to be out of service. This will help limit the duration of the bridge closure, which we understand is important on this project. Also, micropile installation is also a low-vibration inducing construction activity which will limit adverse impacts to the adjacent A1 Diner.

MICROPILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the minimum required nominal micropile resistances in the field. We recommend that Contractor perform one verification load test at each abutment and at the southeast retaining wall (three total). The micropiles should be tested to a nominal resistance equal to the maximum factored axial compressive micropile load divided by a resistance factor equal to 0.7 in accordance with AASHTO LRFD Table 10.5.5.2.5-1. We recommend that the testing be completed prior to production micropile installation.

FOOTING SUBGRADE PREPARATION (A1 DINER AND SITE RETAINING WALL)

The following guidelines are recommended to protect the subgrade soils beneath footings:

- Make final excavations into bearing soils using smooth-bladed equipment to limit disturbance.
- Prevent water from accumulating on soil surfaces to reduce the possibility of soil disturbance. All filling and concreting of footings should be performed in-the-dry. Subgrades that become disturbed due to water infiltration should be re-excavated and stabilized. Subgrade stabilization methods could include placement of crushed stone and geotextile fabric with approval of the design build team's geotechnical engineer.
- Exposed subgrades should be examined in the field by a geotechnical engineer to verify strength and bearing capacity. Over-excavation may be necessary to remove weak, disturbed or otherwise unacceptable soils.
- Exposed granular soils at bearing strata should be proofrolled with a heavy hand-guided vibratory compactor until firm, as determined by a geotechnical engineer. Any soft areas revealed by proofrolling should be excavated and replaced with crushed stone.
- Disturbance due to water and adverse weather could be reduced by maintaining footing excavations at least 12 in. above the final bearing level until immediately before placing footing reinforcement and concrete. Alternatively, it may be desirable to protect the exposed soil subgrade areas as soon as possible after acceptance by a geotechnical engineer, by placing a 2 to 3-in. thick lean concrete mud-mat or 4 to 6 in. of crushed stone.
- Limit equipment traffic across the exposed soil bearing surfaces.
- Soil bearing surfaces below completed foundations must be protected against freezing, before and after foundation construction. If construction is performed during freezing weather, footings should be backfilled to a sufficient depth (up to 5.5 ft) as soon as possible after they are constructed. Alternatively, insulating blankets or other means may be used for protection against freezing.

REUSE OF EXCAVATED ON-SITE SOILS

Based on the test borings drilled at the site we anticipate that the excavated material will consist primarily of in-situ fill. The in-situ fill includes both granular zones and fine-grained zones. If excavated soils are reused, the Contractor will need to segregate the granular and fine-grained soils.

We understand that MaineDOT has reviewed the anticipated excavation quantities and results of environmental testing to determine soil reuse options and prepared a soil management special provision.

PRE-CONSTRUCTION CONDITION SURVEY

Due to proximity of adjacent structures to the existing/replacement bridge, we recommend that MaineDOT conduct a pre-construction condition survey of adjacent structures to document both interior and exterior conditions of these structures prior to the start of construction activities.

INSTRUMENTATION

We recommend that an instrumentation program be developed that will allow MaineDOT to monitor vertical movement and vibrations of the A1 Diner and Frosty's building during construction. Threshold and limiting values for settlement and vibrations would be developed by the design team and incorporated into the contract documents. The Contractor would need to consider these settlement criteria when determining construction means and methods and sequencing of the work in the vicinity of the A1 Diner. However, based on recent discussions with MaineDOT it is our understanding that MaineDOT has decided not to implement an instrumentation program for the project.

SUBMITTAL REVIEWS

The Contract Drawings and Specifications should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. The Specifications require that the Contractor and the Contractor's engineer perform analyses and submit results to MaineDOT for review. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

CONSTRUCTION MONITORING

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction activities is required to enable the geotechnical engineer to confirm that procedures and techniques used by the contractor during construction are appropriate and will not impact the design of the bridge. Therefore, we recommend that an individual representing MaineDOT, qualified by geotechnical training and experience be present at the site to provide monitoring during the foundation construction activities listed below.

- Installation and testing of micropiles.
- Preparation of the footing bearing surfaces.
- Placement and compaction of compacted fills.

Limitations and Closure

This report is prepared for the exclusive use of Stantec relative to the Replacement of Bridge Street Bridge in Gardiner, Maine. There are no intended beneficiaries other than Stantec. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than Stantec for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Stantec and from Haley & Aldrich indicating that the Report is adequate for such other use. Use of this report by such other person or entity without the written authorization of Stantec and Haley & Aldrich shall be at such other person's or entities sole risk, and shall be without legal exposure or liability to Haley & Aldrich.

The analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

We understand that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans and specifications) and that the information contained in the CDs takes precedence over the information provided in this report.

We appreciate the opportunity to provide engineering services on this project. Please do not hesitate to call if you have any questions or comments.

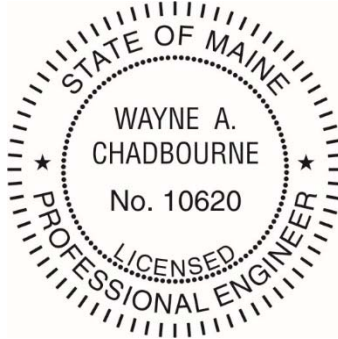
Sincerely yours,
HALEY & ALDRICH, INC.



Erin A. Force, P.E.
Project Manager



Wayne A. Chadbourne, P.E.
Senior Associate



Enclosures:

- Table I – Summary of Subsurface Explorations
- Figure 1 – Project Locus
- Figure 2 – Boring Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Test Boring Logs
- Appendix B – Rock Core Photographs
- Appendix C – Laboratory Test Results
- Appendix D – Historic Bridge Drawings
- Appendix E – A1 Diner Foundations Memorandum
- Appendix F – Calculations

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Table I - Summary of Subsurface Explorations

Bridge Street Bridge

Routes U.S. 201 and 9 over Cobbosseecontee Stream

Gardiner, Maine

MaineDOT WIN 18735.00, Bridge No. 2101

LOCATION DATA:

Test Boring No. ¹	Coordinates ²	
	Easting	Northing
BB-GCS-101	1,141,624	509,739
BB-GCS-101A	1,141,624	509,739
BB-GCS-102	1,141,635	509,813
BB-GCS-102A	1,141,635	509,813
BB-GCS-103	1,141,717	509,942
BB-GCS-201	1,141,577	509,655
BB-GCS-202	1,141,623	509,675
BB-GCS-301	1,141,687	509,631
BB-GCS-401	1,141,691	509,753
BB-GCS-401A	1,141,691	509,755
BB-GCS-402	1,141,675	509,905
BB-GCS-402A	1,141,674	509,903
BB-GCS-403	1,141,699	509,899

SUBSURFACE DATA:

Test Boring No. ¹	Ground Surface Elevation ^{3,4}	Approximate Strata Thickness ⁵ (ft)					Approximate Elevation of Top of Bedrock ^{3,4}	Approximate Elevation of Bottom of Exploration ^{3,4}
		Fill	Alluvial Deposit	Marine Sand	Glacial Till	Weathered Rock		
BB-GCS-101	14.7	12.2	NE	NE	43.7	1.1	-42.3	-52.3
BB-GCS-101A	14.7	> 10.5	--	--	--	--	--	4.2
BB-GCS-102	10.1	> 6.9	--	--	--	--	--	3.2
BB-GCS-102A	10.1	10.5	2.0	NE	50.1	2.4	-54.9	-64.9
BB-GCS-103	30.3	26.2	NE	NE	49.8	NE	-45.7	-45.7
BB-GCS-201	27.8	10.7	NE	> 1.8	--	--	--	15.3
BB-GCS-202	26.8	20.2	NE	> 0.3	--	--	--	6.3
BB-GCS-301	25.3	13.2	NE	NE	> 8.8	--	--	3.3
BB-GCS-401	10.5	9.0	NE	NE	> 27.0	--	--	-25.5
BB-GCS-401A	10.2	9.0	NE	NE	54.3	1.9	-55.0	-70.5
BB-GCS-402	13.0	3.8	5.7	NE	> 15.5	--	--	-12.0
BB-GCS-402A	13.0	3.8	5.7	NE	54.0	NE	-50.5	-66.5
BB-GCS-403	10.5	4.0	3.8	NE	52.2	NE	-49.5	-61.5

Notes:

1. As-drilled test boring locations are shown on Figure 2, Site and Subsurface Exploration Plan.
2. As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment and reference NAD83, Maine 2000 West Zone coordinate system.
3. Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment.
4. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
5. "NE" indicates stratum was not encountered in test boring.
6. "--" indicates test boring was not drilled deep enough to determine presence of stratum.

Prepared By:	EAF	5/23/2017
Checked By:	MLS	6/8/2017
Reviewed By:	WAC	5/26/2017



MAP SOURCE: ESRI

SITE COORDINATES: 44°13'50"N, 69°46'28"W



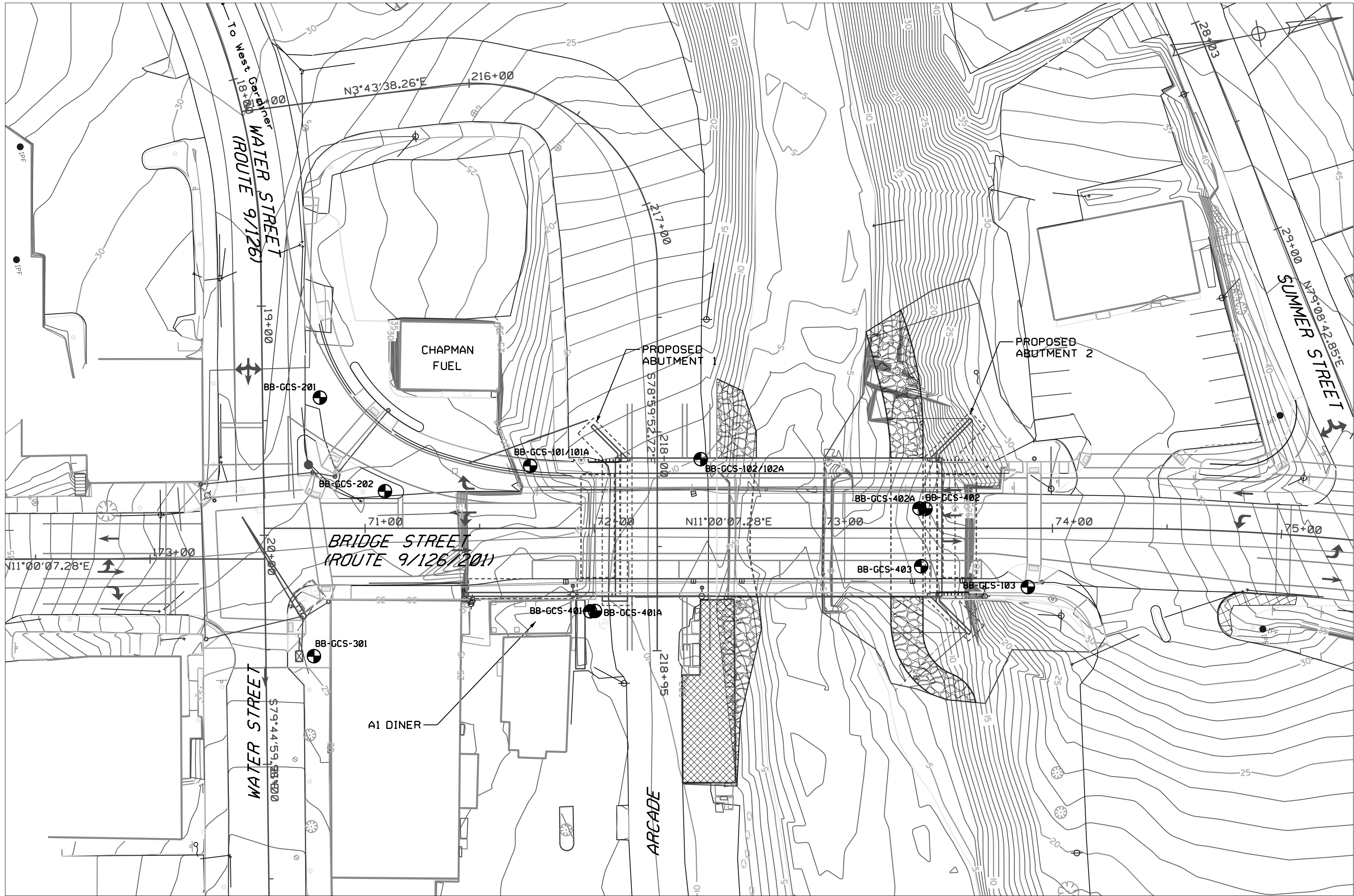
**HALEY
ALDRICH**

BRIDGE STREET BRIDGE
ROUTES U.S. 201 AND 9 OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18735.00, BRIDGE NO. 2101

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
JUNE 2018

FIGURE 1

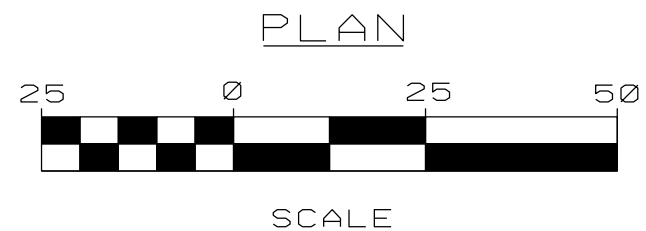


LEGEND

- BB-GCS-101
BB-GCS-201  DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM 9 NOVEMBER TO 1 DECEMBER 2015.
- BB-GCS-301  DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM 24 TO 30 NOVEMBER 2016.
- BB-GCS-401  DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM 10 TO 18 APRIL 2017.

NOTES:

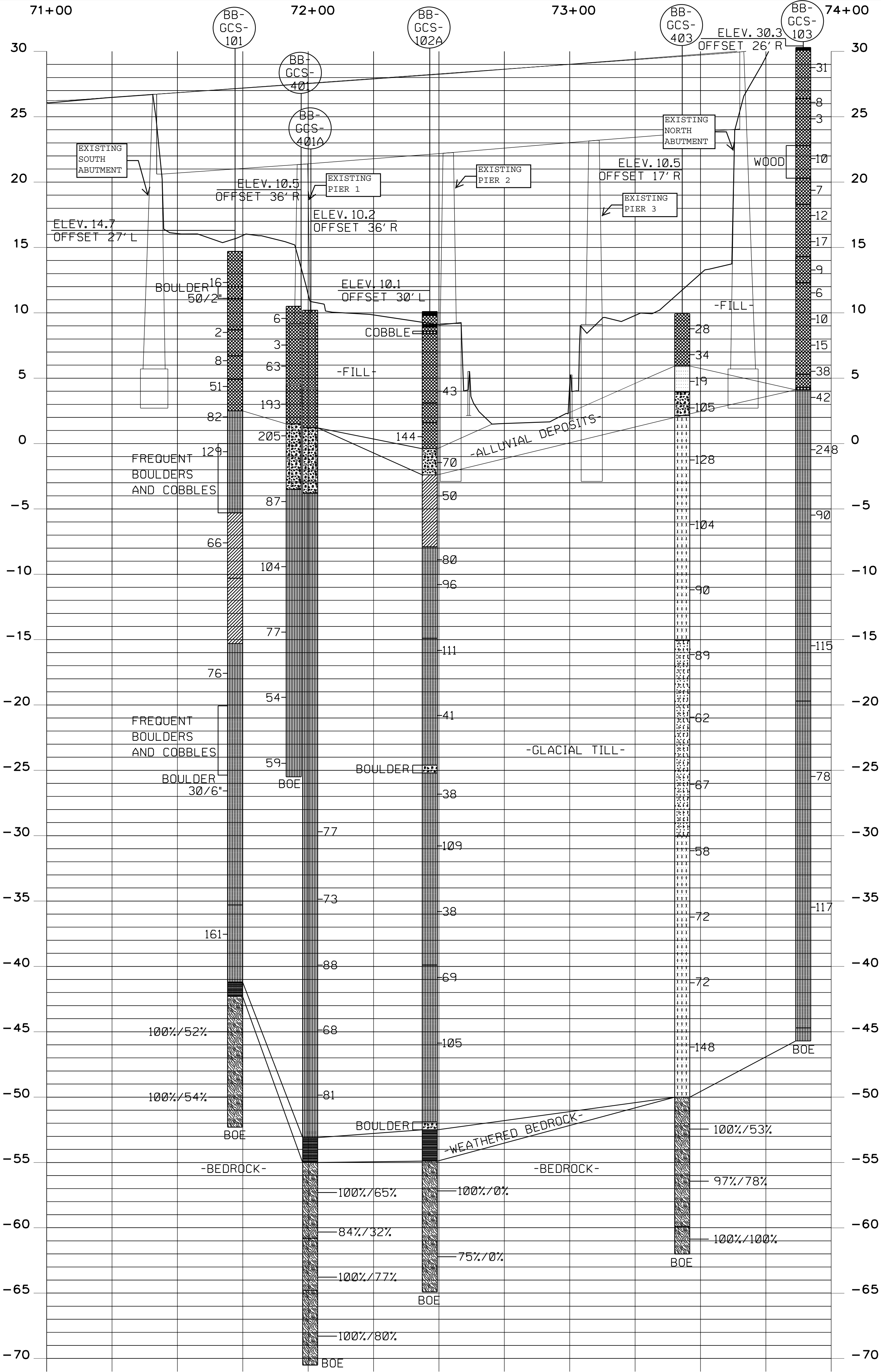
- EXISTING AND PROPOSED SITE CONDITIONS ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY STANTEC ON 12 MAY 2017.
- AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING GPS SURVEY EQUIPMENT.
- SUBSURFACE EXPLORATIONS SHOWN WERE MONITORED BY HALEY & ALDRICH PERSONNEL.
- ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).



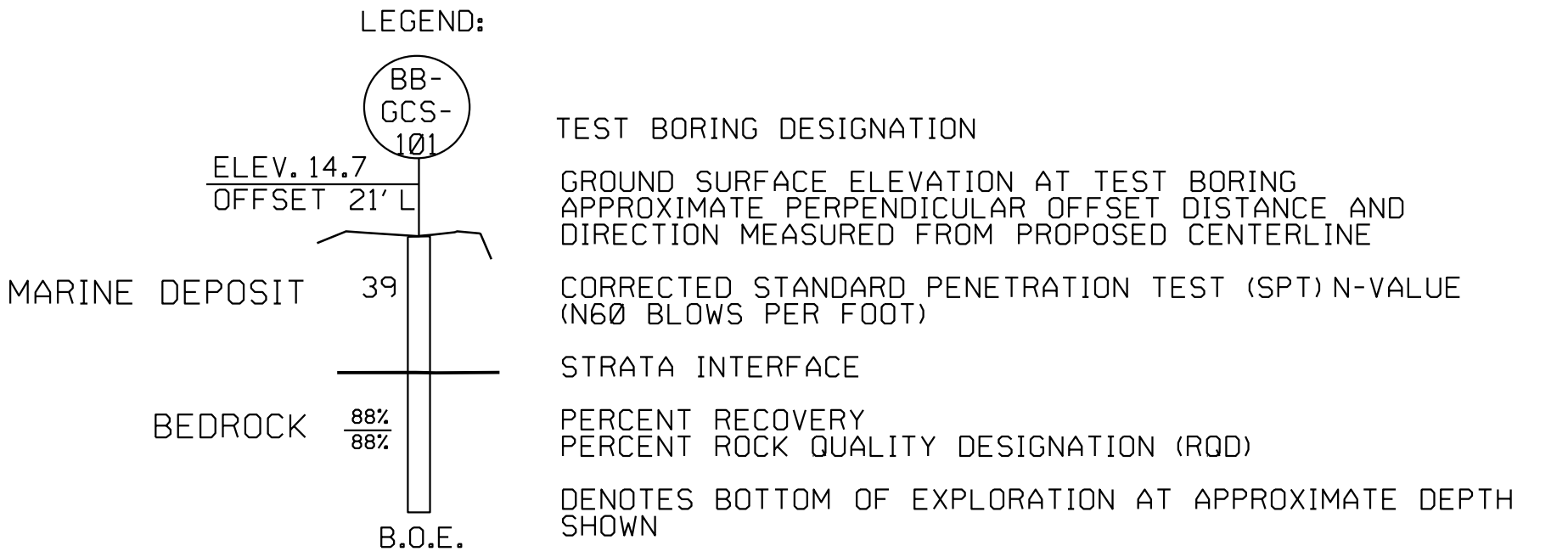
PRELIMINARY
NOT FOR CONSTRUCTION

HALEY
ALDRICH

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	NOT FOR CONSTRUCTION	PROJ. MANAGER	L. TIMBERLAKE	BY	DATE
		CHECKED-REVIEWED	KFP	EAF	5/23/17
BRIDGE STREET BRIDGE COBOSSECONTEE STREAM KENNEBEC COUNTY GARDINER		DESIGNS DETAILED			
		REVISIONS 1			
		REVISIONS 2			
		REVISIONS 3			
SHEET NUMBER		FIELD CHANGES			
2					
OF 3					
STP-1873(500)X		BRIDGE NO. 2101			
WIN		018735.00			
BRIDGE PLANS					

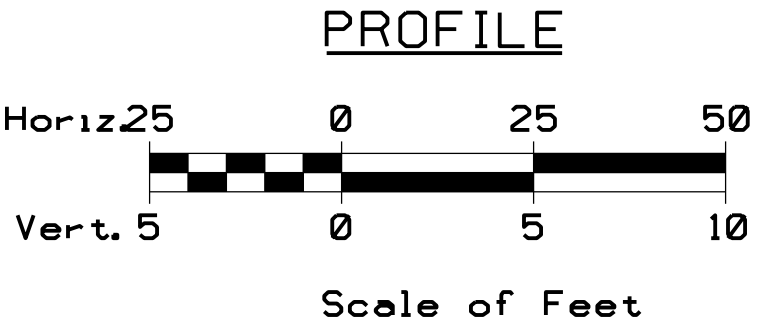


- NOTES:
- 1. THE SUBSURFACE PROFILE IS LOCATED ALONG THE CENTERLINE OF THE PROPOSED BRIDGE.
 - 2. EXISTING GROUND SURFACE/MUDLINE ELEVATIONS AND BRIDGE STRUCTURE GEOMETRY WERE TAKEN FROM THE ELECTRONIC FILES PROVIDED BY STANTEC ON 12 MAY 2017.
 - 3. THE 'AS-DRILLED' TEST BORING LOCATIONS AND GROUND SURFACE ELEVATIONS WERE DETERMINED BY MAINEDOT USING GPS SURVEY EQUIPMENT.
 - 4. LINES REPRESENTING CHANGES IN STRATA ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS. THESE INTERPRETED STRATA LINES DO NOT REPRESENT ACTUAL FIELD CONDITIONS OTHER THAN AT SPECIFIC EXPLORATION LOCATIONS. ACTUAL FIELD CONDITIONS ALONG THE SUBSURFACE PROFILE WILL VARY FROM THOSE SHOWN.
 - 5. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).



- STRATA SYMBOLS
- FILL
 - POORLY GRADED SAND
 - SILTY SAND
 - SILTY SAND AND GRAVEL
 - WEATHERED BEDROCK
 - BEDROCK
 - LOW PLASTICITY ORGANIC SILTS
 - LOW PLASTICITY CLAY
 - FINE GRAVEL
 - WELL GRADED GRAVEL WITH SILT
 - PAVING
 - WELL GRADED SAND
 - WELL GRADED GRAVEL
 - WELL GRADED GRAVEL AND SAND

PRELIMINARY
NOT FOR CONSTRUCTION



HALEY
ALDRICH

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	STP-1873(500)X	WIN 018735.00	BRIDGE NO. 2101	BRIDGE PLANS
NOT FOR CONSTRUCTION				
BRIDGE STREET BRIDGE COBOSSECONTEE STREAM KENNEBEC COUNTY		GARDINER INTERPRETIVE SUBSURFACE PROFILE		
SHEET NUMBER		3 OF 3		

APPENDIX A

Test Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-101 PIN: 18735	
Driller: New England Boring Contractors			Elevation (ft.): 14.7		Auger ID/OD: --		
Operator: M. Porter			Datum: NAVD 88		Sampler: Split Spoon-1.375 in. ID		
Logged By: K. Russ			Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30-HW/NW300#/24		
Date Start/Finish: 11-12-2015/11-17-2015			Drilling Method: HW/NW drive		Core Barrel: NQ-2.0 in. ID		
Boring Location: E1141624; N509739			Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID		Water Level*: 6.0		
Hammer Efficiency Factor: 0.6			Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒				
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows				
0	1D	24/8	0.0 - 2.0	2/7/9/12	16	16	HW				Brown, dry, medium dense, fine to coarse SAND, some gravel, little silt, contains trace concrete, wood chips and roots -FILL-(SW-SM) PID=0.2ppm	G#365344 A-1-b, SW-SM
	2D	8/2	2.0 - 2.7	10/50(2.0")						12.0	Same as 1D, except light brown, very dense due to cobble/boulder PID=0.3ppm	
										11.1	-BOULDER-	
5	3D	24/6	4.0 - 6.0	5/1/1/2	2	2					Dark brown, moist, very loose, fine to coarse SAND, some gravel, little silt, well graded, trace clayey SILT nodules and wood chips -FILL-(SW-SM) PID=0.0ppm	G#365345 A-1-a, SW-SM
	4D	24/1	6.0 - 8.0	4/5/3/12	8	8				8.7	Brown, wet, loose, GRAVEL, trace fine to coarse sand, trace silt, poorly graded -FILL-(GP) PID=0.2ppm	
	5D	24/10	8.0 - 10.0	30/40/11/15	51	51				6.7	Brown, wet, very dense, silty fine to medium SAND, little gravel, trace coarse sand, contains trace wood chips, strong petroleum-like odor at 9.8 ft. -FILL-(SM) PID=0.6ppm	
10	6D	24/7	10.4 - 12.4	15/15/67/74	82	82				4.9	PID=44ppm (9.8-10.0 ft) Similar to 4D -FILL-(GP) PID=16.0ppm (10.4-12.4 ft)	
	7D	24/14	13.0 - 15.0	29/69/60/68	129	129				2.5	Olive-gray, dry, hard, SILT, little fine to coarse sand and gravel, well bonded -GLACIAL TILL-(ML) PID=0.0ppm Note: frequent boulders and cobbles based on drill action and driller observations.	
20	8D	24/21	20.0 - 22.0	23/30/36/50	66	66				-5.3	Olive-gray, dry, hard, CLAY, some fine to medium sand, trace coarse sand, trace gravel, well bonded -GLACIAL TILL-(CL) PID=0.0ppm	G#365346 A-4, CL #365357 WC=9% LL=21, PL=13 PI=8
25												

Remarks:

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

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

Page 1 of 3

Boring No.: BB-GCS-101

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS						Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine			Boring No.: BB-GCS-101 PIN: 18735		
Driller: New England Boring Contractors			Elevation (ft.): 14.7			Auger ID/OD: --					
Operator: M. Porter			Datum: NAVD 88			Sampler: Split Spoon-1.375 in. ID					
Logged By: K. Russ			Rig Type: Mobile B53 Truck			Hammer Wt./Fall: SS-140#/30-HW/NW300#/24					
Date Start/Finish: 11-12-2015/11-17-2015			Drilling Method: HW/NW drive			Core Barrel: NQ-2.0 in. ID					
Boring Location: E1141624; N509739			Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID			Water Level*: 6.0					
Hammer Efficiency Factor: 0.6			Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt			R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person			S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		
Depth (ft.)	Sample Information								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.)			
50	11D	24/24	50.0 - 52.0	29/86/75/163	161	161	NW		Olive-gray, dry, hard, SILT, some gravel, trace fine to coarse sand, well bonded -GLACIAL TILL-(ML) PID=0.0ppm	UC qp=6,343psi	
55								-41.2	Note: Strata change at 55.9 ft based on drill action.		
								-42.3	-WEATHERED BEDROCK-		
	R1	60/60	57.0 - 62.0	RQD = 52%			NQ		Top of Bedrock at El.-42.3 Light gray to gray, fine to medium grained SCHIST with calc-silicate zone from approximately 57.0-57.5 ft and quartz, plagioclase, biotite zone from approximately 57.7-59.6 ft, hard to very hard, slightly weathered. Joints horizontal to low angle. Secondary high angle joint set. Joints very close to close, tight to open, occasional silt infilling. Recovery=100% Rock Mass Quality=Fair R1 Core Times(min/sec): 57.0-58.0'(1:30), 58.0-59.0'(2:00), 59.0-60.0'(2:00), 60.0-61.0'(1:30), 61.0-62.0'(1:30)		
									Gray to dark gray, fine to medium grained SCHIST, hard, slightly weathered. Primary joints dipping at horizontal to low angles, parallel to foliation. High angle secondary joints. Joints very close to close, tight to open, occasional silt infilling. Recovery=100% Rock Mass Quality=Fair R2 Core Times(min:sec): 62.0-63.0'(2:00), 63.0-64.0'(1:30), 64.0-65.0'(2:00), 65.0-66.0'(1:30), 66.0-67.0'(1:30)		
60											
	R2	60/60	62.0 - 67.0	RQD = 53%							
65											
								-52.3	-VASSALBORO FORMATION-		
									Bottom of Exploration at 67.0 feet below ground surface.		
70											
75											
Remarks:											
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-GCS-101	

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-102 PIN: 18735																							
Driller: New England Boring Contractors				Elevation (ft.): 10.1				Auger ID/OD: --																							
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon-1.375 in. ID																							
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30-HW-300#/24																							
Date Start/Finish: 11-09-2015/11-10-2015				Drilling Method: HW Drive				Core Barrel: --																							
Boring Location: E1141635; N509813				Casing ID/OD: HW-4 in. ID				Water Level*: 4.2																							
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒																											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				Su = Insitu Field Vane Shear Strength (psf) Tv = Pocket Torvane Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N60 = SPT N-uncorrected corrected for hammer efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected																							
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Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																					
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows				Elevation (ft.)																				
0							HW	9.6	-BITUMINOUS CONCRETE-																						
	1D	24/12	1.0 - 3.0	24/16/6/5	22	22		6.5	Dark brown, dry, medium dense, fine to coarse SAND, little fine gravel, trace silt -FILL-(SP) PID=0.0ppm		G#365347 A-1-b, SP																				
	2D	7/5	3.0 - 3.6	6/50(1.0")				5.9	Dark brown, moist, very dense, gravelly SAND, little silt, contains brick and concrete fragments, roots -FILL-(SP) PID=0.3ppm																						
5	3D	24/9	4.2 - 6.2	15/19/11/18	30	30		4.1	-BOULDER-																						
	4D	8/7	6.2 - 6.9	30/50(2.0")				3.2	Brown, wet, medium dense, sandy GRAVEL, trace silt, trace roots -FILL-(GW) PID=0.9ppm		G#365348 (4.2'-6.2') A-1-a, GW																				
									Black, wet, medium dense, sandy GRAVEL, little silt, strong petroleum-like odor -FILL-(GP-GM) PID=16.6ppm Similar to above -FILL-(GP-GM) PID=49.9ppm																						
									Bottom of Exploration at 6.9 feet below ground surface.																						
									Note: Could not seat casing and contain water; terminated boring at 6.9 ft. See Test Boring Report BB-GCS-102A.																						
25																															
Remarks:																															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.																															
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.																															
Page 1 of 1										Boring No.: BB-GCS-102																					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-102A PIN: 18735																																																																																																																																																																																																																																																														
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	0							HW SPUN	9.8	-BITUMINOUS CONCRETE-	G#365349 A-6, CL #365358 WC=20% LL=30, PL=15 PI=15									9.1	-FILL-									8.9	-BITUMINOUS CONCRETE-									8.6	-FILL-									8.4	-COBBLE-(Based on drill action)										Note: Begin sampling at 5.0 ft. See Test Boring Report BB-GCS-102 for soil descriptions 0-5.0 ft.	5	1D	24/3	5.0 - 7.0	36/33/10/12	43	43			Brown, wet, dense, sandy GRAVEL, trace silt, contains glass, roots										-FILL-(GP) PID=0.7ppm									3.1	Note: Observed wood, ceramics and probable boulder fragments in drill wash water from approximately 7.0-8.5 ft.		2D	24/8	8.5 - 10.5	30/55/89/22	144	144		1.6	Gray, wet, very dense, fine GRAVEL, some coarse sand, little fine to medium sand, little silt, contains ash, roots and paint chips										-FILL-(GM) PID=0.8ppm	10	3D	24/4	10.5 - 12.5	96/51/19/30	70	70		-0.4	Light gray, wet, very dense, fine GRAVEL, some coarse sand, little medium sand and coarse gravel									-2.4	-ALLUVIAL DEPOSIT-(GW) PID=0.4ppm		4D	24/24	13.0 - 15.0	19/25/25/32	50	50	NW		Olive-gray, dry, hard, silty CLAY, little fine to coarse sand, trace gravel, well bonded										-GLACIAL TILL-(CL) PID=0.0ppm	15											5D	24/24	18.0 - 20.0	23/41/39/55	80	80		-7.9	Olive-gray, dry, hard, clayey SILT, some coarse gravel, trace fine to coarse sand, trace fine gravel, well bonded										-GLACIAL TILL-(ML)	20	MD	24/0	20.0 - 22.0	25/40/56/68	96	96																																		25									
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-102A PIN: 18735																																																																																																																																																																																																																																																																																																																																																																																						
Driller: New England Boring Contractors				Elevation (ft.): 10.1				Auger ID/OD: --																																																																																																																																																																																																																																																																																																																																																																																						
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon-1.375 in. ID																																																																																																																																																																																																																																																																																																																																																																																						
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30-HW/NW300#/24																																																																																																																																																																																																																																																																																																																																																																																						
Date Start/Finish: 11-10-2015/11-12-2015				Drilling Method: HW/NW Drive				Core Barrel: NQ-2.0 in. ID																																																																																																																																																																																																																																																																																																																																																																																						
Boring Location: E1141635; N509813				Casing ID/OD: HW-4 in. ID/NW-3 in. ID				Water Level*: 4.4																																																																																																																																																																																																																																																																																																																																																																																						
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒																																																																																																																																																																																																																																																																																																																																																																																										
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_u(lab) = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																																																																																																																																																																																																																														
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40	9D	24/2	40.0 - 42.0	30/53/56/60	109	109					Olive gray, dry, hard sandy SILT, well bonded -GLACIAL TILL-(ML)																																																																																																																																																																																																																																																																																																																																																																																			
45	10D	24/24	45.0 - 47.0	15/18/20/25	38	38					Olive-gray, dry, hard SILT, little clay, little fine sand, trace medium to coarse sand, trace gravel -GLACIAL TILL-(ML)																																																																																																																																																																																																																																																																																																																																																																																			
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Boring No.: BB-GCS-102A																																																																																																																																																																																																																																																																																																																																																																																														

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-102A PIN: 18735				
Driller: New England Boring Contractors				Elevation (ft.): 10.1				Auger ID/OD: --				
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon-1.375 in. ID				
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30-HW/NW300#/24				
Date Start/Finish: 11-10-2015/11-12-2015				Drilling Method: HW/NW Drive				Core Barrel: NQ-2.0 in. ID				
Boring Location: E1141635; N509813				Casing ID/OD: HW-4 in. ID/NW-3 in. ID				Water Level*: 4.4				
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows				
50	11D	24/4	50.0 - 52.0	17/32/37/62	69	69	NW					
55	12D	24/9	55.0 - 57.0	29/49/56/76	105	105						
60												
65	R1	60/60	65.0 - 70.0	RQD = 0%			NQ					
70	R2	60/45	70.0 - 75.0	RQD = 0%								
75												

Remarks:

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.

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Boring No.: BB-GCS-102A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-102A PIN: 18735					
Driller: New England Boring Contractors		Elevation (ft.) 10.1		Auger ID/OD: --							
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon-1.375 in. ID							
Logged By: K. Russ		Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30-HW/NW300#/24							
Date Start/Finish: 11-10-2015/11-12-2015		Drilling Method: HW/NW Drive		Core Barrel: NQ-2.0 in. ID							
Boring Location: E1141635; N509813		Casing ID/OD: HW-4 in. ID/NW-3 in. ID		Water Level*: 4.4							
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person		S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected		S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test					
Depth (ft.)	Sample Information								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.)			
75								-64.9	Bottom of Exploration at 75.0 feet below ground surface.		
100											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 4 of 4	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GCS-102A	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-103 PIN: 18735	
Driller: New England Boring Contractors		Elevation (ft.): 30.3		Auger ID/OD: --			
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon-1.375 in. ID			
Logged By: K. Russ		Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30-HW/NW300#/24			
Date Start/Finish: 11-17-2015/11-20-2015		Drilling Method: HW/NW Drive		Core Barrel: --			
Boring Location: E1141717; N509942		Casing ID/OD: HW-4 in. ID/NW-3 in. ID		Water Level*: 14.0			
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/8	0.5 - 2.5	14/13/18/18	31	31	HW	30.1		-BITUMINOUS CONCRETE-		
										Brown, dry, dense, gravelly fine SAND, little medium to coarse sand, trace silt		
	2D	24/17	2.5 - 4.5	13/5/3/2	8	8				-FILL-(SW) PID=0.0ppm Brown, dry, loose, fine to coarse SAND, some gravel, little silt	G#365350 A-1-b, SM	
										-FILL-(SM) PID=0.0ppm		
5	3D	24/18	4.5 - 6.5	1/2/1/2	3	3		26.4		3.9	Olive, reddish-brown, moist, soft SILT, little fine gravel, trace fine to coarse sand, contains brick fragments	G#365351 (4.5'-6.5') A-4, ML
										-FILL-(ML) Olive, reddish-brown, moist, very loose, sandy SILT, trace fine gravel, contains brick fragments		
	4D	24/4	7.5 - 9.5	9/6/4/5	10	10		22.8		7.5	-FILL-(ML) PID=1.4ppm	
										-WOOD- PID=0.7ppm		
10	5D	24/18	10.0 - 12.0	7/4/3/3	7	7		20.3		10.0	Olive-gray, moist, medium stiff, silty CLAY, contains wood and brick fragments	
										-FILL-(CL) PID=0.0ppm		
	6D	24/11	12.0 - 14.0	4/7/5/5	12	12		18.3		12.0	Olive-brown, moist, medium dense, sandy SILT, trace gravel, contains wood fragments	G#365352 A-4, ML
										-FILL-(ML) PID=0.0ppm		
15	7D	24/8	14.0 - 16.0	8/9/8/7	17	17		14.3	16.0	Olive-gray, wet, medium dense, silty fine SAND, little coarse gravel, trace clay, trace coarse sand and fine gravel, contains wood fragments		
									-FILL-(SM) PID=0.1ppm			
	8D	24/5	16.0 - 18.0	3/3/6/9	9	9		12.3	18.0	Olive-gray, wet, loose, silty GRAVEL, trace fine to coarse sand, trace clay, contains wood fragments		
									-FILL-(GM)			
20	9D	24/12	18.0 - 20.0	4/3/3/3	6	6				Olive-gray, wet, medium stiff, clayey SILT, little fine sand and gravel, contains wood fragments		
									-FILL-(ML)			
	10D	24/3	20.0 - 22.0	4/6/4/7	10	10				Olive-gray, wet, stiff, clayey SILT, contains wood fragments		
									-FILL-(ML)			
	11D	24/12	22.0 - 24.0	9/6/9/9	15	15				Olive-gray, wet, stiff, fine sandy SILT, trace medium sand, contains wood fragments		
									-FILL-(ML)			
25	12D	24/8	24.0 - 26.0	73/29/9/12	38	38				Olive-gray, wet, hard, fine, sandy SILT, little gravel, trace medium to coarse sand, contains wood fragments		

Remarks:

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.

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Boring No.: BB-GCS-103

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-103 PIN: 18735					
Driller: New England Boring Contractors				Elevation (ft.): 30.3				Auger ID/OD: --					
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon-1.375 in. ID					
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30-HW/NW300#/24					
Date Start/Finish: 11-17-2015/11-20-2015				Drilling Method: HW/NW Drive				Core Barrel: --					
Boring Location: E1141717; N509942				Casing ID/OD: HW-4 in. ID/NW-3 in. ID				Water Level*: 14.0					
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected					
S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test													
Sample Information										Graphic Log		Laboratory Testing Results/ AASHTO and Unified Class.	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N₆₀	Casing Blows	Elevation (ft.)	Visual Description and Remarks				
25							HW	4.3	-FILL-(ML)				
	13D	24/10	26.0 - 28.0	26/20/22/19	42	42		4.1	Olive-brown, wet, dense, coarse GRAVEL, little fine to coarse sand and fine gravel, trace silt, contains roots and wood fragments				
									-FILL-(GW)				
									Olive-gray, dry, hard, clayey SILT, little fine gravel and coarse sand, trace fine to medium sand				
									-GLACIAL TILL-(ML)				
30	14D	18/18	30.0 - 31.5	33/48/200	248	248			Olive-gray, dry, hard, fine sandy SILT, little fine gravel, trace medium to coarse sand, well bonded				
									-GLACIAL TILL-(ML)				
35	15D	24/24	35.0 - 37.0	28/44/46/52	90	90			Olive-gray, dry, hard, fine sandy SILT, little medium sand, trace coarse sand and gravel, well bonded				
									-GLACIAL TILL-(ML)				
45	16D	24/24	45.0 - 47.0	19/60/55/70	115	115			Olive-gray, dry, hard, fine sandy SILT, little coarse gravel, trace medium to coarse sand, trace fine gravel, well bonded				
									-GLACIAL TILL-(ML)				
50													
Remarks:													
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 4			
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GCS-103			

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-103 PIN: 18735																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
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<table><tr><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>Depth (ft.)</th><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>50</td><td></td><td></td><td></td><td></td><td></td><td></td><td>HW</td><td></td><td rowspan="10"></td><td>Note: Losing water at 50.0 ft.</td><td rowspan="10"></td></tr><tr><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></tr><tr><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></tr><tr><td>55</td><td>17D</td><td>24/24</td><td>55.0 - 57.0</td><td>14/33/45/59</td><td>78</td><td>78</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></tr><tr><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></tr><tr><td></td><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></td><td rowspan="10"></td><td>Olive-gray, dry, hard, fine sandy SILT, little coarse sand, trace medium sand and fine to coarse gravel, well bonded -GLACIAL TILL-(ML)</td><td rowspan="10"></td></tr><tr><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></tr><tr><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></tr><tr><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></tr><tr><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></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>65</td><td>18D</td><td>24/24</td><td>65.0 - 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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-103 PIN: 18735					
Driller: New England Boring Contractors		Elevation (ft.) 30.3		Auger ID/OD: --							
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon-1.375 in. ID							
Logged By: K. Russ		Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30-HW/NW300#/24							
Date Start/Finish: 11-17-2015/11-20-2015		Drilling Method: HW/NW Drive		Core Barrel: --							
Boring Location: E1141717; N509942		Casing ID/OD: HW-4 in. ID/NW-3 in. ID		Water Level*: 14.0							
Hammer Efficiency Factor: 0.6		Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person		S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected		S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test					
Depth (ft.)	Sample Information								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.)			
75								-45.7	Note: Advanced roller bit to probable Top of Bedrock at El.-45.7. Bottom of Exploration at 76.0 feet below ground surface.		
100											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 4 of 4	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GCS-103	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-201 PIN: 18735			
Driller: New England Boring Contractors				Elevation (ft.): 27.8				Auger ID/OD: HSA-2-1/4 in. ID			
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon-1.375 in. ID			
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30			
Date Start/Finish: 12-1-2015/12-1-2015				Drilling Method: Auger				Core Barrel: --			
Boring Location: E1141577; N509655				Casing ID/OD: --				Water Level*: None observed			
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/20	0.5 - 2.5	25/17/7/5	24	24	H\$A	27.4		-BITUMINOUS CONCRETE-	
										Olive-brown, dry, medium dense, fine to medium SAND, some gravel, trace coarse sand, trace silt	0.4
	2D	24/9	2.5 - 4.5	6/5/10/8	15	15		25.7		-FILL-(SW) PID=0.4ppm	
										Olive-gray, wet, very stiff, CLAY, trace fine sand, little gravel	2.1
	3D	24/17	4.5 - 6.5	6/7/7/6	14	14		23.3		-FILL-(CL) Olive-brown, wet, stiff, silty CLAY, little fine sand, trace coarse sand, trace gravel, contains brick fragments	
5								22.7		-FILL-(CL) PID=0.0ppm	
	4D	24/8	6.5 - 8.5	8/18/10/13	28	28				Olive-gray, dry, medium dense, silty fine to medium SAND, trace coarse sand and fine gravel	4.5
										-FILL-(SM) PID=0.0ppm	
	5D	24/15	8.5 - 10.5	19/37/36/16	73	73		19.3		Dark brown, dry, stiff, SILT, some fine sand, trace medium sand and coarse gravel, contains wood	5.1
								18.8		-FILL-(ML) PID=0.1ppm	
10	6D	24/24	10.5 - 12.5	6/7/8/10	15	15		17.3		Dark brown, dry, very stiff, SILT, some fine sand, contains wood	
								17.1		-FILL-(ML) PID=0.0ppm	
										Olive-gray, dry, hard, SILT, trace fine to medium sand, trace coarse gravel, contains wood	8.5
										-FILL-(ML) PID=0.0ppm	
15										Olive-brown, dry, very dense, fine to coarse SAND, some silt, little gravel, well graded	9.0
										-FILL-(SW-SM) PID=0.0ppm	
										Olive, moist, medium dense, silty fine SAND	10.5
										-FILL-(SM) PID=0.0ppm	
										Olive, moist, medium dense, silty fine SAND	10.7
										-MARINE DEPOSIT-(SM) PID=0.0ppm	
20											12.5
										Bottom of Exploration at 12.5 feet below ground surface.	
25											
Remarks: No water observed in borehole.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 1 of 1 Boring No.: BB-GCS-201	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-202 PIN: 18735	
Driller: New England Boring Contractors		Elevation (ft.): 26.8		Auger ID/OD: HSA-2-1/4 in. ID			
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon-1.375 in. ID			
Logged By: K. Russ		Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30			
Date Start/Finish: 12-1-2015/12-1-2015		Drilling Method: Auger		Core Barrel: --			
Boring Location: E1141623; N509675		Casing ID/OD: --		Water Level*: 16.8			
Hammer Efficiency Factor: 0.6				Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☒			
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N_{60} = SPT N-uncorrected corrected for hammer efficiency N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected $S_{u(lab)}$ = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/15	0.5 - 2.5	10/12/9/18	21	21	HSA	26.5		-BITUMINOUS CONCRETE-	
								24.8		Gray-brown, dry, medium dense, fine to coarse SAND, little silt, little gravel, well graded	
	2D	24/17	2.5 - 4.5	9/4/3/3	7	7		24.3		-FILL-(SW) PID=1.1ppm	
										Black and olive-brown, dry, very stiff, SILT, little coarse sand and gravel, trace fine to medium sand, contains asphalt, asphalt-like odor	
5	3D	24/8	4.5 - 6.5	4/6/5/7	11	11		22.3		-FILL-(ML) PID=0.8ppm	
										Olive-brown, moist, medium stiff, clayey SILT, trace fine to coarse sand, asphalt-like odor	
	4D	24/2	6.5 - 8.5	5/6/12/9	18	18				-FILL-(ML) PID=1.2ppm	
										Olive-brown, dry, stiff, SILT and fine to coarse SAND, little gravel, asphalt-like odor	
	5D	24/5	8.5 - 10.5	7/9/10/10	19	19				-FILL-(ML/SM) PID=0.6ppm	
										Olive-gray, dry, very stiff, SILT, little fine sand and coarse gravel, trace medium to coarse sand and fine gravel, slight asphalt-like odor, gravel in spoon tip	
10	6D	24/10	10.5 - 12.5	4/4/12/9	16	16			-FILL-(ML) PID=0.4ppm		
									Olive-brown, dry, very stiff, clayey SILT, little coarse gravel, trace fine and coarse sand, slight asphalt-like odor		
	7D	24/8	12.5 - 14.5	7/7/22/52	29	29			-FILL-(ML) PID=0.9ppm		
									Olive-brown, dry, very stiff, SILT, trace fine and coarse sand, trace coarse gravel, slight asphalt-like odor, contains brick and concrete fragments.		
15	8D	24/7	14.5 - 16.5	13/10/4/10	14	14		12.3	-FILL-(ML) PID=1.6ppm		
									Olive-brown, moist, very stiff, SILT, little fine to coarse sand, trace coarse gravel, asphalt-like odor, contains brick fragments		
	9D	24/10	16.5 - 18.5	21/6/12/14	18	18		10.3	-FILL-(ML) PID=1.0ppm		
								10.0			
	10D	24/4	18.5 - 20.5	5/9/15/13	24	24		8.6	Olive-brown, dry, medium dense, coarse GRAVEL, some fine to medium sand, little silt, trace fine gravel, slight asphalt-like odor		
									-FILL-(GM) PID=1.7ppm		
20								6.6	Olive-brown, moist, medium dense, fine SAND, some silt, little gravel, trace medium to coarse sand		
								6.3	-FILL-(SM) PID=0.8ppm		
									Olive-gray, wet, very stiff, gravelly CLAY		
									-FILL-(CL) PID=2.5ppm		
25									Gray, wet, very stiff, fine sandy SILT		

Remarks:

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

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

Page 1 of 2

Boring No.: BB-GCS-202

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-301 PIN: 18735.00					
Driller: New England Boring Contractors		Elevation (ft.): 25.3		Auger ID/OD: --							
Operator: C. Dupuis		Datum: NAVD 88		Sampler: Split Spoon-1.375 in.ID							
Logged By: M. Snow		Rig Type: Mobile B53 Truck		Hammer Wt./Fall: SS-140#/30;HW/NW-300#/24							
Date Start/Finish: 11-29-16/11-30-16		Drilling Method: HW/NW Drive		Core Barrel: NQ-2.0 in.ID							
Boring Location: E1141687; N509631		Casing ID/OD: HW-4.0 in.ID/NW-3.0 in.ID		Water Level*: 19.0							
Hammer Efficiency Factor: 0.569		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0	1D	24/5	0.5 - 2.5	4/7/7/3	14	13	SSA				
5	2D	24/24	5.0 - 7.0	10/8/4/4	12	11	21				
							8				
							10(6.0) SSA				
10	3D	18/16	10.0 - 11.5	33/35/60	95	90	NW Drive				
	4D	18/16	12.0 - 13.5	33/35/50	85	81					
	5D	5/2	14.0 - 14.4	60(5.0)							
15	6D	11/5	15.0 - 15.9	45/70(5.0)							
							Open Hole				
20	7D	24/12	20.0 - 22.0	9/18/21/28							
25											

Remarks:

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.

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Boring No.: BB-GCS-301

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-301 PIN: 18735.00							
Driller: New England Boring Contractors				Elevation (ft.): 25.3				Auger ID/OD: --							
Operator: C. Dupuis				Datum: NAVD 88				Sampler: Split Spoon-1.375 in.ID							
Logged By: M. Snow				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: SS-140#/30;HW/NW-300#/24							
Date Start/Finish: 11-29-16/11-30-16				Drilling Method: HW/NW Drive				Core Barrel: NQ-2.0 in.ID							
Boring Location: E1141687; N509631				Casing ID/OD: HW-4.0 in.ID/NW-3.0 in.ID				Water Level*: 19.0							
Hammer Efficiency Factor: 0.569				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.			
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows								
	25													PID=0.0 ppm	
														Bottom of Exploration at 22.0 feet below ground surface.	
Remarks:															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GCS-301					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-401 PIN: 18735.00			
Driller: New England Boring Contractors				Elevation (ft.): 10.5				Auger ID/OD: --			
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID			
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24			
Date Start/Finish: 04-14-2017/04-15-2017				Drilling Method: HW Drive				Core Barrel: NQ-1.875 in. ID			
Boring Location: E1141691, N509753				Casing ID/OD: HW-4.0 in. ID				Water Level*: Approx. 2.0 ft			
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
0	1D	24/6	0.0 - 2.0	3/2/3/3	5	6	HW Push			Brown, dry, loose, fine to coarse SAND, trace silt, trace gravel -FILL-(SW) PID=0.0 ppm No Recovery (very loose soil) Note: Water encountered at approximately 2.0 ft. Brown, wet, very dense, gravelly fine to coarse SAND, trace silt -FILL-(SW-SM) PID=0.0 ppm Brown to gray, wet, very dense, fine to coarse SAND, little gravel, trace silt, strong petroleum-like odor (gray color approximately 6.3 to 6.5 ft) -FILL-(SW) PID=38 ppm Gray, wet, very dense, GRAVEL, some fine to coarse sand, trace silt, strong petroleum-like odor -FILL-(GW) PID=241 ppm Gray, wet, very dense, GRAVEL, some fine to coarse sand, trace silt, well bonded, slight petroleum-like odor -GLACIAL TILL-(GW) PID=5.6 ppm Note: Wash ahead of casing through boulders from 12 to 14 ft. Gray, wet, very dense, SILT, little fine to medium sand, little gravel, trace coarse sand, well bonded -GLACIAL TILL-(ML) PID=0.0 ppm Note: Wash ahead of casing due to high density of glacial till. Gray, wet, very dense, SILT, little fine sand, little gravel, trace medium to coarse sand, well bonded -GLACIAL TILL-(ML)	(0-8.2"); WC=7.0% ER=992 ohm-cm pH=6.1/5.9 Sulfate=1,616 mg/kg Chloride=320 mg/kg OC=1.5% G#411992 (A-1-a(1)), SW-SM WC=11.2% (12.5-21.0"); WC=7.0% ER=3,306 ohm-cm pH=6.1/6.2 Sulfate=1,616 mg/kg Chloride=320 mg/kg OC=1.5% G#439702 (A-1-a(1)), GP G#411993 (A-4(0)), ML WC=8.7%
	MD	24/0	2.0 - 4.0	1/2/1/1	3	3					
5	2D	16/7	4.0 - 5.3	4/6/50(4.0)	56	63	✓				
							42				
	3D	6/6	6.0 - 6.5	75			115				
	4D	14/7	7.0 - 8.2	27/51/120(2.0)	171	193	181				
							307				
10	5D	24/10	9.0 - 11.0	34/108/74/32	182	205	75				
							175				
							435				
							172+				
							147				
15	6D	24/24	14.0 - 16.0	31/35/42/41	77	87	134				
							63				
							69				
							76				
							79				
20	7D	24/11	19.0 - 21.0	16/41/51/56	92	104	70				
							70				
							87				
							80				
							99				
25	8D	24/24	24.0 - 26.0	10/25/43/52	68	77	70				

Remarks:

 Notes:
 1. ER = Electrical Resistivity
 2. OC = Organic Content

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.

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 Boring No.: BB-GCS-401

[illegible]

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-401A PIN: 18735.00				
Driller: New England Boring Contractors				Elevation (ft.): 10.2				Auger ID/OD: --				
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID				
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;NW-140#/30				
Date Start/Finish: 04-17-2017/04-18-2017				Drilling Method: HSA/NW Drive				Core Barrel: NQ-1.875 in. ID				
Boring Location: E1141691, N509755				Casing ID/OD: HSA-4.25 in. ID/NW-3.0 in. ID				Water Level*:				
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25							37					
							33					
							38					
							61					
							38					
30							29					
							33					
							38					
							32					
							120					
35							415					
							126					
							94					
							95					
	11D	24/8	39.0 - 41.0	16/28/40/49	68	77	180			Gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded -GLACIAL TILL-(ML)		
							62					
							90					
							105					
							90					
	12D	24/24	44.0 - 46.0	21/30/35/55	65	73	162			Gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded -GLACIAL TILL-(ML)		
							133					
							98					
							100					
							117					
50	13D	24/24	49.0 - 51.0	23/33/45/66	78	88	102			Gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded		
Remarks:												
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-GCS-401A		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-401A PIN: 18735.00	
Driller: New England Boring Contractors		Elevation (ft.): 10.2		Auger ID/OD: --			
Operator: B. Enos		Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID			
Logged By: K.Russ/M.Snow		Rig Type: Mobile B53 Track		Hammer Wt./Fall: SS-140#/30;NW-140#/30			
Date Start/Finish: 04-17-2017/04-18-2017		Drilling Method: HSA/NW Drive		Core Barrel: NQ-1.875 in. ID			
Boring Location: E1141691, N509755		Casing ID/OD: HSA-4.25 in. ID/NW-3.0 in. ID		Water Level*:			
Hammer Efficiency Factor: 0.677		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50							118			-GLACIAL TILL-(ML)	G#439704 (A-4(0), ML)	
							130					
							183					
							267					
55	14D	24/24	54.0 - 56.0	16/25/35/57	60	68	218			Gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded		
							142		-GLACIAL TILL-(ML)			
							145					
							165					
							119					
60	15D	24/24	59.0 - 61.0	25/31/41/76	72	81	-		Gray, wet, very dense, SILT, little fine sand, trace medium to coarse sand, little gravel, well bonded			
							-		-GLACIAL TILL-(ML)			
65	16D	2/2	64.0 - 64.2	75(2.0)			-		-53.1	Top of weathered rock at 63.3 ft Note: Advanced roller bit to 64.0 ft. -WEATHERED ROCK-		
	R1	54/54	65.2 - 69.7	RQD = 65%			NQ Core		-55.0	Top of Bedrock El. -55.0 Gray, fine to medium grained GRANOFELS with pegmatite intrusion from 66.8 to 77.4 ft. Moderately hard to hard, fresh to slightly weathered. Joints dipping at moderate angles, moderately close, tight to open, occasional oxidized joint surfaces. Rock is highly fractured due to drill breaks. Rock Mass Quality=Fair Recovery=100% R1 Core Times (min:sec): 65.2-66.2' (2: 06); 66.2-67.2' (2:28); 67.2-68.2' (1:37); 68.2-69.2' (2:22); 69.2-69.7' (1:36) Gray, fine to medium grained GRANOFELS. Moderately hard to hard, moderately weathered. Joints dipping at moderate angles, very close, open, weathered joint surfaces. Rock Mass Quality=Poor Recovery=84% R2 Core Times (min:sec): 69.7-70.7' (1:53); 70.7-71.3' (2:14)		
70	R2	19/16	69.7 - 71.3	RQD = 32%								
	R3	53/53	71.3 - 75.7	RQD = 77%								
75										Light gray to gray, medium grained QUARTZITE. Very hard to hard, slightly weathered. Joints dipping at low to moderate angles, close to moderately close, tight to open, oxidized joint surfaces.		

Remarks:

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.

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Boring No.: BB-GCS-401A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-401A PIN: 18735.00																							
Driller: New England Boring Contractors				Elevation (ft.): 10.2				Auger ID/OD: --																							
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID																							
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;NW-140#/30																							
Date Start/Finish: 04-17-2017/04-18-2017				Drilling Method: HSA/NW Drive				Core Barrel: NQ-1.875 in. ID																							
Boring Location: E1141691, N509755				Casing ID/OD: HSA-4.25 in. ID/NW-3.0 in. ID				Water Level*:																							
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected																							
S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																															
<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</th><th>Blows</th></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
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																							
75	R4	60/60	75.7 - 80.7	RQD = 80%			NQ Core		-64.8		Rock Mass Quality=Good Recovery=100% R3 Core Times (min:sec): 71.3-72.3' (2:01); 72.3-73.3' (1:26); 73.3-74.3' (1:56) Light gray to gray, medium to coarse grained GRANOFELS. Moderately hard to hard, fresh to slightly weathered. Joints dipping at low to moderate angles, close to moderately close, tight to open, occasional <1/16 in. silt infillings. Frequent bands of pegmatite throughout run. Rock Mass Quality=Good Recovery=100% -VASSALBORO FORMATION- Bottom of Exploration at 80.7 feet below ground surface.	UC; qp=3,043psi																			
Remarks:																															
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 4 of 4																					
										Boring No.: BB-GCS-401A																					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-402 PIN: 18735.00				
Driller: New England Boring Contractors				Elevation (ft.): 13.0				Auger ID/OD: --				
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID				
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24				
Date Start/Finish: 04-11-2017/04-12-2017				Drilling Method: HW Drive				Core Barrel: NQ-1.875 in. ID				
Boring Location: E1141675, N509905				Casing ID/OD: HW-4.0 in. ID				Water Level*: 5.0 ft				
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/5	0.0 - 2.0	1/1/1/2	2	2	HW Push			Brown, dry, very loose, fine to medium SAND, some gravel and silt, trace coarse sand -FILL-(SM) PID=0.0 ppm	G#411994 A-2-4(0), SM WC=10.5%	
							5					
	2D	24/7	2.0 - 4.0	4/4/12/4	16	18	12			Brown, moist, medium dense, fine to medium SAND, little coarse sand and gravel, some silt -FILL-(SM) PID=0.0 ppm		
							15					
5	3D	24/7	4.0 - 6.0	5/8/6/5	14	16	23		9.2	*Dark brown, moist, medium dense, fine SAND, some silt, trace organics, rootlets -ALLUVIAL DEPOSIT-(SM)	G#411995 A-1-b(0), SM WC=18.6%	
							65					
	4D	24/8	6.0 - 8.0	3/18/20/15	38	43	87		6.5	Dark brown, wet, dense, silty fine to medium SAND, trace coarse sand, little gravel -ALLUVIAL DEPOSIT-(SM)		
							83					
	5D	20/10	8.0 - 9.7	13/22/31/50(2.0)	53	60	105			Dark brown, wet, dense, fine to coarse GRAVEL, some silt, little fine to coarse sand -ALLUVIAL DEPOSIT-(GW-GM)		
							120		3.5	Dark gray-brown, wet, very dense, fine to coarse GRAVEL, little silt, little fine to coarse sand -ALLUVIAL DEPOSIT-(GW-GM)		
10							64					
							52			COBBLE/GRAVEL layer; washed ahead of casing -GLACIAL TILL-		
							105		1.0			
	6D	24/10	13.0 - 15.0	18/22/28/30	50	56	107			Gray, wet, very dense, SILT, little fine to medium sand, trace coarse sand, trace gravel, well bonded -GLACIAL TILL-(ML)	G#439705 (A-4(0), ML)	
							101					
15							152					
							173			Note: Wash ahead of casing through cobbles from 15 to 18 ft.		
							171					
	7D	14/20	18.0 - 19.2	18/40/75(2.0)						Dark gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded -GLACIAL TILL-(ML)		
20												
	8D	24/22	23.0 - 25.0	24/31/45/59	76	86				Dark gray, wet, very dense, SILT, little fine to coarse sand, little gravel, well bonded -GLACIAL TILL-(ML)		
25												
Remarks: *Lab test results do not reflect field observations. Description reported here is based on field observations.												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 1 of 2 Boring No.: BB-GCS-402		

[illegible]

[illegible]

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-402A PIN: 18735.00																							
Driller: New England Boring Contractors				Elevation (ft.): 13.0				Auger ID/OD: --																							
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID																							
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24																							
Date Start/Finish: 04-12-2017/04-12-2017				Drilling Method: HW/NW Drive				Core Barrel: NQ-1.875 in. ID																							
Boring Location: E1141674, N509903				Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID				Water Level*:																							
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person				S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected																							
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25							NW Spun	-12.0																							
	9D	24/1	28.5 - 30.5	53/33/54/67	87	98																									
30																															
	10D	17/4	33.0 - 34.4	17/36/65(5.0)	101	114																									
35																															
	11D	24/24	38.0 - 40.0	15/24/33/48	57	64																									
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	12D	24/24	43.0 - 45.0	18/30/40/54	70	79																									
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	13D	24/24	48.0 - 50.0	8/13/20/23	33	37																									
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										Page 2 of 4																					
Boring No.: BB-GCS-402A																															

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-402A PIN: 18735.00						
Driller: New England Boring Contractors		Elevation (ft.): 13.0		Auger ID/OD: --								
Operator: B. Enos		Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID								
Logged By: K.Russ/M.Snow		Rig Type: Mobile B53 Track		Hammer Wt./Fall: SS-140#/30;HW-300#/24								
Date Start/Finish: 04-12-2017/04-12-2017		Drilling Method: HW/NW Drive		Core Barrel: NQ-1.875 in. ID								
Boring Location: E1141674, N509903		Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID		Water Level*:								
Hammer Efficiency Factor: 0.677		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐										
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50							NW Spun					
	14D	24/24	53.0 - 55.0	15/25/26/26	51	58						
55												
	15D	24/24	58.0 - 60.0	20/32/45/64	77	87						
60												
	16D	10/6	63.0 - 63.8	30/85(4.0)								
	R1	12/6	64.1 - 65.1	RQD = 0%			NQ Core					
65												
	R2	50/50	65.1 - 69.3	RQD = 22%								
	R3	62/62	69.3 - 74.5	RQD = 66%								
70												
75	R4	60/60	74.5 - 79.5	RQD = 75%								

Remarks:

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

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-402A PIN: 18735.00																																																																																																																																																																										
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-403 PIN: 18735.00				
Driller: New England Boring Contractors				Elevation (ft.): 10.5				Auger ID/OD: --				
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID				
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24				
Date Start/Finish: 04-10-2017/04-11-2017				Drilling Method: HW Drive				Core Barrel: NQ-1.875 in. ID				
Boring Location: E1141699, N509899				Casing ID/OD: HW-4.0 in. ID;NW-3.0 in. ID				Water Level*: 5 ft				
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test												
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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/7	0.0 - 2.0	3/14/11/12	25	28	17				Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, well graded, contains trace concrete fragments -FILL-(SW) PID=0.0 ppm	
							28					
	2D	24/9	2.0 - 4.0	8/13/17/15	30	34	50				Dark brown, moist, medium dense, fine to coarse SAND, little gravel and silt, well graded, contains trace brick fragments -FILL-(SW-SM) PID=0.0 ppm	
							82					
5	3D	24/6	4.0 - 6.0	18/7/10/16	17	19	125				Brown, wet, medium dense, GRAVEL, some fine to coarse sand, trace silt, well graded, contains trace wood fragments -ALLUVIAL DEPOSITS-(GW-GM) PID=0.0 ppm	G#411996 A-1-a(1), GW-GM WC=11.8%
							151					
	4D	24/12	6.0 - 8.0	24/40/53/61	93	105	200+				Brown, wet, very dense, sandy GRAVEL, well graded -ALLUVIAL DEPOSIT-(GW) PID=0.0 ppm	
							200+					
							200+					
10	5D	24/0	10.0 - 12.0	28/38/75(5.0)	113	128	51				No Recovery, rock it spoon tip	
							47					
							66					
							79				Note: Advanced rollerbit ahead of casing to 15 ft, gravelly based on drill action.	
							114					
15	6D	24/24	15.0 - 17.0	24/37/55/51	92	104	34				Olive-gray, moist, very dense, sandy SILT, trace gravel, well bonded -GLACIAL TILL-(ML)	G#411997 A-4(0), ML WC=9.8%
							47					
							37					
							52				Note: Advanced rollerbit ahead of casing to 20 ft, gravelly based on drill action.	
							83					
20	7D	24/13	20.0 - 22.0	22/30/50(4.0)	80	90	118				Olive-gray, moist, very dense, fine to coarse SAND, some gravel, little silt, well graded, well bonded -GLACIAL TILL-(SW-SM)	
							75					
							45					
							62				Note: Advanced rollerbit ahead of casing to 25 ft, gravelly based on drill action.	
							66					
25												
Remarks:												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											Page 1 of 3 Boring No.: BB-GCS-403	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-403 PIN: 18735.00				
Driller: New England Boring Contractors				Elevation (ft.): 10.5				Auger ID/OD: --				
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID				
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24				
Date Start/Finish: 04-10-2017/04-11-2017				Drilling Method: HW Drive				Core Barrel: NQ-1.875 in. ID				
Boring Location: E1141699, N509899				Casing ID/OD: HW-4.0 in. ID;NW-3.0 in. ID				Water Level*: 5 ft				
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	8D	24/19	25.0 - 27.0	16/26/53/40	79	89	110	-14.5		Olive-gray, moist, very dense, fine to coarse SAND, little gravel and silt, poorly graded, well bonded, trace clay -GLACIAL TILL-(SP-SM) Note: Advanced rollerbit ahead of casing to 30 ft.	G#439706 (A-4(0), ML)	
							55					
							85					
							97					
							112					
30	9D	19/19	30.0 - 31.6	10/25/30/50(1.0)	55	62	127					
							159					
							195					
							90					
							78					
35	10D	22/21	35.0 - 36.8	20/26/33/65(4.0)	59	67	121					
							130					
							226					
							216					
							176					
40	11D	24/24	40.0 - 42.0	17/23/28/39	51	58	160	-29.5				
							Open					
45	12D	24/24	45.0 - 47.0	23/31/33/58	64	72						
50												
Remarks:												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3		
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-GCS-403		

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-403 PIN: 18735.00			
Driller: New England Boring Contractors				Elevation (ft.): 10.5				Auger ID/OD: --			
Operator: B. Enos				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID			
Logged By: K.Russ/M.Snow				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS-140#/30;HW-300#/24			
Date Start/Finish: 04-10-2017/04-11-2017				Drilling Method: HW Drive				Core Barrel: NQ-1.875 in. ID			
Boring Location: E1141699, N509899				Casing ID/OD: HW-4.0 in. ID;NW-3.0 in. ID				Water Level*: 5 ft			
Hammer Efficiency Factor: 0.677				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR = weight of rods WO1P = Weight of one person S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing	Blows					
50	13D	24/18	50.0 - 52.0	15/30/34/57	64	72	Open				Olive-gray, moist, very dense, fine to coarse SAND, little gravel and silt, trace clay, well graded, well bonded -GLACIAL TILL-(SW-SM) Note: Advanced rollerbit ahead of casing to 55 ft, occasional gravel.	G#439707 (A-4(0), ML)	
55	14D	24/23	55.0 - 57.0	18/65/66/49	131	148					Olive-gray, moist, very dense, SILT, little fine sand, trace medium to coarse sand, trace gravel, well bonded -GLACIAL TILL-(ML) Note: Advanced rollerbit to probable bedrock at 58.9 ft. Spun N casing to 60.2 ft. Cleaned out to 60 ft.		
60	R1	60/60	60.0 - 65.0	RQD = 88%							Top of Bedrock El. -49.5 Light gray to gray, fine to medium grained SCHIST. Very hard to moderately hard, slightly weathered to fresh. Joints dipping at low angles, parallel with foliation, very close to moderately close, open, weathered joint surfaces, planar and smooth. Rock Mass Quality=Good Recovery=100% -VASSALBORO FORMATION- R1 Core Times (min:sec): 60.0-61.0' (1:55); 61.0-62.0' (2:08); 62.0-63.0' (1:35); 63.0-64.0' (1:29); 64.0-65.0' (1:24) Note: Two attempts for R1. Greenish-gray to gray, fine to coarse grained SCHIST. Very hard to moderately hard, slightly weathered. Joints dipping low angle parallel with foliation, very close to moderately close, open, weathered joint surfaces, planar and rough. Rock Mass Quality=Good Recovery=97% R2 Core Times (min:sec): 65.0-66.0' (1:40); 66.0-67.0' (1:47); 67.0-68.0' (1:38); 68.0-69.0' (1:34); 69.0-70.0' (2:01)		UC; qp=10,719psi
65	R2	60/58	65.0 - 70.0	RQD = 78%									
70	R3	24/24	70.0 - 72.0	RQD = 100%							Yellowish-gray to greenish-gray, coarse grained GRANITE. Very hard, fresh. Joints dipping at low angles, moderately close, open, slight oxidation on joint surfaces, rough and undulating. Rock Mass Quality=Excellent? Recovery=100% R3 Core Times (min:sec): 70.0-71.0' (2:18); 71.0-72.0' (2:32)		
75											Bottom of Exploration at 72.0 feet below ground surface.		

Remarks:

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-GCS-403

APPENDIX B

Rock Core Photographs



BORING ID	RUN ID	RUN DEPTH (FT)	RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2}	
				IN.	%	IN.	%
BB-GCS-102A	R1	65.0 to 70.0	-54.9 to -59.9	60	100	0	0
BB-GCS-102A	R2	70.0 to 75.0	-59.9 to -64.9	45	75	0	0
BB-GCS-101	R1	57.0 to 62.0	-42.3 to -47.3	60	100	31	52
BB-GCS-101	R2	62.0 to 67.0	-47.3 to -52.3	60	100	32	53

NOTES:

1. ELEVATIONS ARE MEASURED IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
2. "RQD" INDICATES ROCK QUALITY DESIGNATION (PERCENT OF ROCK PIECES RECOVERED EQUAL TO OR GREATER THAN 4 IN. IN LENGTH).

**HALEY
ALDRICH**

UNDERGROUND
ENGINEERING &
ENVIRONMENTAL
SOLUTIONS

BRIDGE STREET BRIDGE
MaineDOT WIN 18735.00, BRIDGE NO. 2102
ROUTES 201 & 9 OVER COBBOSSECONTEE STREAM

GARDINER, MAINE

**PHOTOGRAPH OF
BEDROCK CORE
BORINGS: BB-GCS-102A AND
BB-GCS-101**

FILE NO. 42223-004



BORING ID	CORE ID	CORE RUN DEPTH (FT)	CORE RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2}	
				IN.	%	IN.	%
BB-GCS-403	R1	60.0 to 65.0	-49.5 to -54.5	60	100	53	88
BB-GCS-403	R2	65.0 to 70.0	-54.5 to -59.5	58	97	47	78
BB-GCS-403	R3	70.0 to 72.0	-59.5 to -61.5	24	100	24	100

NOTES:

1. ELEVATIONS ARE MEASURED IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
2. "RQD" INDICATES ROCK QUALITY DESIGNATION (PERCENT OF ROCK PIECES RECOVERED EQUAL TO OR GREATER THAN 4 IN. IN LENGTH).



UNDERGROUND
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SOLUTIONS

BRIDGE STREET BRIDGE
MaineDOT WIN 18735.00, BRIDGE NO. 2102
ROUTES 201 & 9 OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
**PHOTOGRAPH OF
BEDROCK CORE
BORING: BB-GCS-403**

FILE NO. 42223-004

MARCH 2018



BORING ID	RUN ID	RUN DEPTH (FT)	RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2}	
				IN.	%	IN.	%
BB-GCS-401A	R1	65.2 to 69.7	-55.0 to -59.5	54	100	35	65
BB-GCS-401A	R2	69.7 to 71.3	-59.5 to -61.1	16	84	6	32
BB-GCS-401A	R3	71.3 to 75.7	-61.1 to -65.5	53	100	41	77
BB-GCS-401A	R4	75.7 to 80.7	-65.5 to -70.5	60	100	48	80

NOTES:

1. ELEVATIONS ARE MEASURED IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
2. "RQD" INDICATES ROCK QUALITY DESIGNATION (PERCENT OF ROCK PIECES RECOVERED EQUAL TO OR GREATER THAN 4 IN. IN LENGTH).



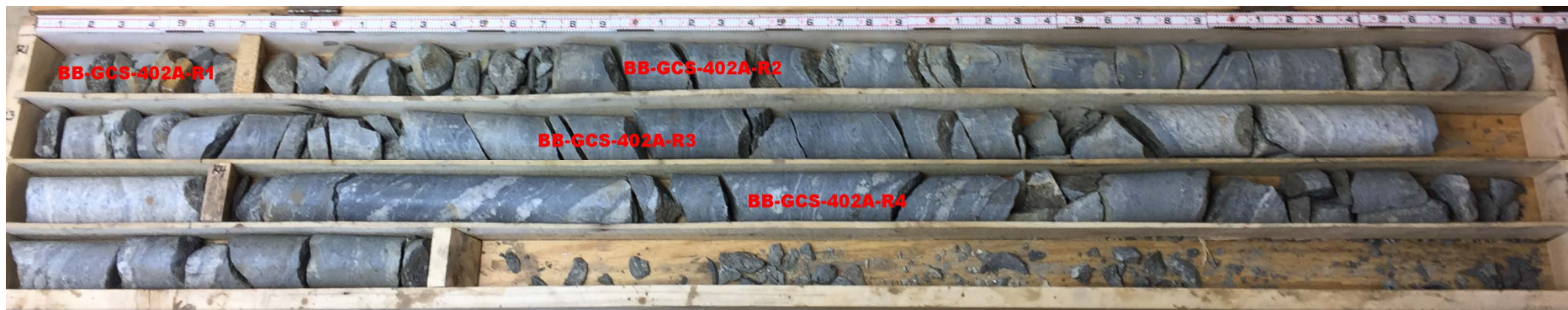
UNDERGROUND
ENGINEERING &
ENVIRONMENTAL
SOLUTIONS

BRIDGE STREET BRIDGE
MaineDOT WIN 18735.00, BRIDGE NO. 2102
ROUTES 201 & 9 OVER COBBOSSECONTEE STREAM
GARDINER, MAINE

**PHOTOGRAPH OF
BEDROCK CORE
BORING: BB-GCS-401A**

FILE NO. 42223-004

MARCH 2018



BORING ID	CORE ID	CORE RUN DEPTH (FT)	CORE RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2}	
				IN.	%	IN.	%
BB-GCS-402A	R1	64.1-65.1	-51.1 to -52.1	6	50	0	0
BB-GCS-402A	R2	65.1 to 69.3	-52.1 to -56.3	50	100	11	22
BB-GCS-402A	R3	69.3 to 74.5	-56.3 to -61.5	62	100	41	66
BB-GCS-402A	R4	74.5 to 79.5	-61.5 to -66.5	60	100	45	75

NOTES:

1. ELEVATIONS ARE MEASURED IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
2. "RQD" INDICATES ROCK QUALITY DESIGNATION (PERCENT OF ROCK PIECES RECOVERED EQUAL TO OR GREATER THAN 4 IN. IN LENGTH).

**HALEY
ALDRICH**

UNDERGROUND
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SOLUTIONS

BRIDGE STREET BRIDGE
MaineDOT WIN 18735.00, BRIDGE NO. 2102
ROUTES 201 & 9 OVER COBBOSSECONTEE STREAM
GARDINER, MAINE

**PHOTOGRAPH OF
BEDROCK CORE
BORINGS: BB-GCS-402A**

FILE NO. 42223-004

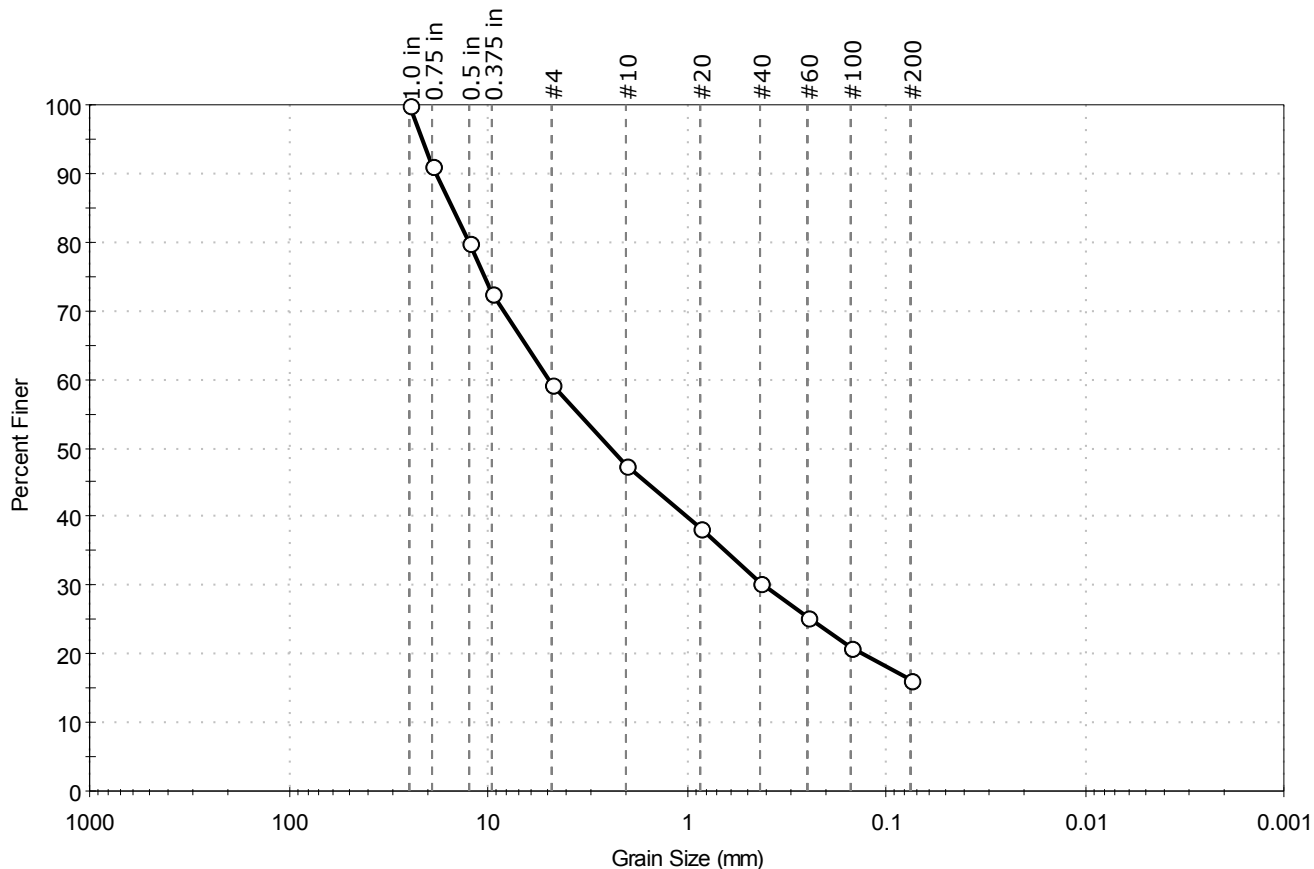
MARCH 2018

APPENDIX C

Laboratory Test Results

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-101	Sample Type:	bag
Sample ID:	1D	Test Date:	02/18/16
Depth :	0.0-2.0	Test Id:	365344
Test Comment:	---		
Visual Description:	Moist, very dark grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	40.8	42.8	16.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	80		
0.375 in	9.50	73		
#4	4.75	59		
#10	2.00	47		
#20	0.85	38		
#40	0.42	31		
#60	0.25	25		
#100	0.15	21		
#200	0.075	16		

Coefficients

$D_{85} = 15.0878$ mm $D_{30} = 0.4034$ mm
 $D_{60} = 4.9625$ mm $D_{15} = \text{N/A}$
 $D_{50} = 2.4171$ mm $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

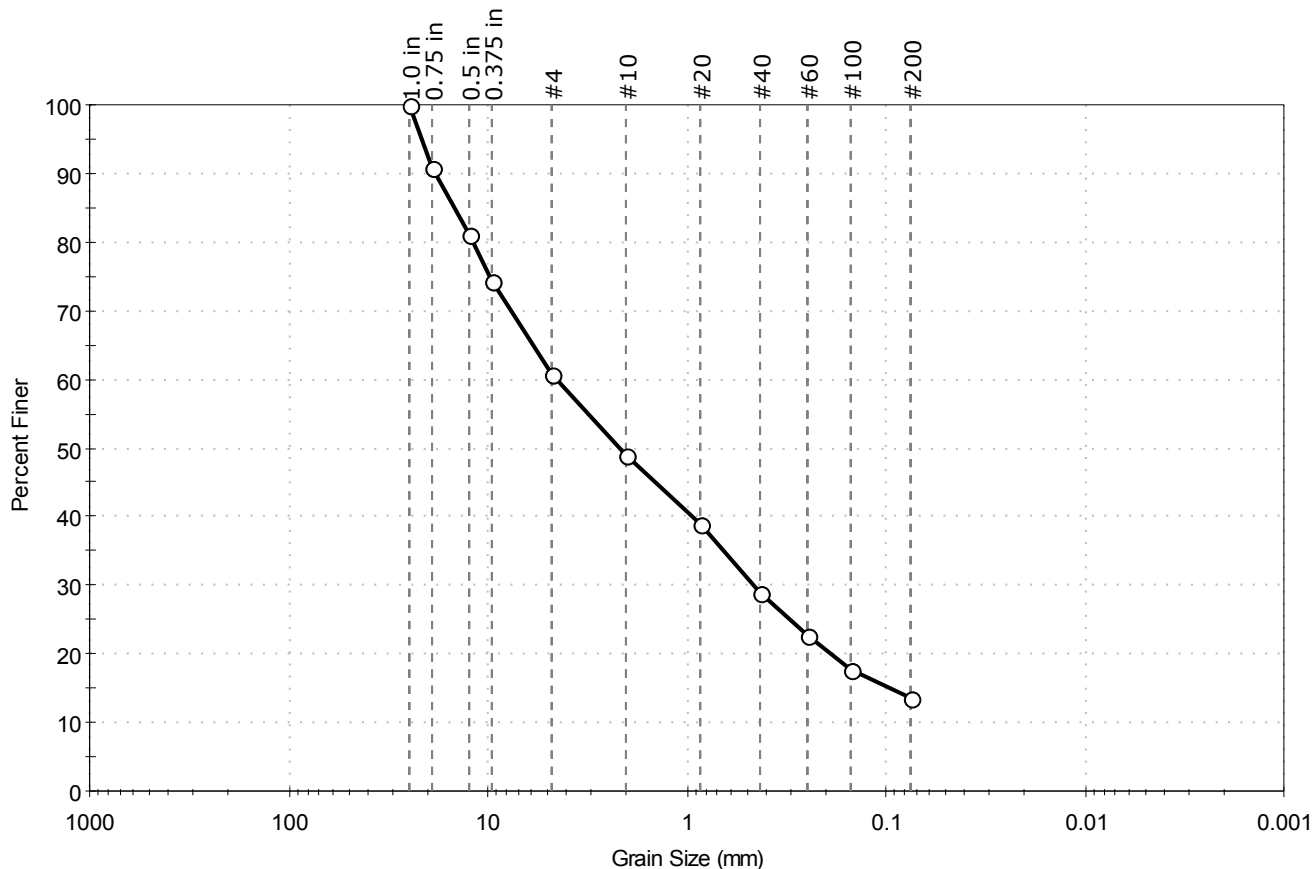
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-101	Sample Type:	bag
Sample ID:	3D	Test Date:	02/18/16
Depth :	4.0-6.0	Test Id:	365345
Test Comment:	---		
Visual Description:	Moist, very dark gray clayey sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.3	47.2	13.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	81		
0.375 in	9.50	74		
#4	4.75	61		
#10	2.00	49		
#20	0.85	39		
#40	0.42	29		
#60	0.25	23		
#100	0.15	18		
#200	0.075	14		

Coefficients

D ₈₅ = 14.7359 mm	D ₃₀ = 0.4567 mm
D ₆₀ = 4.4901 mm	D ₁₅ = 0.0951 mm
D ₅₀ = 2.1424 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

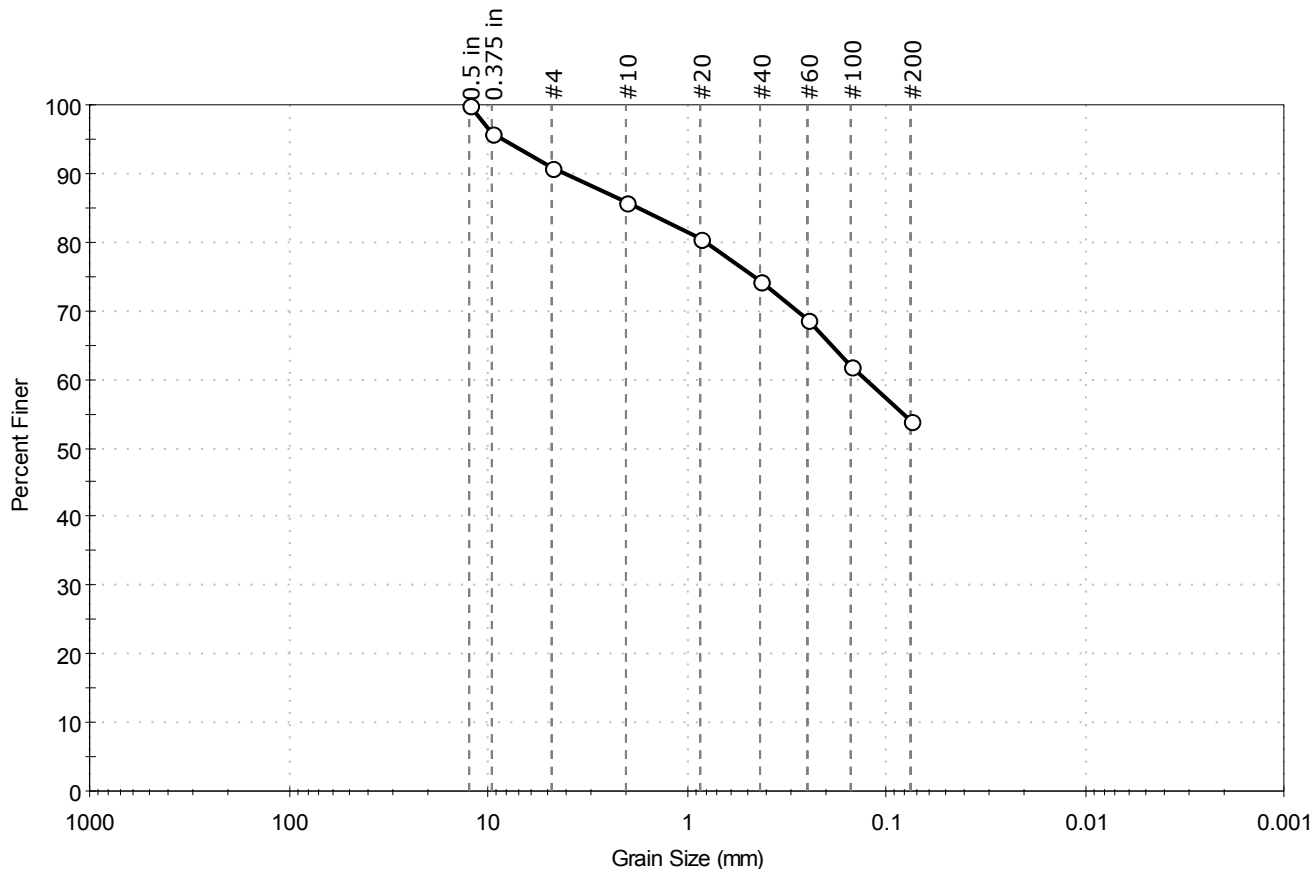
AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-101	Sample Type:	bag
Sample ID:	8D	Test Date:	02/18/16
Depth :	20.0-22.0	Test Id:	365346
Test Comment:	---		
Visual Description:	Moist, dark gray sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.0	37.0	54.0

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	91		
#10	2.00	86		
#20	0.85	81		
#40	0.42	74		
#60	0.25	69		
#100	0.15	62		
#200	0.075	54		

Coefficients

D ₈₅ = 1.7610 mm	D ₃₀ = N/A
D ₆₀ = 0.1259 mm	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM Sandy Lean clay (CL)

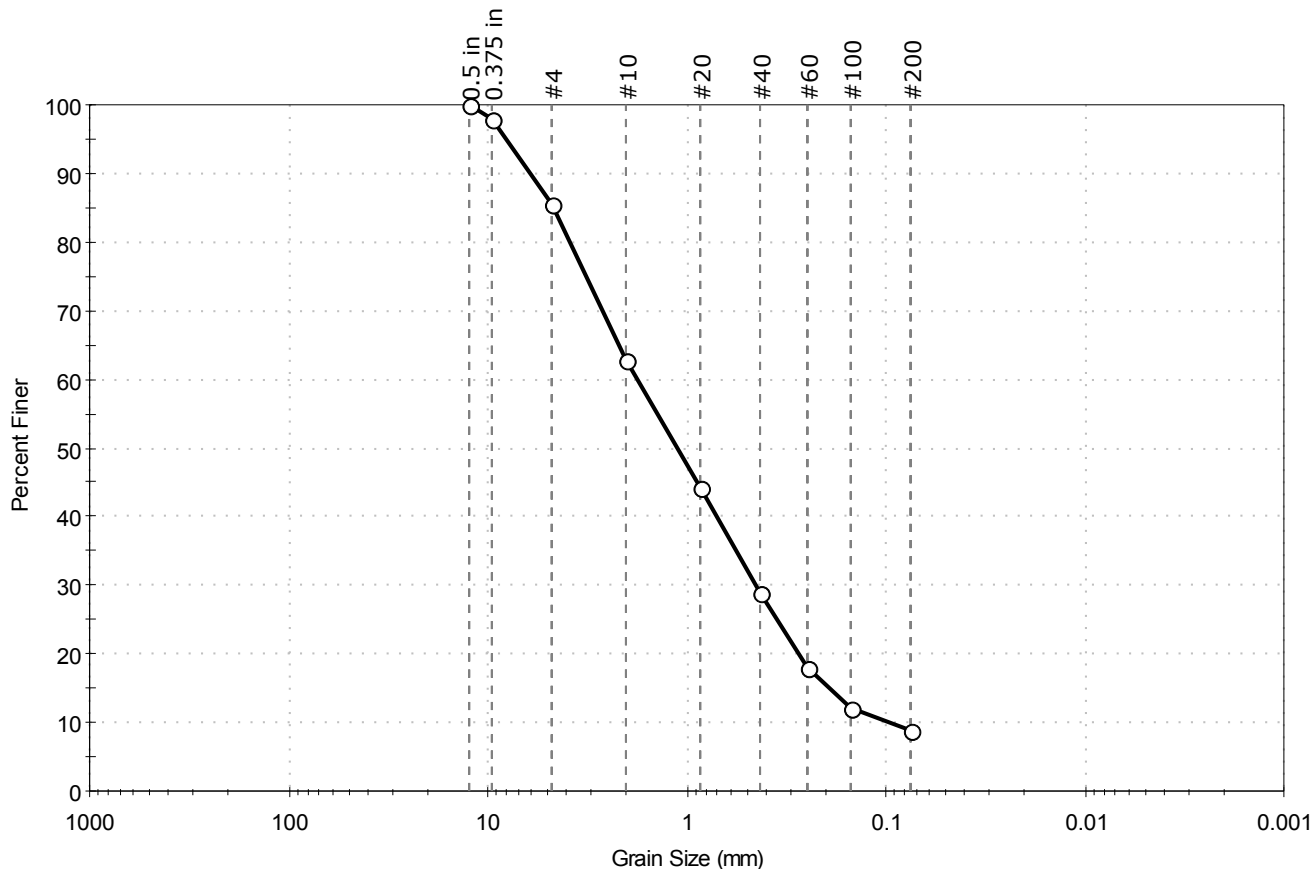
AASHTO Silty Soils (A-4 (1))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-102	Sample Type:	bag
Sample ID:	1D	Test Date:	02/18/16
Depth :	1.0-3.0	Test Id:	365347
Test Comment:	---		
Visual Description:	Moist, very dark gray sand with silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.4	76.7	8.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	86		
#10	2.00	63		
#20	0.85	44		
#40	0.42	29		
#60	0.25	18		
#100	0.15	12		
#200	0.075	8.9		

Coefficients

D ₈₅ = 4.6461 mm	D ₃₀ = 0.4485 mm
D ₆₀ = 1.7638 mm	D ₁₅ = 0.1947 mm
D ₅₀ = 1.1113 mm	D ₁₀ = 0.0962 mm
C _u = 18.335	C _c = 1.185

Classification

ASTM N/A

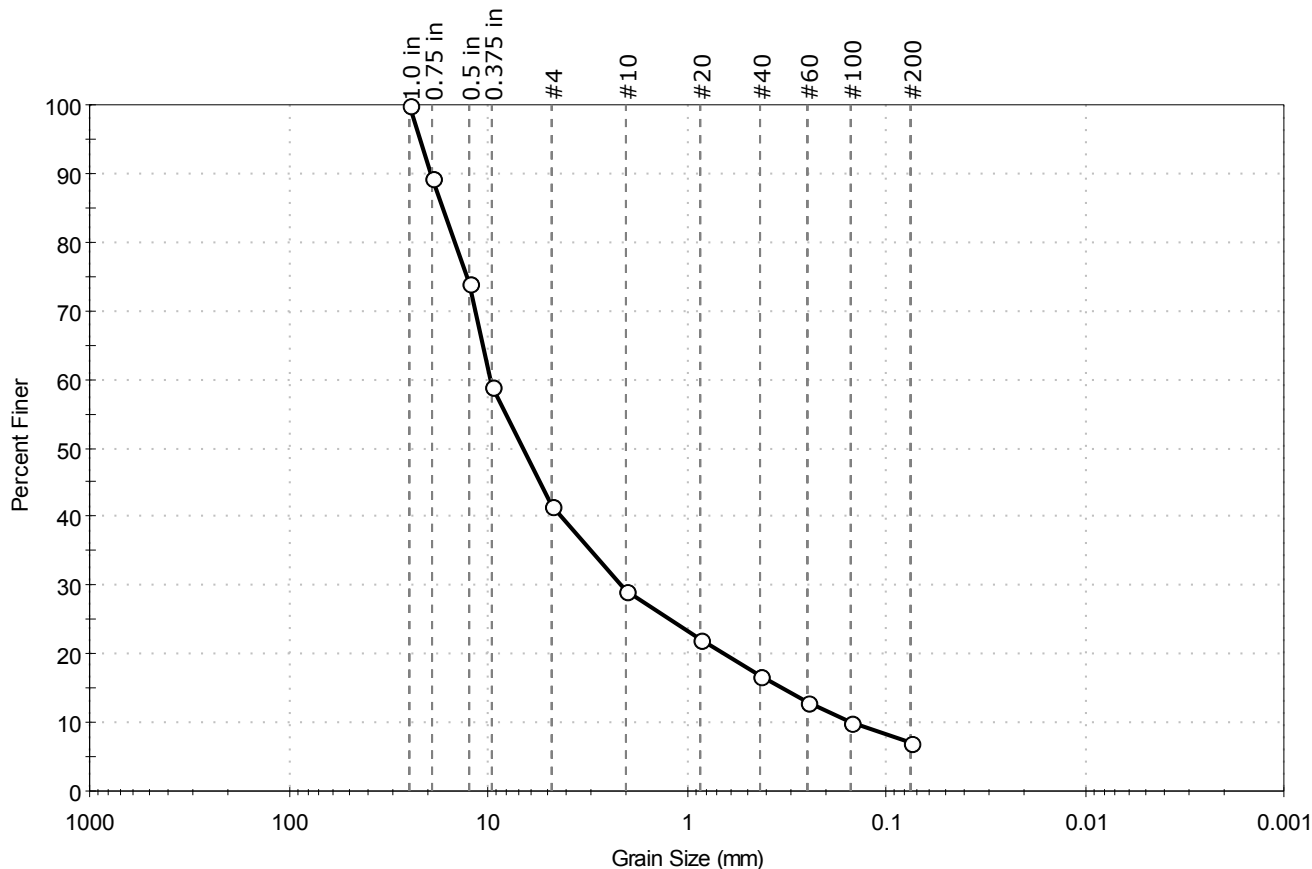
AASHTO Stone Fragments, Gravel and Sand (A-1-b (1))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-102	Sample Type:	bag
Sample ID:	3D	Test Date:	02/18/16
Depth :	4.2-6.2	Test Id:	365348
Test Comment:	---		
Visual Description:	Moist, dark brown gravel with silt and sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	58.4	34.5	7.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	89		
0.5 in	12.50	74		
0.375 in	9.50	59		
#4	4.75	42		
#10	2.00	29		
#20	0.85	22		
#40	0.42	17		
#60	0.25	13		
#100	0.15	10		
#200	0.075	7.1		

Coefficients

D ₈₅ = 16.8355 mm	D ₃₀ = 2.1277 mm
D ₆₀ = 9.6543 mm	D ₁₅ = 0.3293 mm
D ₅₀ = 6.6227 mm	D ₁₀ = 0.1467 mm
C _u = 65.810	C _c = 3.196

Classification

ASTM N/A

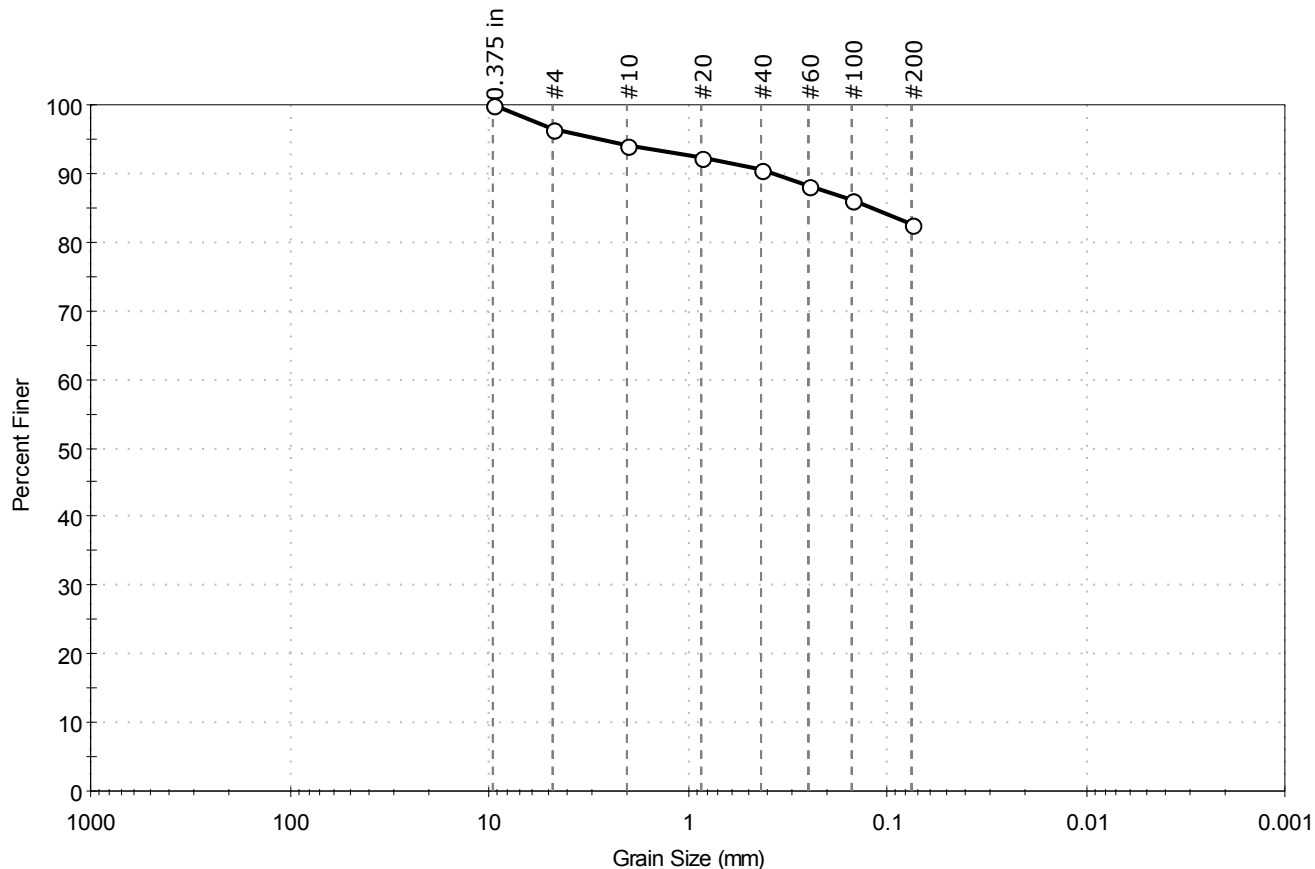
AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-102A	Sample Type:	bag
Sample ID:	4D	Test Date:	02/18/16
Depth :	13.0-15.0	Test Id:	365349
Test Comment:	---		
Visual Description:	Moist, dark gray clay with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.6	13.7	82.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	96		
#10	2.00	94		
#20	0.85	92		
#40	0.42	90		
#60	0.25	88		
#100	0.15	86		
#200	0.075	83		

Coefficients

$D_{85} = 0.1194 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = \text{N/A}$ $D_{15} = \text{N/A}$
 $D_{50} = \text{N/A}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM Lean clay with sand (CL)

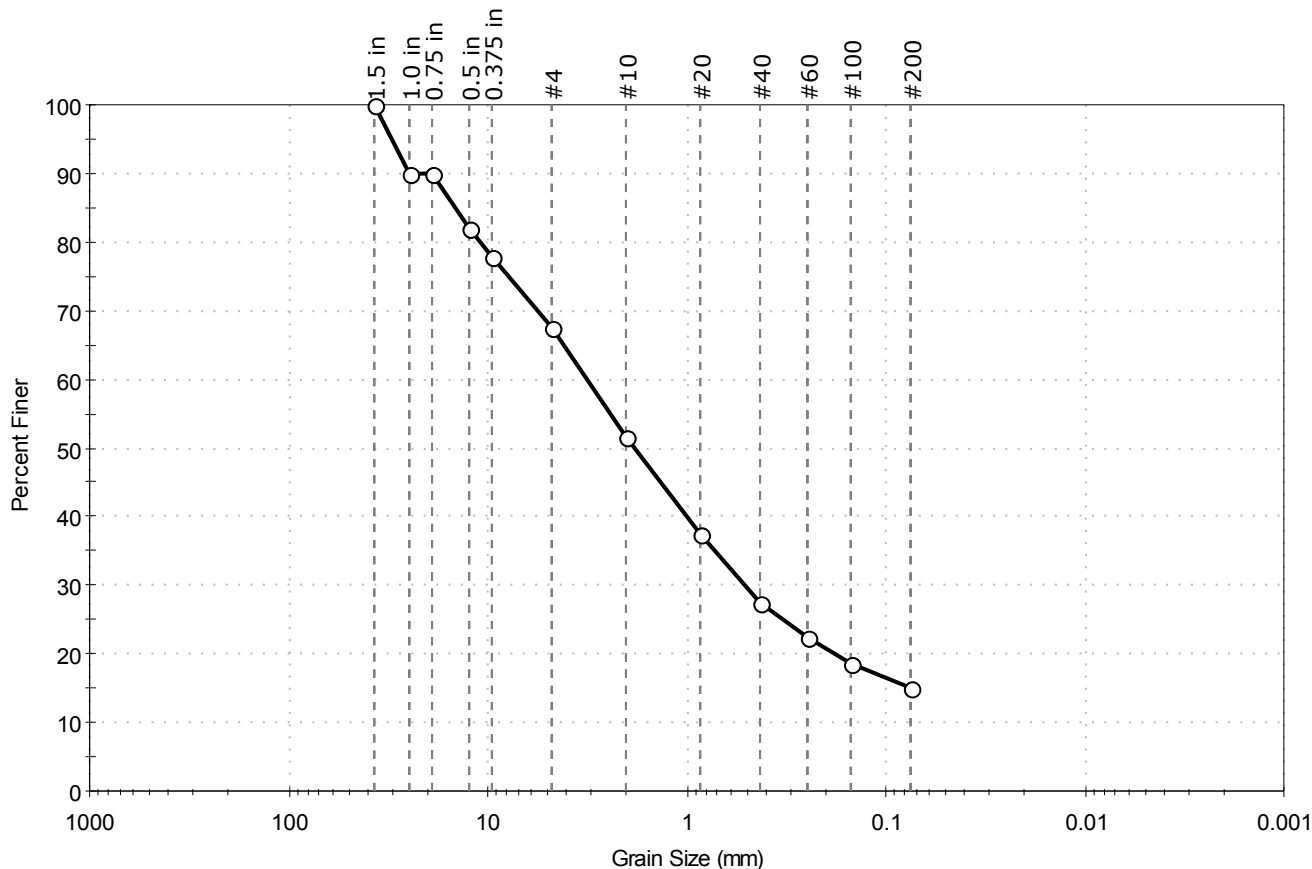
AASHTO Clayey Soils (A-6 (11))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-103	Sample Type:	bag
Sample ID:	2D	Test Date:	02/18/16
Depth :	2.5-4.5	Test Id:	365350
Test Comment:	---		
Visual Description:	Moist, dark grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	32.4	52.7	14.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1.0 in	25.00	90		
0.75 in	19.00	90		
0.5 in	12.50	82		
0.375 in	9.50	78		
#4	4.75	68		
#10	2.00	52		
#20	0.85	38		
#40	0.42	27		
#60	0.25	22		
#100	0.15	18		
#200	0.075	15		

Coefficients

D ₈₅ = 14.5829 mm	D ₃₀ = 0.5059 mm
D ₆₀ = 3.1497 mm	D ₁₅ = 0.0762 mm
D ₅₀ = 1.8128 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

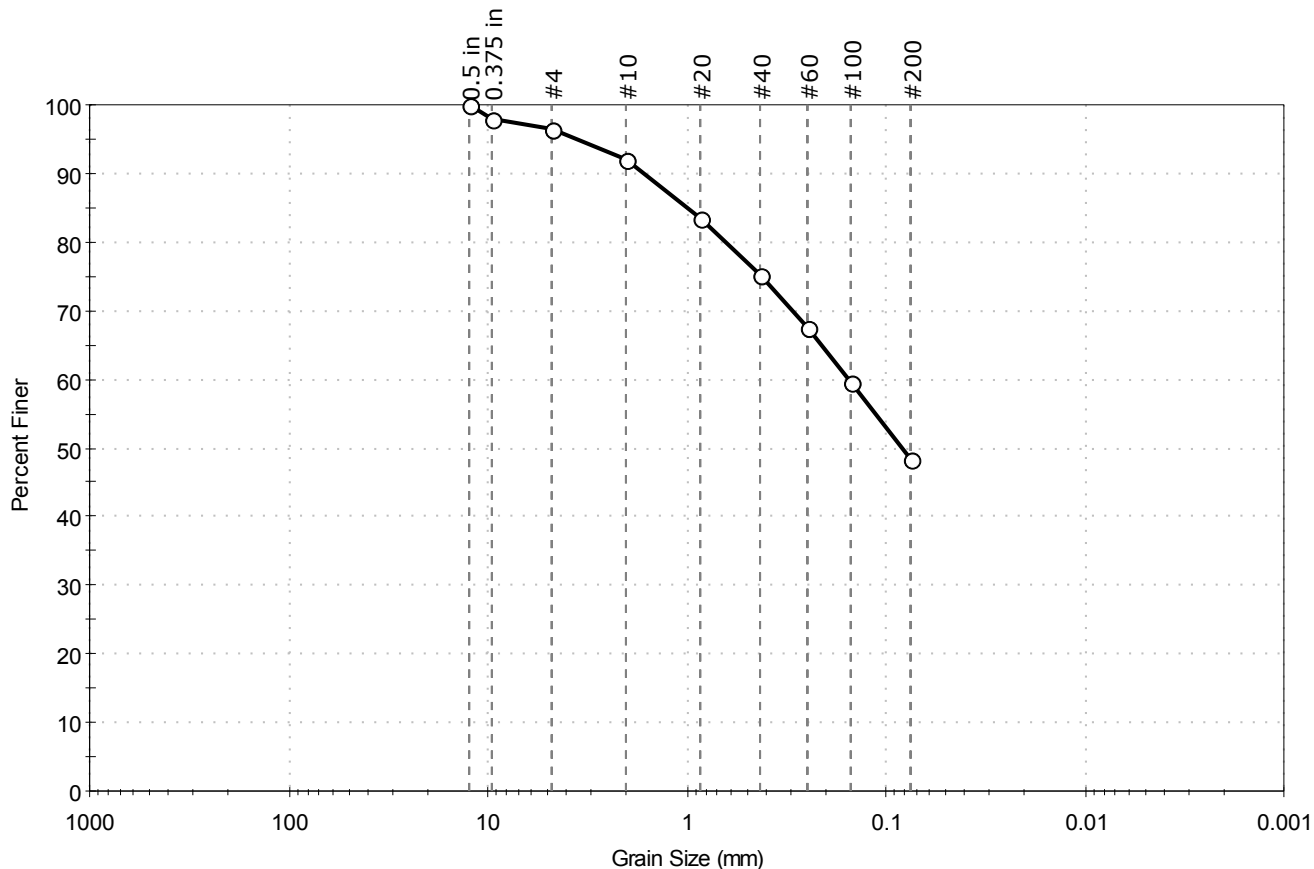
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-103	Sample Type:	bag
Sample ID:	3D	Test Date:	02/18/16
Depth :	4.5-6.5	Test Id:	365351
Test Comment:	---		
Visual Description:	Moist, dark grayish brown clayey sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.5	48.0	48.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	96		
#10	2.00	92		
#20	0.85	83		
#40	0.42	75		
#60	0.25	67		
#100	0.15	60		
#200	0.075	48		

Coefficients

$D_{85} = 0.9943 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = 0.1542 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.0826 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

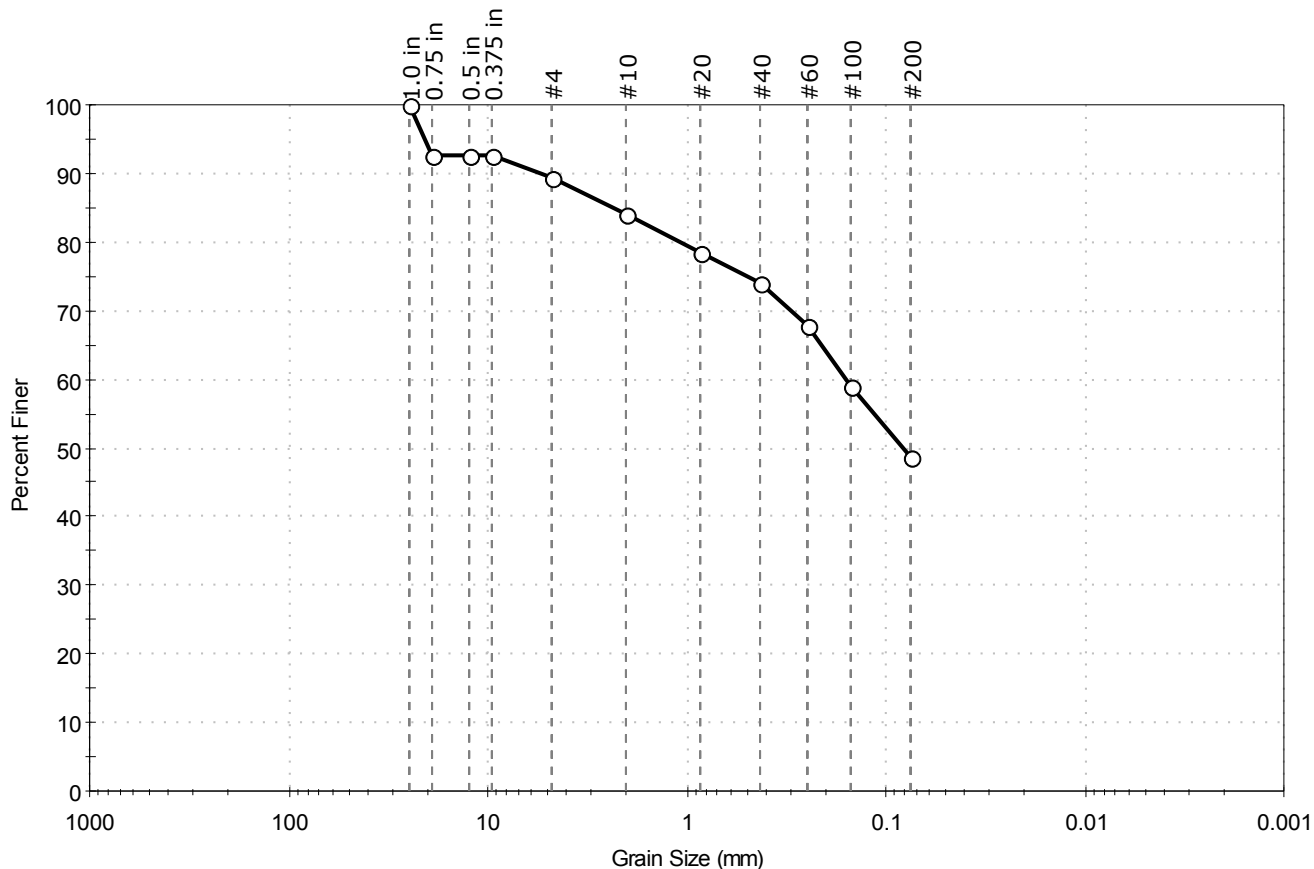
AASHTO Silty Soils (A-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-103	Sample Type:	bag
Sample ID:	6D	Test Date:	02/18/16
Depth :	12.0-14.0	Test Id:	365352
Test Comment:	---		
Visual Description:	Moist, olive gray clayey sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.5	40.8	48.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	93		
0.5 in	12.50	93		
0.375 in	9.50	93		
#4	4.75	90		
#10	2.00	84		
#20	0.85	79		
#40	0.42	74		
#60	0.25	68		
#100	0.15	59		
#200	0.075	49		

Coefficients

$D_{85} = 2.2887 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = 0.1594 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.0817 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

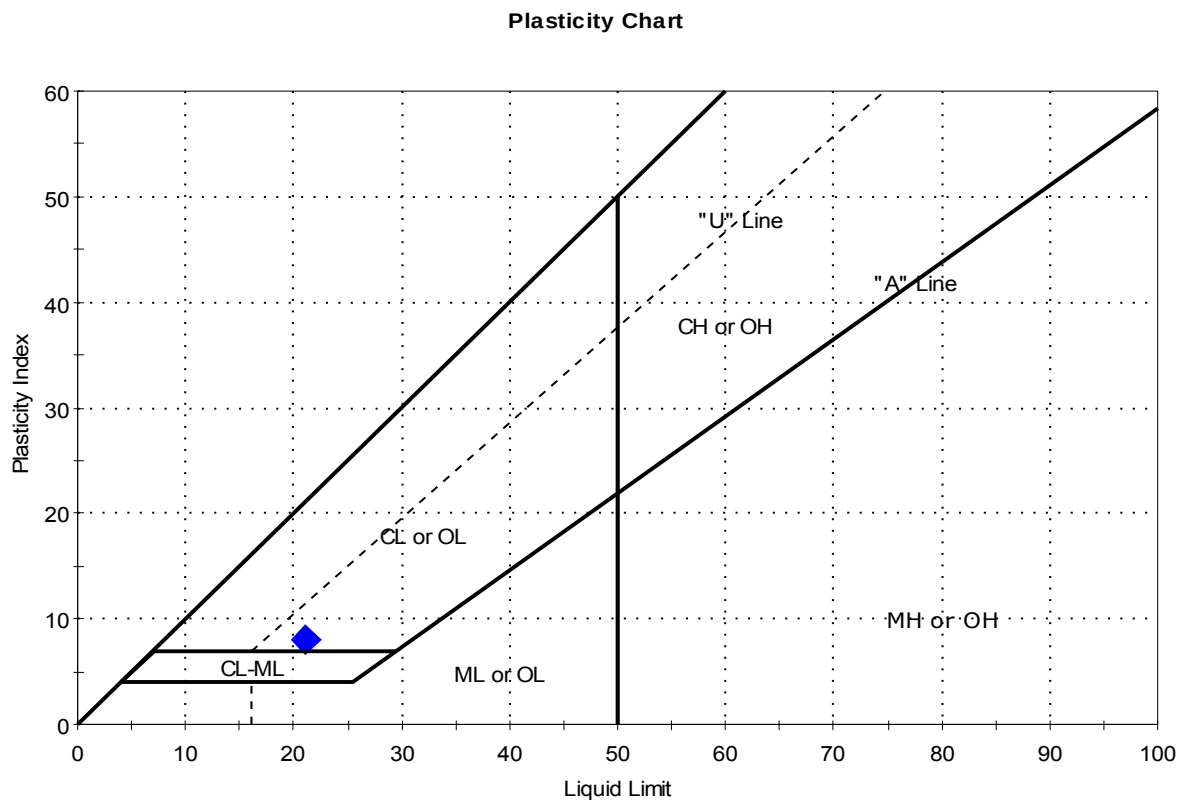
AASHTO Silty Soils (A-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-101	Sample Type:	bag
Sample ID:	8D	Test Date:	02/17/16
Depth :	20.0-22.0	Test Id:	365357
Test Comment:	---		
Visual Description:	Moist, dark gray sandy clay		
Sample Comment:	---		

Atterberg Limits - ASTM D4318



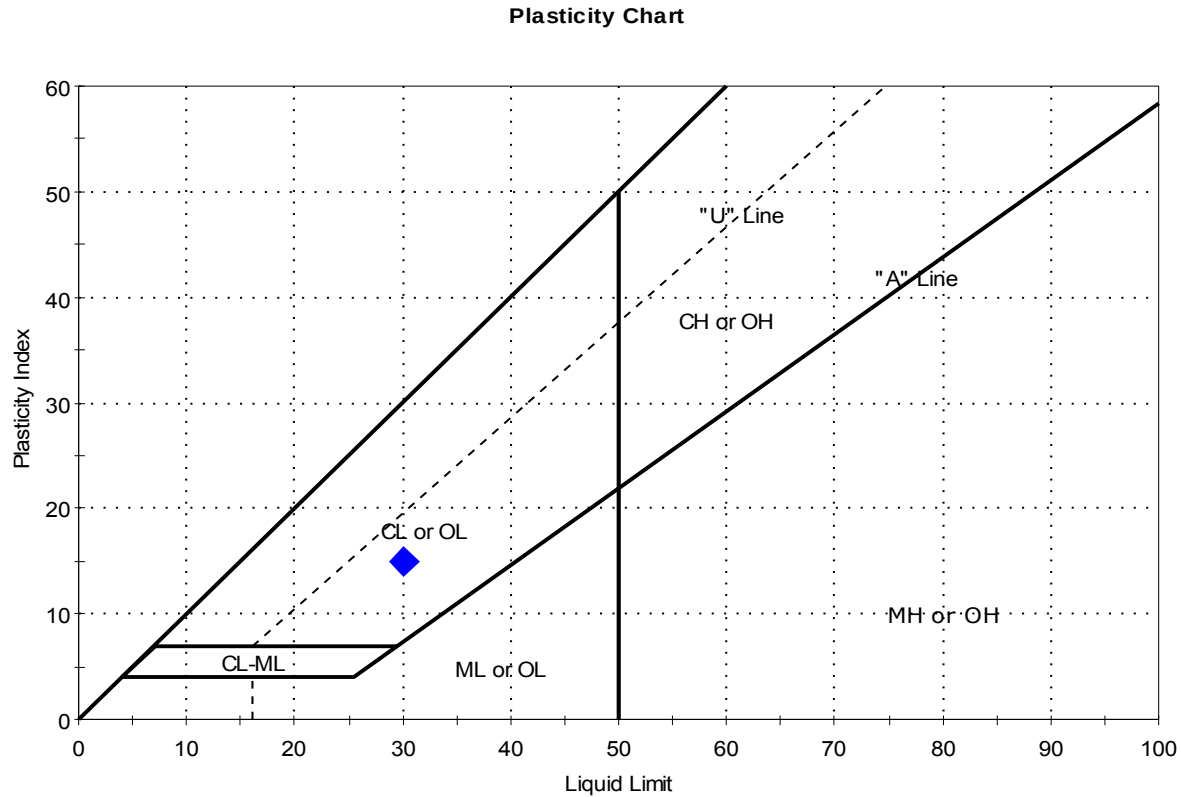
Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	8D	B-GCS-101	20.0-22.0	9	21	13	8	-0.5	Sandy Lean clay (CL)

Sample Prepared using the WET method
 26% Retained on #40 Sieve
 Dry Strength: HIGH
 Dilatancy: NONE
 Toughness: MEDIUM



Client:	Haley & Aldrich, Inc.			Project No:	GTX-304437
Project:	U.S. 201 & 9 over the Cobbossee Stream				
Location:	Gardiner, Maine				
Boring ID:	BB-GCS-102A	Sample Type:	bag	Tested By:	GA
Sample ID:	4D	Test Date:	02/18/16	Checked By:	jdt
Depth :	13.0-15.0	Test Id:	365358		
Test Comment:	---				
Visual Description:	Moist, dark gray clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



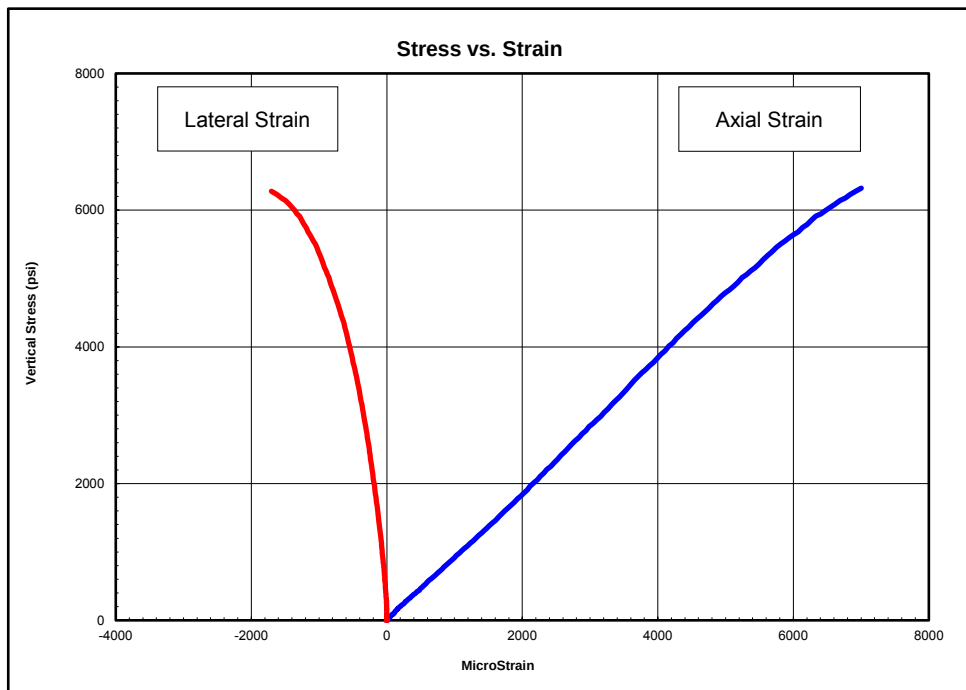
Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	BB-GCS-102A	13.0-15.0	20	30	15	15	0.3	Lean clay with sand (CL)

Sample Prepared using the WET method
 10% Retained on #40 Sieve
 Dry Strength: HIGH
 Dilatancy: NONE
 Toughness: MEDIUM



Client:	Haley & Aldrich, Inc.
Project Name:	U.S. 201 & 9 over the Cobbossee Stream
Project Location:	Gardiner, Maine
GTX #:	304437
Test Date:	2/15/2016
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-GCS-101
Sample ID:	R2
Depth, ft:	65.50-65.87
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 6,343 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
600-2300	929,000	0.11
2300-4000	1,000,000	0.19
4000-5700	885,000	0.32

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

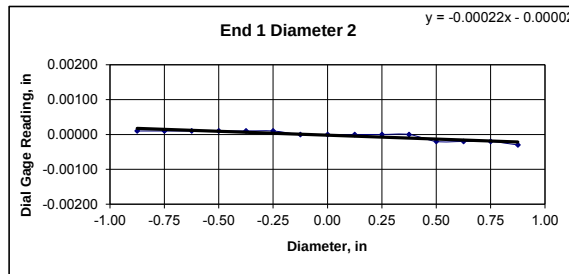
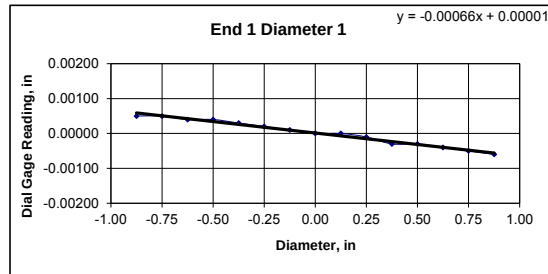


Client:	Haley & Aldrich, Inc.	Test Date:	2/12/2016
Project Name:	U.S. 201 & 9 over the Cobbossee Stream	Tested By:	daa/rlc
Project Location:	Gardiner, Maine	Checked By:	jsc
GT#:	304437		
Boring ID:	BB-GCS-101		
Sample ID:	R2		
Depth:	65.50-65.87 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.28	4.28	4.28	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.97	1.97	1.97	Straightness Tolerance Met? YES	
Specimen Mass, g:	577				
Bulk Density, lb/ft ³ :	168				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

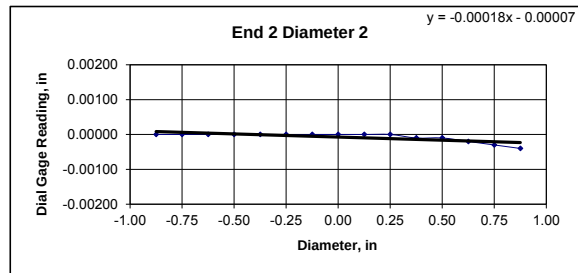
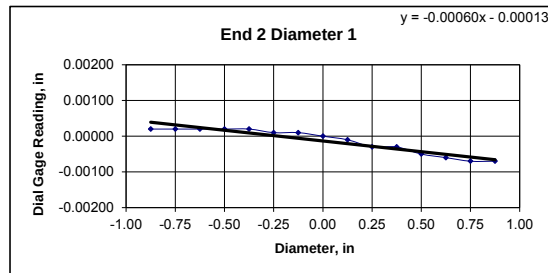
END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00050	0.00050	0.00040	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00030	-0.00030	-0.00040	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00020	-0.00030
Difference between max and min readings, in:															
0° = 0.00110 90° = 0.00040															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	-0.00010	-0.00030	-0.00030	-0.00050	-0.00060	-0.00070	-0.00070
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040
Difference between max and min readings, in:															
0° = 0.0009 90° = 0.0004															
Maximum difference must be $<$ 0.0020 in. Difference = $\pm$ 0.00055															
Flatness Tolerance Met? YES															



DIAMETER 1

End 1:		
Slope of Best Fit Line	-0.00066	
Angle of Best Fit Line:	-0.03782	
End 2:		
Slope of Best Fit Line	-0.00060	
Angle of Best Fit Line:	-0.03438	
Maximum Angular Difference:	0.00344	

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:		
Slope of Best Fit Line	-0.00022	
Angle of Best Fit Line:	-0.01261	
End 2:		
Slope of Best Fit Line	-0.00018	
Angle of Best Fit Line:	-0.01031	
Maximum Angular Difference:	0.00229	

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq$ 0.25°
Diameter 1, in	0.00110	1.970	0.00056	0.032	YES		
Diameter 2, in (rotated 90°)	0.00040	1.970	0.00020	0.012	YES		
						Perpendicularity Tolerance Met?	YES
END 2							
Diameter 1, in	0.00090	1.970	0.00046	0.026	YES		
Diameter 2, in (rotated 90°)	0.00040	1.970	0.00020	0.012	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	U.S. 201 & 9 over the Cobbossee Stream
Project Location:	Gardiner, Maine
GTX #:	304437
Test Date:	2/15/2016
Tested By:	daa/rlc
Checked By:	jsc
Boring ID:	BB-GCS-101
Sample ID:	R2
Depth, ft:	65.50-65.87



After cutting and grinding



After break



Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	05/24/17
Depth :	---	Test Id:	411991
		Tested By:	jbr
		Checked By:	jdt

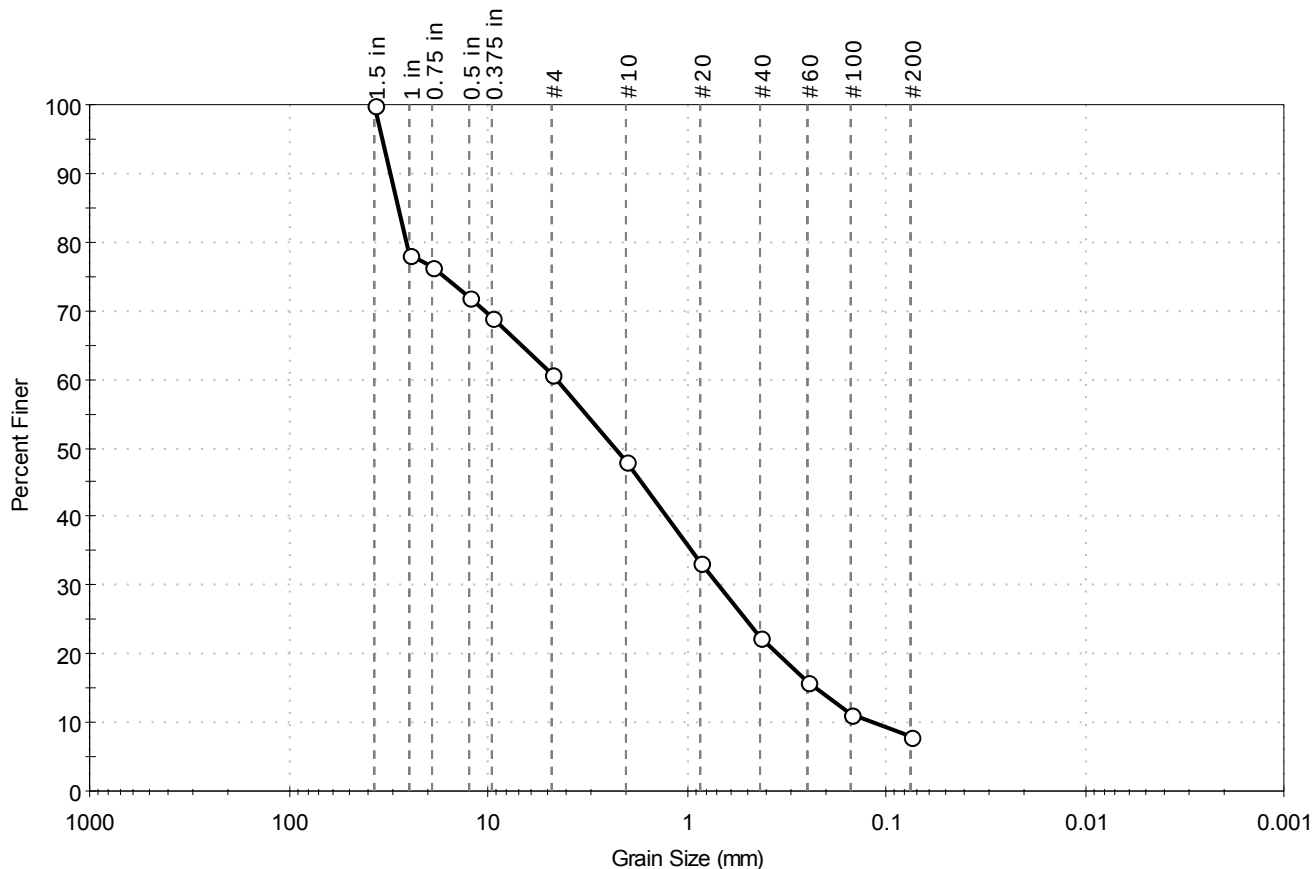
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-GCS-401	2D	4.0-6.0 ft	Moist, dark grayish brown sand with silt and gravel	11.2
BB-GCS-401	6D	14.0-16.0 ft	Moist, olive sandy clay	8.7
BB-GCS-402	1D	0-2.0 ft	Moist, dark brown silty sand with gravel	10.5
BB-GCS-402	3D	4.0-6.0 ft	Moist, dark grayish brown silty gravel with sand	18.6
BB-GCS-403	3D	4.0-6.0 ft	Moist, dark brown gravel with silt and sand	11.8
BB-GCS-403	6D	15.0-17.0 ft	Moist, olive sandy clay	9.8

Notes: Temperature of Drying : 110° Celsius

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-401	Sample Type:	bag
Sample ID:	2D	Test Date:	05/25/17
Depth:	4.0-6.0 ft	Test Id:	411992
Test Comment:	---		
Visual Description:	Moist, dark grayish brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.1	52.9	8.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	78		
0.75 in	19.00	77		
0.5 in	12.50	72		
0.375 in	9.50	69		
#4	4.75	61		
#10	2.00	48		
#20	0.85	33		
#40	0.42	23		
#60	0.25	16		
#100	0.15	11		
#200	0.075	8.0		

Coefficients

D ₈₅ = 28.3380 mm	D ₃₀ = 0.6899 mm
D ₆₀ = 4.4736 mm	D ₁₅ = 0.2246 mm
D ₅₀ = 2.2625 mm	D ₁₀ = 0.1136 mm
C _u = 39.380	C _c = 0.937

Classification

ASTM N/A

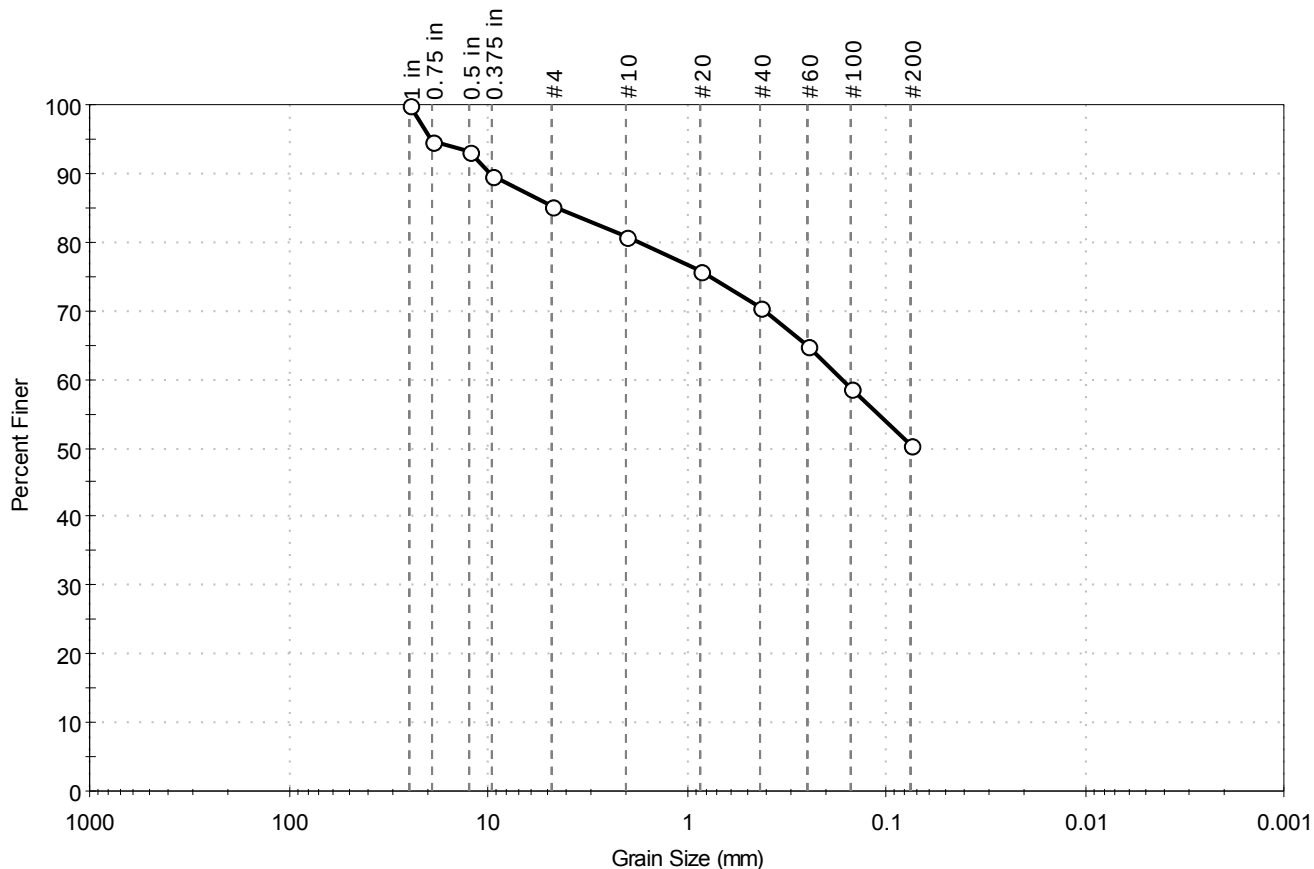
AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-401	Sample Type:	bag
Sample ID:	6D	Test Date:	05/25/17
Depth :	14.0-16.0 ft	Test Id:	411993
Test Comment:	---		
Visual Description:	Moist, olive sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.6	34.8	50.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	95		
0.5 in	12.50	93		
0.375 in	9.50	90		
#4	4.75	85		
#10	2.00	81		
#20	0.85	76		
#40	0.42	70		
#60	0.25	65		
#100	0.15	59		
#200	0.075	51		

Coefficients

D ₈₅ = 4.4433 mm	D ₃₀ = N/A
D ₆₀ = 0.1665 mm	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

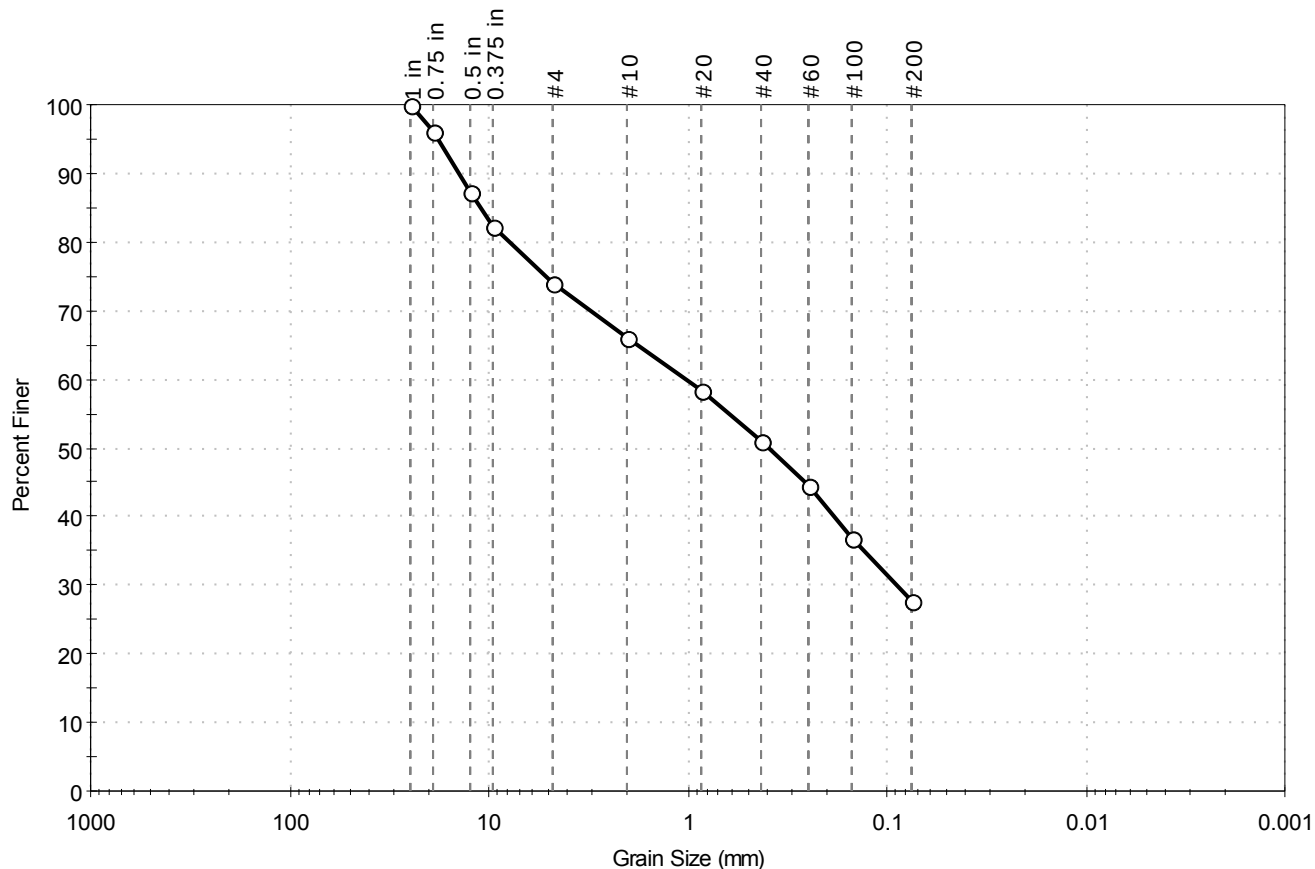
AASHTO Silty Soils (A-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-402	Sample Type:	bag
Sample ID:	1D	Test Date:	05/25/17
Depth :	0-2.0 ft	Test Id:	411994
Test Comment:	---		
Visual Description:	Moist, dark brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.9	46.3	27.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	96		
0.5 in	12.50	87		
0.375 in	9.50	82		
#4	4.75	74		
#10	2.00	66		
#20	0.85	58		
#40	0.42	51		
#60	0.25	45		
#100	0.15	37		
#200	0.075	28		

Coefficients

D ₈₅ = 10.9466 mm	D ₃₀ = 0.0891 mm
D ₆₀ = 1.0176 mm	D ₁₅ = N/A
D ₅₀ = 0.3876 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

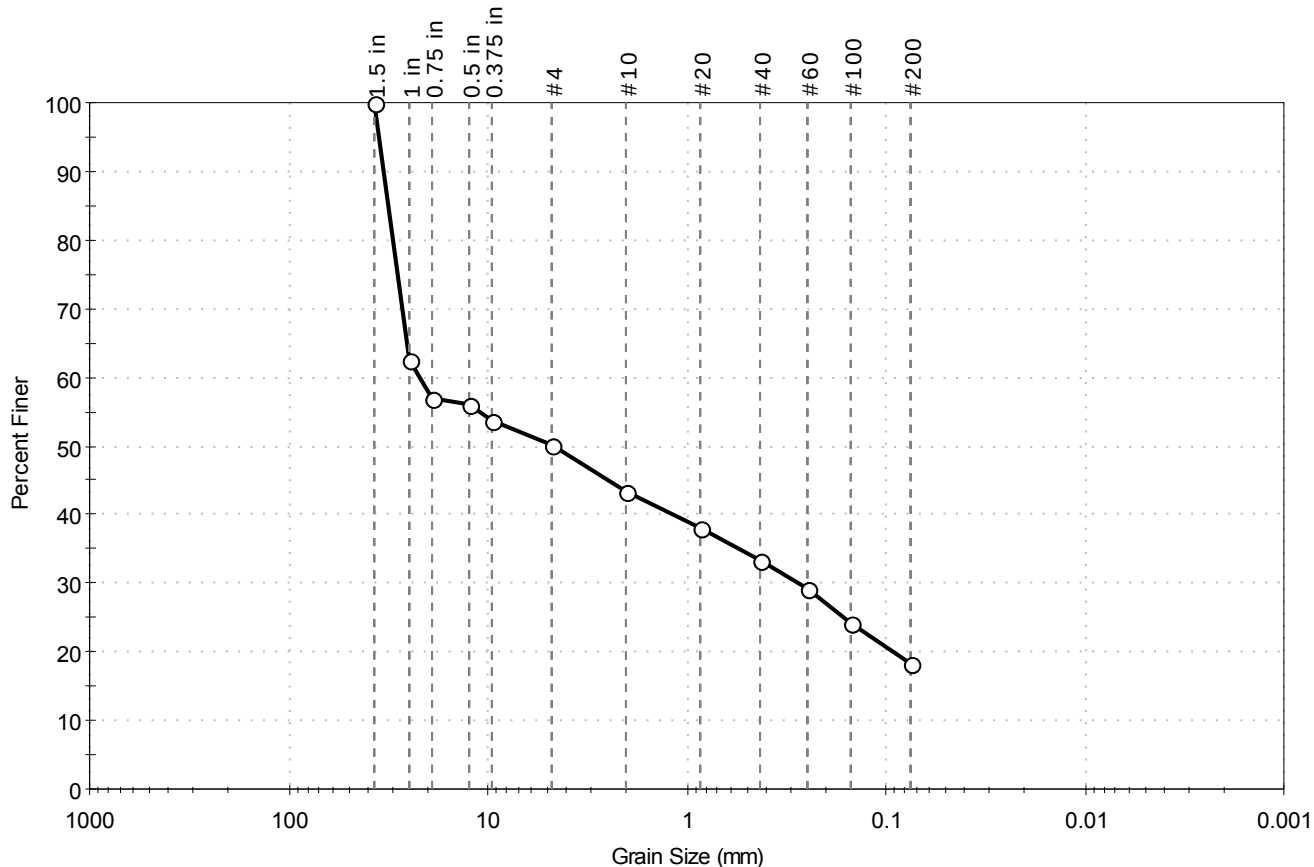
AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-402	Sample Type:	bag
Sample ID:	3D	Test Date:	05/25/17
Depth:	4.0-6.0 ft	Test Id:	411995
Test Comment:	---		
Visual Description:	Moist, dark grayish brown silty gravel with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	49.7	32.1	18.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	63		
0.75 in	19.00	57		
0.5 in	12.50	56		
0.375 in	9.50	54		
#4	4.75	50		
#10	2.00	43		
#20	0.85	38		
#40	0.42	33		
#60	0.25	29		
#100	0.15	24		
#200	0.075	18		

Coefficients

D ₈₅ = 31.8705 mm	D ₃₀ = 0.2782 mm
D ₆₀ = 22.0036 mm	D ₁₅ = N/A
D ₅₀ = 4.5782 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

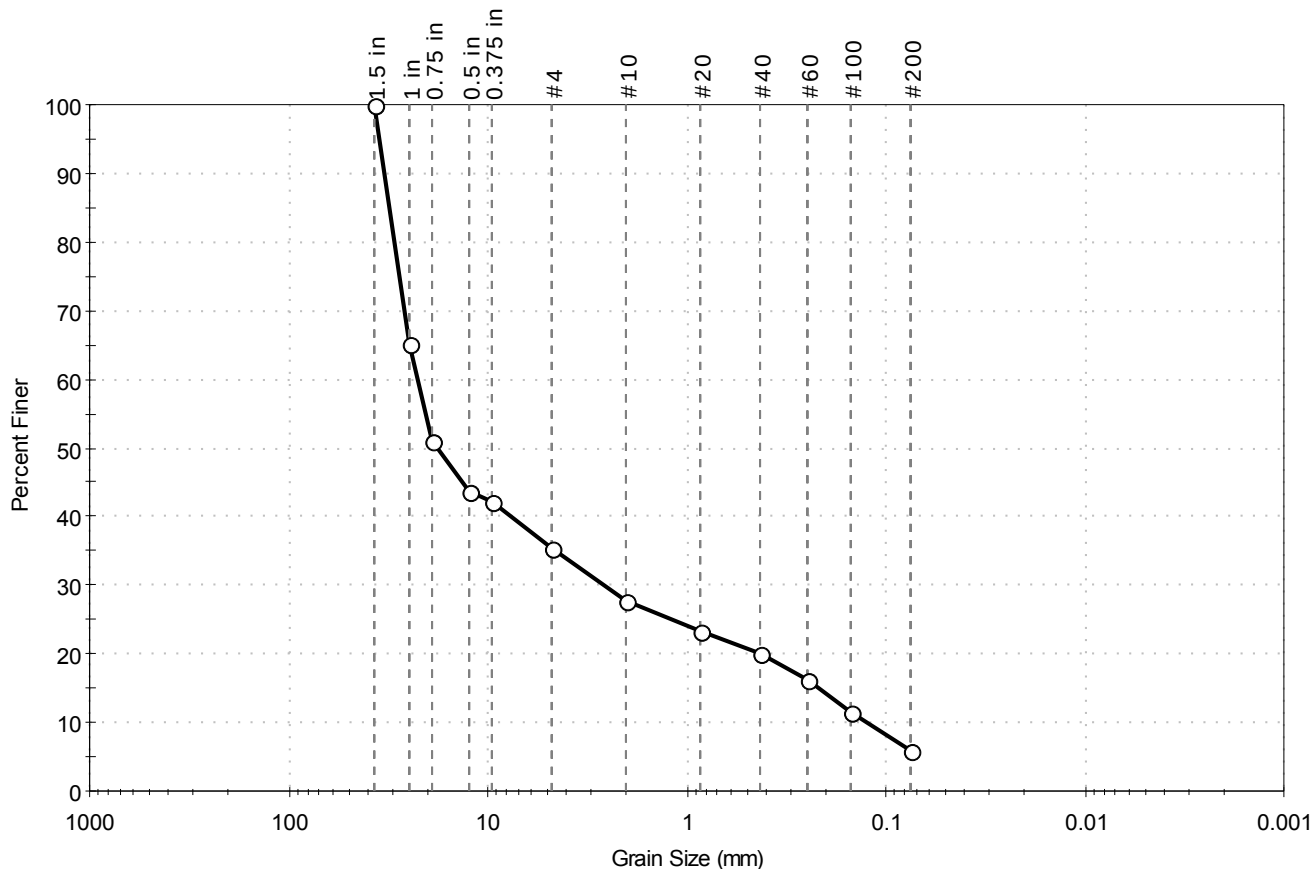
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-403	Sample Type:	bag
Sample ID:	3D	Test Date:	05/25/17
Depth:	4.0-6.0 ft	Test Id:	411996
Test Comment:	---		
Visual Description:	Moist, dark brown gravel with silt and sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	64.7	29.3	6.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	65		
0.75 in	19.00	51		
0.5 in	12.50	44		
0.375 in	9.50	42		
#4	4.75	35		
#10	2.00	28		
#20	0.85	23		
#40	0.42	20		
#60	0.25	16		
#100	0.15	12		
#200	0.075	6.0		

Coefficients

D ₈₅ = 31.4887 mm	D ₃₀ = 2.5610 mm
D ₆₀ = 22.5875 mm	D ₁₅ = 0.2167 mm
D ₅₀ = 17.8135 mm	D ₁₀ = 0.1233 mm
C _u = 183.191	C _c = 2.355

Classification

ASTM N/A

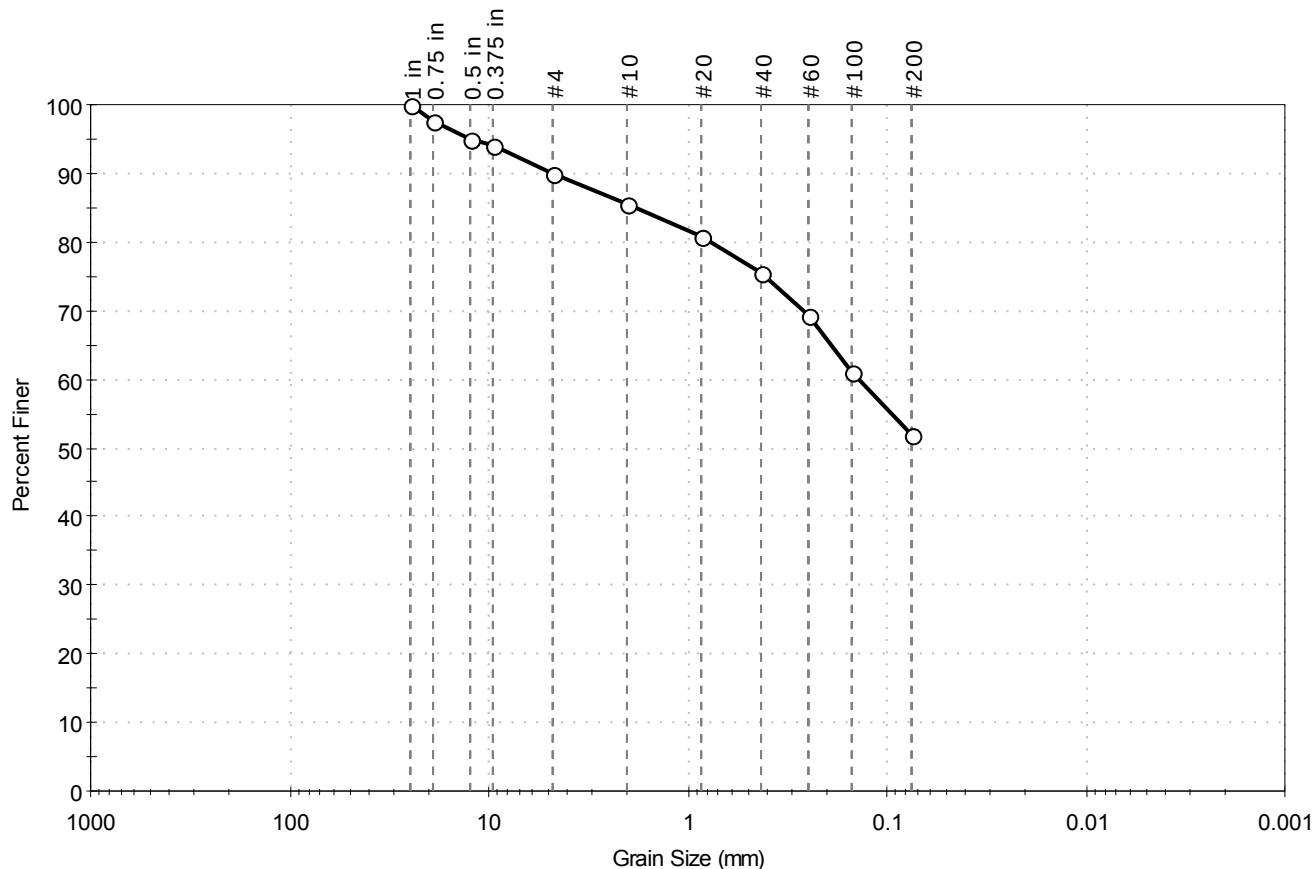
AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	U.S. 201 & 9 over the Cobbossee Stream		
Location:	Gardiner, Maine	Project No:	GTX-304437
Boring ID:	BB-GCS-403	Sample Type:	bag
Sample ID:	6D	Test Date:	05/25/17
Depth :	15.0-17.0 ft	Test Id:	411997
Test Comment:	---		
Visual Description:	Moist, olive sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.9	38.1	52.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	98		
0.5 in	12.50	95		
0.375 in	9.50	94		
#4	4.75	90		
#10	2.00	86		
#20	0.85	81		
#40	0.42	75		
#60	0.25	69		
#100	0.15	61		
#200	0.075	52		

Coefficients

D ₈₅ = 1.8151 mm	D ₃₀ = N/A
D ₆₀ = 0.1394 mm	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 : ANGULAR
Sand/Gravel Hardness : HARD



Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID: ---	Sample Type: ---	Tested By:	cam
Sample ID: ---	Test Date: 01/22/18	Checked By:	emm
Depth : ---	Test Id: 439716		

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
BB-GCS-104	S5+S6+S7	10.8-17	Moist, very dark gray sand with gravel	6	99.7	.3
BB-GCS-105	11D+12D+13D	20-29	Moist, dark gray sand with gravel	3	99.7	.3
BB-GCS-303	2D+3D	5-12	Moist, gray clay	22	98.8	1.2
BB-GCS-304	4D+5D+6D	12.5-21	Moist, dark gray clay with gravel	16	98.8	1.2
BB-GCS-401	1D+3D+4D	0-8.2	Moist, dark brown sand with gravel	7	98.5	1.5

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass;
dried to a constant mass at temperature of 105° C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440° C



Client:	Haley & Aldrich, Inc.				
Project:	US 201 & 9 over the Cobbossee Stream				
Location:				Project No:	GTX-307546
Boring ID: ---	Sample Type: ---		Tested By:	jbr	
Sample ID: ---	Test Date: 01/22/18		Checked By:	emm	
Depth : ---	Test Id: 439721				

pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
BB-GCS-104	S5+S6+S7	10.8-17	Moist, very dark gray sand with gravel	6.1	5.8
BB-GCS-105	11D+12D+13E	20-29	Moist, dark gray sand with gravel	6.3	6.1
BB-GCS-303	2D+3D	5-12	Moist, gray clay	7.7	6.9
BB-GCS-304	4D+5D+6D	12.5-21	Moist, dark gray clay with gravel	6.6	6.2
BB-GCS-401	1D+3D+4D	0-8.2	Moist, dark brown sand with gravel	6.1	5.9

Notes: Sample Preparation: screened through #10 sieve
Method A, pH meter used



Client:	Haley & Aldrich, Inc.
Project:	US 201 & 9 over the Cobbossee Stream
Location:	---
GTX#:	307546
Test Date:	01/22/18
Tested By:	jbr
Checked By:	emm

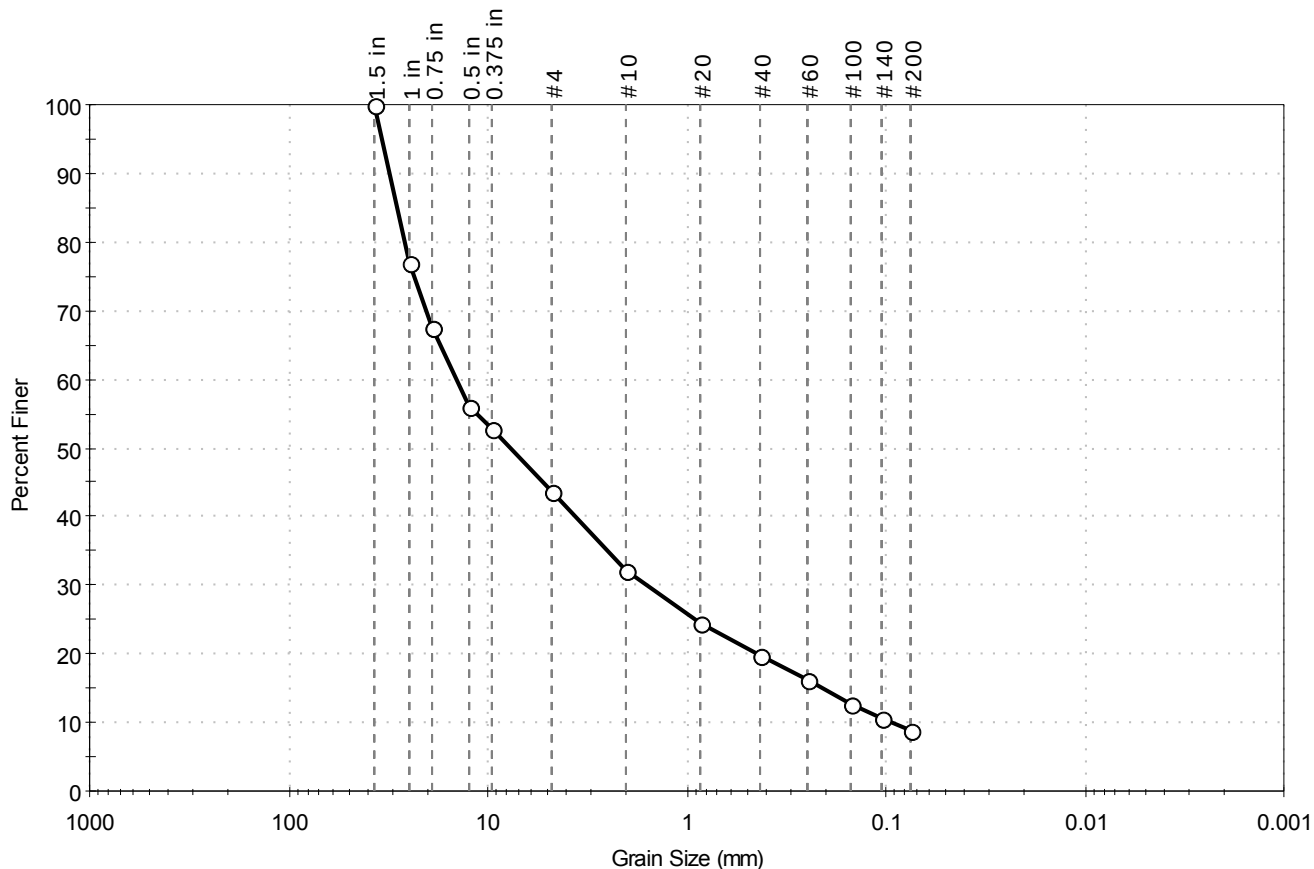
Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, (ohm-cm) ⁻¹
BB-GCS-104	S5+S6+S7	10.8-17	Moist, very dark gray sand with gravel	3,306	3.03E-04
BB-GCS-105	11D+12D+13D	20-29	Moist, dark gray sand with gravel	3,822	2.62E-04
BB-GCS-303	2D+3D	5-12	Moist, gray clay	744	1.34E-03
BB-GCS-304	4D+5D+6D	12.5-21	Moist, dark gray clay with gravel	3,306	3.03E-04
BB-GCS-401	1D+3D+4D	0-8.2	Moist, dark brown sand with gravel	992	1.01E-03

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
 Water added to sample to create a thick slurry prior to testing (saturated condition).
 Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
 Test conducted in standard laboratory atmosphere: 68-73 F

Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID:	BB-GCS-401	Sample Type:	jar
Sample ID:	5D	Test Date:	01/18/18
Depth:	9-11	Test Id:	439702
Test Comment:	---		
Visual Description:	Moist, reddish gray gravel with silt and sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	56.4	34.6	9.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	77		
0.75 in	19.00	67		
0.5 in	12.50	56		
0.375 in	9.50	53		
#4	4.75	44		
#10	2.00	32		
#20	0.85	25		
#40	0.42	20		
#60	0.25	16		
#100	0.15	13		
#140	0.11	11		
#200	0.075	9.0		

Coefficients

D ₈₅ = 28.7510 mm	D ₃₀ = 1.5838 mm
D ₆₀ = 14.4446 mm	D ₁₅ = 0.2116 mm
D ₅₀ = 7.6937 mm	D ₁₀ = 0.0920 mm
C _u = 157.007	C _c = 1.888

Classification

ASTM N/A

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

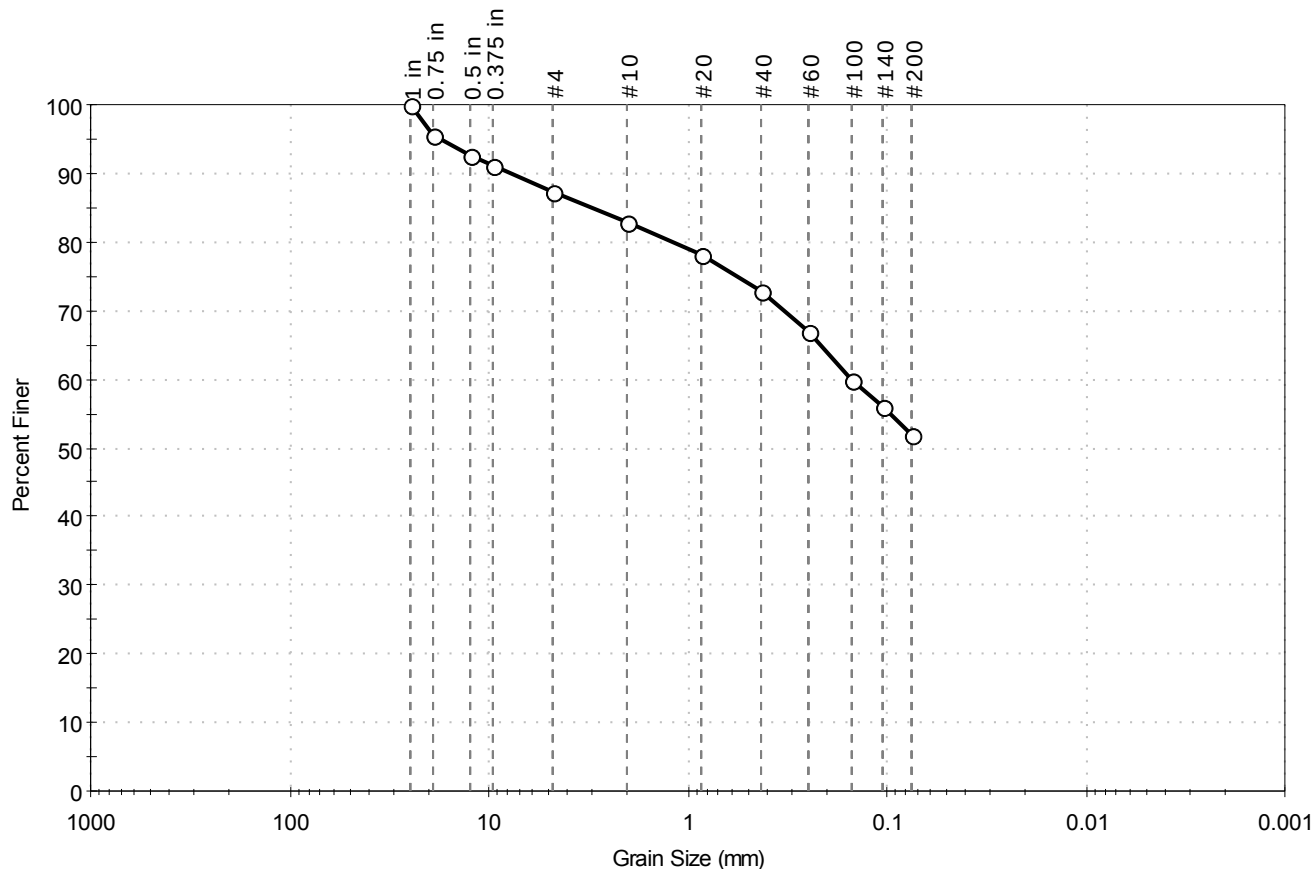
Sample / Test Description

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



Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID:	BB-GCS-401	Sample Type:	jar
Sample ID:	10D	Test Date:	01/18/18
Depth :	34-36	Test Id:	439703
Test Comment:	---		
Visual Description:	Moist, dark gray sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.5	35.6	51.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	96		
0.5 in	12.50	93		
0.375 in	9.50	91		
#4	4.75	87		
#10	2.00	83		
#20	0.85	78		
#40	0.42	73		
#60	0.25	67		
#100	0.15	60		
#140	0.11	56		
#200	0.075	52		

Coefficients

D ₈₅ = 2.9739 mm	D ₃₀ = N/A
D ₆₀ = 0.1497 mm	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

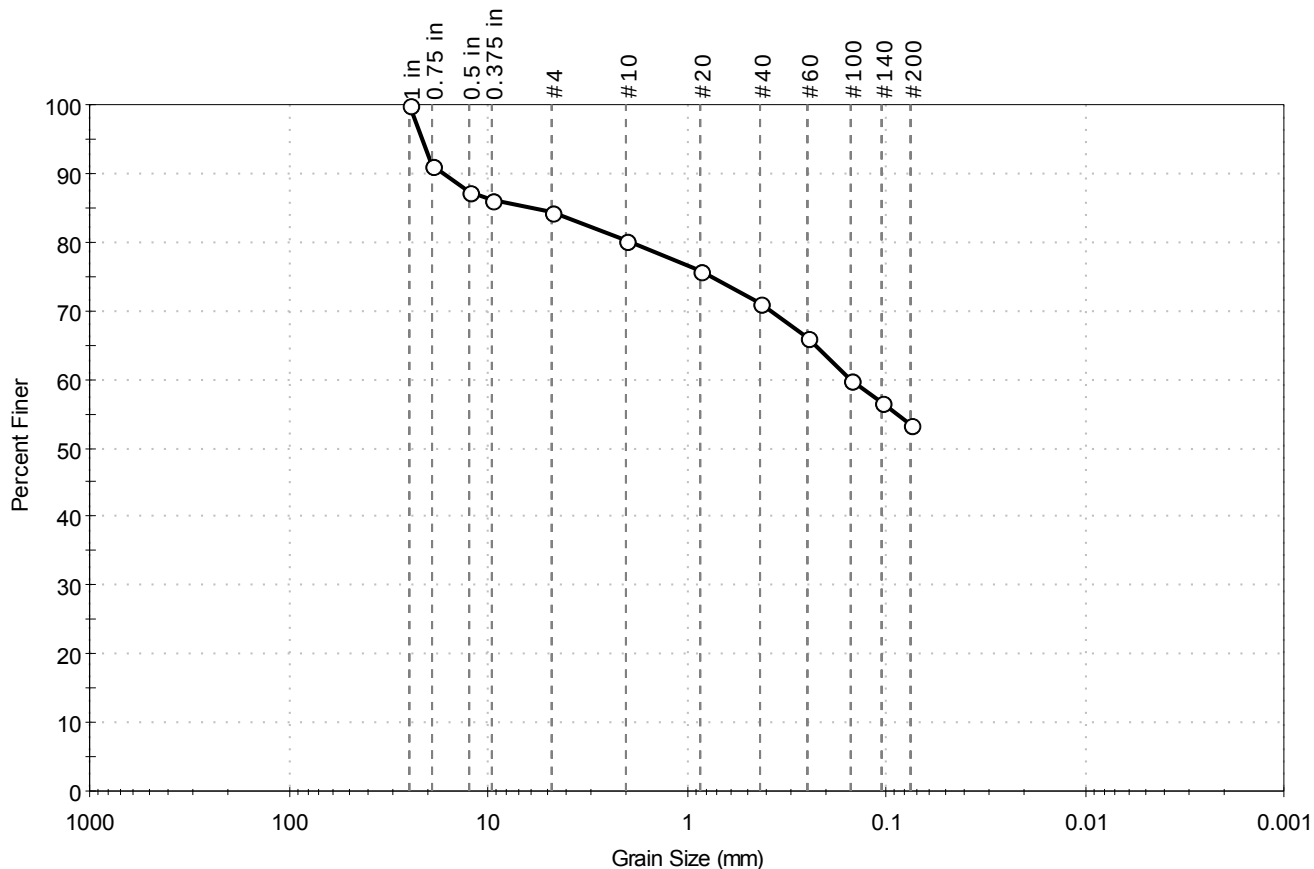
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD



Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID: BB-GCS-401A	Sample Type: jar	Tested By: jbr	
Sample ID: 15D	Test Date: 01/18/18	Checked By: emm	
Depth : 59-61	Test Id: 439704		
Test Comment:	---		
Visual Description:	Moist, dark gray sandy clay with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	15.6	31.1	53.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	87		
0.375 in	9.50	86		
#4	4.75	84		
#10	2.00	80		
#20	0.85	76		
#40	0.42	71		
#60	0.25	66		
#100	0.15	60		
#140	0.11	57		
#200	0.075	53		

Coefficients

D ₈₅ = 6.0648 mm	D ₃₀ = N/A
D ₆₀ = 0.1499 mm	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

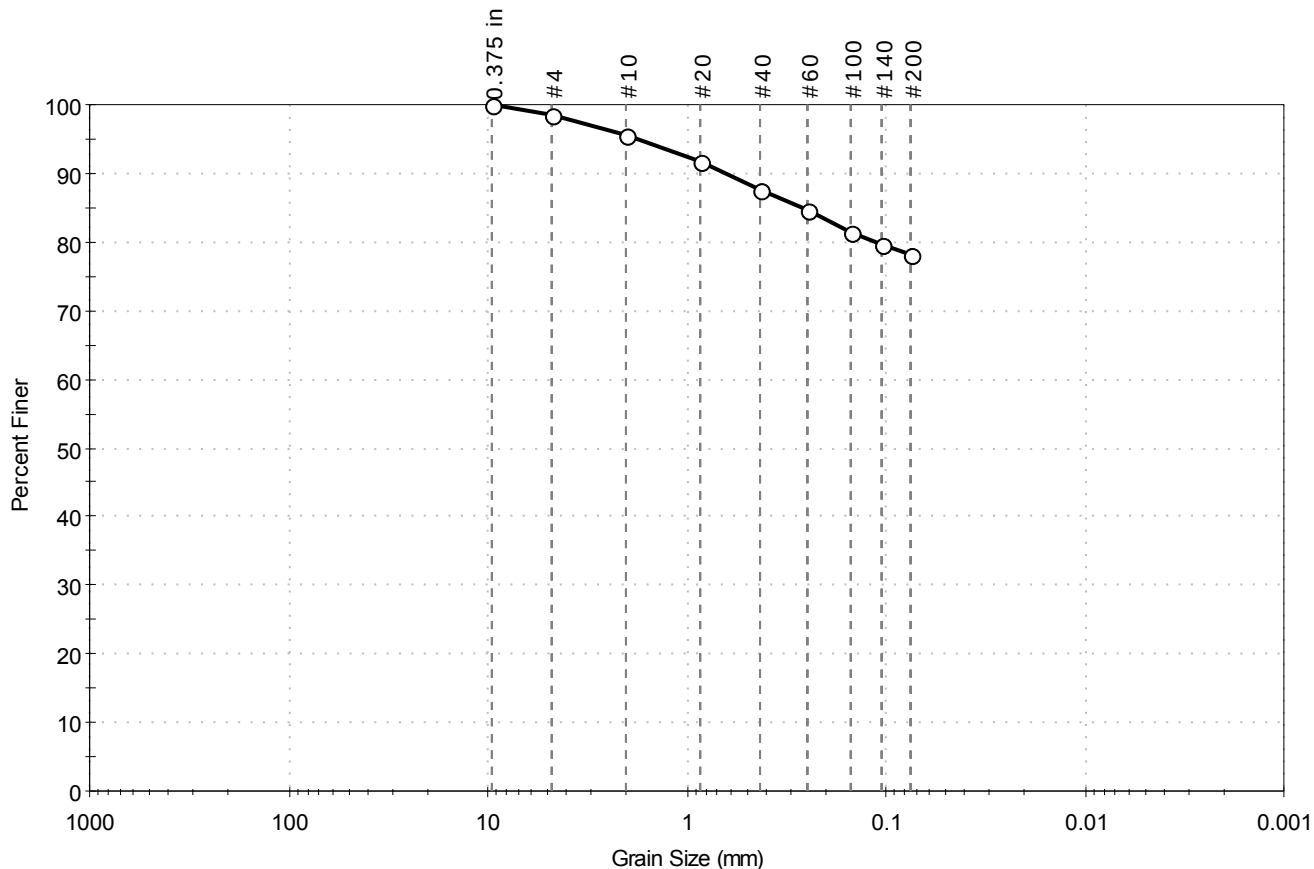
AASHTO Silty Soils (A-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID:	BB-GCS-402	Sample Type:	jar
Sample ID:	6D	Test Date:	01/18/18
Depth :	13-15	Test Id:	439705
Test Comment:	---		
Visual Description:	Moist, gray clay with sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.4	20.3	78.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	96		
#20	0.85	92		
#40	0.42	88		
#60	0.25	85		
#100	0.15	81		
#140	0.11	80		
#200	0.075	78		

Coefficients

$D_{85} = 0.2683 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = \text{N/A}$ $D_{15} = \text{N/A}$
 $D_{50} = \text{N/A}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

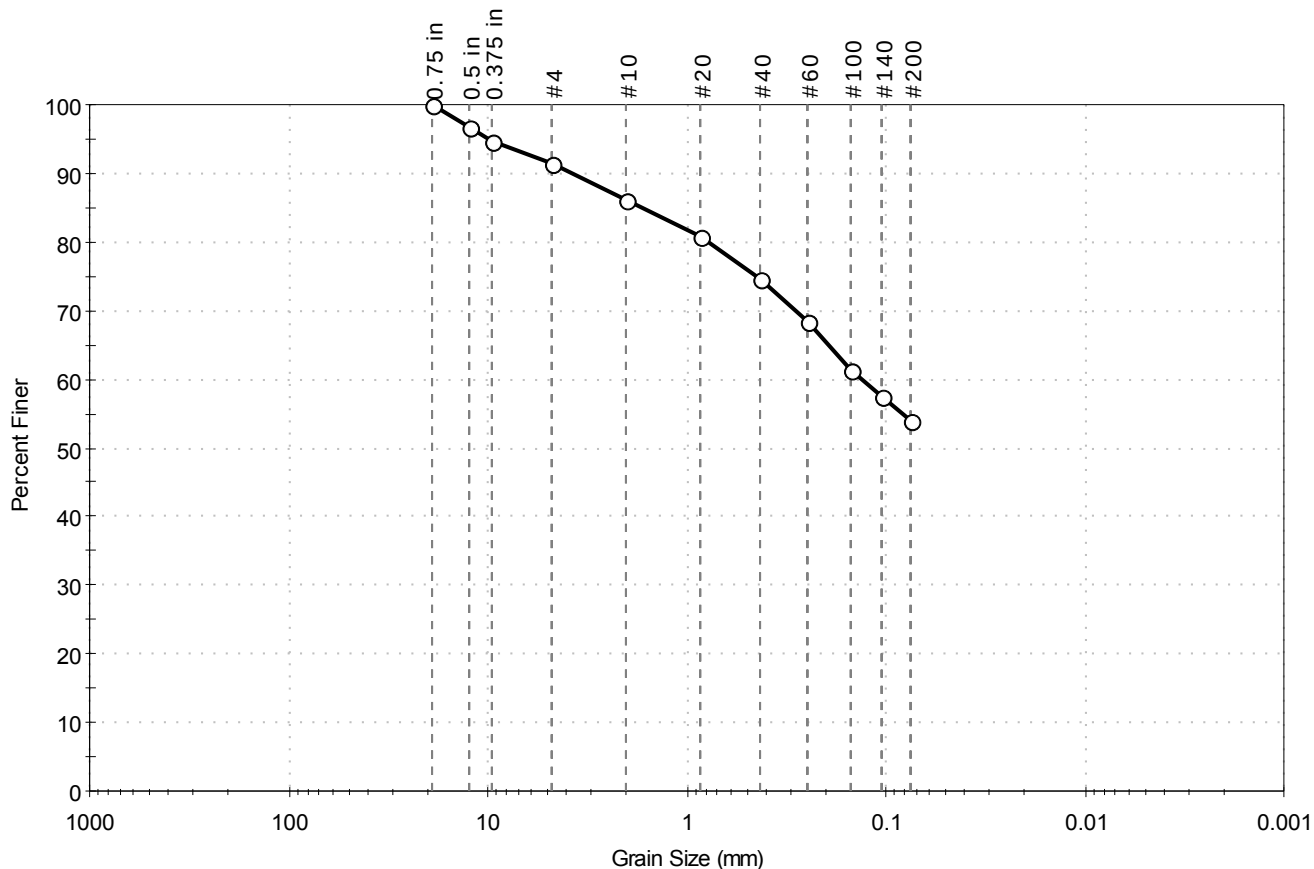
Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID:	BB-GCS-403	Sample Type:	jar
Sample ID:	10D	Test Date:	01/18/18
Depth :	35-36.8	Test Id:	439706
Test Comment:	---		
Visual Description:	Moist, gray sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	8.4	37.7	53.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	95		
#4	4.75	92		
#10	2.00	86		
#20	0.85	81		
#40	0.42	75		
#60	0.25	69		
#100	0.15	61		
#140	0.11	57		
#200	0.075	54		

Coefficients

D ₈₅ = 1.6769 mm	D ₃₀ = N/A
D ₆₀ = 0.1328 mm	D ₁₅ = N/A
D ₅₀ = N/A	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

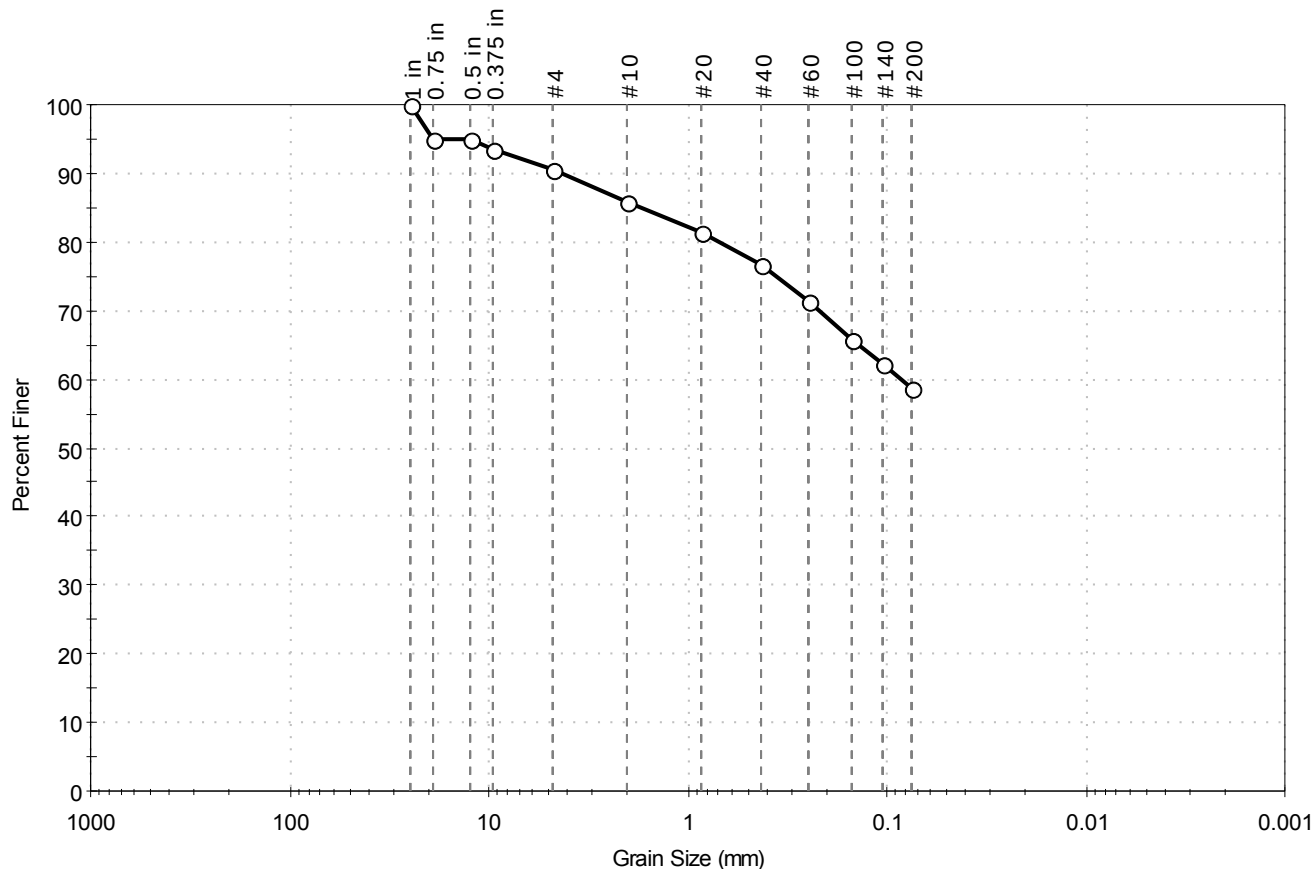
AASHTO Silty Soils (A-4 (0))

Sample / Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	US 201 & 9 over the Cobbossee Stream		
Location:		Project No:	GTX-307546
Boring ID:	BB-GCS-403	Sample Type:	jar
Sample ID:	14D	Test Date:	01/18/18
Depth :	55-57	Test Id:	439707
Test Comment:	---		
Visual Description:	Moist, gray sandy clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.5	31.8	58.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	95		
0.5 in	12.50	95		
0.375 in	9.50	94		
#4	4.75	90		
#10	2.00	86		
#20	0.85	81		
#40	0.42	77		
#60	0.25	71		
#100	0.15	66		
#140	0.11	62		
#200	0.075	59		

Coefficients

D ₈₅ = 1.6967 mm	D ₃₀ = N/A
D ₆₀ = 0.0849 mm	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))

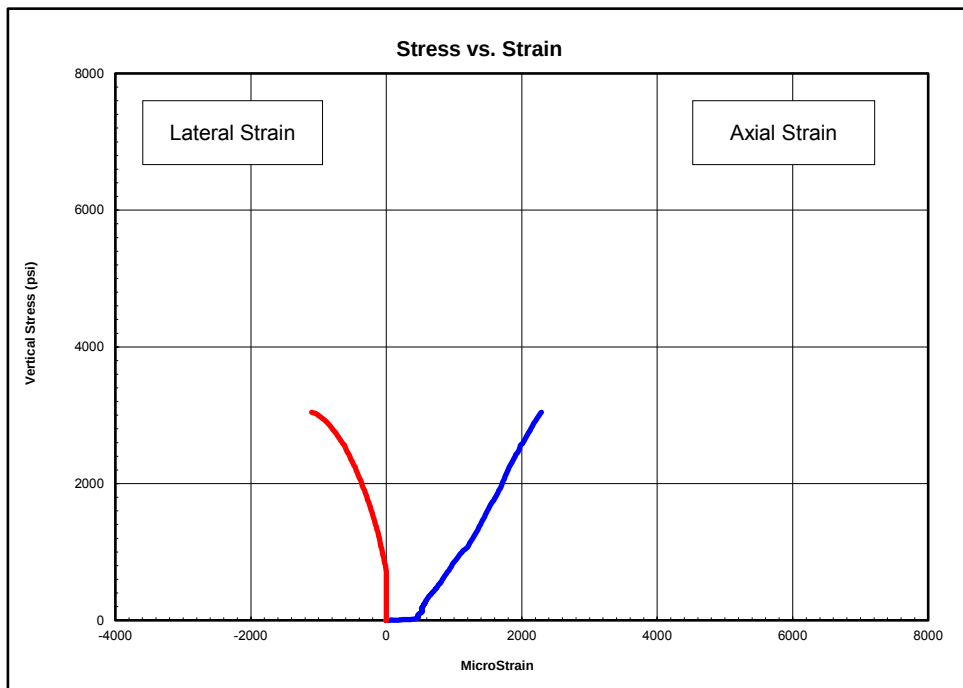
Sample / Test Description

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



Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-401A
Sample ID:	R4
Depth, ft:	78.9-79.3
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 3,043 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
300-1100	1,340,000	0.15
1100-1900	1,810,000	---
1900-2700	1,950,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

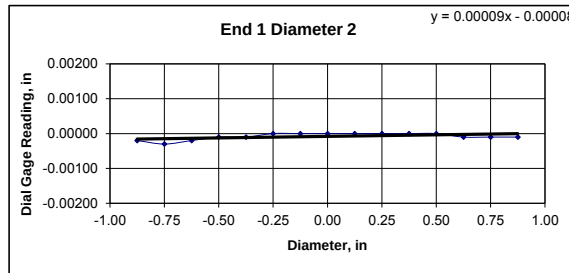
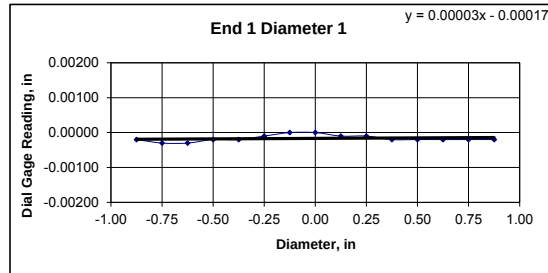


Client:	Haley & Aldrich, Inc.	Test Date:	1/17/2018
Project Name:	US 201 & 9 over the Cobbessee Stream	Tested By:	trm
Project Location:	---	Checked By:	jsc
GTX #:	307546		
Boring ID:	BB-GCS-401A		
Sample ID:	R4		
Depth:	78.9-79.3 ft		
Visual Description:	See photographs		

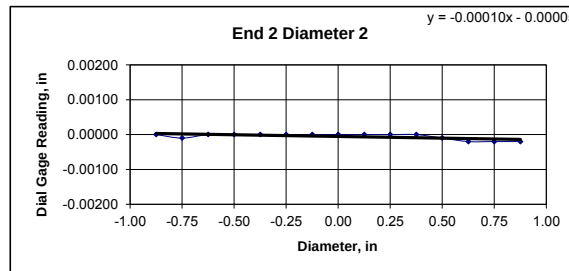
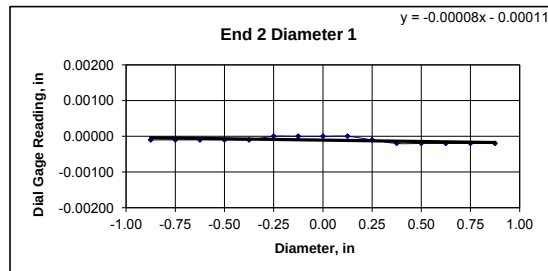
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? NO	
Specimen Length, in:	3.75	3.75	3.75	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? NO	
Specimen Mass, g:	508.31				
Bulk Density, lb/ft ³ :	166				
Length to Diameter Ratio:	1.9	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? NO			

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00030	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	-0.00020	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Difference between max and min readings, in:															
0° = 0.00030 90° = 0.00030															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00020
Difference between max and min readings, in:															
0° = 0.0002 90° = 0.0002															
Maximum difference must be $<$ 0.0020 in. Difference = $\pm$ 0.00015															
Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00003
Angle of Best Fit Line:	0.00164
End 2:	
Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00442
Maximum Angular Difference:	0.00278
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00009
Angle of Best Fit Line:	0.00507
End 2:	
Slope of Best Fit Line	0.00010
Angle of Best Fit Line:	0.00557
Maximum Angular Difference:	0.00049
Parallelism Tolerance Met? Spherically Seated	YES

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq$ 0.25°	
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES		
END 2							
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-401A
Sample ID:	R4
Depth, ft:	78.9-79.3 ft



After cutting and grinding

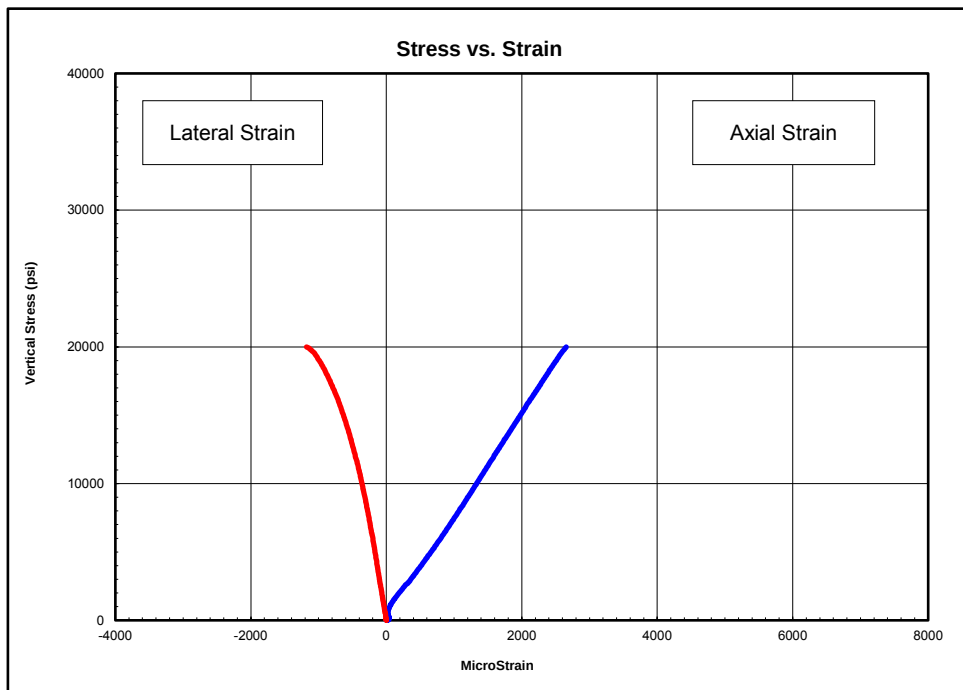


After break



Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-402A
Sample ID:	R4
Depth, ft:	76.1-76.7
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 20,003 psi

One axial strain gauge failed to record meaningful data. Young's Modulus and Poisson's Ratio reported based on results of a single axial strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2000-7300	6,700,000	0.23
7300-12700	7,720,000	0.33
12700-18000	7,660,000	---

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

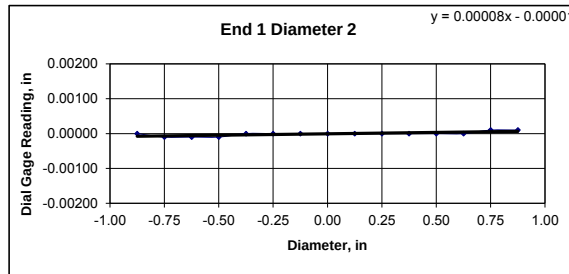
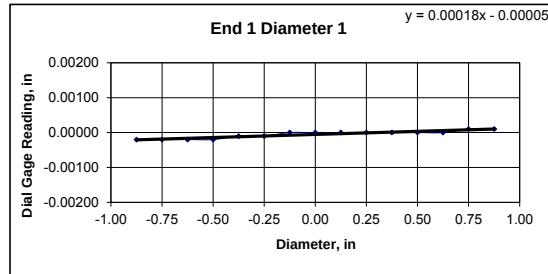


Client:	Haley & Aldrich, Inc.	Test Date:	1/17/2018
Project Name:	US 201 & 9 over the Cobbessee Stream	Tested By:	trm
Project Location:	---	Checked By:	jsc
GTX #:	307546		
Boring ID:	BB-GCS-402A		
Sample ID:	R4		
Depth:	76.1-76.7 ft		
Visual Description:	See photographs		

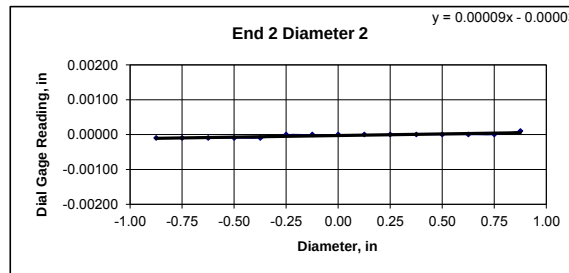
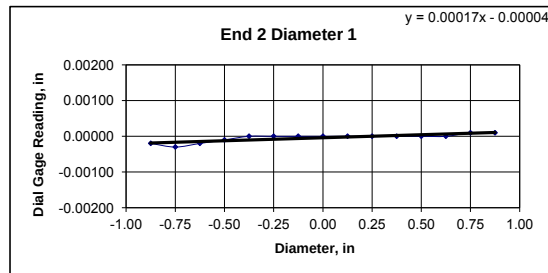
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.09	4.09	4.09	Maximum difference must be < 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	555.56				
Bulk Density, lb/ft ³ :	166				
Length to Diameter Ratio:	2.1	Minimum Diameter Tolerance Met? YES	Length to Diameter Ratio Tolerance Met? YES		

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Difference between max and min readings, in: 0° = 0.00030 90° = 0.00020															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010
Difference between max and min readings, in: 0° = 0.0004 90° = 0.0002 Maximum difference must be < 0.0020 in. Difference = ± 0.00020															
Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01015
End 2:	
Slope of Best Fit Line	0.00017
Angle of Best Fit Line:	0.00966
Maximum Angular Difference:	0.00049
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00458
End 2:	
Slope of Best Fit Line	0.00009
Angle of Best Fit Line:	0.00524
Maximum Angular Difference:	0.00065
Parallelism Tolerance Met? Spherically Seated	YES

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)						Maximum angle of departure must be $\leq$ 0.25°	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		
END 2							
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES		
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-402A
Sample ID:	R4
Depth, ft:	76.1-76.7 ft



After cutting and grinding

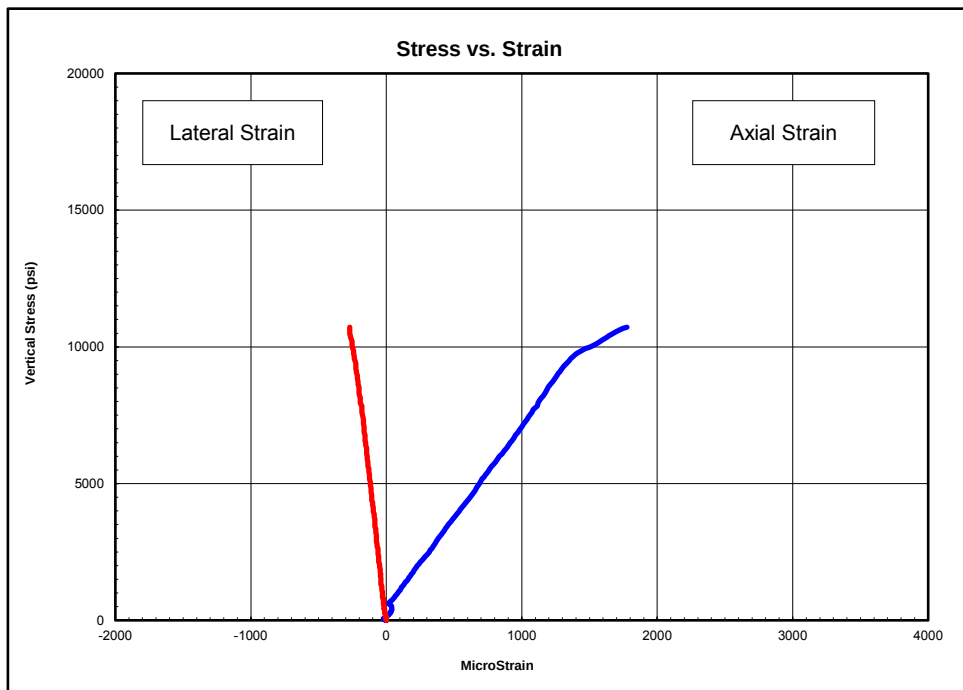


After break



Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-403
Sample ID:	R1
Depth, ft:	63.45-63.81 ft
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 10,719 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-3900	6,590,000	0.14
3900-6800	6,680,000	0.16
6800-9600	7,060,000	0.20

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

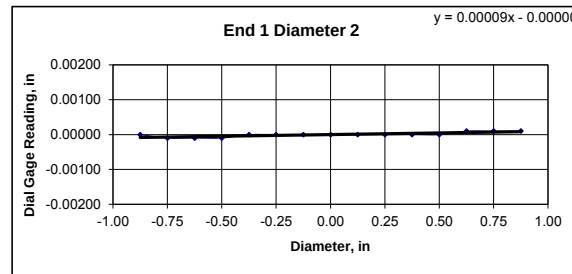
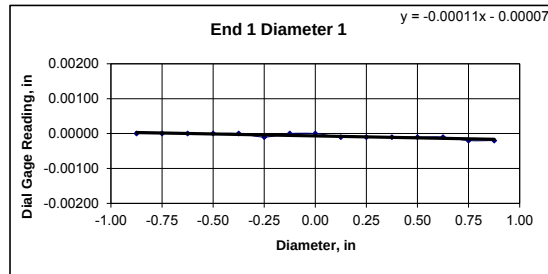


Client:	Haley & Aldrich, Inc.	Test Date:	1/17/2018
Project Name:	US 201 & 9 over the Cobbessee Stream	Tested By:	trm
Project Location:	---	Checked By:	jsc
GTX #:	307546		
Boring ID:	BB-GCS-403		
Sample ID:	R1		
Depth:	63.45-63.81 ft		
Visual Description:	See photographs		

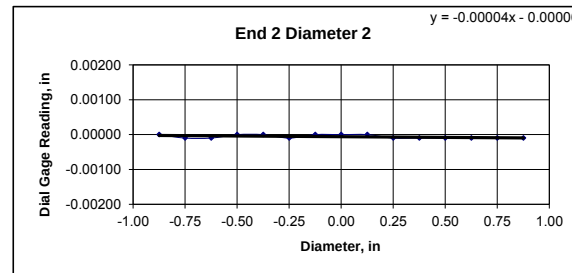
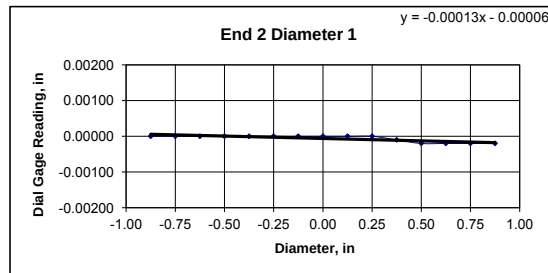
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.07	4.07	4.07	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	551.66				
Bulk Density, lb/ft ³ :	166				
Length to Diameter Ratio:	2.0				
		Minimum Diameter Tolerance Met?	YES		
		Length to Diameter Ratio Tolerance Met?	YES		

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Difference between max and min readings, in:															
0° = 0.00020 90° = 0.00020															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010
Difference between max and min readings, in:															
0° = 0.0002 90° = 0.0001															
Maximum difference must be $<$ 0.0020 in. Difference = $\pm$ 0.00010															
Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00638
End 2:	
Slope of Best Fit Line	0.00013
Angle of Best Fit Line:	0.00769
Maximum Angular Difference:	0.00131
Parallelism Tolerance Met?	YES
Spherically Seated	



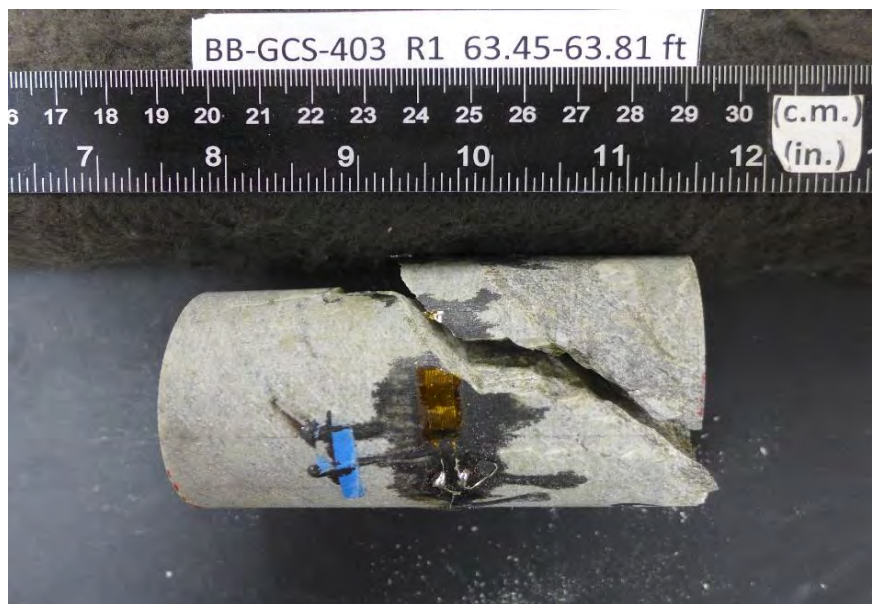
DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00009
Angle of Best Fit Line:	0.00540
End 2:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00229
Maximum Angular Difference:	0.00311
Parallelism Tolerance Met?	YES
Spherically Seated	

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq$ 0.25°
Diameter 1, in		0.00020	1.990	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)		0.00020	1.990	0.00010	0.006	YES	Perpendicularity Tolerance Met? YES
END 2							
Diameter 1, in		0.00020	1.990	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)		0.00010	1.990	0.00005	0.003	YES	

Client:	Haley & Aldrich, Inc.
Project Name:	US 201 & 9 over the Cobbossee Stream
Project Location:	---
GTX #:	307546
Test Date:	1/17/2018
Tested By:	trm
Checked By:	jsc
Boring ID:	BB-GCS-403
Sample ID:	R1
Depth, ft:	63.45-63.81



After cutting and grinding



After break



6100 HILLCROFT
PHONE (713) 369-5400

HOUSTON, TEXAS 77081
FAX (713) 369-5518

RESULTS OF TESTS

PROJECT: US 201-9 COBBOSSEE STREAM (GTX 307546)
SAMPLE ID: BB-GCS-401, 1D + 3D+ 4D, 0 – 8.2

FOR: GEOTESTING EXPRESS, INC.
125 NAGOG PARK ACTION, MA 01720

REPORTED TO: ETHAN MARRO

LAB NUMBER: 0125038

REPORT DATE: 01-26-18

CLIENT NUMBER:

JOB NUMBER: 04.1115-0003

REPORT NUMBER:

DATE SAMPLED:

TIME SAMPLED:

SAMPLED BY: CLIENT

DATE RECEIVED: 01-25-18

TIME RECEIVED: 1230

RECEIVED BY: SD

PARAMETER	RESULTS	UNITS	METHOD	TIME/DATE	ANALYST
Sulfate, Soluble	1,616 *	mg/kg	ASTM D-516 **	1200/01-26-18	SD
Chloride, Soluble	320 *	mg/kg	ASTM D-512 **	1230/01-26-18	SD

SO4CL 009-18

Respectfully submitted,

* Dry weight basis

Steve DeGregorio
Chemist

SD

** WATER EXTRACTION PERFORMED BY USING A 1:10 RATIO OF SAMPLE AND REAGENT WATER FOLLOWED BY CENTRIFUGE AND VACUUME FILTRATION. THE WATER EXTRACT IS THEN ANALYZED USING THE ASTM D-512 AND D-516 METHODS.

THE RESULTS RELATE AS TO THE LOCATION TESTED AND NO OTHER REFERENCE SHALL BE MADE.
THIS REPORT SHALL NOT BE REPRODUCED EXCEPT IN FULL WITHOUT THE WRITTEN APPROVAL OF THE LABORATORY.

END OF REPORT

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.

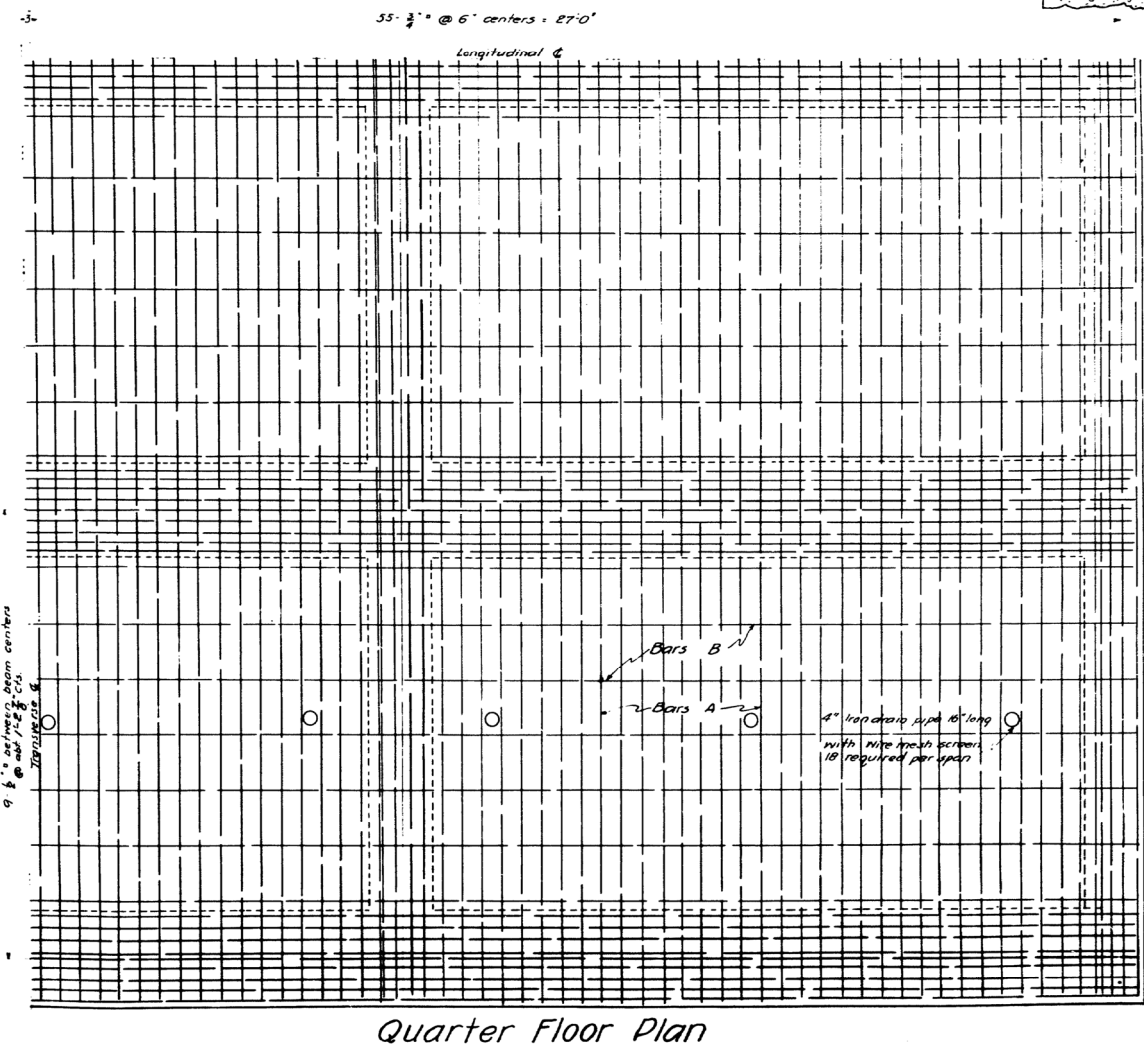
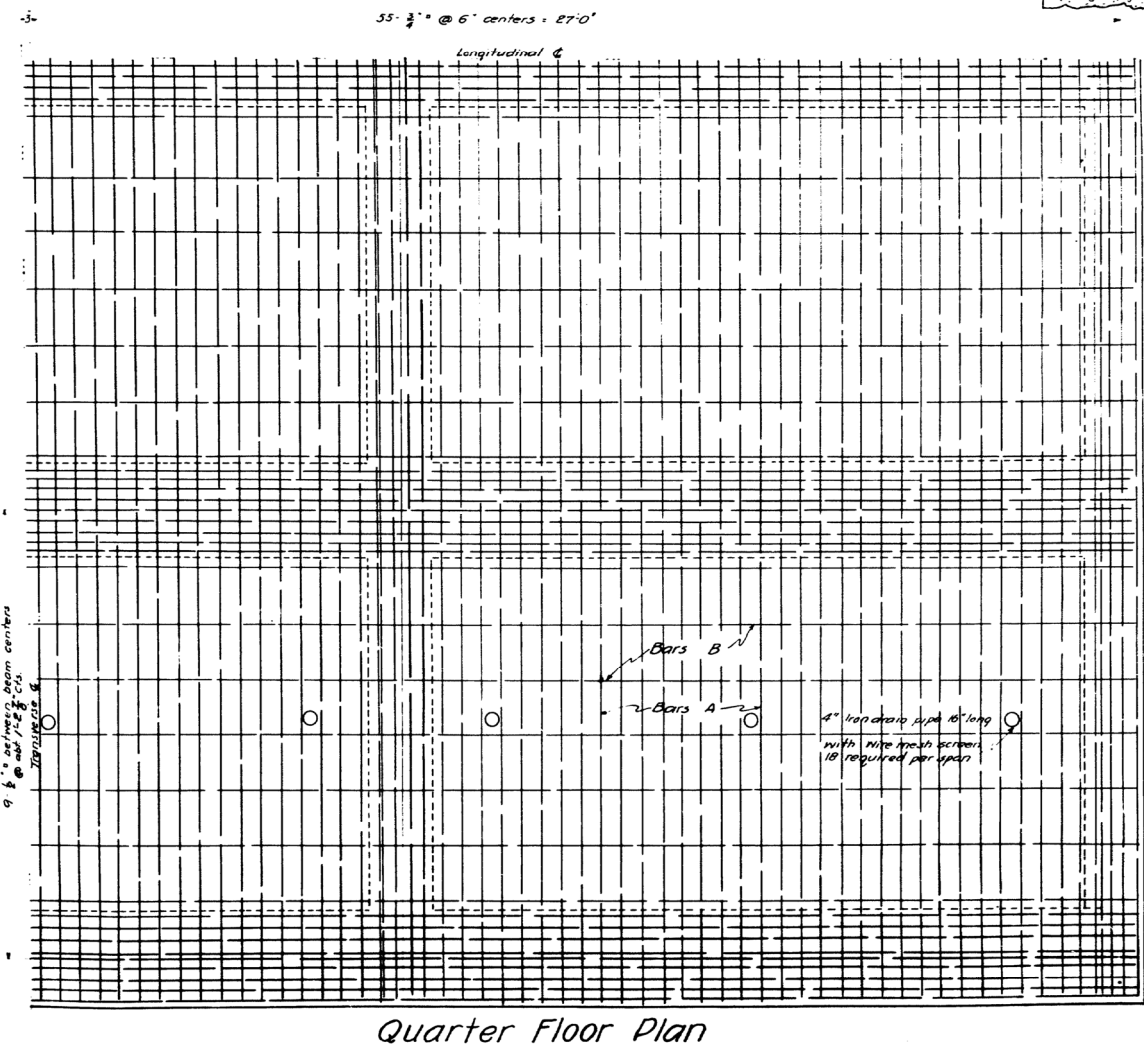
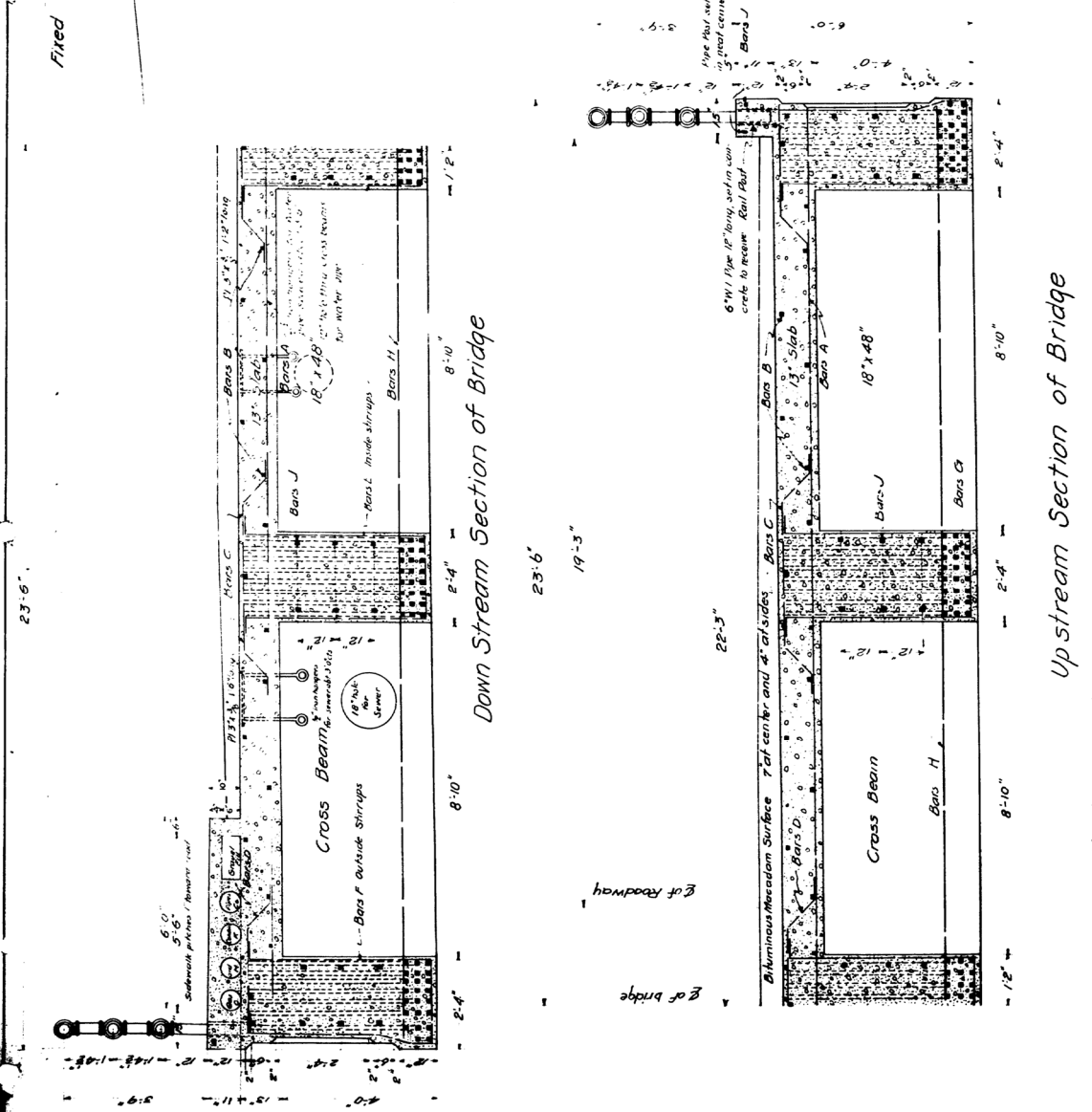
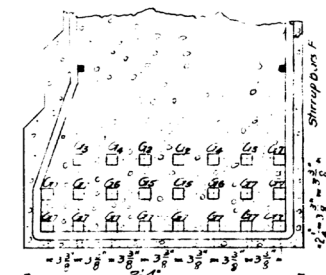
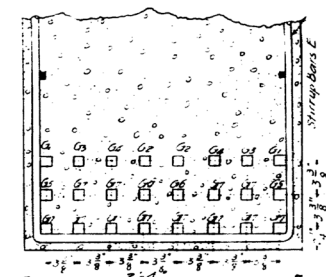
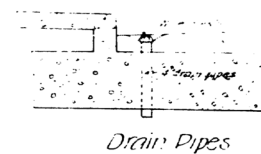
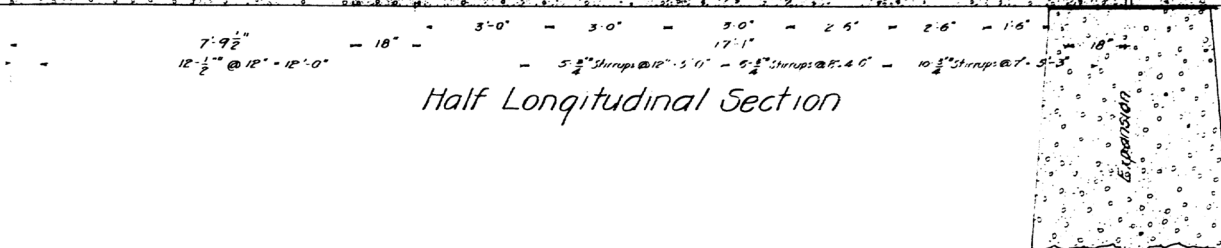
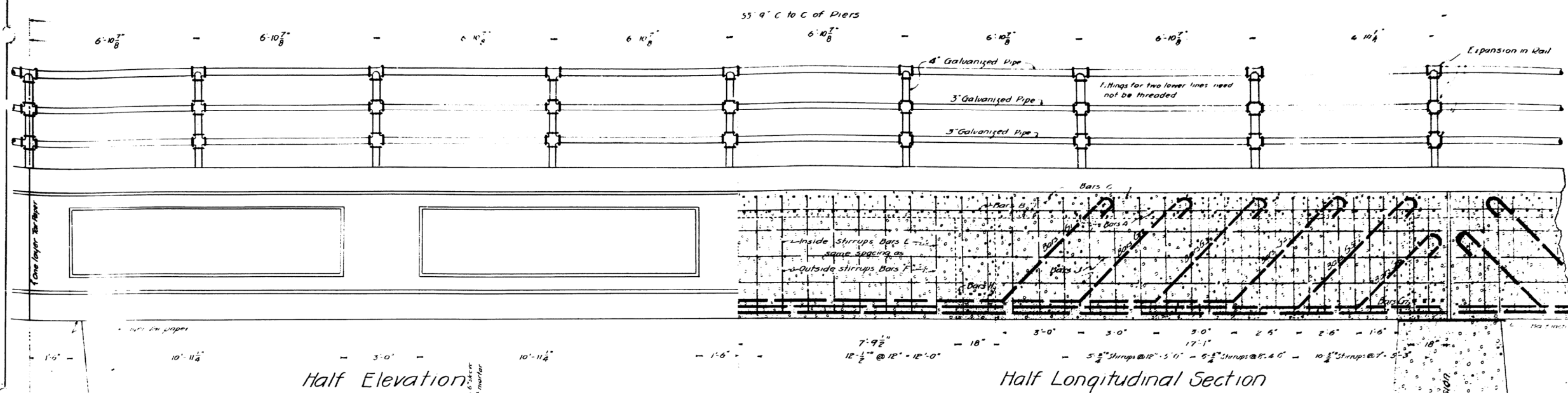
GTX's liability will be limited to correcting or repeating a test which fails our warranty. GTX's liability for damages to the Purchaser of testing services for any cause whatsoever shall be limited to the amount GTX received for the testing services. GTX will not be liable for any damages, or for any 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.

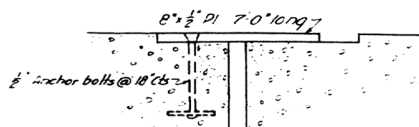
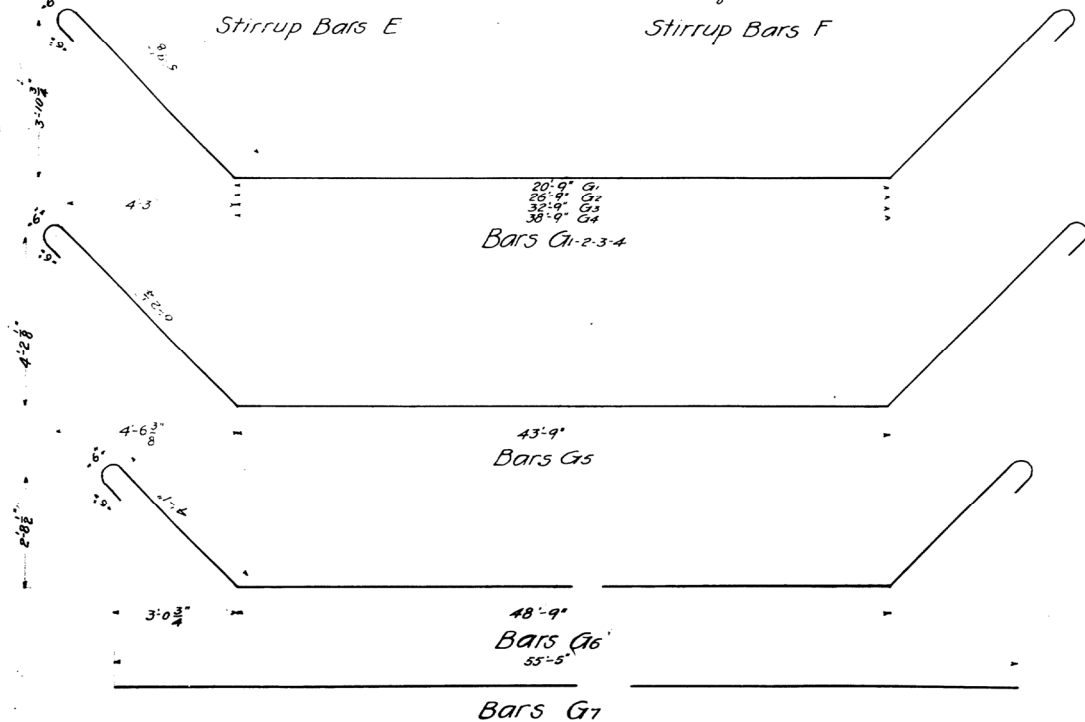
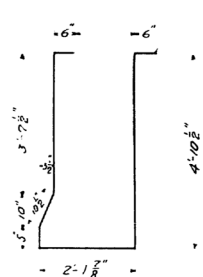
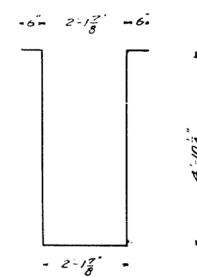
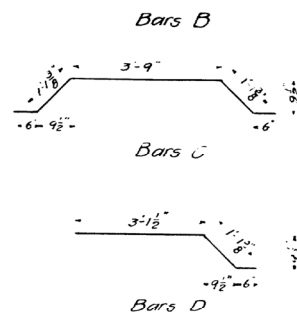
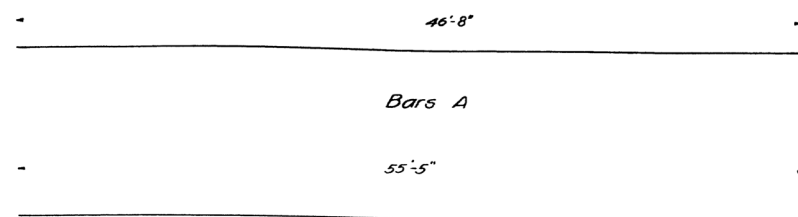
Commonly Used Symbols

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		

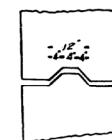
APPENDIX D

Historic Bridge Drawings

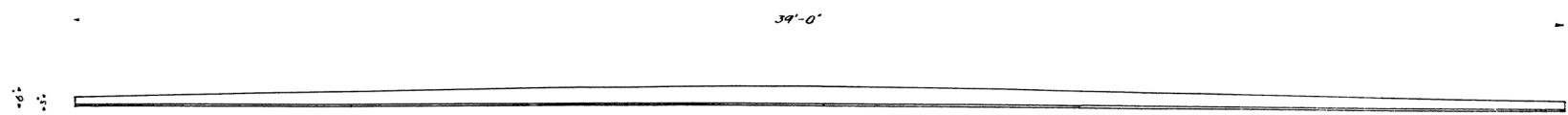




Section of Sidewalk Expansion

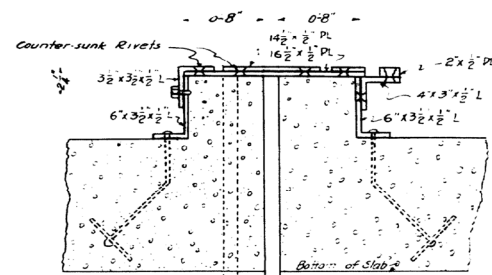


Detail of Joint at Floor Beam

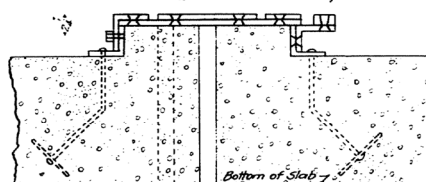


Detail of Bottom Angle of Expansion Joint Size 6"x3 1/2"x 1/2"

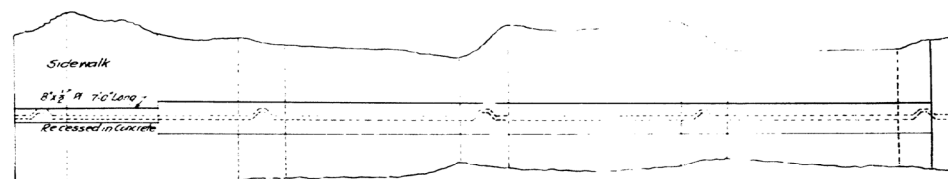
Two required for each joint. One to be drilled with a line of holes 24" centers 1 1/4" from top of flange and topped for 3/4" bolts. The 3/4" leg to be drilled with 3/8" holes 3'-0" centers in both angles. The 6" leg of both angles to be sheared to fit crown of road as shown.



Section of Expansion Joint at Center of Roadway



Section of Expansion Joint at Curb



Plan of Expansion

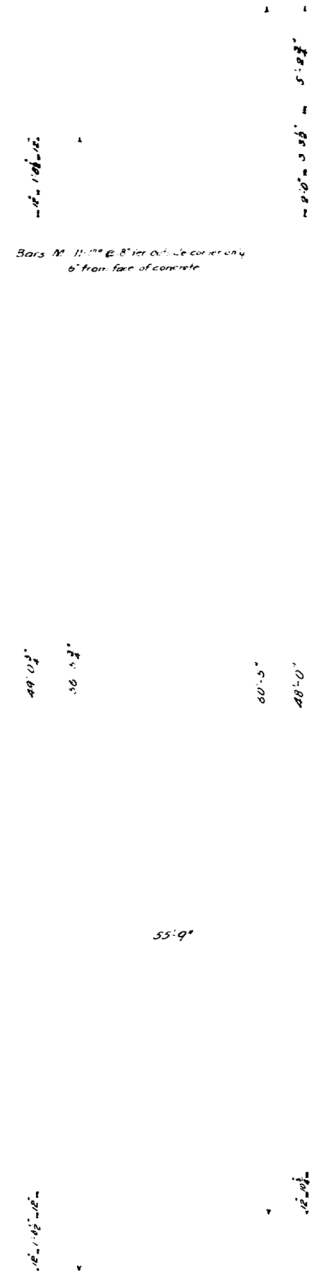
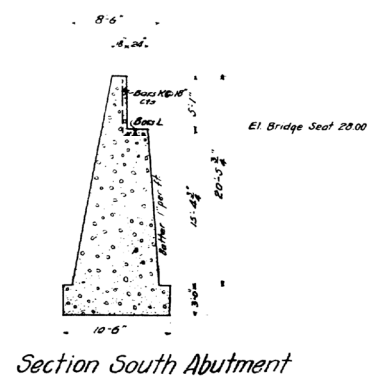
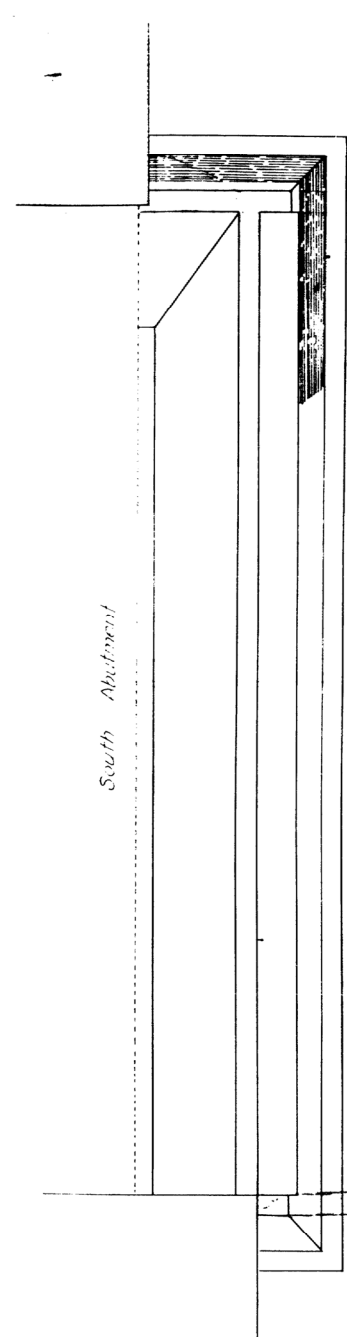
Schedule of Steel in Main and Cross Beams

Mark	Size	Length	No. Req'd	Weight	Location
A-1	3/4"	16'-8"	1	300	Slab
A-2	3/4"	16'-8"	1	300	Slab
A-3	3/4"	16'-8"	1	300	Slab
A-4	3/4"	16'-8"	1	300	Slab
A-5	3/4"	16'-8"	1	300	Slab
A-6	3/4"	16'-8"	1	300	Slab
A-7	3/4"	16'-8"	1	300	Slab
A-8	3/4"	16'-8"	1	300	Slab
A-9	3/4"	16'-8"	1	300	Slab
A-10	3/4"	16'-8"	1	300	Slab
A-11	3/4"	16'-8"	1	300	Slab
A-12	3/4"	16'-8"	1	300	Slab
A-13	3/4"	16'-8"	1	300	Slab
A-14	3/4"	16'-8"	1	300	Slab
A-15	3/4"	16'-8"	1	300	Slab
A-16	3/4"	16'-8"	1	300	Slab
A-17	3/4"	16'-8"	1	300	Slab
A-18	3/4"	16'-8"	1	300	Slab
A-19	3/4"	16'-8"	1	300	Slab
A-20	3/4"	16'-8"	1	300	Slab
A-21	3/4"	16'-8"	1	300	Slab
A-22	3/4"	16'-8"	1	300	Slab
A-23	3/4"	16'-8"	1	300	Slab
A-24	3/4"	16'-8"	1	300	Slab
A-25	3/4"	16'-8"	1	300	Slab
A-26	3/4"	16'-8"	1	300	Slab
A-27	3/4"	16'-8"	1	300	Slab
A-28	3/4"	16'-8"	1	300	Slab
A-29	3/4"	16'-8"	1	300	Slab
A-30	3/4"	16'-8"	1	300	Slab
A-31	3/4"	16'-8"	1	300	Slab
A-32	3/4"	16'-8"	1	300	Slab
A-33	3/4"	16'-8"	1	300	Slab
A-34	3/4"	16'-8"	1	300	Slab
A-35	3/4"	16'-8"	1	300	Slab
A-36	3/4"	16'-8"	1	300	Slab
A-37	3/4"	16'-8"	1	300	Slab
A-38	3/4"	16'-8"	1	300	Slab
A-39	3/4"	16'-8"	1	300	Slab
A-40	3/4"	16'-8"	1	300	Slab
A-41	3/4"	16'-8"	1	300	Slab
A-42	3/4"	16'-8"	1	300	Slab
A-43	3/4"	16'-8"	1	300	Slab
A-44	3/4"	16'-8"	1	300	Slab
A-45	3/4"	16'-8"	1	300	Slab
A-46	3/4"	16'-8"	1	300	Slab
A-47	3/4"	16'-8"	1	300	Slab
A-48	3/4"	16'-8"	1	300	Slab
A-49	3/4"	16'-8"	1	300	Slab
A-50	3/4"	16'-8"	1	300	Slab
A-51	3/4"	16'-8"	1	300	Slab
A-52	3/4"	16'-8"	1	300	Slab
A-53	3/4"	16'-8"	1	300	Slab
A-54	3/4"	16'-8"	1	300	Slab
A-55	3/4"	16'-8"	1	300	Slab
A-56	3/4"	16'-8"	1	300	Slab
A-57	3/4"	16'-8"	1	300	Slab
A-58	3/4"	16'-8"	1	300	Slab
A-59	3/4"	16'-8"	1	300	Slab
A-60	3/4"	16'-8"	1	300	Slab
A-61	3/4"	16'-8"	1	300	Slab
A-62	3/4"	16'-8"	1	300	Slab
A-63	3/4"	16'-8"	1	300	Slab
A-64	3/4"	16'-8"	1	300	Slab
A-65	3/4"	16'-8"	1	300	Slab
A-66	3/4"	16'-8"	1	300	Slab
A-67	3/4"	16'-8"	1	300	Slab
A-68	3/4"	16'-8"	1	300	Slab
A-69	3/4"	16'-8"	1	300	Slab
A-70	3/4"	16'-8"	1	300	Slab
A-71	3/4"	16'-8"	1	300	Slab
A-72	3/4"	16'-8"	1	300	Slab
A-73	3/4"	16'-8"	1	300	Slab
A-74	3/4"	16'-8"	1	300	Slab
A-75	3/4"	16'-8"	1	300	Slab
A-76	3/4"	16'-8"	1	300	Slab
A-77	3/4"	16'-8"	1	300	Slab
A-78	3/4"	16'-8"	1	300	Slab
A-79	3/4"	16'-8"	1	300	Slab
A-80	3/4"	16'-8"	1	300	Slab
A-81	3/4"	16'-8"	1	300	Slab
A-82	3/4"	16'-8"	1	300	Slab
A-83	3/4"	16'-8"	1	300	Slab
A-84	3/4"	16'-8"	1	300	Slab
A-85	3/4"	16'-8"	1	300	Slab
A-86	3/4"	16'-8"	1	300	Slab
A-87	3/4"	16'-8"	1	300	Slab
A-88	3/4"	16'-8"	1	300	Slab
A-89	3/4"	16'-8"	1	300	Slab
A-90	3/4"	16'-8"	1	300	Slab
A-91	3/4"	16'-8"	1	300	Slab
A-92	3/4"	16'-8"	1	300	Slab
A-93	3/4"	16'-8"	1	300	Slab
A-94	3/4"	16'-8"	1	300	Slab
A-95	3/4"	16'-8"	1	300	Slab
A-96	3/4"	16'-8"	1	300	Slab
A-97	3/4"	16'-8"	1	300	Slab
A-98	3/4"	16'-8"	1	300	Slab
A-99	3/4"	16'-8"	1	300	Slab
A-100	3/4"	16'-8"	1	300	Slab

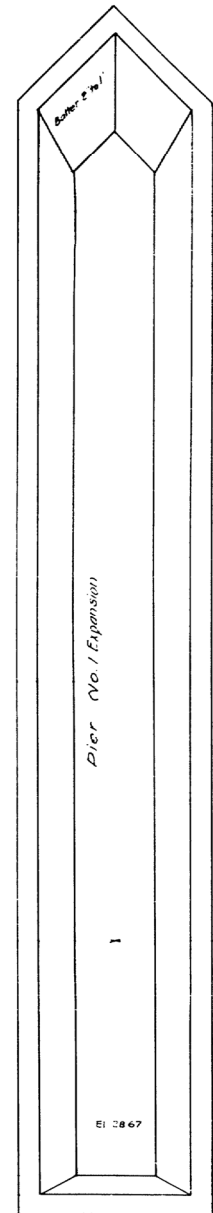
Schedule of Reinforcing Steel

Mark	Size	Length	No. Required	Location	Weight	Remarks
A	3/4"	46'-8"	330	Slab	29663	Staircase
B	3/4"	55'-5"	132	Slab	6220	Staircase
C	3/4"	7'-0"	1320	Slab	17685	Staircase
D	3/4"	4'-9"	880	Slab	3000	Staircase
E	3/4"	12'-10 3/8"	504	Beams	12420	Inside Staircase
F	3/4"	12'-10 3/8"	288	Beams	3152	Inside Staircase
G	3/4"	12'-11 1/2"	336	Beams	8488	Outside Staircase
H	3/4"	12'-11 1/2"	192	Beams	2120	Outside Staircase
I	3/4"	34'-10 1/2"	24	Beams	3600	Beams
J	3/4"	40'-10 1/2"	40	Beams	7038	Beams
K	3/4"	46'-10 1/2"	40	Beams	8068	Beams
L	3/4"	52'-10 1/2"	40	Beams	9100	Beams
M	3/4"	58'-8 1/2"	40	Beams	10104	Beams
N	3/4"	59'-6"	40	Beams	10240	Beams
O	3/4"	55'-5"	240	Beams	59000	Staircase
P	3/4"	46'-6"	48	Cross Beams	9603	Staircase
Q	3/4"	55'-5"	140	Beams	6532	Staircase
R	3/4"	20'-6"	11	Anchor Rods	770	Anchor Rods
S	3/4"	26'-0"	26	Anchor Rods	2300	Anchor Rods
T	3/4"	16'-0"	8	Anchor Rods	434	Anchor Rods

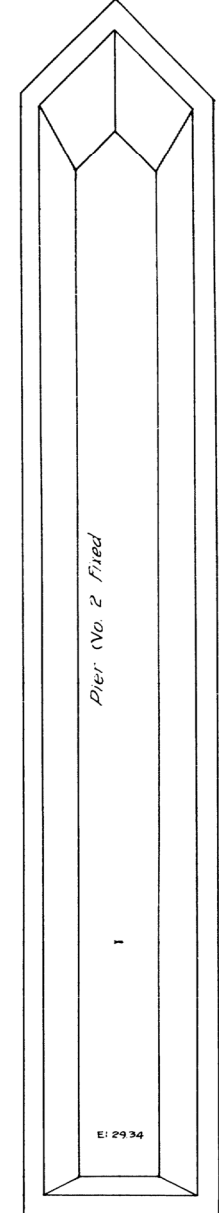
Maine Highway Commission
 Bridge Over
COBBOSSEECONTEE STREAM
 In the Town of
GARDINER, KENNEBEC CO., ME
 August 31, 1917 General Bridge Law
 Sheet No. 3



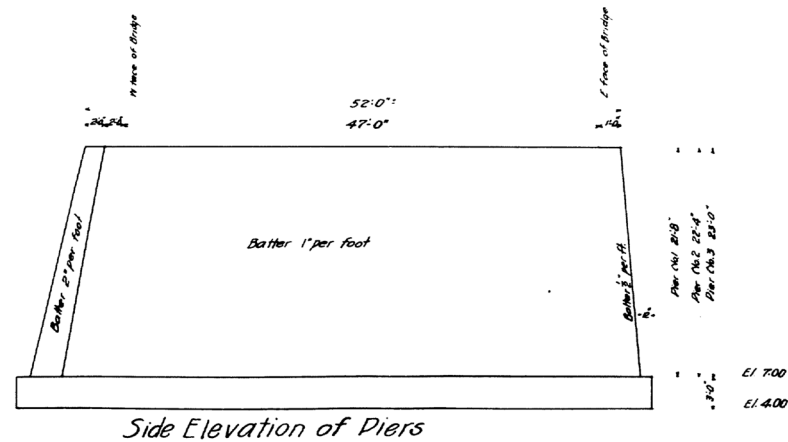
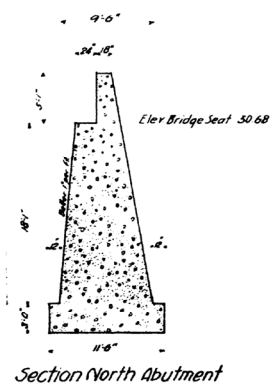
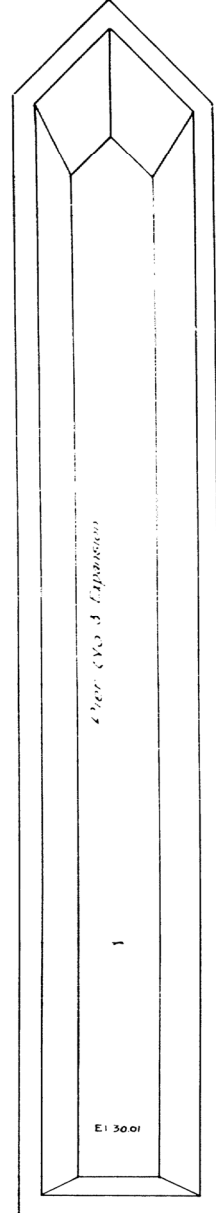
12'-1 1/2" 4'-0" 12'-1 1/2" 9'-7 1/2"



12'-1 1/2" 4'-0" 12'-1 1/2" 9'-8 1/2"

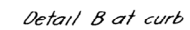
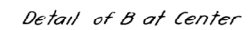
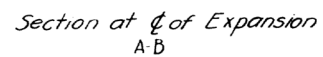
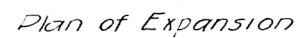
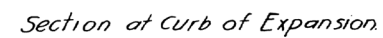
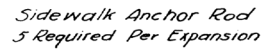
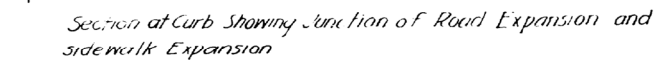


12'-1 1/2" 4'-0" 12'-1 1/2" 9'-10"

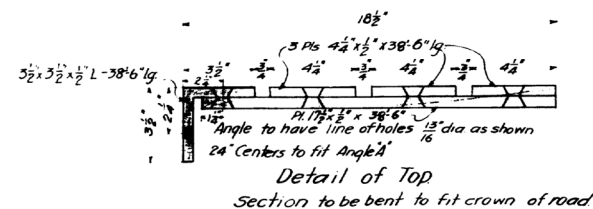


0 1 2 3 4 5 INCHES

Maine Highway Commission
Bridge over
COBBOSSECONTEE STREAM
In the City of
GARDINER, KENNEBEC CO., ME.
August 31, 1917 General Bridge Law
Sheet No. 4 of 4



(NOTE: The $6 \times 3\frac{1}{2} \times \frac{1}{2}$ L to be sheared to fit crown of Road, the other parts to be bent to fit.
The $3\frac{1}{2}$ " leg to have a line of $\frac{9}{16}$ " holes spaced as shown in detail of Angle "A"



Maine Highway Commission
Bridge Over
COBBOSEECONTEE STREAM
In the City of
GARDINER, KENNEBEC CO., ME.
Sept. 25 '1917 Sheet No. 5 General Bridge Law

APPENDIX E

A1 Diner Foundations Memorandum



HALEY & ALDRICH, INC.
75 Washington Avenue
Suite 1A
Portland, ME 04101
207.482.4600

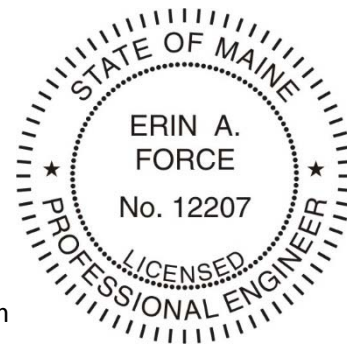
MEMORANDUM

29 September 2017
File No. 42223-000

TO: Stantec
Tim Merritt, P.E.

FROM: Haley & Aldrich, Inc.
Erin A. Force, P.E., Wayne A. Chadbourne, P.E.

SUBJECT: A1 Diner Foundations
Bridge Street Bridge
Routes U.S. 201 and 9 over Cobbosseecontee Stream
Gardiner, Maine
MaineDOT WIN 18735.00, Bridge No. 2101



This memorandum provides the results of a recently completed supplemental subsurface exploration program (test pits) at the A1 Diner and a discussion of the impacts of the A1 Diner foundations on the proposed bridge and retaining wall design and construction. This work has been completed in accordance with our proposal dated 15 June 2015, our amendment dated 16 March 2017, and your subsequent authorization.

We presented the results of previous subsurface explorations, summarized the soil, bedrock and groundwater conditions encountered at the site, and provided geotechnical design recommendations and construction considerations in a report titled "Preliminary Geotechnical Design Report, Bridge Street Bridge, Routes U.S. 201 and 9 over Cobbosseecontee Stream, Gardiner, Maine, MaineDOT WIN 18735.00, Bridge No. 2101," dated 8 June 2017 (PGDR).

Background

The proposed Bridge Street replacement bridge will be located approximately along the same alignment as the existing bridge, as shown on Figure 1, Project Locus. The replacement bridge is proposed to be a 135-ft single span structure with semi-integral abutments. The south abutment will be located immediately north of existing Pier 1. The north abutment will be located between the existing Pier 3 and north abutment.

An approximately 70-ft long retaining wall is proposed to extend from the proposed south abutment to the existing south abutment, along the east side of the roadway, to retain new embankment fill. The wall will be located immediately adjacent to the foundations for the elevated A1 Diner. The plan limits,

depth and condition of the A1 Diner foundations were not previously known. A ground penetrating radar (GPR) survey was performed in spring 2017, but the survey did not provide conclusive information on the plan limits of the A1 Diner foundations. Therefore a program of exploratory test pits was performed in summer 2017 to determine the A-1 Diner foundation type, foundation limits, foundation bearing elevations and foundation condition.

Elevation Datum

Elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

Subsurface Exploration Program

GROUND PENETRATING RADAR SURVEY

As part of a previous phase of explorations and as mentioned above, a GPR survey was performed in the area of the A1 Diner. Hager-Richter Geoscience, Inc. of Salem, New Hampshire performed the GPR survey at the site on 18 April 2017, under contract with MaineDOT. Their report outlining the details of the procedures, equipment used to complete the study, and results of the survey is included in Appendix B.

As described in the report in Appendix B, the GPR signal penetration was generally good, and is estimated to have been about 3 to 6.5 ft below ground surface. As shown in Figure 3 of the GPR report, the GPR survey identified shallow buried objects in the A1 Diner area ranging from about 2 ft by 2 ft to about 5 ft by 15 ft; the tops of the objects are approximately 1 to 2 ft below grade.

EXPLORATORY TEST PITS

Exploratory test pits were excavated on 6 July 2017 adjacent to the four western supports for the elevated A1 Diner and adjacent to the existing bridge Pier 1 in an effort to determine the foundation type (i.e., concrete or stacked blocks) and foundation bearing elevation, and to observe and document the existing foundation condition.

In general, the test pits were excavated to depths ranging from approximately 2 to 6 ft below ground surface (BGS) by McGee Construction, LLC (McGee) of West Gardiner, Maine using a Yanmar B22 mini-excavator. Each test pit was backfilled using excavated material upon completion. The excavated material was placed in lifts and tamped into place with the excavator bucket.

A Haley & Aldrich geotechnical engineer was on-site to monitor the excavations and record the soil, groundwater and foundation conditions encountered. Test pit locations are shown on Figure 2. Plan and profile sketches and photographs of the test pit excavations are provided in Appendix A.

Where visual and olfactory evidence of petroleum-impacted soil were observed, soil samples were screened with a photoionization detector (PID) to assess the presence of volatile organic compounds

(VOCs). There were several instances of elevated PID readings in test pits TP4 and TP5, with PID readings ranging from 455 to 827 ppm. Refer to individual test pit logs provided in Appendix A for PID readings.

Existing Foundation Conditions

Based on the conditions encountered in the test pit excavations, we have made the following general observations regarding the existing foundation system supporting the A1 Diner and existing bridge Pier 1:

Foundation Location	Foundation Type	Foundation Bearing Level		Comments
		Approx. Depth below Ground Surface (ft)	Approx. Elevation (ft)	
A1 Diner - Footing 1 (southwest corner)	cast-in-place concrete pier/footing	4.9	El. 8.9	–
A1 Diner – Footing 2	footing consisting of unmortared, stacked granite blocks	2.2	El. 11.6	blocks loosely stacked, occasionally chinked with smaller stones
A1 Diner – Footing 3	footing consisting of unmortared granite blocks with concrete pier above	ND* (bottom of concrete pier at 5.2 ft)	ND* (bottom of concrete pier El. 8.4)	unmortared granite blocks extend below bottom of pier concrete
A1 Diner – Footing 4 (northwest corner)	cast-in-place concrete pier/footing	6.1	El. 6.3	petroleum-impacted soil encountered at 4.4 ft BGS
Existing Bridge – Pier 1	footing consisting of a single large granite block with concrete pier above	ND* (bottom of concrete pier at 4.3 ft)	ND* (bottom of concrete pier El. 6.2)	granite block extends below bottom of pier concrete; petroleum-impacted soil encountered at 4.6 ft BGS

Note: *The bearing level for Footing 3 of the A1 Diner and Pier 1 of the existing bridge extended below the bottom of the excavation. Determining the foundation bearing level for the footings at these locations was not possible due to the dense nature of the sand and gravel around the blocks (refusal to mini-excavator and shovel) and due to the depth of excavation (depth limit of mini-excavator and safe access of people in excavation).

Groundwater was not encountered within the test pits.

Test pit excavations were stopped at the bottom of existing foundations to avoid undermining. We believe the foundations are likely soil supported (not pile supported). At locations where we were able to excavate to the bottom of the existing footings, we used a steel rod to probe beneath the footings to define the footing bearing level.

Please note that based on the GPR results and observations of the sidewalls of the test pits, there are buried granite blocks located throughout the A1 Diner area. Although we identified the blocks directly beneath several of the A1 Diner supports, it is likely that there are additional granite blocks below/beyond the limits of our test pits excavations.

Detailed information regarding the type, condition and geometry of foundation elements exposed in the test pits as well as photographs are provided in Appendix A.

Impacts of Existing Foundation Conditions on Development

Based on the existing conditions encountered in the subsurface explorations referenced herein and our current understanding of the project, we have the following general geotechnical “observations” regarding future development at the site:

- A1 Diner Footing 2 and Footing 3 interfere with the proposed retaining wall alignment. As described below, there are several possible alternatives to resolve this conflict. Evaluations and selection of the preferred alternative will be made during the final design phase of the project.
 - Expose Footings 2 and 3 and cut off the portion of the footings that interfere with construction of the proposed retaining wall. The design team (Stantec and Haley & Aldrich) would need to verify that the reduced footing sizes would continue to provide adequate support for the A1 Diner. Note: Footing 2 is unmortared, which could cause constructability challenges (i.e., stability issues) while cutting off portions of the blocks.
 - Temporarily underpin portions of Footings 2 and 3 with a Contractor-designed system (i.e., micropiles) and remove the portions of footings that interfere with construction of the proposed retaining wall. Reconstruct new footings as needed prior to decommissioning the temporary underpinning system.
 - Temporarily support the entire A1 Diner with a Contractor-designed system (e.g., micropiles, cribbing and needle beams) and remove the portions of existing footings that interfere with construction of the proposed retaining wall. Reconstruct new footings as needed prior to decommissioning the temporary support system.
 - Permanently underpin the A1 Diner with a new foundation system designed by the design team and remove portions or all of Footings 2 and 3 that interfere with construction of the proposed retaining wall.
 - Modify the plan location and alignment of the proposed retaining wall to avoid conflicts with the A1 Diner Footings 2 and 3.
- To minimize excavation and potential impacts to the A1 Diner foundation, we recommend supporting the proposed retaining wall on micropiles bearing in glacial till or bedrock. Feasibility of shallow foundation support of the proposed retaining wall will also be evaluated during final design.
- We anticipate that some level of protection of the A1 Diner foundation, such as support of excavation and/or underpinning, will be required to construct the retaining wall. During final design, requirements to monitor and limit settlement of the A1 Diner will be established by the design team. Furthermore, threshold and limiting values for settlement of the A1 Diner should be developed by the design team and incorporated into the contract documents for the project. Selection and design of temporary excavation support and/or temporary underpinning systems used to protect the A1 Diner should be the responsibility of the Contractor and should be provided as a

submittal to be reviewed by the design team. Permanent underpinning of the A1 Diner, if selected as the preferred protection option, should be designed by Stantec/Haley & Aldrich.

Closure

We appreciate the opportunity to provide geotechnical engineering services for this phase of the project. Please do not hesitate to call us if you have any questions or comments.

Enclosures:

- Figure 1 – Project Locus
- Figure 2 – Site and Subsurface Exploration Location Plan
- Figure 3 – Approximate Limits of Foundations
- Appendix A – Test Pit Sketches and Photographs
- Appendix B – Geophysical Survey

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MAP SOURCE: ESRI

SITE COORDINATES: 44°13'50"N, 69°46'28"W



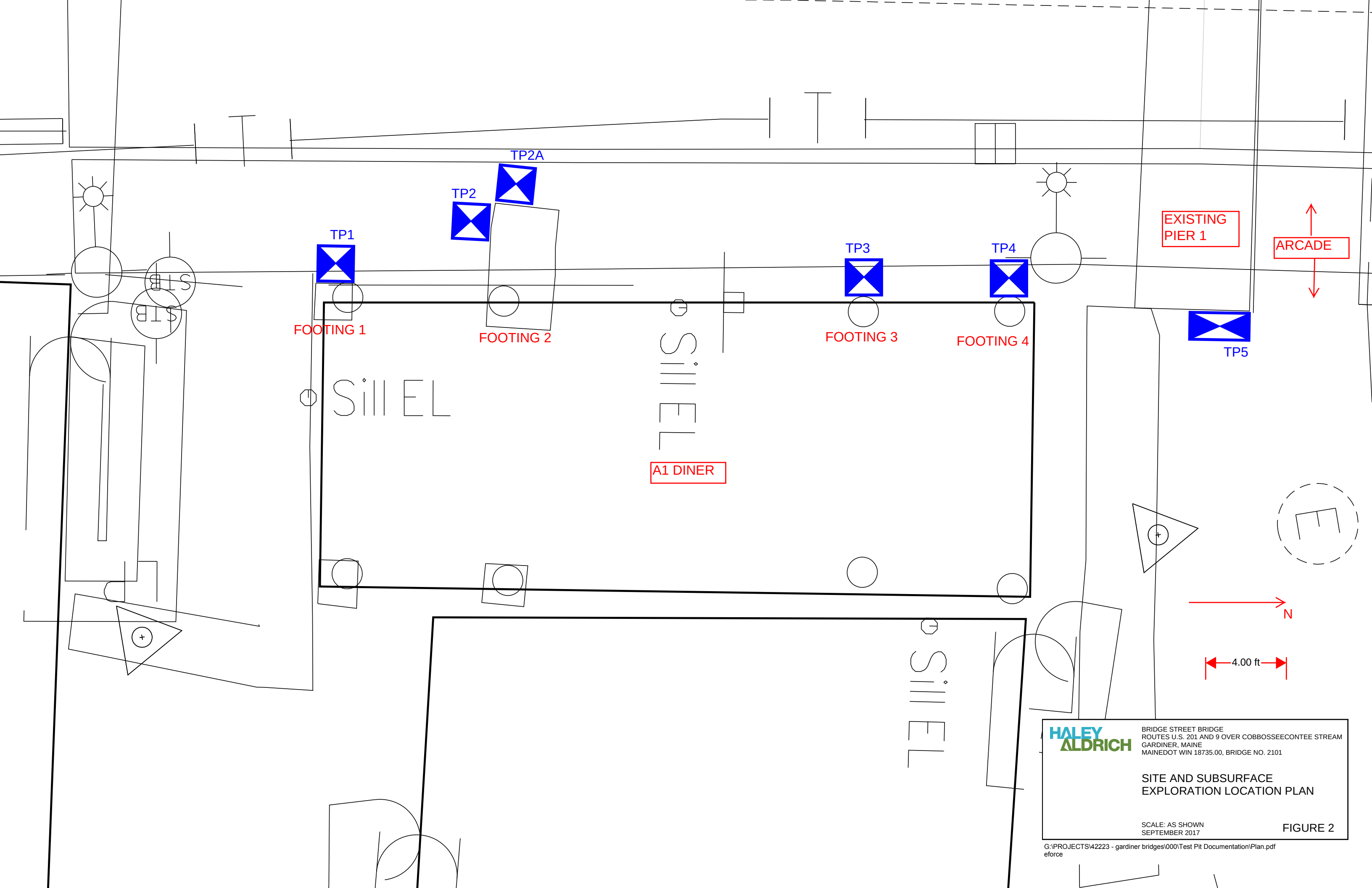
**HALEY
ALDRICH**

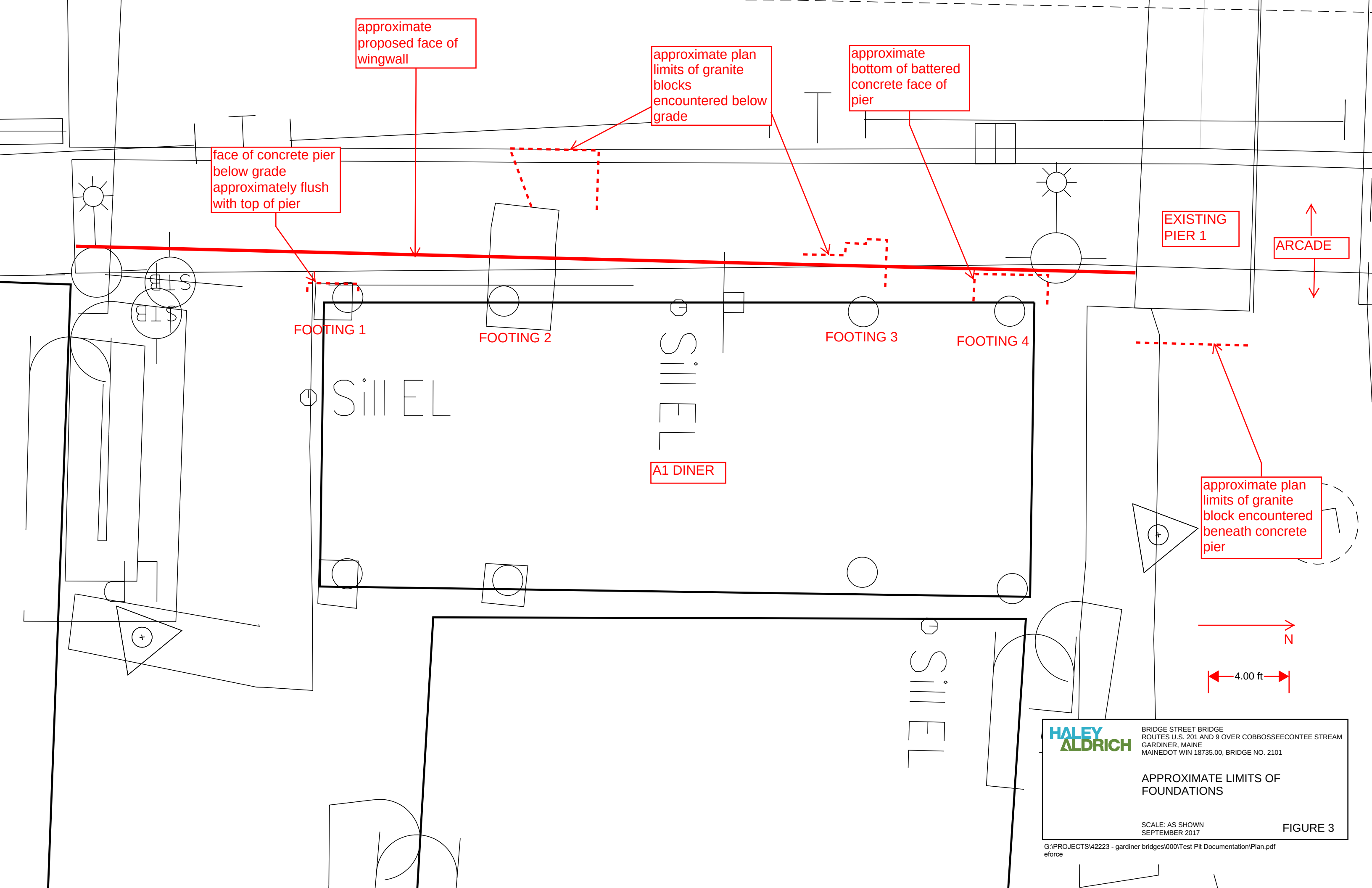
BRIDGE STREET BRIDGE
ROUTES U.S. 201 AND 9 OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18735.00, BRIDGE NO. 2101

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
SEPTEMBER 2017

FIGURE 1





approximate
proposed face of
wingwall

approximate plan
limits of granite
blocks
encountered below
grade

approximate
bottom of battered
concrete face of
pier

face of concrete pier
below grade
approximately flush
with top of pier

EXISTING
PIER 1

ARCADE

FOOTING 1

FOOTING 2

FOOTING 3

FOOTING 4

A1 DINER

approximate plan
limits of granite
block encountered
beneath concrete
pier

N

4.00 ft

HALEY
ALDRICH

BRIDGE STREET BRIDGE
ROUTES U.S. 201 AND 9 OVER COBBOSSECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18735.00, BRIDGE NO. 2101

APPROXIMATE LIMITS OF
FOUNDATIONS

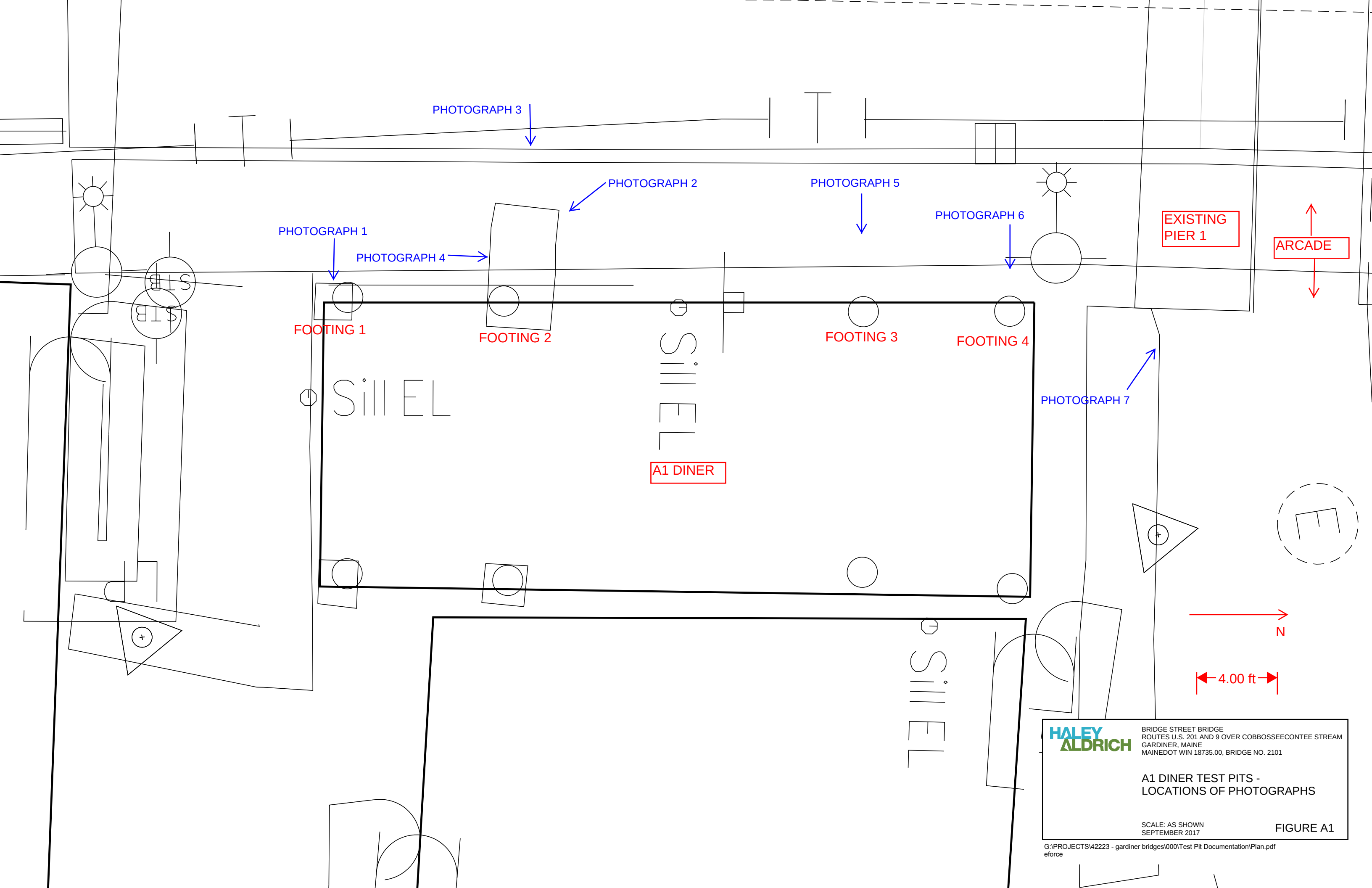
SCALE: AS SHOWN
SEPTEMBER 2017

FIGURE 3

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APPENDIX A

Test Pit Sketches and Photographs



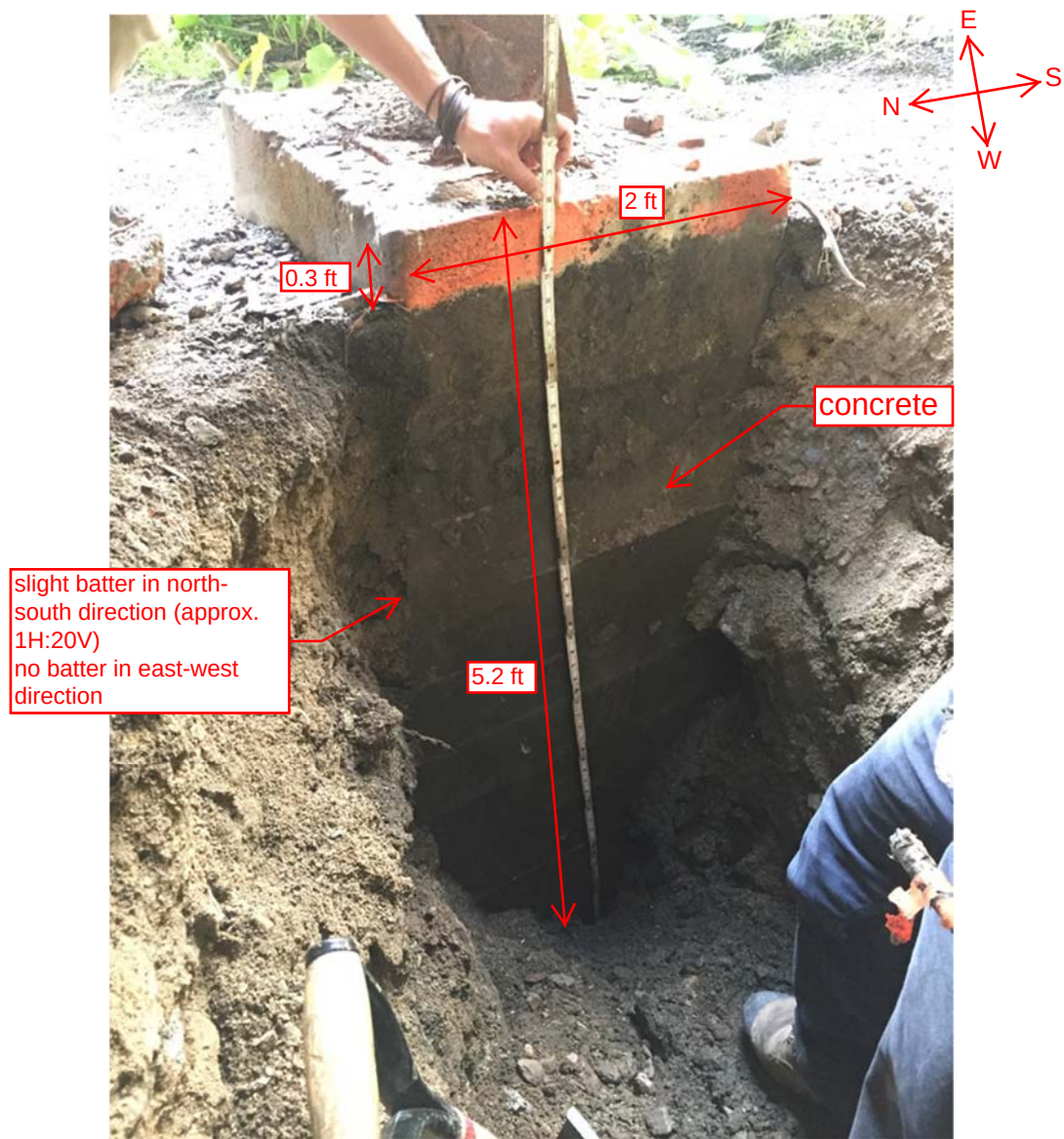


BRIDGE STREET BRIDGE
ROUTES U.S. 201 AND 9 OVER COBBOSSECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18735.00, BRIDGE NO. 2101

A1 DINER TEST PITS -
LOCATIONS OF PHOTOGRAPHS

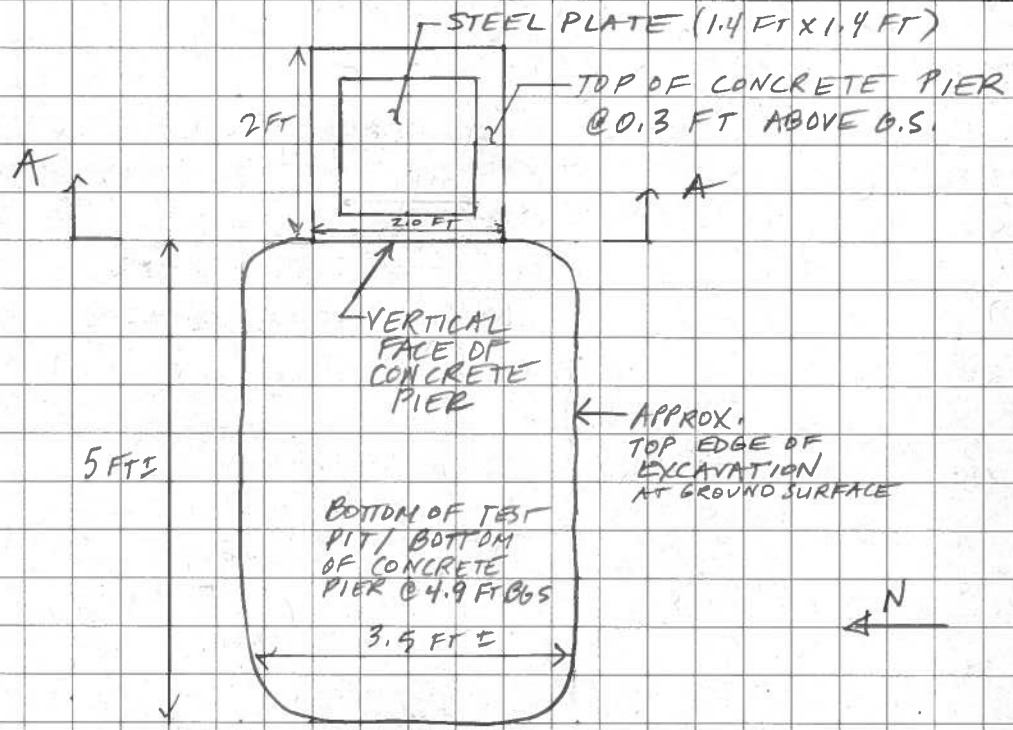
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SEPTEMBER 2017

FIGURE A1

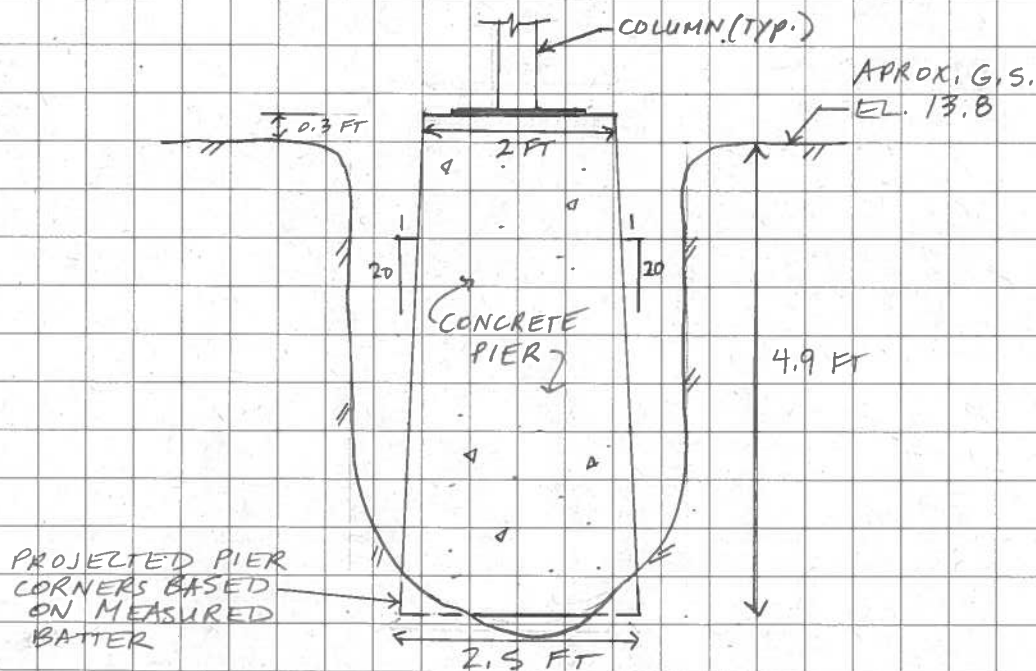


Photograph 1. TP1, A1 Diner - Footing 1 (southwest corner), facing east

Client STANTEL
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT TPI (A1 DINER, FOOTING 1) SKETCHES



PLAN VIEW
 SCALE: 1" = 2'



SECTION A-A VIEW
 SCALE: 1" = 2'



Photograph 2. A1 Diner – Footing 2, facing east, area of TP2 and TP2A



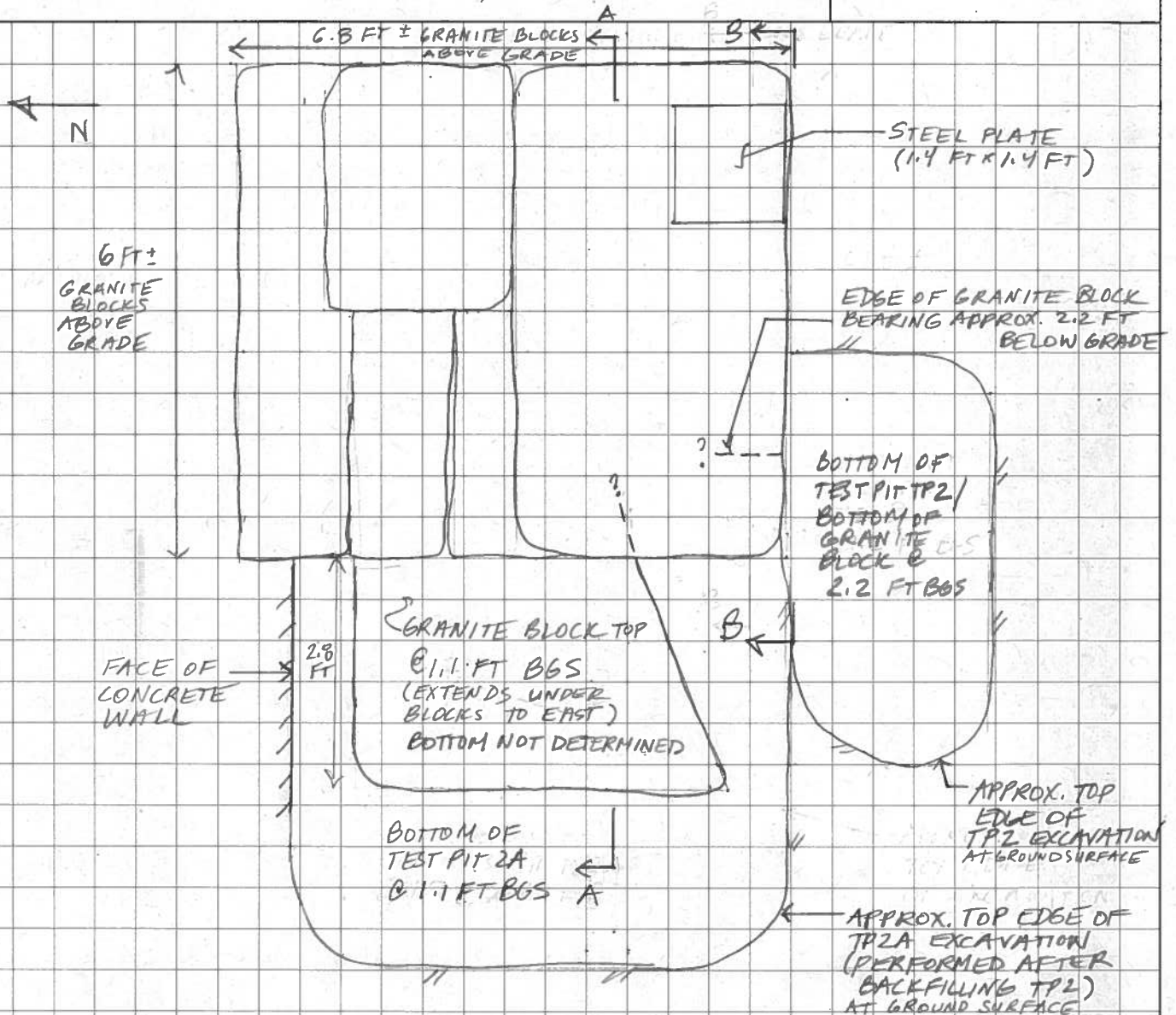
Photograph 3. TP2A, A1 Diner - Footing 2, facing east



Photograph 4. TP2, A1 Diner - Footing 2, facing north

File No. 42223-000
 Sheet 1 of 2
 Date 6 July 2017
 Computed By EAF
 Checked By WAC

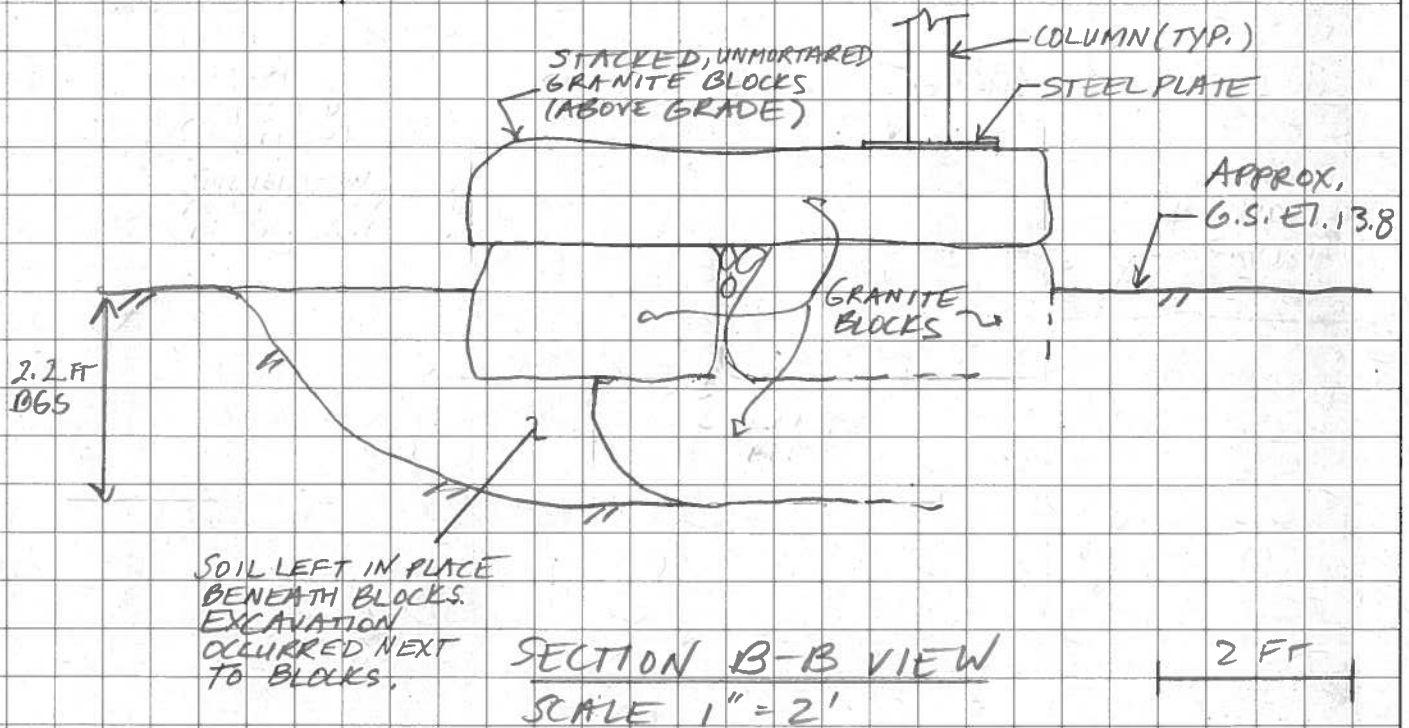
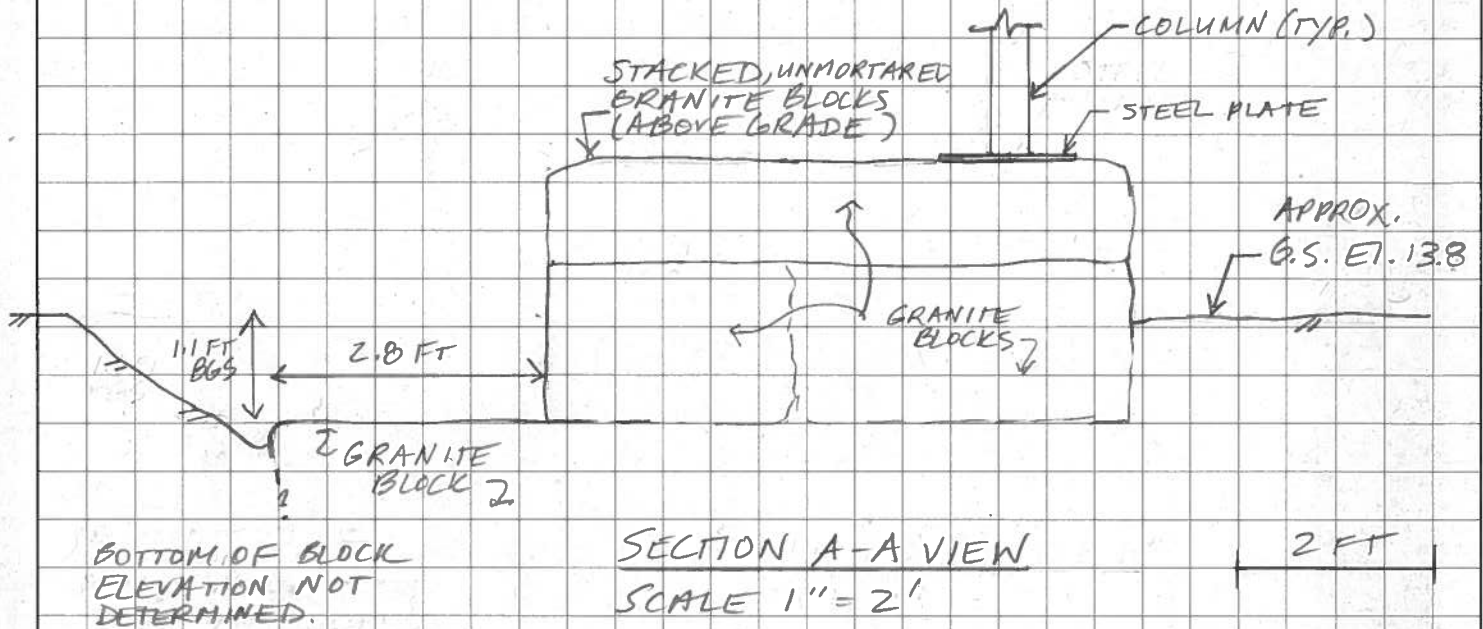
Client STANTEC
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT TP2A (A1 DINER, FOOTING 2) SKETCHES

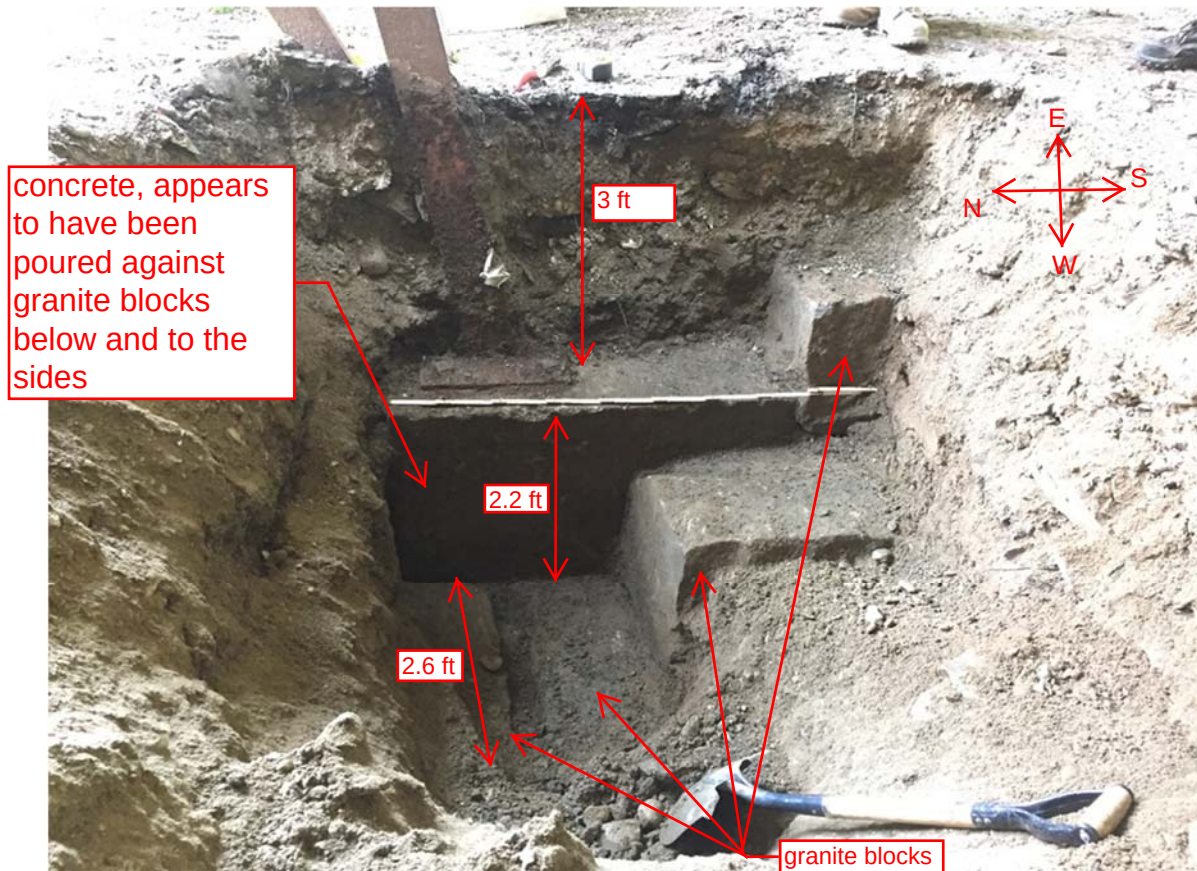


PLAN VIEW
 SCALE: 1" = 2'

2 FT

Client STANTEC
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT TP2 : TP2A (A1 DINER, FOOTING 2) SKETCHES

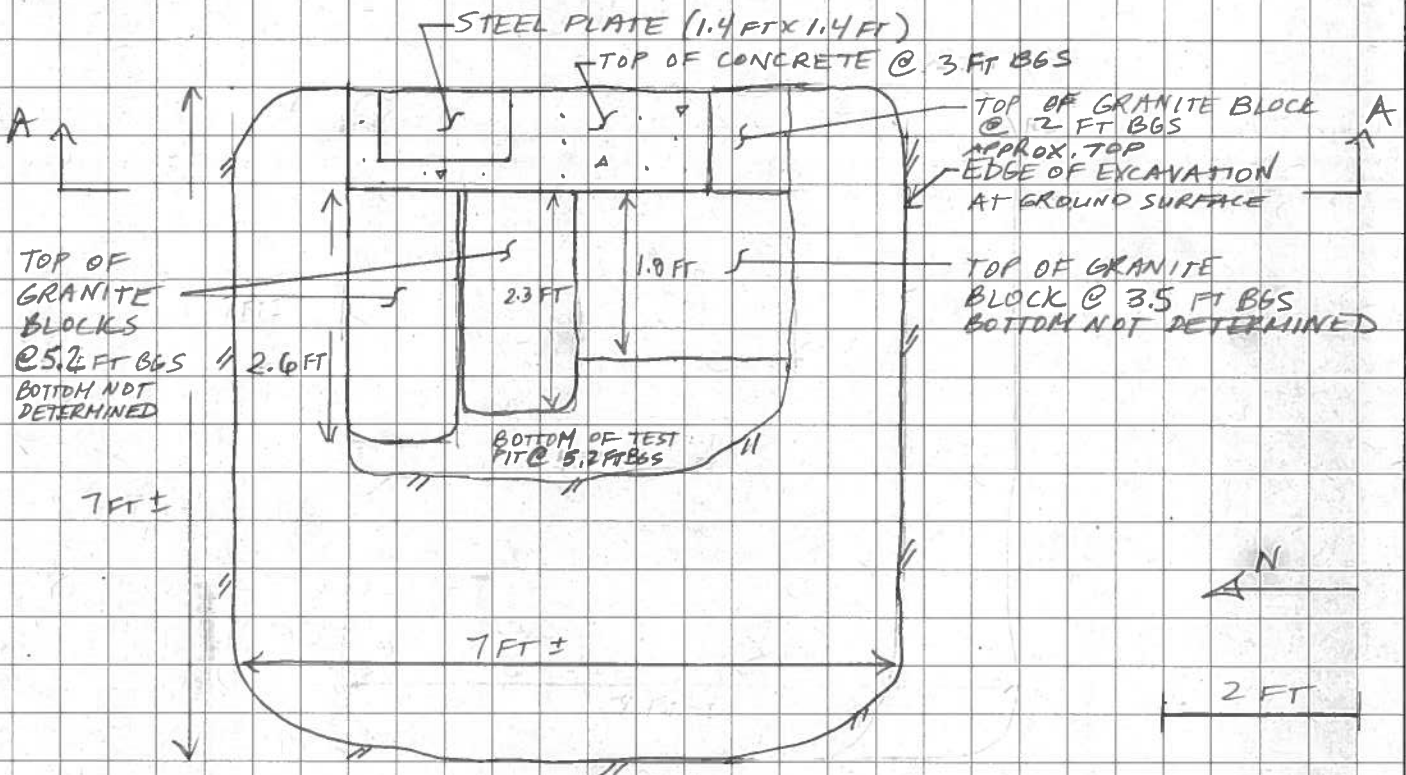




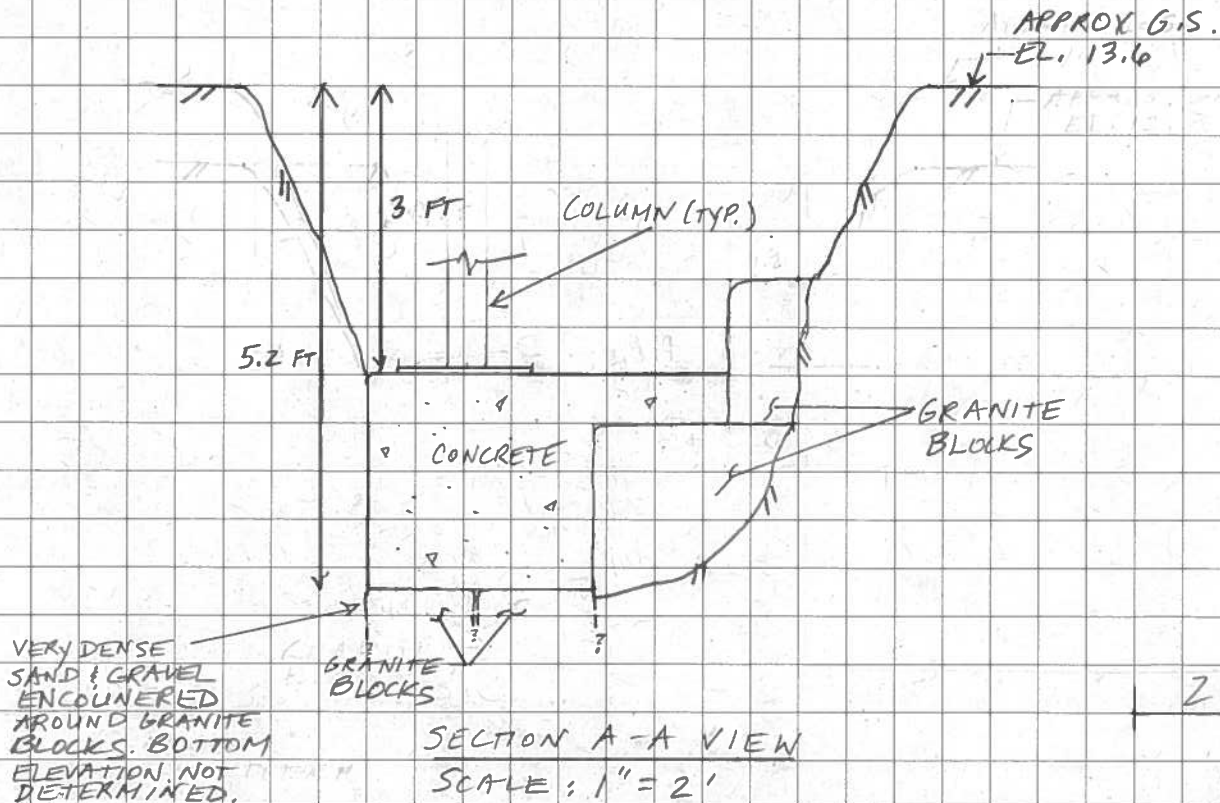
Photograph 5. TP3, A1 Diner – Footing 3, facing east

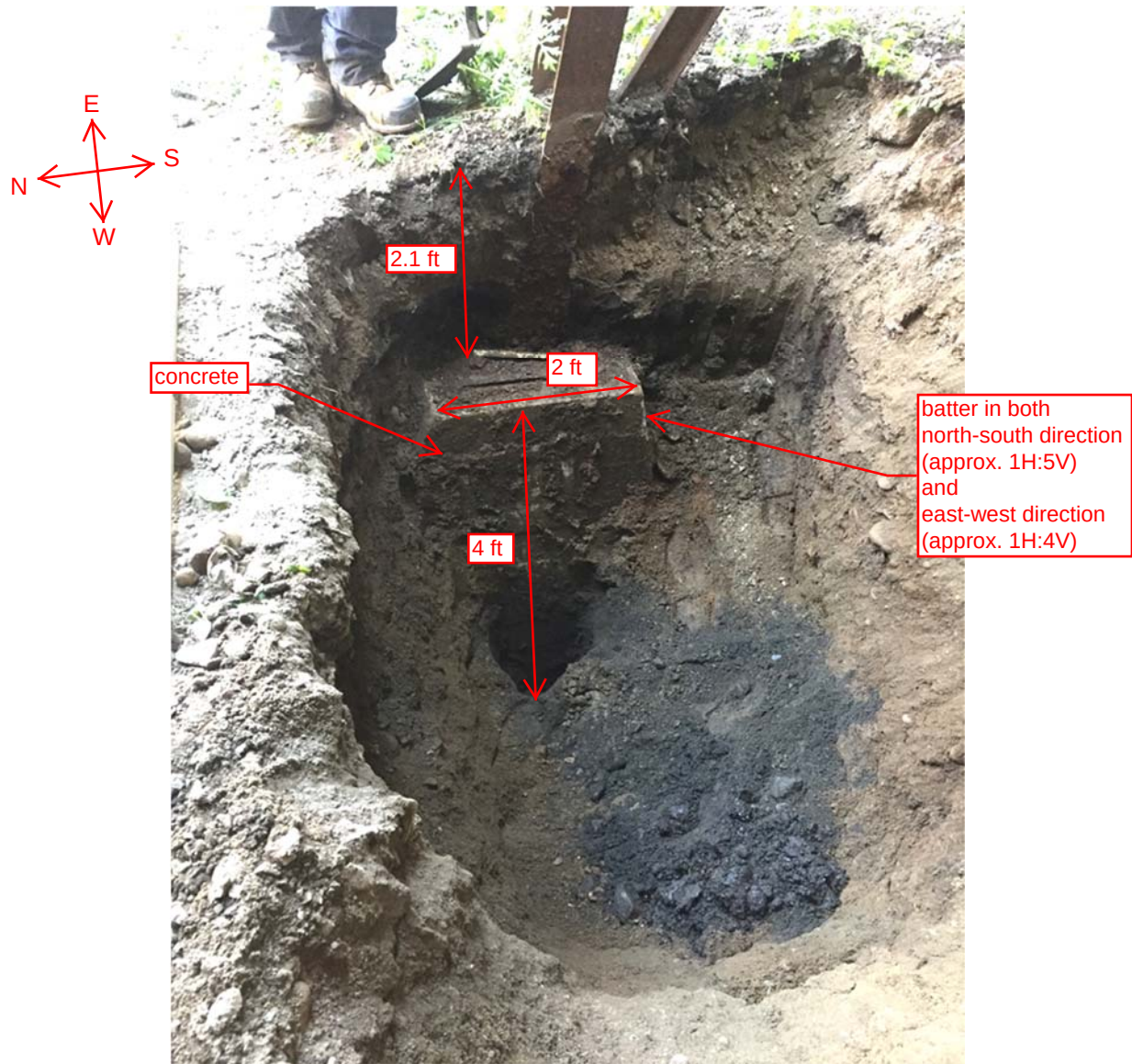
Client STANTEC
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT. TP3 (AI DINER, FOOTING 3) SKETCHES

File No. 42223-000
 Sheet 1 of 1
 Date 6 JULY 2017
 Computed By EQF
 Checked By WAC



PLAN VIEW
 SCALE: 1" = 2'

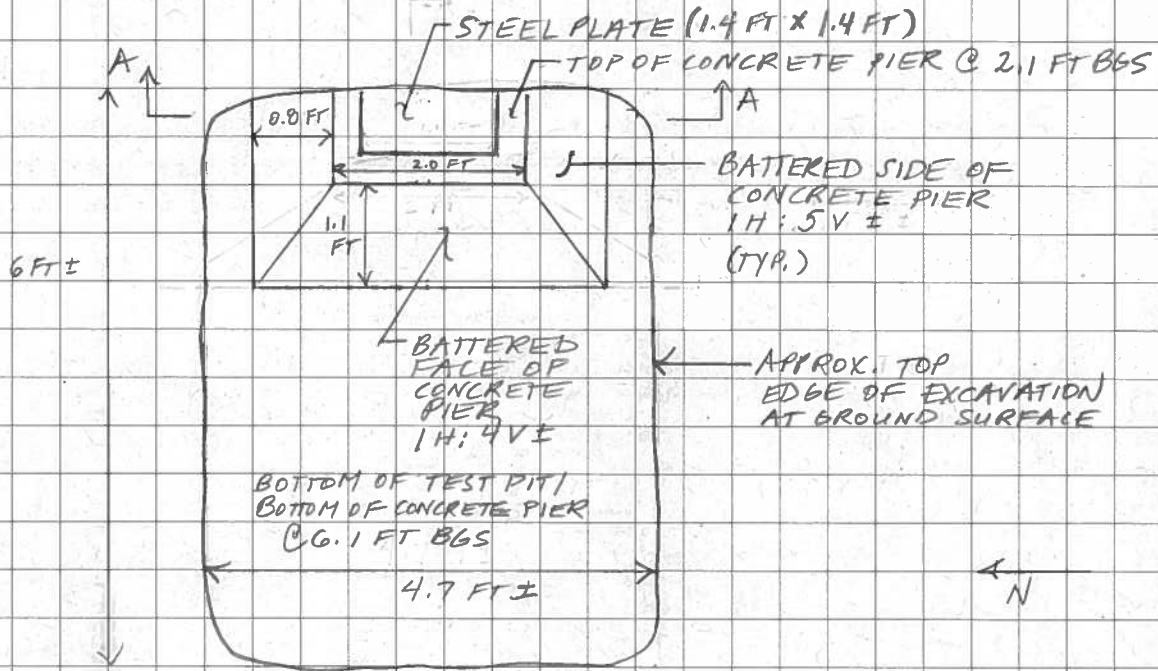




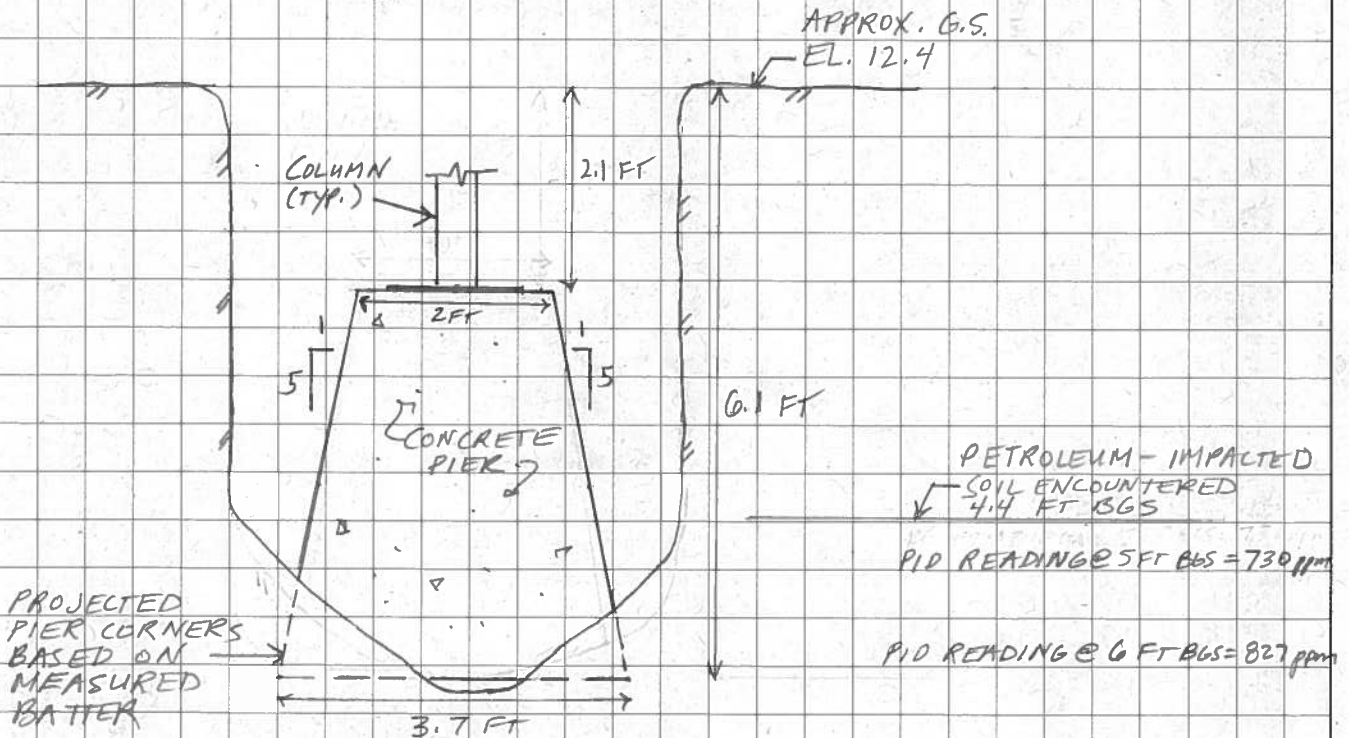
Photograph 6. TP4, A1 Diner - Footing 4 (northwest corner), facing east

Client STANTEC
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT TP4 (A1 DINER, FOOTING 4) SKETCHES

File No. 42223-000
 Sheet 1 of 1
 Date 6 JULY 2017
 Computed By EAF
 Checked By WAC



PLAN VIEW
 SCALE: 1" = 2'

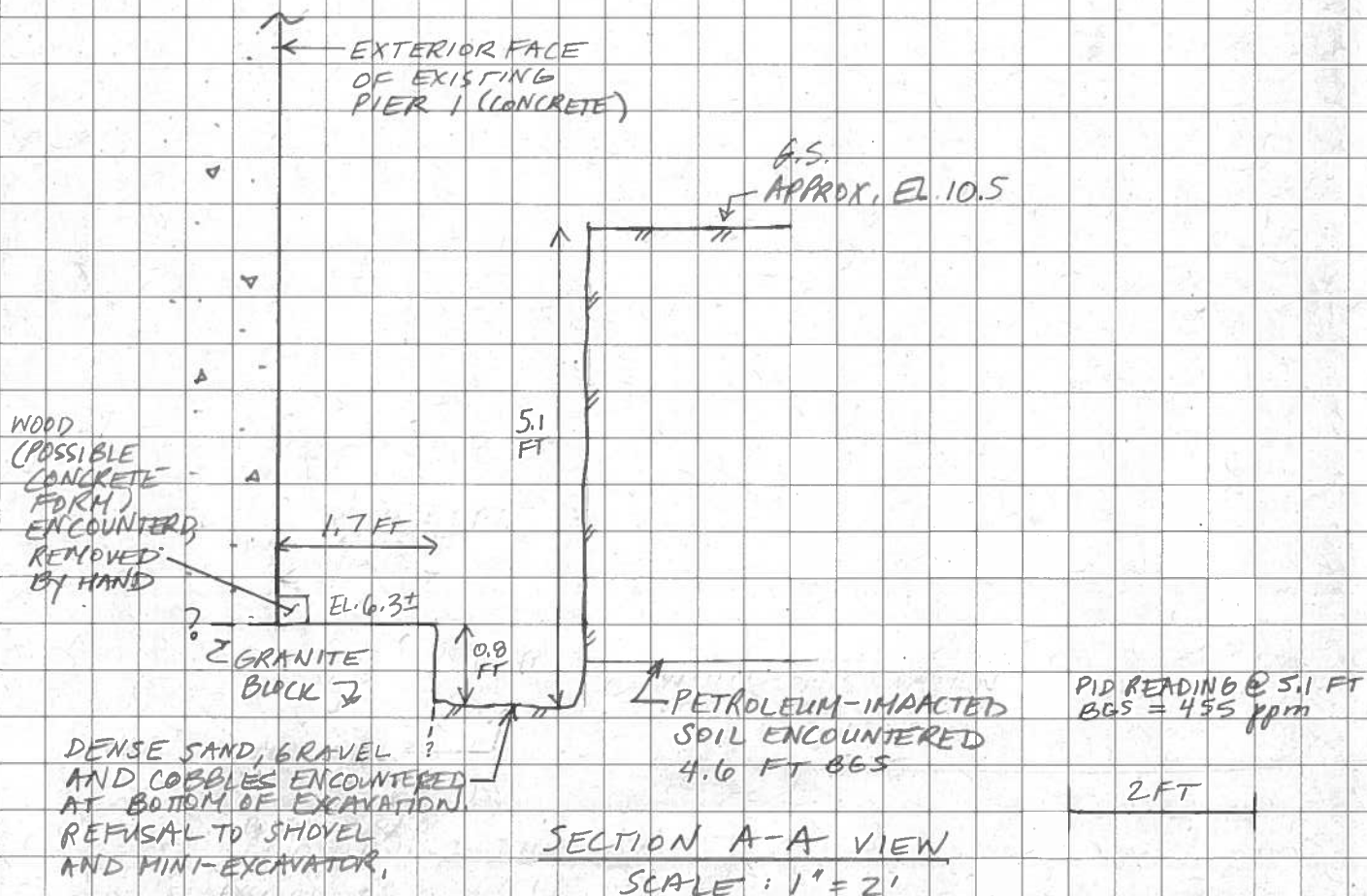
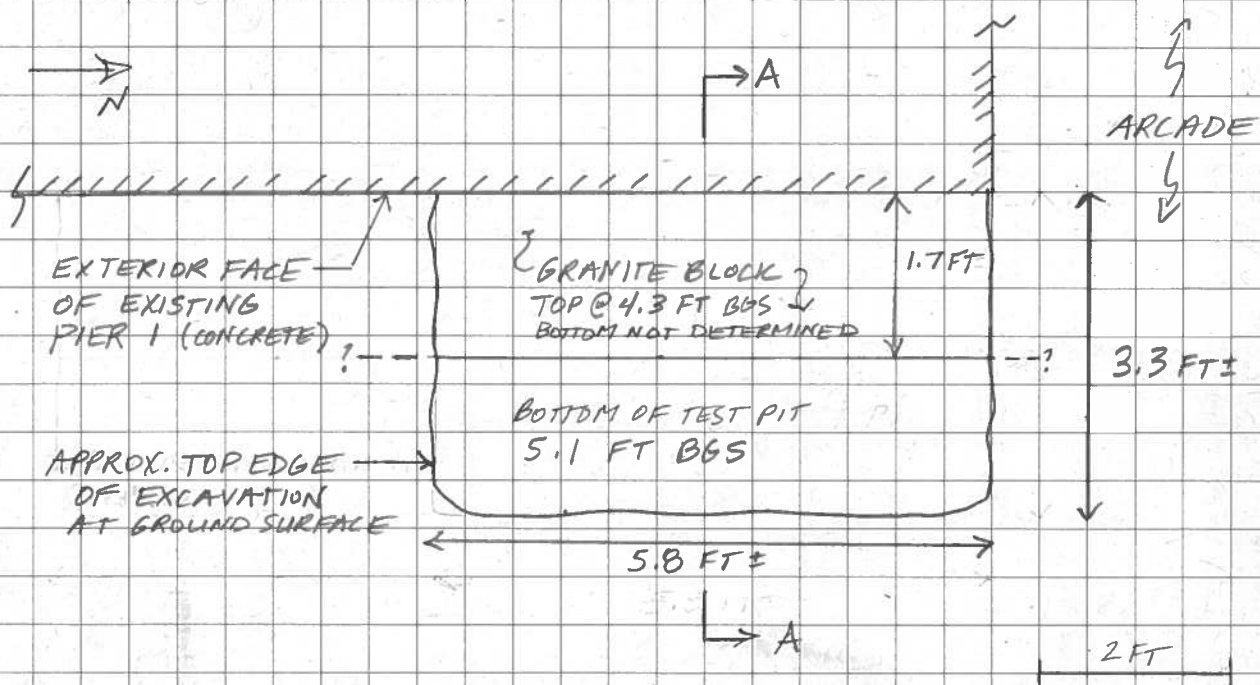


SECTION A-A VIEW
 SCALE: 1" = 2'



Photograph 7. TP5, Existing Bridge Pier 1, facing west

Client STANTEC
 Project BRIDGE STREET BRIDGE, GARDINER, MAINE
 Subject TEST PIT TPS (PIER 1) SKETCHES



APPENDIX B

Geophysical Survey

**GEOPHYSICAL SURVEY
BRIDGE STREET BRIDGE
GARDINER, MAINE**

**MaineDOT MultiPIN Contract
No. 20150715000000000068
WIN 18735.00**

Prepared for:

Maine Department of Transportation
Bridge Program
State House Station 16
Augusta, Maine 04333-0016

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 17J39
May, 2017

HAGER-RICHTER GEOSCIENCE, INC.

CONSULTANTS IN GEOLOGY AND GEOPHYSICS

8 INDUSTRIAL WAY - D10
SALEM, NEW HAMPSHIRE 03079-5820

TELEPHONE (603) 893-9944

FAX (603) 893-8313

May 16, 2017

File 17J39

Laura Krusinski, P.E.
Senior Geotechnical Engineer
Maine Department of Transportation
Bridge Program
State House Station 16
Augusta, Maine 04333-0016

Dir: 207-624-3441

Tel: 207-215-7641

Email: Laura.Krusinski@maine.gov

RE: **MaineDOT MultiPIN Contract
No. 20150715000000000068**

Geophysical Survey
Bridge Street Bridge
Gardiner, Maine
WIN 18735.00

Dear Ms. Krusinski:

In this letter, we report the results of a geophysical survey conducted by Hager-Richter Geoscience, Inc. (H-R) in support of a geotechnical investigation for the above referenced project located on Bridge Street in Gardiner, Maine in April 2017. The geophysical survey was performed under Maine Department of Transportation (MaineDOT) MultiPIN Contract No. 20150715000000000068.

INTRODUCTION

The site is the Bridge Street Bridge over the Cobbosseecontee Stream in Gardiner, Maine. The general location of the Site is shown on Figure 1. According to information provided by MaineDOT, the bridge was constructed in 1917 and is planned for replacement. As part of that investigation, MaineDOT requested a geophysical survey to detect existing foundations supporting the A1 Diner (a small, elevated building) and to characterize subsurface features (especially buried historical buildings or waterfront structures) in the areas of the proposed temporary abutment locations where the new bridge superstructure will be constructed prior to being moved into its permanent place.

The areas of interest are located under and adjacent to the existing bridge in the vicinity of the existing north and south abutments and at the foundation for the A1 Diner located near the south abutment. The areas total about 8,000 square feet in size and are generally open and accessible. Figure 2 is a site plan showing the locations of the geophysical survey.

OBJECTIVE

The objective of the geophysical survey was to investigate existing foundations supporting the A1 Diner as well and to characterize subsurface features (especially possible buried historical buildings or waterfront structures) in the areas of the proposed temporary abutment locations for the Bridge Street Bridge replacement project.

THE SURVEY

The geophysical survey was conducted using the ground penetrating radar (GPR) method. Michael Howley, P.G., and Bryan Carnahan of Hager-Richter conducted the geophysical survey on April 18, 2017. The fieldwork was coordinated with Ms. Laura Krusinski, P.E., of MaineDOT. Ms. Krusinski, Ms. Erin Force of Haley & Aldrich, and Ms. Lauren Flanders of Stantec were on site for a portion of the survey and specified the areas of interest. Data analysis and interpretation were completed at the Hager-Richter offices. Original data and field notes will be retained in the Hager-Richter files for a minimum of three (3) years.

GPR data were acquired in the vicinity of both the north and the south existing abutments. GPR data were acquired between the Cobbosseecontee Stream and the north abutment, mostly under the bridge deck, along mutually perpendicular traverses spaced 5 feet apart. The area investigated near the north abutment was limited by high water and vegetation.

GPR data were acquired between the stream and the south abutment along mutually perpendicular traverses spaced 5 feet apart. The main area investigated near the south abutment extended both west and east from under the bridge deck and included a portion directly west of the abutment. GPR data were also acquired under the A1 Diner. Access to the area directly under the A1 Diner was limited by the support structures.

Geophysical survey grids, one for each abutment, were tied to local site features, such as intersections between the bridge deck and an abutment, and located on CAD drawings provided by MaineDOT. Figure 2 is a modified site plan showing the locations of the GPR traverses, the existing abutments, the existing bridge deck, and other site features.

EQUIPMENT

GPR. The GPR survey was conducted using a Geophysical Survey Systems, Inc. (GSSI) UtilityScan DF digital dual-frequency subsurface imaging radar system. The UtilityScan DF acquires data simultaneously from 800 MHz and 300 MHz antennas. Data were acquired by rolling the GPR antenna along straight lines on the surface of the area(s) to be investigated. The system includes a survey wheel that triggers the recording of data at fixed intervals, thereby

ensuring the accuracy of the locations of features detected along the survey lines, and the real time images of the data are immediately available for field use. For this survey, the depth ranges were approximately 6 feet and 12 feet for the 800 MHz and 300 MHz antennas, respectively, and a dielectric constant of 14 was used.

GPR uses a high-frequency electromagnetic pulse (referred to herein as “radar signal”) transmitted from a radar antenna to probe the subsurface. The transmitted radar signals are reflected from subsurface interfaces of materials with contrasting electrical properties. Travel times of the radar signal can be converted to approximate depth below the surface by correlation with targets of known depths and by a curve matching routine. We monitor the acquisition of GPR data in the field and record the GPR data digitally for subsequent processing. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

Data from the GPR survey were processed using RADAN 7.4 GPR processing software from Geophysical Survey Systems, Inc. We reviewed profile images and created plan view time slice maps of the GPR data. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

LIMITATIONS OF THE METHOD

HAGER-RICHTER GEOSCIENCE, INC. MAKES NO GUARANTEE THAT ALL TARGETS OF INTEREST WERE DETECTED IN THIS SURVEY. HAGER-RICHTER GEOSCIENCE, INC. IS NOT RESPONSIBLE FOR DETECTING TARGETS THAT CANNOT BE DETECTED BY THE METHOD EMPLOYED OR BECAUSE OF SITE CONDITIONS. GPR SIGNAL PENETRATION MIGHT NOT BE SUFFICIENT TO DETECT ALL TARGETS OF INTEREST.

GPR. There are limitations of the GPR technique: (1) surface conditions, (2) electrical conductivity and thickness of the subsurface layers, (3) electrical properties of the target(s), and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us in most cases.

The condition of the survey surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. For exterior sites, a surface covered with obstacles such as automobiles, dumpsters, thick leaf debris, materials piles, etc. limit the survey access. Similarly, for interior sites, a surface covered with obstacles such as desks, benches, laboratory equipment, etc. also limit access. Some floor coverings may limit the coupling of the GPR antenna with the subsurface.

The electrical conductivity of the subsurface determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. The GPR signal does not penetrate clay-rich soils or soils contaminated with road salt. In some cases, the GPR signal may not penetrate below concrete pavement, and some asphalts are electrically conducting.

A strong contrast in the electrical conductivities of the ground and the target (for examples, UST, pipe, void, dry well, drum, contaminant plume) is required to obtain a reflection of the GPR signal. If the contrast is too small, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted on the basis of the size of the target.

RESULTS

The geophysical survey consisted of GPR to detect existing foundations supporting the A1 Diner (a small, elevated building) as well as to characterize subsurface features (especially any buried historical buildings or waterfront structures) in the areas of the proposed temporary abutment locations where the new bridge superstructure will be constructed prior to being moved into its permanent place. The locations of the GPR traverses and the results of the GPR survey are shown in Figure 3.

Apparent GPR signal penetration was generally good for all areas surveyed, with the exception of the north portions of the South Abutment Area, where GPR signal penetration was limited. For most areas surveyed, two-way traveltime reflections were received for 22 to 50 ns for the GPR records recorded with the 300 MHz antenna. Based on site specific time-to-depth conversions for the GPR signal, the GPR signal penetration is estimated to have been approximately 3 to 6.5 feet. In areas with limited GPR signal penetration, two-way traveltime reflections were received from no more than approximately 15 ns, or about 2 feet. GPR signal penetration for the 800 MHz antenna was somewhat less than for the 300 MHz antenna.

A1 Diner Area. The GPR records for the area located under and west of the A1 Diner exhibit reflections consistent with shallow buried objects ranging in size from about 2 ft x 2 ft to about 5 ft x 15 ft, and their locations are shown on Figure 3. The tops of the objects are approximately 1 to 2 feet deep. Three of the buried objects located in the northeast portion of the area are located near some of the west supports of the A1 Diner, and it is possible that they are related to foundation elements of the supports, although the identity of the objects cannot be determined on the basis of the GPR data alone. Access to the area directly under the A1 Diner was limited by the support structures.

South Abutment Area. The GPR records for the South Abutment Area contain reflections consistent with subsurface utilities, and their locations are shown in Figure 3. The tops of the utilities are approximately 1 to 2 feet deep. The identity of the subsurface utilities cannot be determined on the basis of the GPR data.

North Abutment Area. The GPR records for the North Abutment Area contain reflections consistent with two possible buried structures that are approximately 30 to 40 feet long, approximately 3 feet wide, and no more than about 2 feet deep. The location of the possible buried structures are shown on Figure 3. In addition, a possible buried object, a possible buried metal plate, and a possible utility were detected at depths of 1 to 2 feet in the North Abutment Area, and their locations are shown in Figure 3.

CONCLUSIONS

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc., at the Bridge Street Bridge over the Cobbosseecontee Stream in Gardiner, Maine in April, 2017, we conclude:

- Possible buried shallow objects are present in the A1 Diner Area. It is possible that several may be related to possible foundations for the A1 Diner, but their identity cannot be determined on the basis of the GPR data alone.
- Possible buried utilities are present in the South Abutment Area.
- Possible buried structures, objects, utilities, and a metal plate are present in the North Abutment Area.

LIMITATIONS

This letter report was prepared for the exclusive use of the Maine Department of Transportation (Client). No other party shall be entitled to rely on this Report or any information, documents, records, data, interpretations, advice or opinions given to Client by Hager-Richter Geoscience, Inc. (H-R) in the performance of its work. The Report relates solely to the specific project for which H-R has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of H-R. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to H-R.

H-R has used reasonable care, skill, competence and judgment in the performance of its services for this project consistent with professional standards for those providing similar

services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by H-R should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

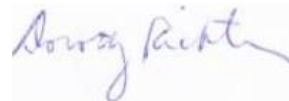
Except as expressly provided in this limitations section, H-R makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed.

If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with you on this project. We look forward to working with you again in the future.

Sincerely yours,
HAGER-RICHTER GEOSCIENCE, INC.



Michael Howley, P.G.
Geophysicist



Dorothy Richter, P.G.
President

Attachments: Figures 1-3



APPROXIMATE SCALE (feet)



LOCATION

NOTE:

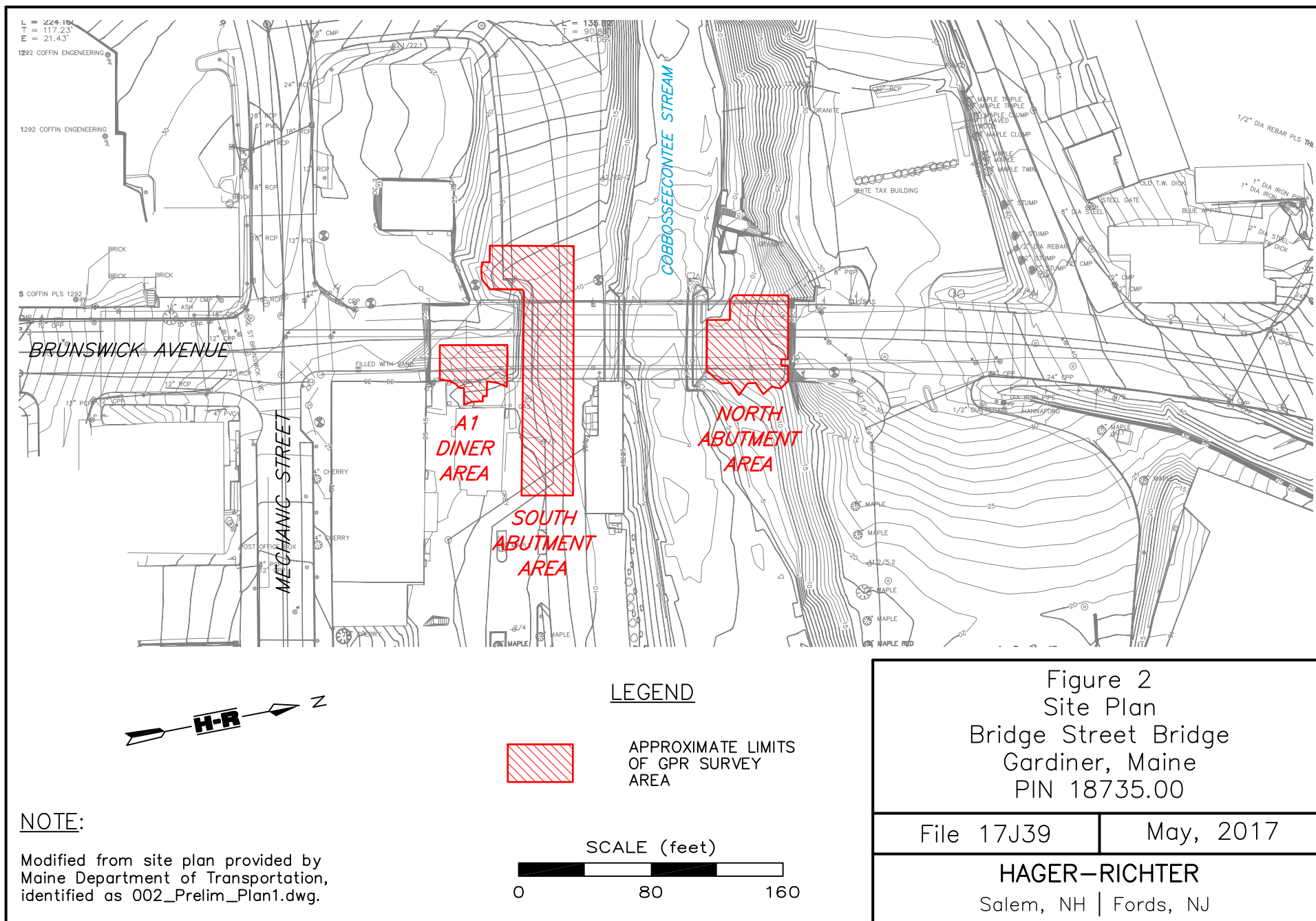
Modified from Google Earth Pro aerial photograph.

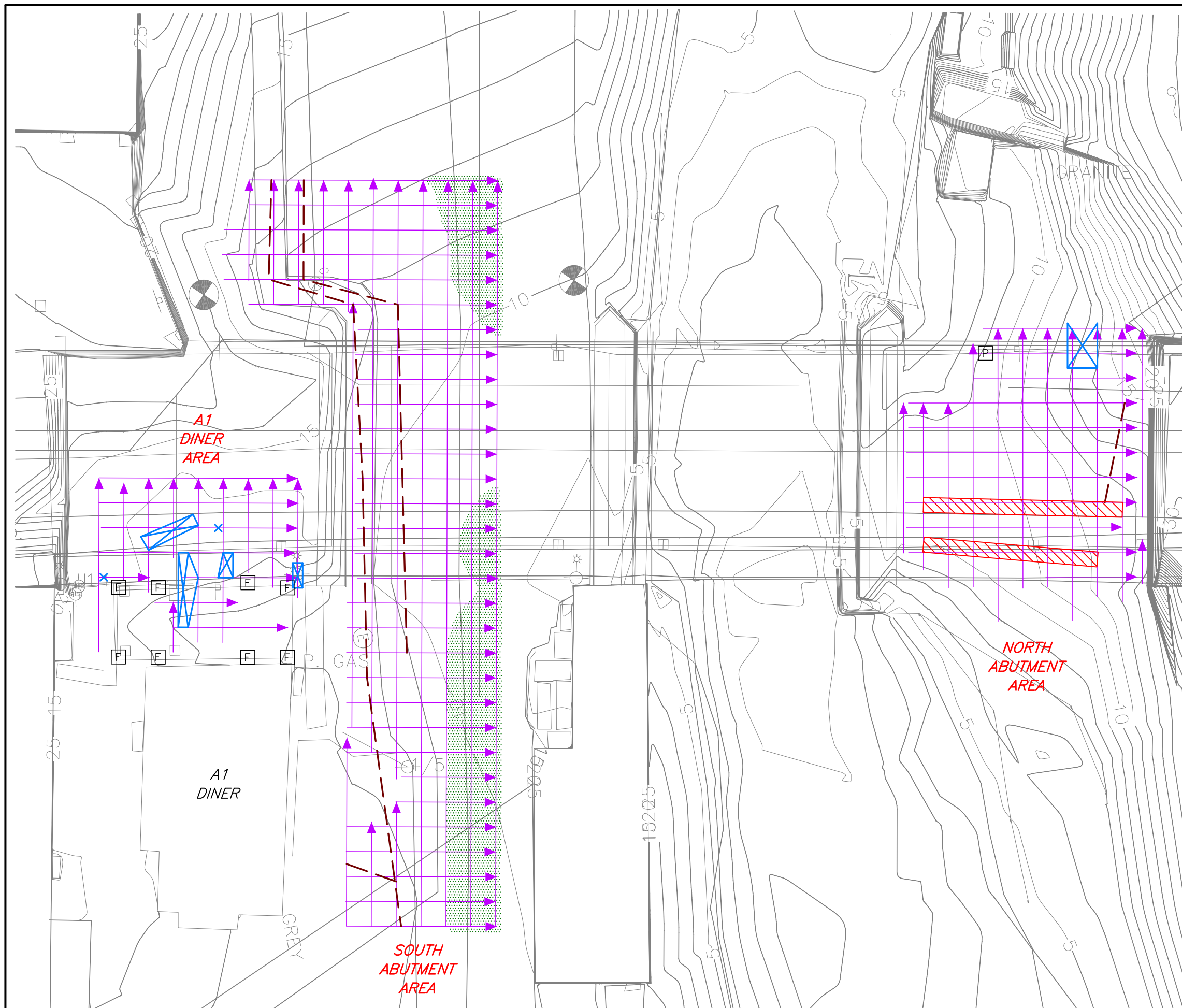
Figure 1
General Site Location
Bridge Street Bridge
Gardiner, Maine
PIN 18735.00

File 17J39

May, 2017

HAGER-RICHTER
Salem, NH | Fords, NJ





LEGEND

-  GPR TRAVERSE
-  POSSIBLE BURIED STRUCTURE
-  POSSIBLE UTILITY
-  UNIDENTIFIED BURIED OBJECT
-  POSSIBLE BURIED METAL PLATE
-  AREA OF LIMITED GPR SIGNAL PENETRATION (SEE TEXT)
-  FOUNDATION

SCALE (feet)



NOTE:

Modified from site plan provided by
Maine Department of Transportation,
identified as 002_Prelim_Plan1.dwg.

Figure 3
GPR Survey
Bridge Street Bridge
Gardiner, Maine
PIN 18735.00

File 17J39

May, 2017

HAGER-RICHTER

Salem, NH | Fords, NJ

APPENDIX F

Calculations

Table of Contents

- Frost Protection
- Approach Embankment Settlement
- Approach Embankment Global Stability
- Seismic Site Class
- Liquefaction Analysis
- Lateral Micropile Evaluation
- Axial Micropile Evaluation
- Micropile Elastic Compression
- Lateral Soil Pressures
- A1 Diner Footing Bearing Resistance
- Site Retaining Wall Footing Bearing Resistance
- Footing Sliding Resistance
- Stair Foundation Resistance

Frost Protection

Client:	MAINEDOT
Project:	BRIDGE STREET BRIDGE, GARDINER, MAINE
Subject:	FROST SUSCEPTIBILITY AND MAX. DEPTH OF FROST PENETRATION

OBJECTIVE:

Evaluate maximum depth of frost penetration based on soil and groundwater conditions, as well as geographic site location.

REFERENCES:

1. MaineDOT Bridge Design Guide, 2003 with interim revisions through 2017.
2. Preliminary Geotechnical Design Report prepared by Haley & Aldrich, Inc.
3. MaineDOT drawing set prepared by Stantec, dated August, 2017.

EVALUTATION:

1. Gather relevant information from test borings performed near proposed bridge abutment locations:

STRUCTURE	TEST BORING NO.	GROUND SURFACE	GROUND WATER	SAMPLE No. AND ELEVATION	AASHTO/ USCS	FINES CONTENT
ABUTMENT NO. 1	BB-GCS-101	El. 14.7	El. 8.7	1D El. 12.7-14.7	A-1-B / SW-SM	16.4
				3D El. 8.7-10.7	A-1-a / SW-SM	13.5
	BB-GCS-401	El. 10.5	El. 8.5	2D El. 5.2-6.5	A-1-a(1) / SW-SM	8.0
ABUTMENT NO. 2	BB-GCS-402	El. 13	El. 8.0	1D El. 11.0-13.0	A-2-4(0) / SM	27.8
				3D El. 7.0-9.0	A-1-b(0) / SM	18.2
	BB-GCS-403	El. 10.5	El. 5.5	3D El. 4.5-6.5	A-1-a(1) / GW-GM	6.0

Note: Ground water elevations summarized above were determined in the field and may have been influenced by the drilling process. Ground water elevations may vary throughout the year due to seasonal variations and precipitation events.

2. Water levels at the site are as follows:

Q1.1	El. 7.0
Q50	El. 11.1
Q100	El. 11.6

3. Proposed bottom of abutment elevations are as follows:

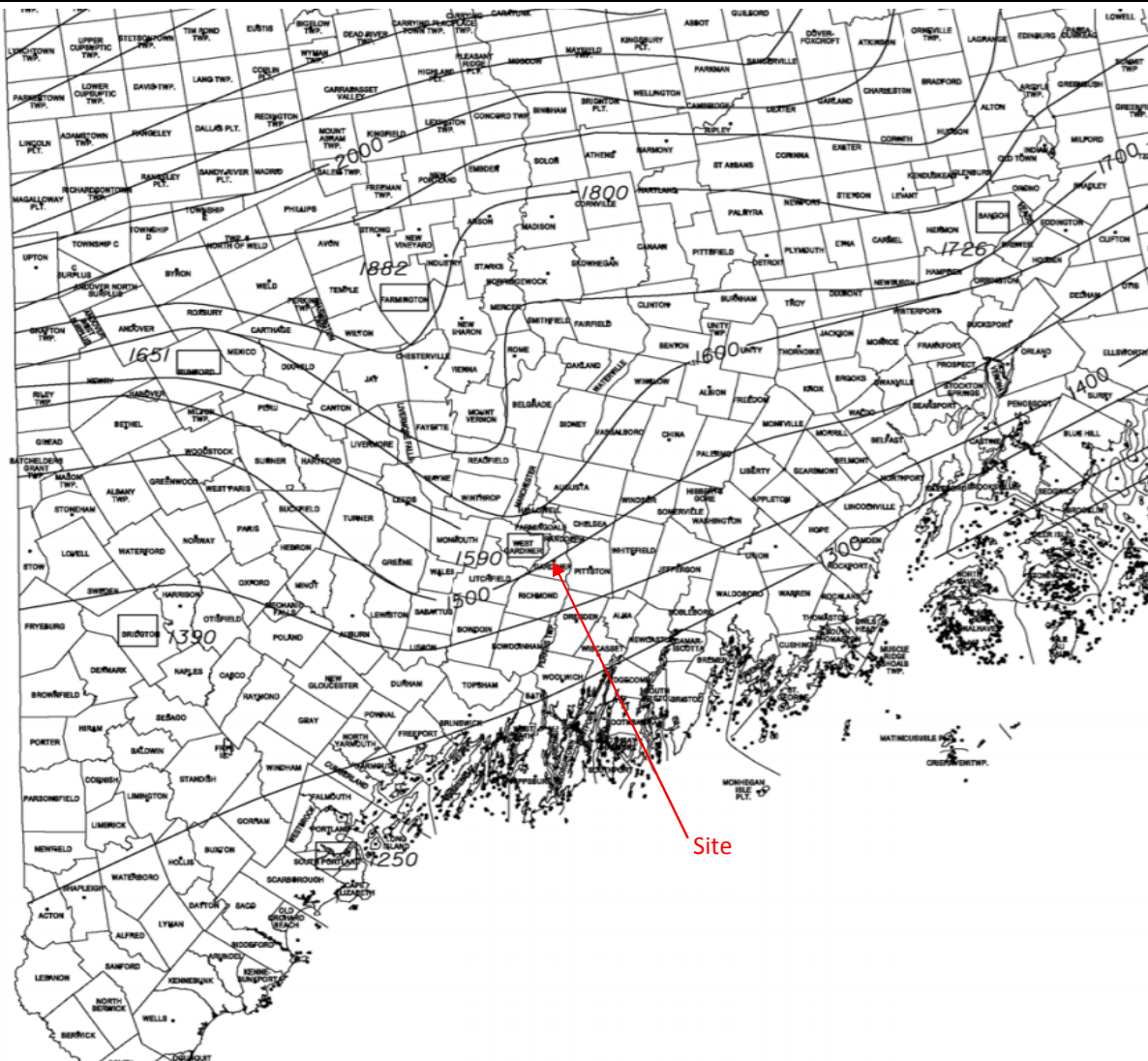
Abutment No. 1	El. 4.0
Abutment No. 2	El. 3.0

4. From MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1500 °F - days

5. Estimate range in frost penetration using MaineDOT Bridge Design Guide Table 5-1, the design freezing index above and an assumed range in water content since site-specific laboratory test data does not exist

From Table 5-1, frost penetration depths vary between approximately 5 ft (w=30%), 5.5 ft (w=20%) and 7 ft (w=10%) for coarse-grained soils.

Recommend 5.5 ft (w=20%)


Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Approach Embankment Settlement

Client	Maine Department of Transportation
Project	Bridge Street Bridge, Routes U.S. 201 & 9 over Cobbosseecontee Stream, Gardiner, Maine
Subject	Embankment Settlement

PROBLEM STATEMENT & OBJECTIVE

Estimate the elastic settlement for the proposed embankment for the bridge replacement.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

AVAILABLE INFORMATION

1. Draft plan set titled, "Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream" by Stantec, labeled 90% PS&E Submittal.
2. Boring logs BB-GCS-101, BB-GCS-101A, BB-GCS-401, and BB-GCS-401A for the south side
3. Boring logs BB-GCS-402, BB-GCS-402A, and BB-GCS-403 for the north side.

ASSUMPTIONS

1. Width of additional fill embankment at south side (Station 72+00) is about 60 ft.
2. Width of additional fill embankment at north side (Station 73+50) is about 60 ft.
3. Length of additional fill embankment at south side is about 64 ft.
4. Length of additional fill embankment at north side is about 9 ft.
5. Greatest fill height at south side (Station 72+00) is about 13 ft.
6. Greatest fill height at north side (Station 73+50) is about 16 ft.
7. New embankment fill unit weight is 125 pcf.

LOADING

1. South side addition pressure is $13 \text{ ft} * 125 \text{ pcf} = 1,625 \text{ psf}$.
2. North side additional pressure is $16 \text{ ft} * 125 \text{ pcf} = 2,000 \text{ psf}$.

ITERATIONS

1. South side (width = 60 ft, length = 64 ft, pressure = 1.625 ksf)
2. North side (width = 60 ft, length = 9 ft, pressure = 2.0 ksf)

CALCULATIONS

See pages 2 and 3.

CONCLUSIONS

Elastic settlements will range from 1 to 2 in. from the new fill placement.

Client Maine Department of Transportation

Date 4-Jun-18

Project Bridge Street Bridge, Routes U.S. 201 & 9 over Cobbosseecontee Stream, Gardiner, Maine

Computed by EAF

Subject Embankment Settlement

Checked by MMB

SOUTH EMBANKMENT
CALCULATION:

$$S_e = \frac{q_0(1 - \nu^2)\sqrt{A'}}{144 E_s \beta_z} \quad \text{Equation 10.6.2.4.2-1}$$

where

 q_0 = applied vertical stress (ksf)

 A' = effective area of footing (*** use full area

 E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of insitu or laboratory tests (ksi)

 β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

 ν = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of ν are not available from the results of insitu or laboratory tests (dim)

 S_e = elastic settlement (ft)

Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

FIXED SERVICE LIMIT STATE PARAMETERS

E_s (ksi)	4	ksi from Table C10.4.6.3-1
ν	0.3	Table C10.4.6.3-1
L (ft)	64	
β_z	1.06	Table 10.6.2.4.2-1
q_0	1.625	ksf

VARIABLE SERVICE LIMIT STATE PARAMETERS

B (ft)	A (ft ²)	S_e (in.)
60	3840	1.80

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Soil Type		
Clay: Soft sensitive Medium stiff to stiff Very stiff	0.347–2.08 2.08–6.94 6.94–13.89	0.4–0.5 (undrained)
Loess Silt	2.08–8.33 0.278–2.78	0.1–0.3 0.3–0.35
Fine Sand: Loose Medium dense Dense	1.11–1.67 1.67–2.78 2.78–4.17	0.25
Sand: Loose Medium dense Dense	1.39–4.17 4.17–6.94 6.94–11.11	0.20–0.36 0.30–0.40
Gravel: Loose Medium dense Dense	4.17–11.11 11.11–13.89 13.89–27.78	0.20–0.35 0.30–0.40
Estimating E_s from SPT N Value		
Soil Type	E_s (ksi)	
Silts, sandy silts, slightly cohesive mixtures	0.056 $N_{1,60}$	
Clean fine to medium sands and slightly silty sands	0.097 $N_{1,60}$	
Coarse sands and sands with little gravel	0.139 $N_{1,60}$	
Sandy gravel and gravels	0.167 $N_{1,60}$	
Estimating E_s from q_c (static cone resistance)		
Sandy soils	0.028 q_c	

Client Maine Department of Transportation

Date 4-Jun-18

Project Bridge Street Bridge, Routes U.S. 201 & 9 over Cobbosseecontee Stream, Gardiner, Maine

Computed by EAF

Subject Embankment Settlement

Checked by MMB

SOUTH EMBANKMENT
CALCULATION:

$$S_e = \frac{q_0(1 - \nu^2)\sqrt{A'}}{144 E_s \beta_z} \quad \text{Equation 10.6.2.4.2-1}$$

where

 q_0 = applied vertical stress (ksf)

 A' = effective area of footing (*** use full area

 E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of insitu or laboratory tests (ksi)

 β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

 ν = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of ν are not available from the results of insitu or laboratory tests (dim)

 S_e = elastic settlement (ft)

Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

FIXED SERVICE LIMIT STATE PARAMETERS

E_s (ksi)	4	ksi from Table C10.4.6.3-1
ν	0.3	Table C10.4.6.3-1
L (ft)	9	
β_z	1.28	Table 10.6.2.4.2-1
q_0	2	ksf

VARIABLE SERVICE LIMIT STATE PARAMETERS

B (ft)	A (ft ²)	S_e (in.)
60	540	0.69

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Soil Type		
Clay: Soft sensitive Medium stiff to stiff Very stiff	0.347–2.08 2.08–6.94 6.94–13.89	0.4–0.5 (undrained)
Loess Silt	2.08–8.33 0.278–2.78	0.1–0.3 0.3–0.35
Fine Sand: Loose Medium dense Dense	1.11–1.67 1.67–2.78 2.78–4.17	0.25
Sand: Loose Medium dense Dense	1.39–4.17 4.17–6.94 6.94–11.11	0.20–0.36 0.30–0.40
Gravel: Loose Medium dense Dense	4.17–11.11 11.11–13.89 13.89–27.78	0.20–0.35 0.30–0.40
Estimating E_s from SPT N Value		
Soil Type	E_s (ksi)	
Silts, sandy silts, slightly cohesive mixtures	0.056 $N_{1,60}$	
Clean fine to medium sands and slightly silty sands	0.097 $N_{1,60}$	
Coarse sands and sands with little gravel	0.139 $N_{1,60}$	
Sandy gravel and gravels	0.167 $N_{1,60}$	
Estimating E_s from q_c (static cone resistance)		
Sandy soils	0.028 q_c	

Approach Embankment Global Stability

Client: Maine Department of Transportation

Project: Bridge Street Bridge over Cobbosseecontee Stream, Gardiner, Maine

Subject: Global Stability for Abutment 1 and Abutment 2

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for Abutment 1 and Abutment 2.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. Slide version 7.0 by RocScience.
3. Maine DOT Bridge Design Guide, 2003, with 2014 updates.

AVAILABLE INFORMATION

1. Plan set titled, "State of Maine Department of Transportation, Gardiner Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream," dated August 2017.

ASSUMPTIONS

1. Water level will be modeled at Q1.1 at El. 7.0 (MHW).
2. Seismic cases will have a seismic force of $A_s/2$ ($0.124g/2$) = 0.062 g based on the seismic site class calculations .
3. A 250 psf traffic surcharge will be modeled.

SOIL PROPERTIES

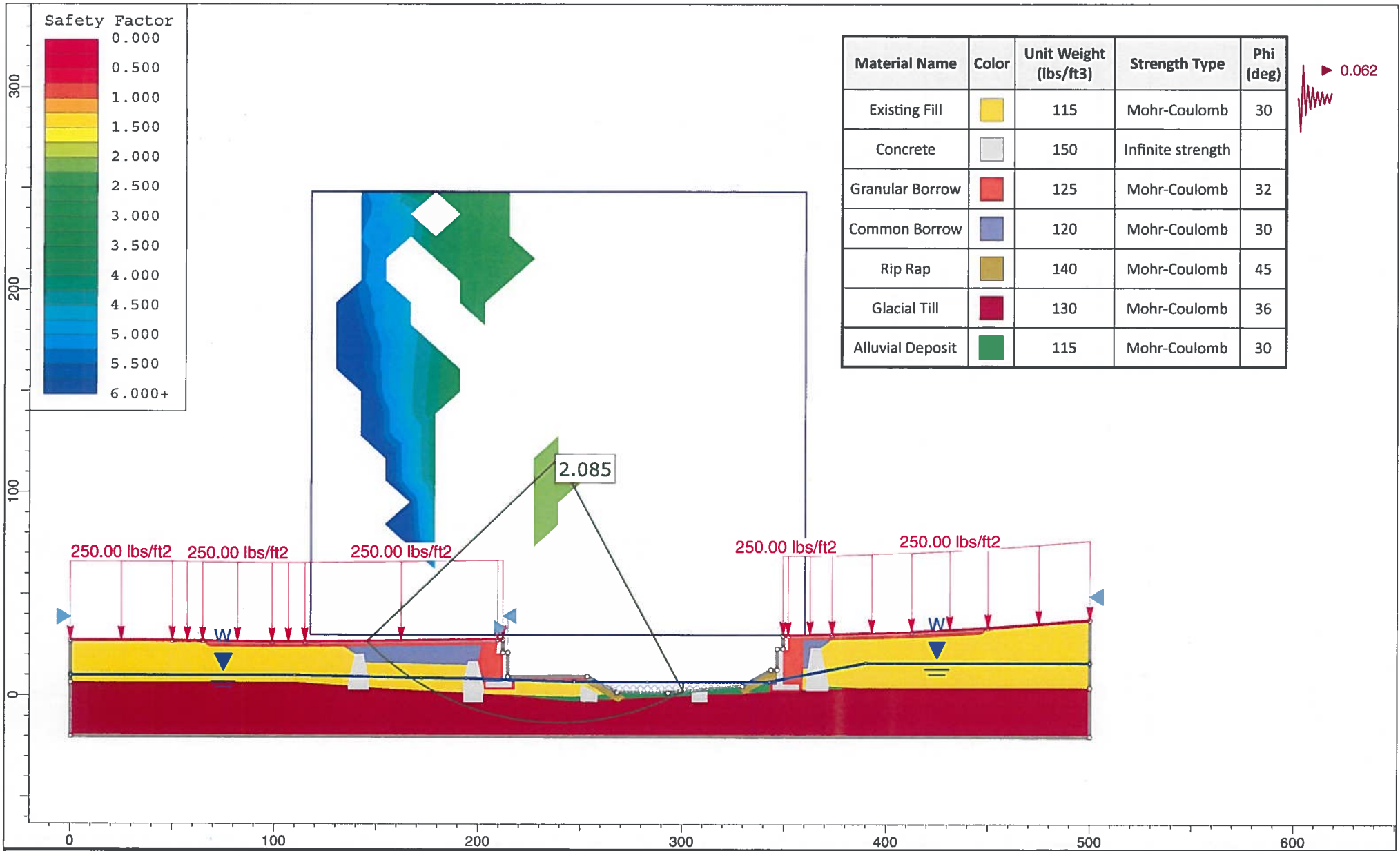
Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Riprap	140	45	0
Granular Borrow	125	32	0
Common Borrow	120	30	0
Existing Fill	115	30	0
Alluvial Deposit	115	30	0
Glacial Till/Weathered Bedrock	130	36	0

RESULTS AND CONCLUSIONS

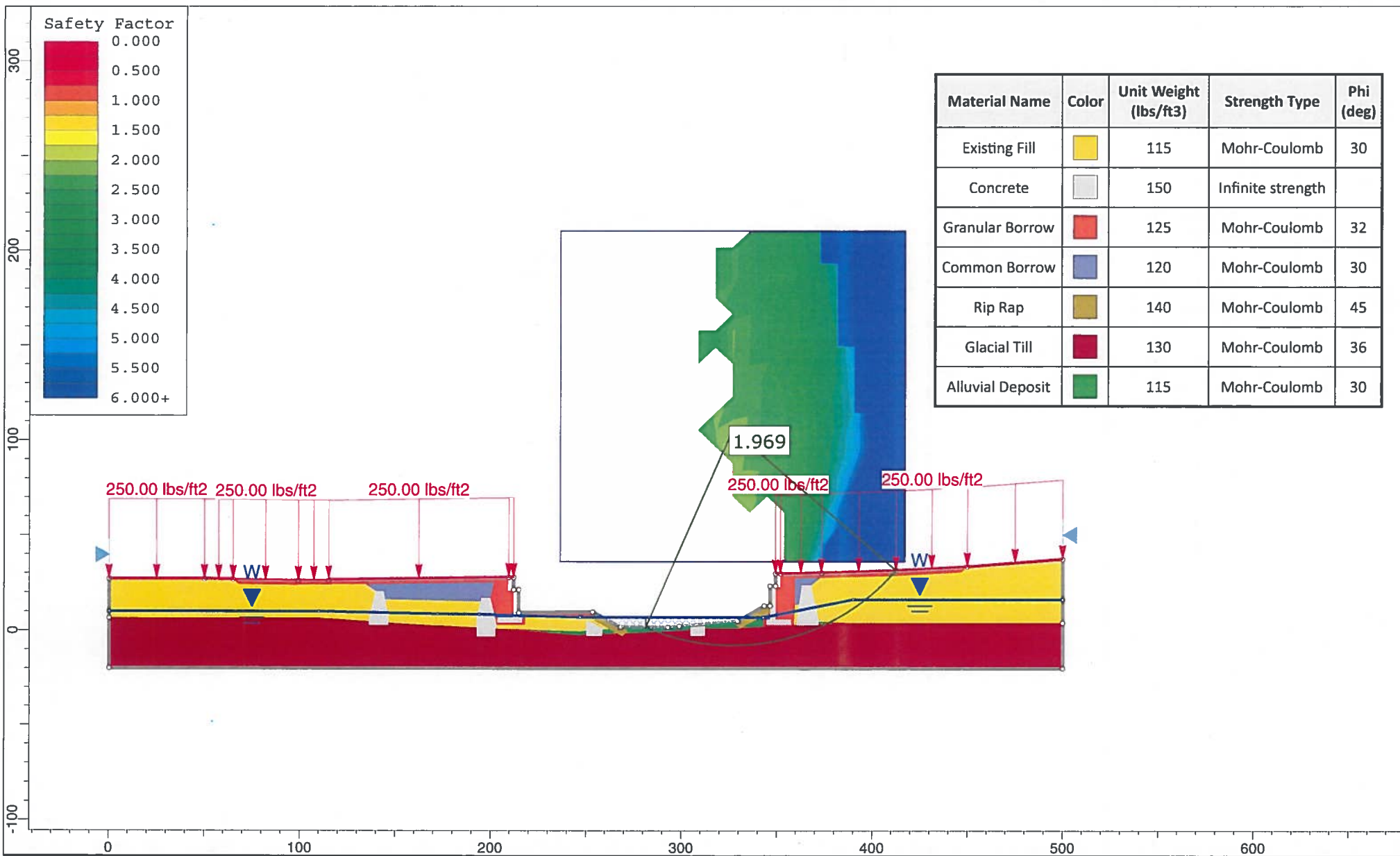
	Factor of Safety	
	Static	Seismic
Abutment 1	2.7	2.1
Abutment 2	2.0	1.6

Based on AASHTO LRFD Section 11.6.2.3, an acceptable resistance factor for where the slope contains or supports a structure is 0.65 (F.S. = $1/0.65 = 1.5$).

Based on Maine DOT Bridge Design Guide Section 5.9.4, a minimum seismic factor of safety of 1.0 is acceptable for slope stability.

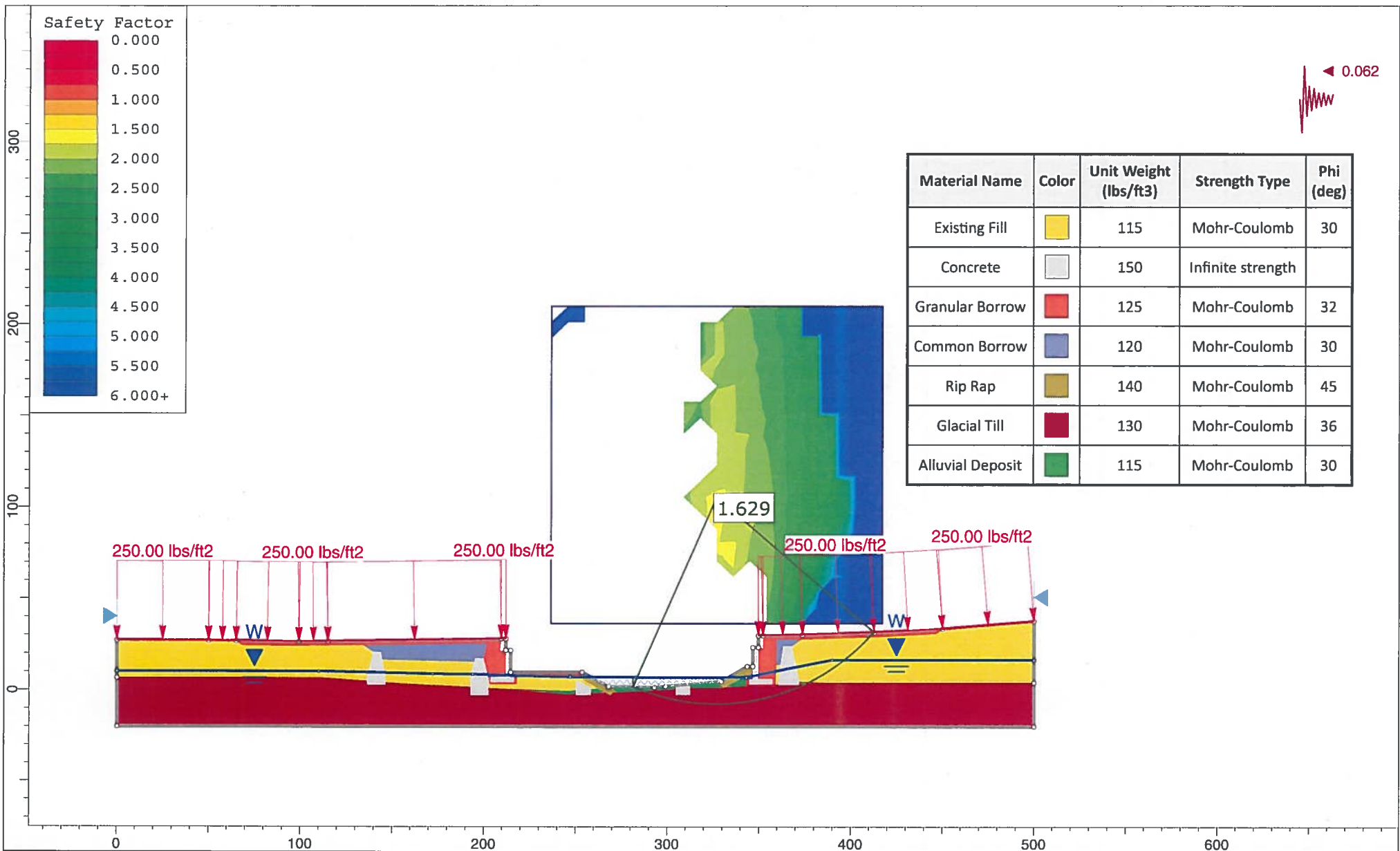


	Project			Route 201 and 9 over Cobbosseecontee Stream, Gardiner, Maine	
	Analysis Description			Global Stability - South - Seismic	
	Drawn By	Kevin Russ	Scale	1:781	Company
	Date	6/5/2018, 8:08:03 AM	File Name	2018_0605_Global Stability_S_Seismic.slim	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Phi (deg)
Existing Fill		115	Mohr-Coulomb	30
Concrete		150	Infinite strength	
Granular Borrow		125	Mohr-Coulomb	32
Common Borrow		120	Mohr-Coulomb	30
Rip Rap		140	Mohr-Coulomb	45
Glacial Till		130	Mohr-Coulomb	36
Alluvial Deposit		115	Mohr-Coulomb	30

	Project			
	Route 201 and 9 over Cobbosseecontee Stream, Gardiner, Maine			
	Analysis Description			
	Global Stability - North - Static			
	Drawn By	Kevin Russ	Scale	1:840
	Date	6/5/2018, 8:08:03 AM	Company	Haley & Aldrich, Inc.
SLIDEINTERPRET 7.018			File Name	2018_0605_Global Stability_N_Static.slim



	Project			
	Route 201 and 9 over Cobbosseecontee Stream, Gardiner, Maine			
	Analysis Description			
	Global Stability - North - Seismic			
SLIDEINTERPRET 7.018	Drawn By	Kevin Russ	Scale	1:869
	Date	6/5/2018, 8:08:03 AM	Company	Haley & Aldrich, Inc.
			File Name	2018_0605_Global Stability_N_Seismic.slim

Seismic Site Class

CALCULATIONS

File No.	42223-004
Sheet	1 of 3
Date	6/4/2018
Computed By	KAR
Checked By	MMB

Client Maine Department of Transportation

Project 201 and 9 over Cobbosseecontee Stream; WIN 18732.00 Bridge NO. 2101

Subject Seismic Site Class & Design Parameters

OBJECTIVE:

Assess the seismic hazard at the bridge site in accordance with the AASHTO LRFD Bridge Design Specifications 7th Edition 2014 and the MaineDOT Bridge Design Guide 2003 with interim revisions through August 2008.

METHODOLOGY:

1. Determine geographic location of the bridge site (latitude-longitude coordinates).
2. Determine Site Class in accordance with Section 3.10.3.1 (N method for cohesionless soils) considering only test borings drilled at proposed substructure locations. These test borings were drilled into bedrock and/or penetrated a minimum of 100 ft below existing ground surface/mudline. For sublayers with an SPT N60 value equal to 0 bpf or in excess of 100 bpf, default values of 1 and 100 bpf were used, respectively.
3. Determine Site Factors and Elastic Seismic Response Coefficients based on (1) and (2) in accordance with Section 3.10.3.2. Factors are determined using the AASHTO earthquake ground motion parameters application v. 2.10 for a seismic event having a 7 percent probability of exceedance in 75 years.
4. Determine Seismic Design Category based on (3) in accordance with Section 3.5.

1. GEOGRAPHIC LOCATION:

Determine latitude-longitude site coordinates using Google Earth.

Latitude =	44° 13' 50" North	Convert to decimal degrees.	Latitude =	44.2306° North
Longitude =	69° 46' 28" West		Longitude =	69.7744° West





CALCULATIONS

File No.	42223-004
Sheet	2 of 3
Date	6/4/2018
Computed By	KAR
Checked By	MMB

Client	Maine Department of Transportation
Project	201 and 9 over Cobbosseecontee Stream; WIN 18732.00 Bridge NO. 2101
Subject	Seismic Site Class & Design Parameters

2. SITE CLASS:

BB-GCS-101:	14.7	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
13.7	1	16	3	34
9.7	5	2	3	
7.7	7	8	2	
5.7	9	51	2.2	
3.3	11.4	82	2	
0.7	14	100	5.3	
-6.3	21	66	8.5	
-16.3	31	76	15	
-36.3	51	100	16	
-42.3	57	100	43	
Total =			100	

BB-GCS-401/A	10.5	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
9.5	1	6	2	46
7.5	3	3	2	
5.8	4.7	63	2.2	
2.9	7.6	100	2.8	
0.5	10	100	3.5	
-4.5	15	87	5	
-9.5	20	100	5	
-14.5	25	77	5	
-19.5	30	54	5	
-24.5	35	59	5	
-29.5	40	77	5	
-34.5	45	73	5	
-39.5	50	88	5	
-44.5	55	68	5	
-49.5	60	81	7.7	
-54.7	65.2	100	34.8	
Total =			100	

BB-GCS-102/A	#	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
8.1	2	22	4	66
4.1	6	43	3.75	
0.6	9.5	100	2.75	
-1.4	11.5	70	2	
-3.9	14	50	4	
-8.9	19	80	3.5	
-10.9	21	96	3.5	
-15.9	26	100	5	
-20.9	31	41	5.5	
-26.9	37	38	5	
-30.9	41	100	4.5	
-35.9	46	38	5	
-40.9	51	69	5	
-45.9	56	100	11.5	
-54.9	65	100	35	
Total =			100	

BB-GCS-402/A	13.0	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
12.0	1	2	2	40
10.0	3	18	1.8	
8.0	5	16	2.2	
6.0	7	43	2	
4.1	8.9	60	1.5	
-1.0	14	56	9.5	
-11.0	24	86	7.75	
-16.5	29.5	98	5	
-20.7	33.7	100	4.45	
-26.0	39	64	5.3	
-31.0	44	79	5	
-36.0	49	37	5	
-41.0	54	58	5	
-46.0	59	87	7	
-50.5	63.5	100	36.5	
Total =			100	

BB-GCS-103:	30.3	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
28.8	1.5	31	2.5	26
26.8	3.5	8	2	
24.8	5.5	3	2.5	
21.8	8.5	10	2.75	
19.3	11	7	2.25	
17.3	13	12	2	
15.3	15	17	2	
13.3	17	9	2	
11.3	19	6	2	
9.3	21	10	2	
7.3	23	15	2	
5.3	25	38	2.2	
3.3	27	42	2.8	
-0.5	30.8	100	4.25	
-5.7	36	90	7.75	
-15.7	46	100	10	
-25.7	56	78	10	
-35.7	66	100	15	
-45.7	76	100	24	
Total =			100	

BB-GCS-403	10.5	(existing ground surface elevation)		
Elevation (ft, NAVD 88)	Depth (ft, BGS)	N ₆₀ (bpf)	Thickness (ft)	Weighted SPT N60 (bpf)
9.5	1	28	2	76
7.5	3	34	2	
5.5	5	19	2	
3.5	7	100	1.8	
-0.5	11	100	5.4	
-5.5	16	100	5.3	
-10.5	21	90	4.7	
-15.5	26	89	5.3	
-20.3	30.8	62	4.8	
-25.4	35.9	67	5.1	
-30.5	41	58	5.1	
-35.5	46	72	5	
-40.5	51	72	5	
-45.5	56	100	6.5	
-49.5	60	100	40	
Total =			100	

- Notes:
- For test boring locations where bedrock was encountered within 100 ft of ground surface/mudline, the thickness of the rock layer was adjusted so that the total thickness of material encountered at each test boring location is equal to 100 ft.
 - For sublayers where the average SPT N60 value was in excess of 100 bpf a default value of 100 bpf was used in accordance with AASHTO LRFD. Applicable cells are highlighted in orange.
 - For sublayers where the SPT N60 value was equal to 0 bpf a default value of 1 bpf was used. Applicable cells are highlighted in green.



CALCULATIONS

Client

Maine Department of Transportation

Project

201 and 9 over Cobbosseecontee Stream; WIN 18732.00 Bridge NO. 2101

Subject

Seismic Site Class & Design Parameters

File No.

42223-004

Sheet

3 of 3

Date

6/4/2018

Computed By

KAR

Checked By

MMB

Summarize weighted SPT N60 values for each test boring....

Test Boring No.	Weighted SPT N60 (bpf)	Site Class
BB-GCS-101	34	D
BB-GCS-102/A	66	C
BB-GCS-103	26	D
BB-GCS-401/A	46	D
BB-GCS-402/A	40	D
BB-GCS-403	76	C

In accordance with Table 3.10.3.1-1 for SPT > 50 bpf the site should be considered Site Class C.
In accordance with Table 3.10.3.1-1 for SPT 15 < N60 < 50 bpf the site should be considered Site Class D.

Determine site factors and elastic seismic response coefficients based on geographic location and Site Class

3. SITE FACTORS & ELASTIC SEISMIC RESPONSE COEFFICIENTS:

Determine parameters using the Seismic Design Parameters v. 2.0 application developed by USGS and included in the AASHTO LRFD Bridge Design Specifications. The values summarized below are based on the AASHTO response spectra for an event having a 7 percent probability of exceedance in 75 years.

SITE CLASS D - "Stiff Soil"	
Design Parameter	Design Value
Site Factor for short-period range of acceleration response spectrum, F_a =	1.600
Site Factor for long-period range of acceleration response spectrum, F_v =	2.400
Horizontal response spectral acceleration coefficient at 0.2-s period on rock, S_s =	0.160
Horizontal response spectral acceleration coefficient at 1.0-s period on rock, S_1 =	0.045
Peak seismic ground acceleration coefficient on rock, PGA =	0.077
Site factor at zero-period on acceleration response spectrum, F_{pga} =	1.600
Horizontal response spectral acceleration coefficient at 0.2-s period modified by F_a , S_{DS} =	0.256
Horizontal response sepctral acceleration coefficient at 1.0-s period modified by F_v , S_{D1} =	0.108

In accordance with Table 3.10.6-1, since $SD1 \leq 0.15$, the bridge should be designed based on the requirements of **Seismic Zone 1 (AASHTO LRFD 2014)**.
Equivalent to previously...
Seismic Design Category A (AASHTO LRFD 2011)

Liquefaction Analysis

PROBLEM STATEMENT & OBJECTIVE

Calculate factor of safety against liquefaction using the procedure in Youd and Idriss (2001) for SPT N data.

EXECUTIVE SUMMARY

All factors of safety are greater than 1.0, so liquefaction is unlikely. Most test samples are considered non-liquefiable due to location above MHW groundwater level or test blow counts > 30.

REFERENCES

- 1. Youd and Idriss, "Liquefaction Resistance of Soils: Summary Report for the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils," 2001.
- 2. AASHTO LRFD Bridge Design Specifications 7th Edition, 2014.

AVAILABLE INFORMATION

- 1. Maine Department of Transportation boring logs BB-CGS-101, BB-CGS-102, BB-CGS-102a, and BB-CGS-103 from November 2015.
- 2. The Peak Ground Acceleration, As, is 0.123g from the seismic site class calculations completed by Haley & Aldrich (SITE CLASS D).
- 3. Deaggregation plot of the site from the United States Geological Survey (USGS) Earthquake Hazards website (<https://earthquake.usgs.gov/hazards/interactive/>) for 975-yr return period.
- 4. Geotechnical laboratory test results for sieve gradation by GeoTesting Express.

ASSUMPTIONS

- 1. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
- 2. Fines content for samples from borings BB-cgs-101, BB-cgs-102a and BB-cgs-103 were estimated based on field geologist log descriptions correlated to the Maine Department of Transportation key to soil and rock description and terms.
- 3. Fines content for following samples was determined from sieve gradation laboratory testing:

Boring No.	Sample	Depth	% Fines
BB-CGS-101	1D	0-2	16.4
BB-CGS-101	3D	4-6	13.5
BB-CGS-101	8D	20-22	54
BB-CGS-102	1D	1-3	8.9
BB-CGS-102	3D	4.2-6.2	7.1
BB-CGS-102a	4D	13-15	82.7
BB-CGS-103	2D	2.5-4.5	14.9
BB-CGS-103	3D	4.5-6.5	48.5
BB-CGS-103	6D	12-14	48.7

- 4. Total unit weights used in the analysis were estimated as follows: Fill 115 pcf, Marine Clay 110 pcf, Marine Deposit 120 pcf, and Glacial Till 130 pcf.
- 5. SPT N₆₀ values were provided on the boring logs. The SPT N₆₀ values were used in the analysis.
- 6. An SPT value of "1" was assigned to samples with WOR or WOH. An SPT value of "100" was assigned to samples that encountered refusal.
- 7. Assume 90% fines for CL layer
- 8. Magnitude M=5.1 based on deaggregation plot and corresponding MSF=2.68 based on equation (24) in Youd & Idriss (2001).
- 9. Groundwater elevation based on test boring data.
- 10. Minimum required factor of safety against liquefaction is 1.2 (AASHTO LRFD Section 10.5.4.2).

Client: Maine Department of Transportation
Computed by: JMK Checked by: JLL
Sheet: 1 of 5

HALEY ALDRICH File No.: 42223-000
Date: 31-May-17
Project: Gardiner- Bridge 2101-U.S. 201 & 9 over the Cobbossee Stream
Subject: Liquefaction Analysis

PROCEDURE

1. Evaluate the Cyclic Stress Ratio:

The Cyclic Stress Ratio (CSR) is defined as $CSR = (\tau_{av}/\sigma'_{vo}) = 0.65(\frac{a_{max}}{g})(\frac{\sigma_{vo}}{\sigma'_{vo}})r_d$

a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake (A_s * g)
g = acceleration of gravity
 σ_{vo} = total vertical overburden stress
 σ'_{vo} = effective vertical overburden stress
 r_d = stress reduction coefficient
 $r_d = 1 - 0.00765z$ for $z \leq 9.15$ m
 $r_d = 1.174 - 0.0267z$ for 9.15 m $< z \leq 23$ m
z = depth below ground surface in meters

2. Evaluate the Cyclic Resistance Ratio:

The Cyclic Resistance Ratio for a magnitude 7.5 earthquake ($CRR_{7.5}$) is defined as $CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10*(N_1)_{60} + 45]^2} - \frac{1}{200}$ for clean sands

where $(N_1)_{60}$ = SPT blow count normalized to an overburden pressure of approximately 100 KPa (1 tsf) and a hammer energy ratio or hammer efficiency of 60%.

$(N_1)_{60} = C_N * N_{60}$

$C_N = \frac{P_a^{0.5}}{\sigma'_{vo}} (\leq 1.7)$

The above is valid for $(N_1)_{60} < 30$
for $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable (NL)

To consider the influence of fines content (FC), a correction to the $(N_1)_{60}$ to an equivalent clean sand is made $(N_1)_{60\text{cs}} = \alpha + \beta(N_1)_{60}$
where α and β are coefficients determined from the following relationships:

- $\alpha = 0$ for $FC \leq 5\%$
- $\alpha = \exp[1.76 - (190/FC^2)]$ for $5\% < FC < 35\%$
- $\alpha = 5.0$ for $FC \geq 35\%$
- $\beta = 1.0$ for $FC \leq 5\%$
- $\beta = [0.99 + (FC^{1.5}/1000)]$ for $5\% < FC < 35\%$
- $\beta = 1.2$ for $FC \geq 35\%$

3. Evaluate the Factor of Safety Against Liquefaction:

where $F.S. = \frac{CRR_{7.5}}{CSR} * MSF$

MSF = magnitude scaling factor (equation (24) in Youd & Idriss, 2001)
 M_w = moment magnitude of earthquake

CALCULATIONS

Boring No.	BB-GCS-101
Approximate Ground Surface Elevation (NAVD88)	14.7
Depth of Groundwater	6ft
Unit Weight of Seawater	64pcf
$A_s = F_{PGA} * PGA$	0.123g
Magnitude (based on modal magnitude from deaggregation plot)	5.1
Magnitude Scaling Factor	2.68

Sample No.	Depth (ft)	Elevation	Description	SPT N_{60}	Y_t (pcf)	σ_v (psf)	σ_w (psf)	σ'_v (psf)	C_N	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60CS}$	$CRR_{7.5}$	r_d	CSR	FS
1D	1	13.7	Assumed Fill	16	115	115	0	115	1.70	27	16.4	2.87	1.06	32	NL	1.00	0.080	#N/A
3D	5	9.7	Assumed Fill	2	115	575	0	575	1.70	3	13.5	2.05	1.04	6	0.076	0.99	0.079	2.6
4D	7	7.7	Assumed Fill	8	115	805	64	741	1.68	13	5	0.00	1.00	13	0.145	0.98	0.085	4.5
5D	9	5.7	Assumed Fill	51	115	1,035	192	843	1.57	80	35	4.98	1.20	101	NL	0.98	0.096	#N/A
6D	11.4	3.3	Assumed Fill	82	115	1,311	346	965	1.47	121	35	4.98	1.20	149	NL	0.97	0.106	#N/A
7D	14	0.7	Glacial Till	100	130	1,649	512	1137	1.36	136	80	5.00	1.20	168	NL	0.97	0.112	#N/A
8D	21	-6.3	Glacial Till	66	130	2,559	960	1599	1.14	75	54	5.00	1.20	96	NL	0.95	0.122	#N/A
9D	31	-16.3	Glacial Till	76	130	3,859	1600	2259	0.96	73	70	5.00	1.20	93	NL	0.92	0.126	#N/A
10D	51	-36.3	Glacial Till	100	130	6,459	2880	3579	0.76	76	70	5.00	1.20	97	NL	0.76	0.110	#N/A
11D	55.9	-41.2	Weathered Bedrock															
			Bedrock															

Boring No.	BB-GCS-102A
Approximate Ground Surface Elevation (NAVD88)	10.1
Depth of Groundwater	4.4ft
Unit Weight of Seawater	64pcf
$A_s = F_{PGA} * PGA$	0.123g
Magnitude (based on modal magnitude from deaggregation plot)	5.1
Magnitude Scaling Factor	2.68

Sample No.	Depth (ft)	Elevation	Description	SPT N_{60}	Y_t (pcf)	σ_v (psf)	σ_w (psf)	σ'_v (psf)	C_N	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60CS}$	$CRR_{7.5}$	r_d	CSR	FS
	2	8.1	Fill	22	115	230	0	230	1.70	37	8.9	0.53	1.02	39	NL	1.00	0.080	#N/A
	5.2	4.9	Fill	30	115	598	51	547	1.70	51	7.1	0.13	1.01	52	NL	0.99	0.086	#N/A
1D	6	4.1	Fill	43	115	690	102	588	1.70	73	5	0.00	1.00	73	NL	0.99	0.093	#N/A
2D	9.5	0.6	Fill	100	115	1,093	326	766	1.65	165	15	2.50	1.05	176	NL	0.98	0.111	#N/A
3D	11.5	-1.4	Alluvial	70	120	1,333	454	878	1.54	108	5	0.00	1.00	108	NL	0.97	0.118	#N/A
4D	14	-3.9	Glacial Till	50	130	1,658	614	1043	1.41	71	82.7	5.00	1.20	90	NL	0.97	0.123	#N/A
5D	19	-8.9	Glacial Till	80	130	2,308	934	1373	1.23	99	83	5.00	1.20	123	NL	0.96	0.128	#N/A
MD	21	-10.9	Glacial Till	96	130	2,568	1062	1505	1.18	113	83	5.00	1.20	141	NL	0.95	0.130	#N/A
6D	26	-15.9	Glacial Till	100	130	3,218	1382	1835	1.07	107	83	5.00	1.20	133	NL	0.94	0.132	#N/A
7D	31	-20.9	Glacial Till	41	130	3,868	1702	2165	0.98	40	83	5.00	1.20	53	NL	0.92	0.132	#N/A
8D	37	-26.9	Glacial Till	38	130	4,648	2086	2561	0.90	34	70	5.00	1.20	46	NL	0.87	0.127	#N/A
9D	41	-30.9	Glacial Till	100	130	5,168	2342	2825	0.86	86	83	5.00	1.20	108	NL	0.84	0.123	#N/A
10D	46	-35.9	Glacial Till	38	130	5,818	2662	3155	0.81	31	70	5.00	1.20	42	NL	0.80	0.118	#N/A
11D	51	-40.9	Glacial Till	69	130	6,468	2982	3485	0.77	53	70	5.00	1.20	69	NL	0.76	0.113	#N/A
12D	56	-45.9	Glacial Till	100	130	7,118	3302	3815	0.74	74	60	5.00	1.20	94	NL	0.72	0.107	#N/A

Client: Maine Department of Transportation

Computed by: JMK

Checked by: JLL

Sheet: 3 of 5

File No.: 42223-000

Date: 31-May-17

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Project: Gardiner- Bridge 2101-U.S. 201 & 9 over the Cobbossee Stream

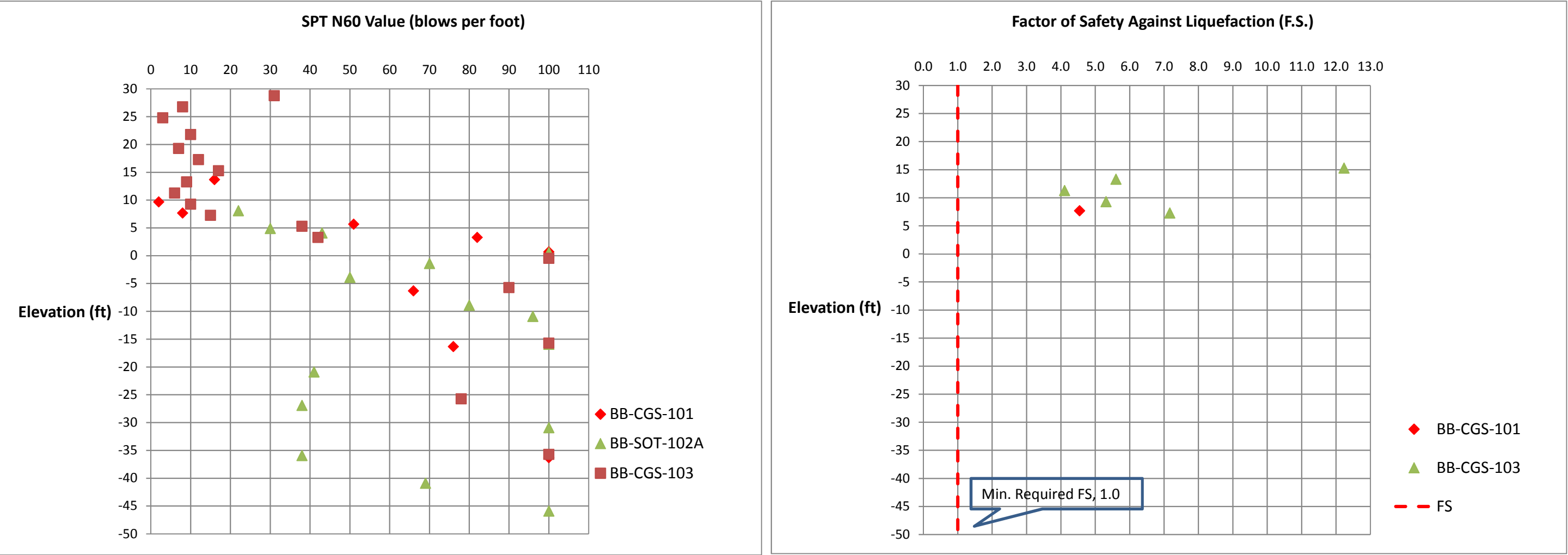
Subject: Liquefaction Analysis

CALCULATIONS

Boring No.	BB-GCS-103
Approximate Ground Surface Elevation (NAVD88)	30.3
Depth of Groundwater	14ft
Unit Weight of Seawater	64pcf
$A_s = F_{PGA} * PGA$	0.123g
Magnitude (based on modal magnitude from deaggregation plot)	5.1
Magnitude Scaling Factor	2.68

Sample No.	Depth (ft)	Elevation	Description	SPT N_{60}	γ_t (pcf)	σ_v (psf)	σ_w (psf)	σ'_v (psf)	C_N	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60CS}$	$CRR_{7.5}$	r_d	CSR	FS
1D	1.5	28.8	Fill	31	115	173	0	173	1.70	53	15	2.50	1.05	58	NL	1.00	0.080	#N/A
2D	3.5	26.8	Fill	8	115	403	0	403	1.70	14	14.9	2.47	1.05	17	0.178	0.99	0.079	6.0
3D	5.5	24.8	Fill	3	115	633	0	633	1.70	5	48.5	5.00	1.20	11	0.123	0.99	0.079	4.2
4D	8.5	21.8	Fill	10	115	978	0	978	1.46	15	40	5.00	1.20	23	0.250	0.98	0.078	8.6
5D	11	19.3	Fill	7	115	1,265	0	1265	1.28	9	80	5.00	1.20	16	0.168	0.97	0.078	5.8
6D	13	17.3	Fill	12	115	1,495	0	1495	1.18	14	48.7	5.00	1.20	22	0.242	0.97	0.078	8.4
7D	15	15.3	Fill	17	115	1,725	64	1661	1.12	19	40	5.00	1.20	28	0.365	0.97	0.080	12.2
8D	17	13.3	Fill	9	115	1,955	192	1763	1.09	10	35	4.98	1.20	17	0.178	0.96	0.085	5.6
9D	19	11.3	Fill	6	115	2,185	320	1865	1.06	6	80	5.00	1.20	13	0.137	0.96	0.090	4.1
10D	21	9.3	Fill	10	115	2,415	448	1967	1.03	10	80	5.00	1.20	17	0.185	0.95	0.093	5.3
11D	23	7.3	Fill	15	115	2,645	576	2069	1.00	15	80	5.00	1.20	23	0.258	0.95	0.097	7.2
12D	25	5.3	Fill	38	115	2,875	704	2171	0.98	37	80	5.00	1.20	50	NL	0.94	0.100	#N/A
13D	27	3.3	Glacial Till	42	130	3,135	832	2303	0.95	40	80	5.00	1.20	53	NL	0.94	0.102	#N/A
14D	30.75	-0.45	Glacial Till	100	130	3,623	1072	2551	0.90	90	80	5.00	1.20	114	NL	0.92	0.105	#N/A
15D	36	-5.7	Glacial Till	90	130	4,305	1408	2897	0.85	76	80	5.00	1.20	97	NL	0.88	0.105	#N/A
16D	46	-15.7	Glacial Till	100	130	5,605	2048	3557	0.77	77	80	5.00	1.20	97	NL	0.80	0.101	#N/A
17D	56	-25.7	Glacial Till	78	130	6,905	2688	4217	0.70	55	80	5.00	1.20	71	NL	0.72	0.094	#N/A
18D	66	-35.7	Glacial Till	100	130	8,205	3328	4877	0.65	65	80	5.00	1.20	84	NL	0.64	0.086	#N/A

PLOTS



Note: Non-liquefied samples (NL) are not shown above.

CONCLUSIONS AND RECOMMENDATIONS

All factors of safety are greater than 1.2, so liquefaction is unlikely.

Lateral Micropile Evaluation

Client: Maine Department of Transportation

Date: 20SEP2018

Project: Bridge Street Bridge over the Cobbosseecontee Stream, Gardiner, Maine

Computed by: MMB

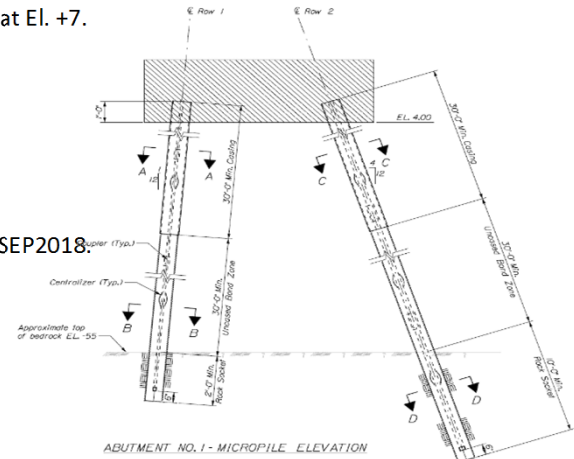
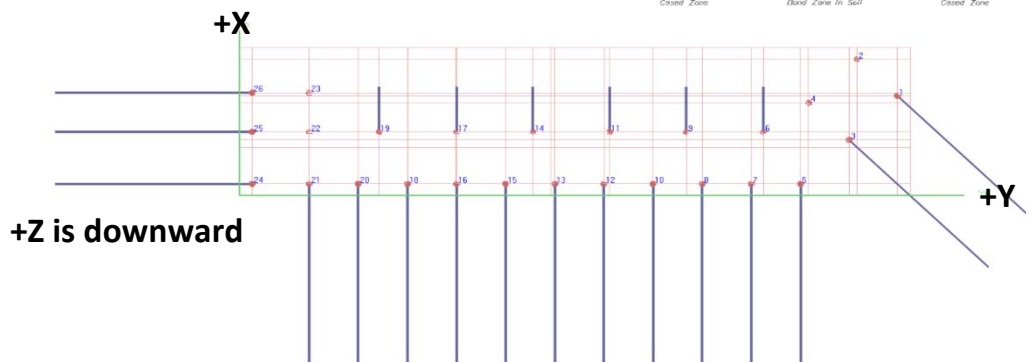
Subject: FB-MPier Analysis of Abutment 1

Checked by: JLL

ABUTMENT 1 ASSUMPTIONS

1. Bottom of cap/top of piles and ground surface at El. +4. Water at El. +7.
2. Pile cap is 3 ft thick and assumed rigid and weightless.
3. Pinned pile-to-cap connection.
4. Loads were applied at location on cap indicated by Stantec.
5. Corrosion assumed to reduce casing thickness by 1/16 in.
6. Piles are assumed to be linear elastic and with infinite strength.
7. Pile Lateral group effects can be assumed negligible.
8. Pile loads, cross-sections, and layout provided by Stantec on 05SEP2018.
9. Piles are drilled micropiles with sections and properties shown:

Pile Section	AE total (kip)	EI total (kip*in ²)
A-A	640360	4984194
B-B	242274	928197
C-C	727309	5028432
D-D	329223	972436

**FB-MPIER MODEL****SUMMARY OF APPLIED LOADS (FB-MPIER COORDINATE AXES)**

Limit State	FX kip	FY kip	FZ kip	MX kip*ft	MY kip*ft	MZ kip*ft
Service I	-895	-85	3519	-3007	8606	1025
Strength I (Max. Comp.)	-1293	-153	4847	-4633	12152	1692
Strength I (Max. Tension)	-1293	-153	3511	613	11696	1692

SUMMARY OF RESULTS**PINNED PILE-TO-CAP CONNECTION**

Limit State	Min Fz (kip)	Max Fz (kip)	Max Mx (kip*ft)	Max My (kip*ft)	Max Vx (kip)	Max Vy (kip)	Max Ux (in)	Max Uy (in)
Service I	2	241	7	23	5	2	0.38	0.09
Strength I (Max. Comp.)	4	339	13	35	8	3	0.62	0.20
Strength I (Max. Tension)	-48	297	21	54	11	5	0.99	0.34

Notes:

1. x = parallel to bridge alignment, y = parallel to bridge transverse direction, z = vertical direction.
2. Positive Fz is compression, negative Fz is tension or uplift.
3. Vx and Vy are pile internal shear forces.
4. Mx and My are pile internal bending moments.
5. Ux and Uy are pile lateral deflections.
6. Maximum and minimum force and moment values are for an individual pile within the group.
7. Maximum forces, shears and bending moments may not occur within the same pile for a particular load case.
8. Structural/Bridge Engineer should check pile section structural resistance under the combined loading.

Client: Maine Department of Transportation

Date: 20SEP2018

Project: Bridge Street Bridge over the Cobbosseecontee Stream, Gardiner, Maine

Computed by: MMB

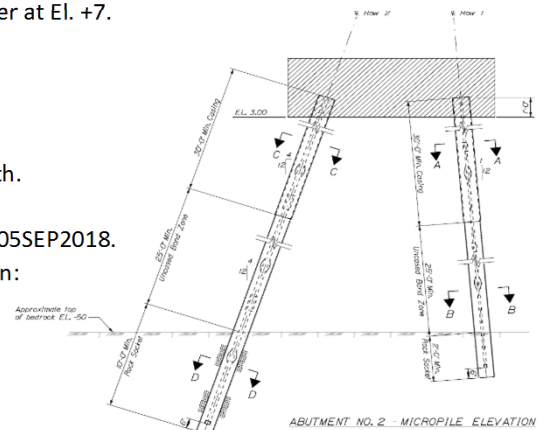
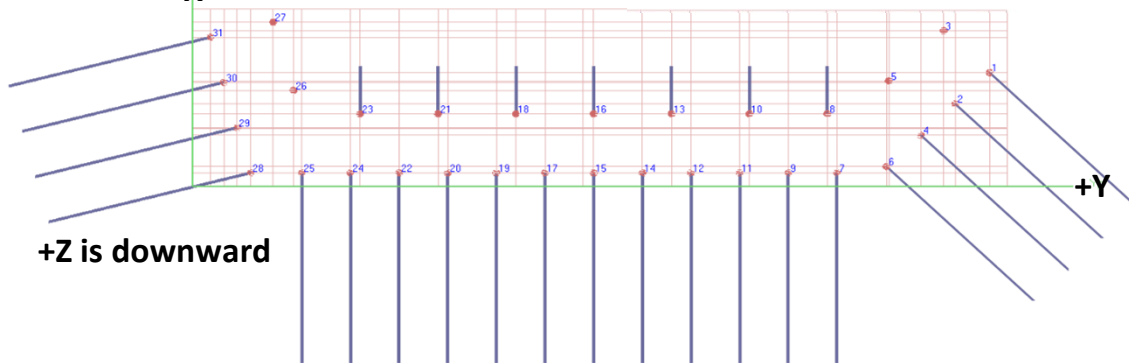
Subject: FB-MPIER Analysis of Abutment 2

Checked by: JLL

ABUTMENT 2 ASSUMPTIONS

1. Bottom of cap/top of piles and ground surface at El. +3. Water at El. +7.
2. Pile cap is 3 ft thick and assumed rigid and weightless.
3. Pinned pile-to-cap connection.
4. Loads were applied at location on cap indicated by Stantec.
5. No corrosion reduction in casing thickness.
6. Piles are assumed to be linear elastic and with infinite strength.
7. Pile Lateral group effects can be assumed negligible.
8. Pile loads, cross-sections, and layout provided by Stantec on 05SEP2018.
9. Piles are drilled micropiles with sections and properties shown:

Pile Section	AE total (kip)	EI total (kip*in ²)
A-A	694811	5606595
B-B	242274	928197
C-C	781759	5650834
D-D	329223	972436


FB-MPIER MODEL
+X

SUMMARY OF APPLIED LOADS (FB-MPIER COORDINATE AXES)

Limit State	FX kip	FY kip	FZ kip	MX kip*ft	MY kip*ft	MZ kip*ft
Service I	-1171	-39	3847	-703	12479	1418
Strength I (Max. Comp.)	-1710	-84	5281	-1638	17767	2210
Strength I (Max. Tension)	-1710	-84	3830	2584	17136	2210

SUMMARY OF RESULTS
PINNED PILE-TO-CAP CONNECTION

Limit State	Min Fz (kip)	Max Fz (kip)	Max Mx (kip*ft)	Max My (kip*ft)	Max Vx (kip)	Max Vy (kip)	Max Ux (in)	Max Uy (in)
Service I	-18	230	4	29	8	2	0.34	0.03
Strength I (Max. Comp.)	-25	328	10	46	12	4	0.58	0.07
Strength I (Max. Tension)	-62	291	12	63	16	4	0.83	0.11

Notes:

1. x = parallel to bridge alignment, y = parallel to bridge transverse direction, z = vertical direction.
2. Positive Fz is compression, negative Fz is tension or uplift.
3. Vx and Vy are pile internal shear forces.
4. Mx and My are pile internal bending moments.
5. Ux and Uy are pile lateral deflections.
6. Maximum and minimum force and moment values are for an individual pile within the group.
7. Maximum forces, shears and bending moments may not occur within the same pile for a particular load case.
8. Structural/Bridge Engineer should check pile section structural resistance under the combined loading.



PROJECT Gardiner Bridge St Bridge

PROJECT # 11468

WIN 18735

MADE BY LSF

DATE 9/4/2018

CHECKED BY DDT

DATE 9/5/2018

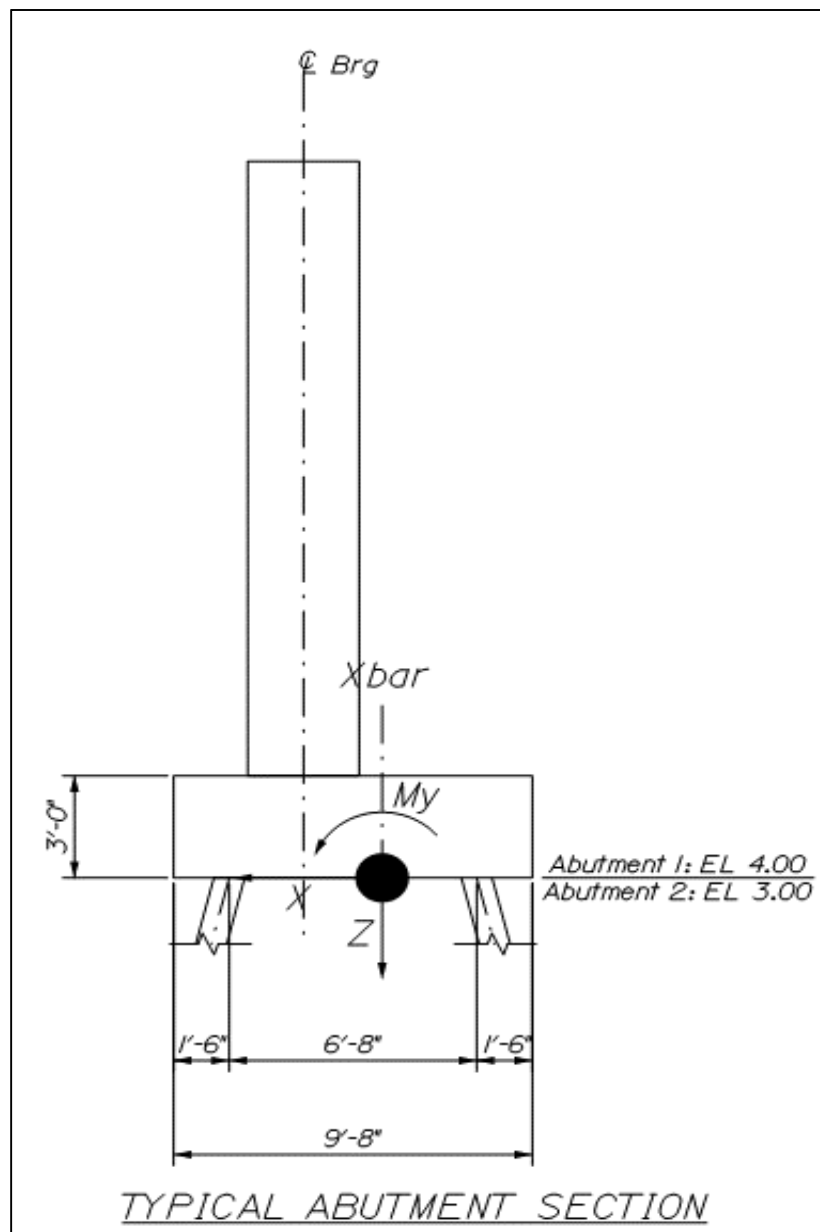
REFERENCES:

AASHTO LRFD 7th Edition (2014) with 2015-2016 Interims

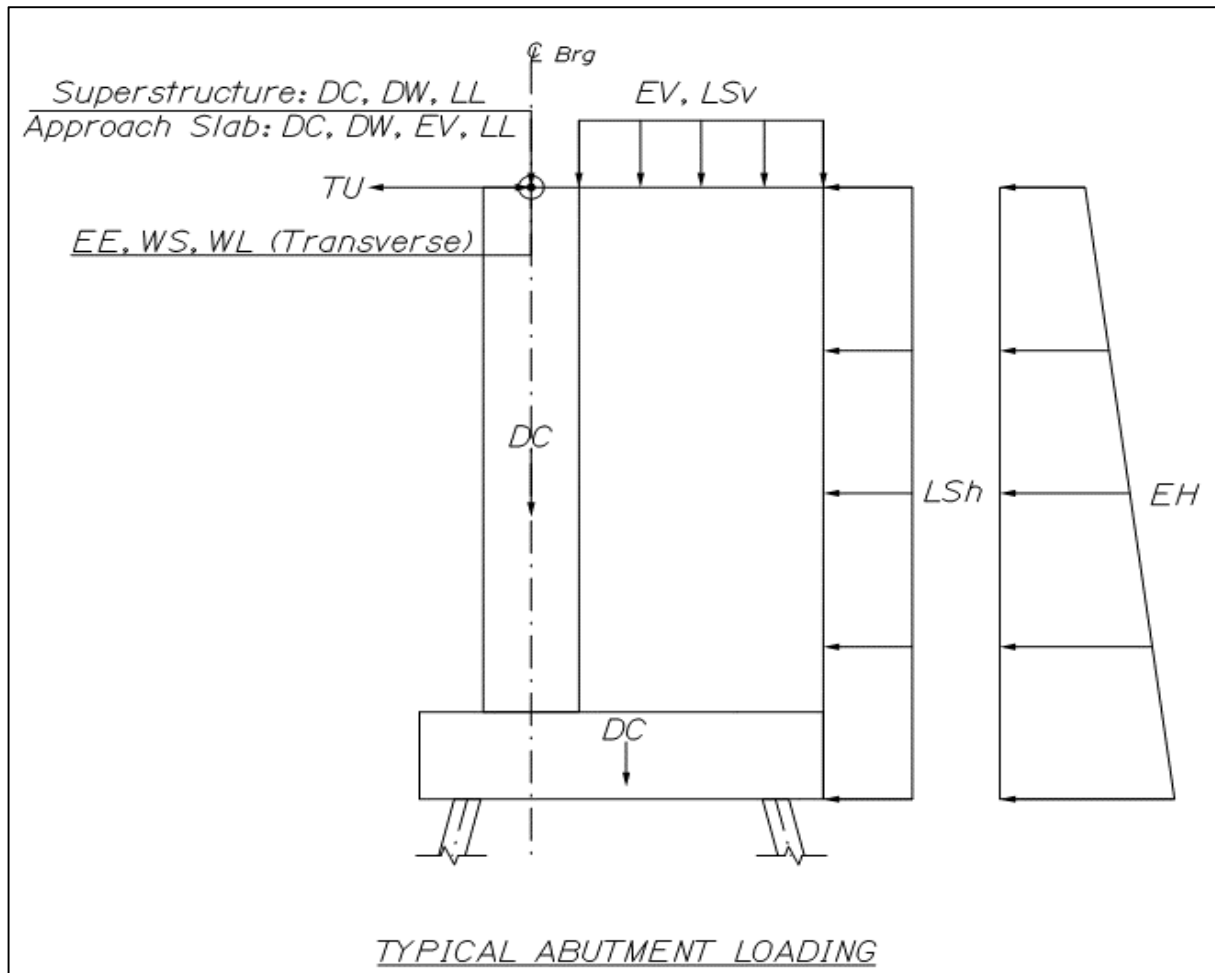
MaineDOT Bridge Design Guide (2014)

MassDOT Bridge Design Manual (2013)

TYPICAL SECTION:



LOADING DIAGRAM:



NOTES:

1. All loads and moments are in units of kips and feet.
2. ~~All battered piles are 3H:12V.~~ ← See attached plans
3. Load signs are based on the sign convention shown.

LOADING SUMMARY:

ABUTMENT 1						
Load Case	F _x	F _y	F _z	M _x	M _y	M _z
	kip	kip	kip	kip-ft	kip-ft	kip-ft
Service I	895	-85	3519	-3007	8606	1025
Strength I (max comp.)	1293	-153	4847	-4633	12152	1692
Strength I (max tension)	1293	-153	3511	613	11696	1692

ABUTMENT 2						
Load Case	F _x	F _y	F _z	M _x	M _y	M _z
	kip	kip	kip	kip-ft	kip-ft	kip-ft
Service I	1171	-39	3847	-703	12479	1418
Strength I (max comp.)	1710	-84	5281	-1638	17767	2210
Strength I (max tension)	1710	-84	3830	2584	17136	2210

PILE PROPERTIES:

Outside Diameter =	9.625	in
Casing thickness =	0.545	in
Bar diameter =	2.5	in
Bar area =	4.91	in ²
Composite Area (with casing) =	21.95	in ²
Composite Moment of Inertia (with casing) =	167.11	in ⁴
Noncomposite area (no casing) =	12.38	in ²
Noncomposite moment of inertia (no casing) =	38.86	in ⁴

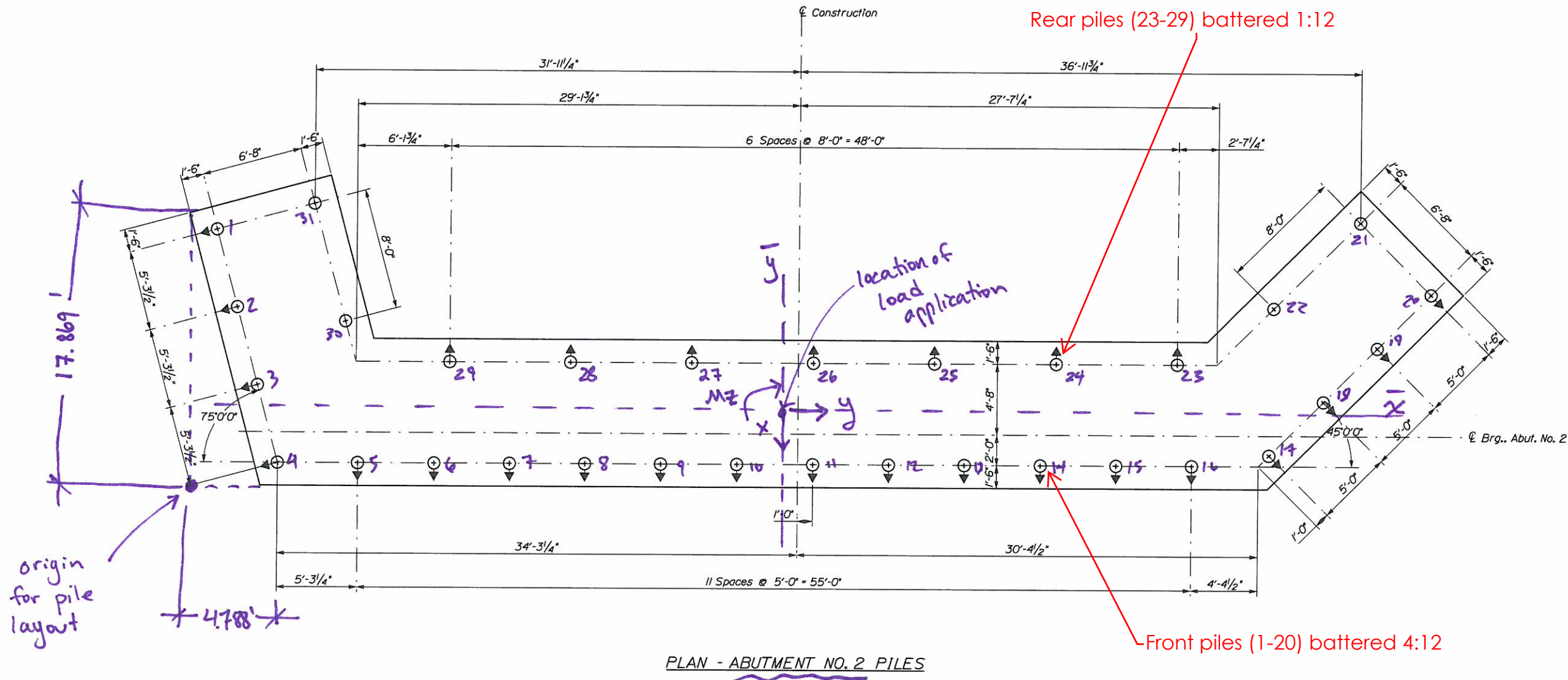
PILE LAYOUT:

ABUTMENT 1		
Number of Piles =		26
Pile #	Distance from Origin in X	Distance from Origin in Y
1	-13.20	1.50
2	-8.17	1.50
3	-1.50	1.50
4	-1.50	8.17
5	-1.50	13.92
6	-1.50	19.67
7	-1.50	25.42
8	-1.50	31.17
9	-1.50	36.92
10	-1.50	42.67
11	-1.50	48.42
12	-1.50	54.17
13	-1.50	59.92
14	-1.50	65.70
15	-7.16	71.36
16	-12.80	77.02
17	-17.50	72.30
18	-11.90	66.64
19	-8.17	61.33
20	-8.17	52.33
21	-8.17	43.33
22	-8.17	34.33
23	-8.17	25.33
24	-8.17	16.33
25	-8.17	8.17
26	-13.20	8.17
Xbar		Ybar
-6.12		36.43

Point of load application Abut 1

ABUTMENT 2		
Number of Piles =		31
Pile #	Distance from Origin in X	Distance from Origin in Y
1	-16.80	1.83
2	-11.71	3.20
3	-6.60	4.57
4	-1.50	5.94
5	-1.50	11.20
6	-1.50	16.20
7	-1.50	21.20
8	-1.50	26.20
9	-1.50	31.20
10	-1.50	36.20
11	-1.50	41.20
12	-1.50	46.20
13	-1.50	51.20
14	-1.50	56.20
15	-1.50	61.20
16	-1.50	66.20
17	-2.21	71.27
18	-5.74	74.82
19	-9.28	78.35
20	-12.81	81.89
21	-17.53	77.17
22	-11.87	71.52
23	-8.17	65.20
24	-8.17	57.20
25	-8.17	49.20
26	-8.17	41.20
27	-8.17	33.20
28	-8.17	25.20
29	-8.17	17.20
30	-10.80	10.35
31	-18.50	8.27
Xbar		Ybar
-6.47		40.06

Point of load application Abut 2



Axial Micropile Evaluations

File No.:	42223-004
Sheet:	1 of 5
Date:	25-Sep-2018
Computed by:	JLL
Checked by:	EAF

Client:	MaineDOT
Project:	ROUTE 24 OVER COBBOSSEECONTEE STREAM
Subject:	Micropile Geotechnical Resistance

PROBLEM STATEMENT & OBJECTIVE

Determine the geotechnical resistance of drilled micropiles in glacial till and bedrock.

EXECUTIVE SUMMARY

Refer to tables on sheet 4 and 5 for capacity for a range of embedment depths.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 7th Edition, with 2015 & 2016 Interim Revisions.

AVAILABLE INFORMATION

1. Test boring logs BB-GCS-101/101A, BB-GCS-401, BB-GCS-402/402A, BB-GCS-403.
2. Draft plan set titled, "Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream" by Stantec, labeled 90% PS&E Submittal.

ASSUMPTIONS

1. Micropile diameter = 9.625 in (OD). Socket diameter below casing = 9.0 in.
2. Micropile diameter = 7.625 in (OD). Socket diameter below casing = 7.0 in.
3. Micropile is socketed in glacial till or glacial till and bedrock with glacial till nominal unit friction resistance of 4 ksf and bedrock nominal unit friction resistance of 19.8 ksf.
4. Axial group effects are not considered.
5. Resistance from soils above glacial till are not considered. Micropile end-bearing resistance is assumed zero.

PROCEDURE

1. Determine nominal grout-to-ground resistance based on embedment within Glacial Till and Bedrock.
2. Determine the rock socket perimeter and calculate the factored geotechnical resistance per unit length.

Client: MaineDOT

Date: 25-Sep-2018

Project: ROUTE 24 OVER COBBOSSECONTEE STREAM

Computed by: JLL

Subject: Micropile Geotechnical Resistance

Checked by: EAF

CALCULATIONS

AASHTO LRFD Nominal Geotechnical Unit Resistance

Table C10.9.3.5.2-1—Summary of Typical α_u Values (Grout-to-Ground Bond) for Preliminary Micropile Design (modified after Sabatini, et al., 2005)

Soil/Rock Description	Typical Range of Grout-to-Ground Bond Nominal Resistance for Micropile Types ⁽¹⁾ (ksf)				
	Type A	Type B	Type C	Type D	Type E
Silt & Clay (some sand) (soft medium plastic)	0.7–1.4	0.7–2.0	0.7–2.5	0.7–3.0	0.7–2.0
Silt & Clay (some sand) (stiff, dense to very dense)	0.7–2.5	1.4–4.0	2.0–4.0	2.0–4.0	1.4–4.0
Sand (some silt) (fine, loose-medium dense)	1.4–3.0	1.4–4.0	2.0–4.0	2.0–5.0	1.4–5.0
Sand (some silt, gravel) (fine-coarse, medium-very dense)	2.0–4.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Gravel (some sand) (medium-very dense)	2.0–5.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	2.0–4.0	2.0–6.5	2.5–6.5	2.5–7.0	2.0–6.5
Soft Shales (fresh-moderate fracturing, little to no weathering)	4.3–11.5	N/A	N/A	N/A	N/A
Slates and Hard Shales (fresh- moderate fracturing, little to no weathering)	10.8–28.8	N/A	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	21.6–43.2	N/A	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	10.8–36.0	N/A	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	28.8–87.7	N/A	N/A	N/A	N/A

⁽¹⁾ Refer to Article 10.9.1 for description of micropile types.

Glacial Till Nominal Unit Resistance

4 ksf

Bedrock Nominal Unit Resistance

19.8 ksf

- Type A micropiles are constructed by placing a sand-cement mortar or neat cement grout in the pile under a gravity head only;
- Type B micropiles are constructed by injecting a neat cement grout under pressure (typically 6–21 ksf) into the drilled hole while the temporary drill casing or auger is withdrawn;
- Type C micropiles are grouted as for Type A, followed 15–25 min after primary grouting by injection of additional grout under pressure (typically greater than 21 ksf) via a preplaced sleeved grout pipe.
- Type D micropiles are grouted similar to Type C, but the primary grout is allowed to harden before injecting the secondary grout under pressure (typically 42–170 ksf) with a packer to achieve treatment of specific pile intervals or material horizons; or
- Type E micropiles are constructed by drilling with grout injection through a continuous-thread, hollow-core steel bar. The grout injection serves to flush cuttings, achieve grout penetration into the ground and stabilize the drill hole. Often the initial grout has a high water to cement ratio and is then replaced with a thicker structural grout near the completion of drilling.

Client: MaineDOT

Date: 25-Sep-2018

Project: ROUTE 24 OVER COBBOSSECONTEE STREAM

Computed by: JLL

Subject: Micropile Geotechnical Resistance

Checked by: EAF

CALCULATIONS

AASHTO LRFD STR Limit State Resistance Factors for Micropiles

Table 10.5.5.2.5-1—Resistance Factors for Geotechnical Resistance of Axially Loaded Micropiles

Limit State	Method/ Ground Condition	Resistance Factor
Compression Resistance of Single Micropile, ϕ_{stat}	Side Resistance (Bond Resistance): Presumptive Values	0.55 ⁽¹⁾
	Tip Resistance on Rock O'Neill and Reese (1999)	0.50
	Side Resistance and Tip Resistance Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Block Failure, ϕ_{bl}	Clay	0.60
Uplift Resistance of Single Micropile, ϕ_{up}	Presumptive Values	0.55 ⁽¹⁾
	Tension Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Group Uplift Resistance, ϕ_{ug}	Sand & Clay	0.50

⁽¹⁾ Apply to presumptive grout-to-ground bond values for preliminary design only in Article C10.9.3.5.2.

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles

	Condition/Resistance Determination Method	Resistance Factor
Nominal Bearing Resistance of Single Pile—Dynamic Analysis and Static Load Test Methods, ϕ_{dyn}	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing* conducted on 100% of production piles	0.75
	Driving criteria established by dynamic testing,* quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
	Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)	0.10

Resistance Factor (Load Test)

0.7

Glacial Till Factored Unit Resistance

2.8 ksf

Bedrock Factored Unit Resistance

13.9 ksf

Client: MaineDOT

Date: 25-Sep-2018

Project: ROUTE 24 OVER COBBOSSEECONTEE STREAM

Computed by: JLL

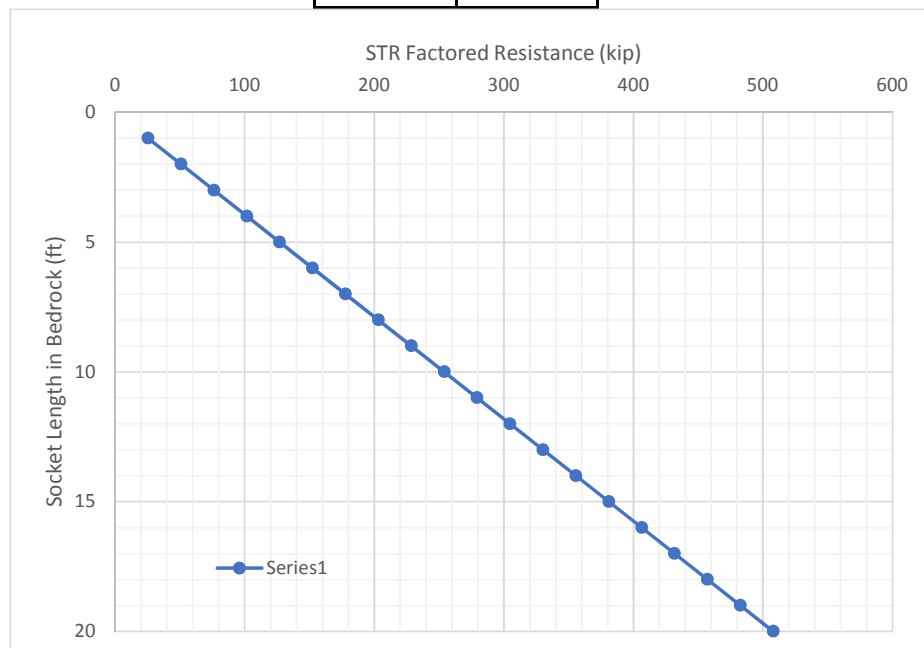
Subject: Micropile Geotechnical Resistance

Checked by: EAF

CALCULATIONS

Rock Socket Diameter (in) = 7
 Rock Socket Perimeter (ft) = 1.8
 Rock Socket Side Area (ft²/ft) = 1.8
 Bedrock STR Factored Unit Side Resistance (ksf) = 13.9
 Bedrock STR Factored Resistance per foot (kip/ft) = 25.4

Rock Socket Length in Bedrock (ft)	Bedrock Factored Geotechnical Resistance (kip)
1	25
2	51
3	76
4	102
5	127
6	152
7	178
8	203
9	229
10	254
11	279
12	305
13	330
14	356
15	381
16	406
17	432
18	457
19	483
20	508



Client:	MaineDOT
Project:	ROUTE 24 OVER COBBOSSECONTEE STREAM
Subject:	Micropile Geotechnical Resistance

CALCULATIONS

Rock Socket Diameter (in) = 9
 Rock Socket Perimeter (ft) = 2.4
 Rock Socket Side Area (ft²/ft) = 2.4
 Bedrock STR Factored Unit Side Resistance (ksf) = 13.9
 Bedrock STR Factored Resistance per foot (kip/ft) = 32.7

Rock Socket Length in Bedrock (ft)	Bedrock Factored Geotechnical Resistance (kip)
1	33
2	65
3	98
4	131
5	163
6	196
7	229
8	261
9	294
10	327
11	359
12	392
13	425
14	457
15	490
16	523
17	555
18	588
19	620
20	653



Micropile Elastic Compression

Client: Stantec

Date: 21SEP2018

Project: Bridge Street Bridge over the Cobbosseecontee Stream, Gardiner, Maine

Computed by: JLL

Subject: Elastic Deformation of Abutment Micropiles

Checked by: EAF

ABUTMENT 1 ASSUMPTIONS & CALCULATIONS

- Corrosion assumed to reduce casing thickness by 1/16 inch.
- Piles are drilled micropiles with sections and properties shown (see attached figure also):

Pile Position	Pile Section	AE total (kip)	Length (ft)	AE/L (kip/in)
Back Row	A-A	640360	30	1779
	B-B	242274	35	577
Front Row	C-C	727309	30	2020
	D-D	329223	45	610

- Effective vertical stiffness (k_v) of piles are estimated as:

Pile Position	k_v (kip/in)
Back	436
Front	468

- Max. Service Limit State pile loads from H&A and Stantec pile group analyses are as follows:

	H&A	Stantec
Max. Compression (kip)	241	234
Max. Uplift (kip)	None	50

- Assuming that the max. compression load occurs at a front pile, the pile elastic compression is estimated to be 0.5 inch.
- Assuming that the max. uplift load occurs at a back pile, the pile elastic elongation is estimated to be 0.1 inch.

ABUTMENT 2 ASSUMPTIONS & CALCULATIONS

- No corrosion reduction in casing thickness.
- Piles are drilled micropiles with sections and properties shown (see attached figure also):

Pile Position	Pile Section	AE total (kip)	Length (ft)	AE/L (kip/in)
Back Row	E-E	718184	30	1995
	F-F	265648	29	763
Front Row	C-C	781759	30	2172
	D-D	329223	39	703

- Effective vertical stiffness (k_v) of piles are estimated as:

Pile Position	k_v (kip/in)
Back	552
Front	531

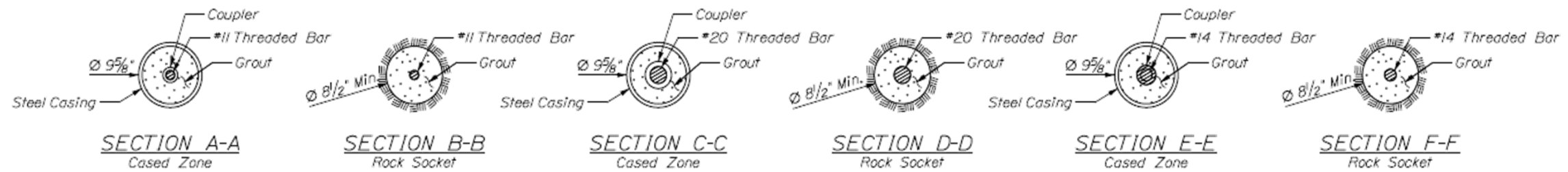
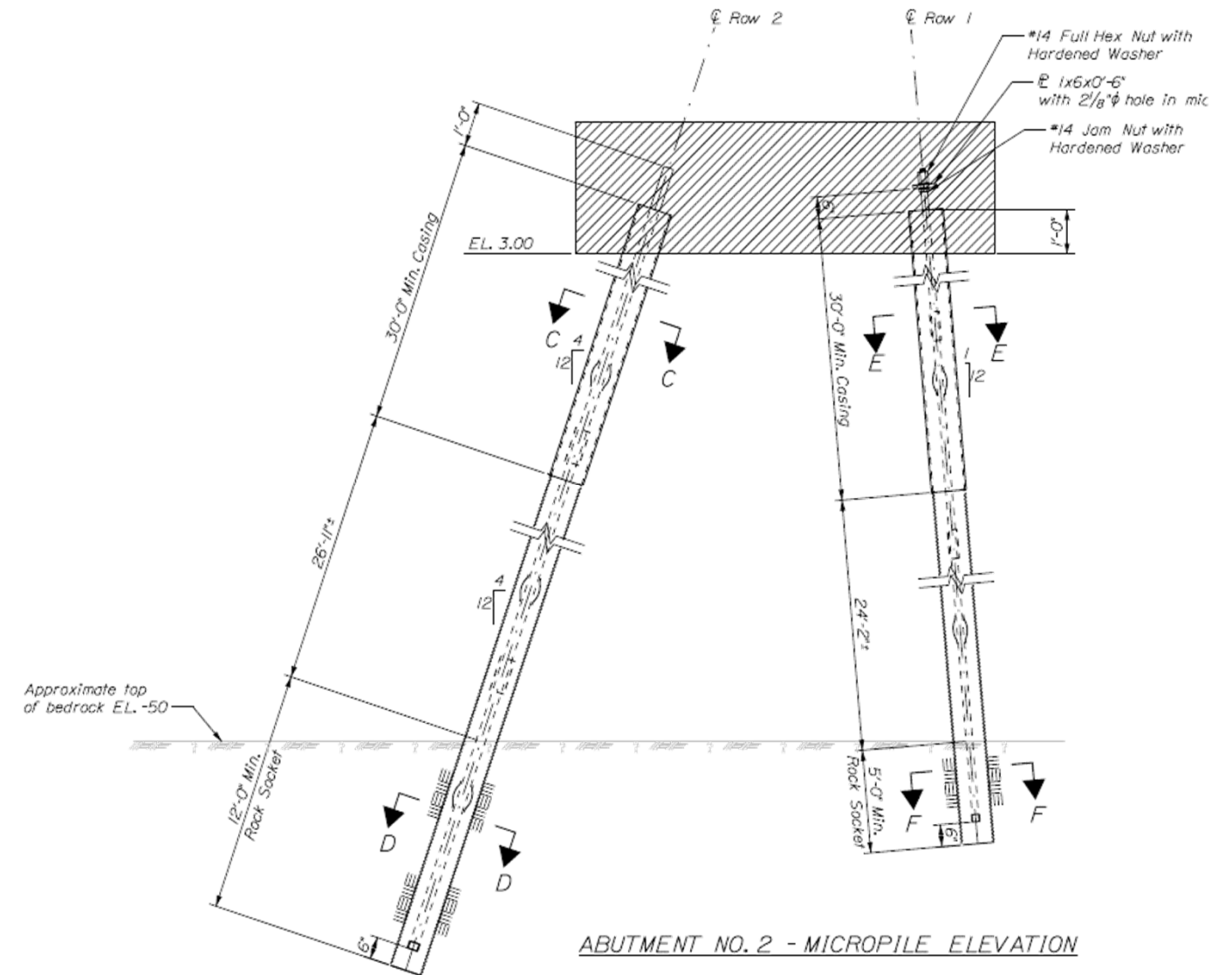
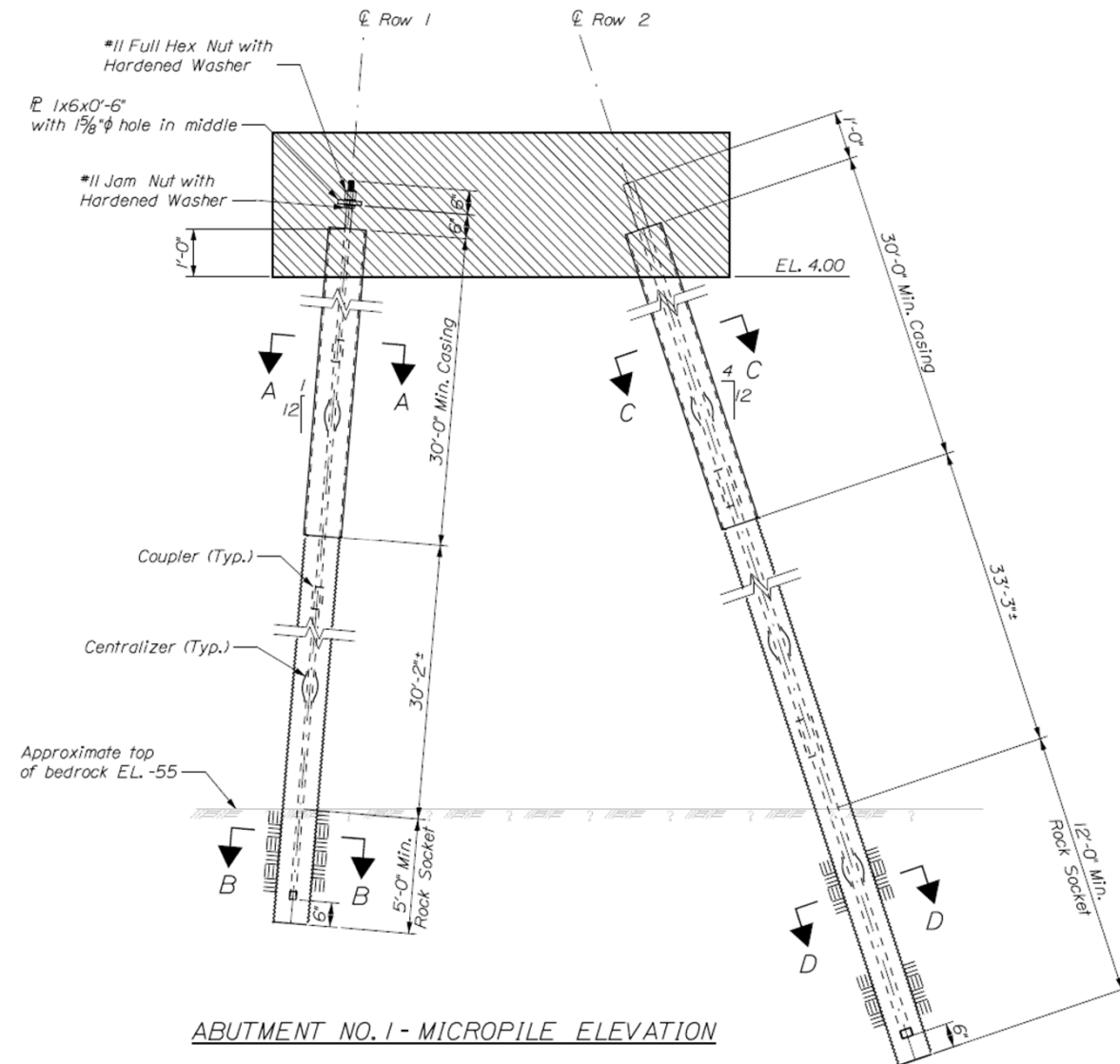
- Max. Service Limit State pile loads from H&A and Stantec pile group analyses are as follows:

	H&A	Stantec
Max. Compression (kip)	230	232
Max. Uplift (kip)	18	74

- Assuming that the max. compression load occurs at a front pile, the pile elastic compression is estimated to be 0.4 inch.
- Assuming that the max. uplift load occurs at a back pile, the pile elastic elongation is estimated to be 0.1 inch.

Notes:

- AE total is the sum of the steel AE and grout AE for each section.
- The effective vertical stiffness is calculated as $k_v = 1 / (1/k_{v1} + 1/k_{v2})$ where k_{v1} and k_{v2} are the AE/L of the sections making up the micropile.
- Pile loads are from H&A FB-MPier analyses dated 20SEP2018, and Stantec's Group evaluations dated 23SEP2018.
- Pile cross-sections from Stantec dated 25SEP2018 (see attached figure)



- Notes:
1. Pile cross-sections from Stantec dated 25SEP2018.

HALEY
ALDRICH

BRIDGE STREET BRIDGE OVER THE COBBOSSECONTEE STREAM
GARDINER, MAINE

PILE CROSS-SECTIONS
FOR ABUTMENT 1 AND 2
FROM STANTEC 25SEP2018

SEPTEMBER 2018

Lateral Soil Pressures

Client: Stantec

Date: 26-Sep-2018

Project: Bridge Street Bridge, Gardiner, Maine

Computed by: JLL

Subject: Lateral Earth Pressures

Checked by: EAF

ACTIVE LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 with interim revisions. Excerpt below.

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma [\sin^2 \theta \sin(\theta - \delta)]} \quad (3.11.5.3-1)$$

in which:

$$\Gamma = \left[1 + \frac{\sin(\phi'_f + \delta) \sin(\phi'_f - \beta)}{\sin(\theta - \delta) \sin(\theta + \beta)} \right]^2 \quad (3.11.5.3-2)$$

where:

- δ = friction angle between fill and wall taken as specified in Table 3.11.5.3-1 (degrees)
- β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
- θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
- ϕ'_f = effective angle of internal friction (degrees)

CALCULATIONS

					Coulomb	Rankine
ϕ'_f (deg)	θ (deg)	δ (deg)	β (deg)	Γ	k_a	k_a
32	90	24	0	2.9	0.27	0.31

Note: Rankine k_a is based on $\delta=0$ (no wall to soil friction). k_a should only be applied to effective earth pressures (i.e., do not apply to hydrostatic pressures).

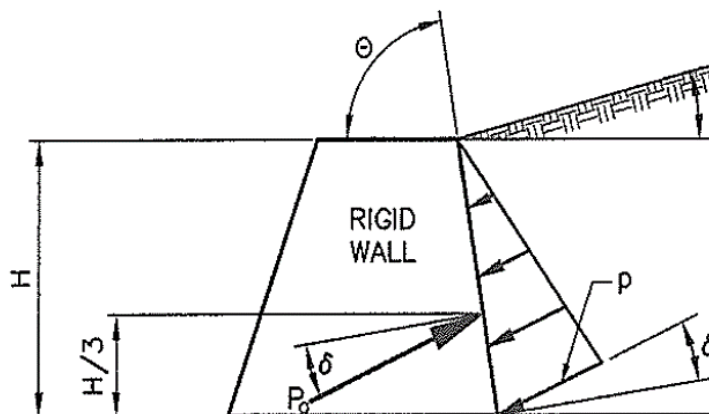


Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure

Client: Stantec

Project: Bridge Street Bridge, Gardiner, Maine

Computed by: JLL

Subject: Lateral Earth Pressures

Checked by: EAF

PASSIVE LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 with interim revisions. See plot below.

ϕ'_f (deg)	θ (deg)	δ (deg)	$-\delta/\phi'_f$	k_p	R	$R \cdot k_p$
32	90	24	-0.75	7.7	0.86	6.6

Note: $R \cdot k_p$ should only be applied to effective earth pressures (i.e., do not apply to hydrostatic pressures).

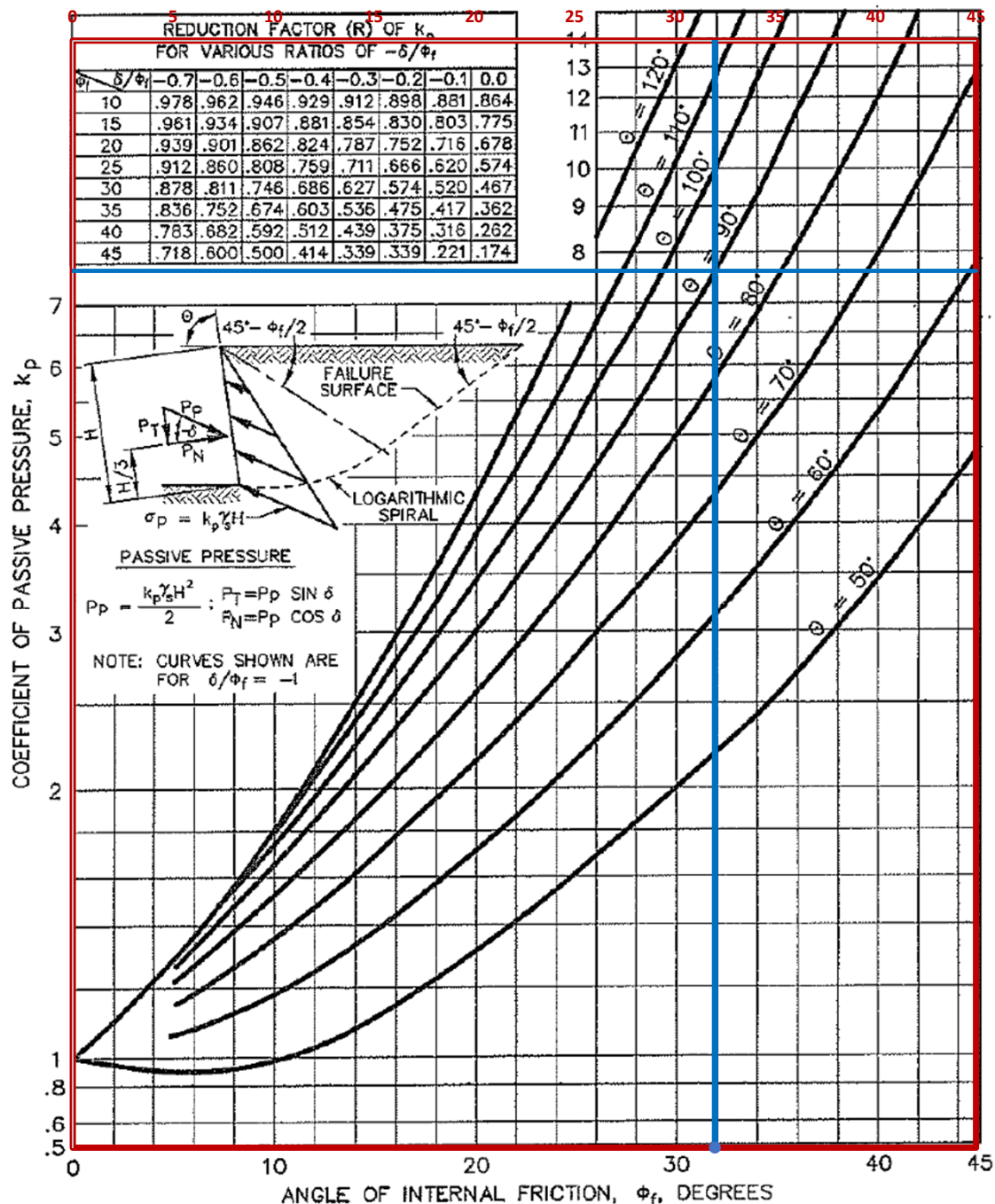


Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)

Client: Stantec

Date: 26-Sep-2018

Project: Bridge Street Bridge, Gardiner, Maine

Computed by: JLL

Subject: Lateral Earth Pressures

Checked by: EAF

SEISMIC LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 with interim revisions. Excerpts below.

$$K_{AE} = \frac{\cos^2(\phi - \theta_{MO} - \beta)}{\cos \theta_{MO} \cos^2 \beta \cos(\delta + \beta + \theta_{MO})} \times \left[1 + \frac{\sin(\phi + \delta) \sin(\phi - \theta_{MO} - i)}{\cos(\delta + \beta + \theta_{MO}) \cos(i - \beta)} \right]^{-2} \quad (A11.3.1-1)$$

where:

- K_{AE} = seismic active earth pressure coefficient (dim)
 γ = unit weight of soil (kcf)
 H = height of wall (ft)
 h = height of wall at back of wall heel considering height of sloping surcharge, if present (ft)
 ϕ_f = friction angle of soil (degrees)
 θ_{MO} = arc tan [$k_h/(1 - k_v)$] (degrees)
 δ = wall backfill interface friction angle (degrees)
 k_h = horizontal seismic acceleration coefficient (dim.)
 k_v = vertical seismic acceleration coefficient (dim.)
 i = backfill slope angle (degrees)
 β = slope of wall to the vertical, negative as shown (degrees)

Note that K_{AE} includes the static (k_a) and dynamic (Δk_{AE}) components of the lateral soil pressure. According to Section A11.3.1 of AASHTO LRFD, a reasonable approach for routine walls is to apply the combined resultant of the static and seismic force at the same location as the static earth pressure but no less than $h/3$. Assuming hydrodynamic effects are negligible, total soil unit weight should be used when calculating the static+seismic force using K_{AE} .

CALCULATIONS

ϕ'_f (deg)	PGA (g)	F_{PGA}	k_h (g)	k_v (g)	θ_{MO} (deg)	δ (deg)	β (deg)	i (deg)	K_{AE}
32	0.077	1.60	0.12	0.00	7.0	24	0.0	0.00	0.36

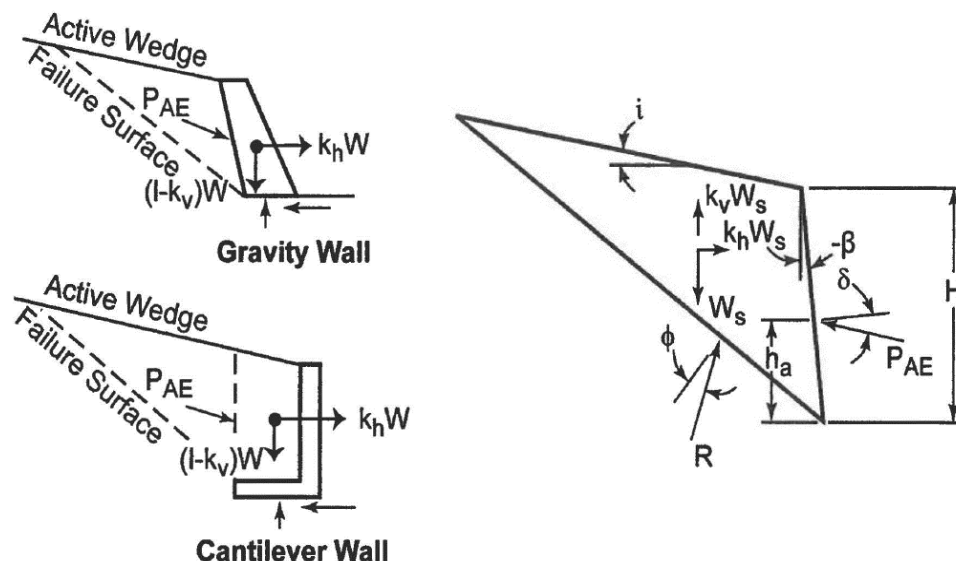


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams

A1 Diner Footing Bearing Resistance

Client Maine Department of Transportation

Date 23-Jan-18

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by MMB

Subject Bearing Resistance of Replacement Footings for A1 Diner

Checked by JLL

PROBLEM STATEMENT & OBJECTIVE

Calculate the Strength and Service Limit State bearing resistance of replacement footings for the A1 Diner.

EXECUTIVE SUMMARY

- | | | |
|--|-----|------|
| 1. The factored bearing resistance at the Strength Limit State is | 3.4 | ksf. |
| 2. The factored bearing resistance at the Service Limit State for 1 in. of settlement is | 1.8 | ksf. |

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

AVAILABLE INFORMATION

1. Plan set titled, "Gardiner Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream," dated August 2017.

ASSUMPTIONS

1. The vertical load eccentricity only applies in one direction (i.e., overturning moment only in one direction). The maximum eccentricity assumed is B/3 based on AASHTO Section 10.6.3.3.
2. Fully saturated soils beneath footing (area has history of flooding).
3. Bottom of footing No. 2 at El. 8 and Bottom of footing No. 3 at El. 6.
4. Subsurface conditions based on borings BB-GCS-401, BB-GCS-101, and BB-GCS-401A (boring logs are in the plan set).
5. Bottom of footing is between about 5.5 and 7 ft below ground surface - assume 5.5 ft in the calculations.
6. Footing sizes considered: 3 ft x 3 ft and 4 ft x 4 ft.
7. Footings will bear on loose granular fill material.
8. Soil properties for granular fill will be 115 pcf (unit weight) and 30 degrees (phi angle).
9. Full area will be considered for service limit state.
10. Fill consisting of generally SW and SM with blow counts of 3 and 6 from boring BB-GCS-401 and 2 and 8 from boring BB-GCS-101 were considered.

PROCEDURE FOR STRENGTH LIMIT STATE

AASHTO LRFD Equation 10.6.3.1.2a- Basic Formulation for Nominal Bearing Resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma} \quad \text{Equation 10.6.3.1.2a-1}$$

$$N_{cm} = N_c s_c i_c \quad \text{Equation 10.6.3.1.2a-2}$$

$$N_{qm} = N_q s_q d_q i_q \quad \text{Equation 10.6.3.1.2a-3}$$

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma \quad \text{Equation 10.6.3.1.2a-4}$$

- q_n = nominal strength limit state bearing resistance (ksf)
 RF = resistance factor from Table 10.5.5.2.2-1
 q_R = factored strength limit state bearing resistance (ksf) = $RF \times q_n$
 c = cohesion, taken as undrained shear strength (ksf)
 N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
 N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
 N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)
 γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

Client Maine Department of Transportation

Date 23-Jan-18

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by MMB

Subject Bearing Resistance of Replacement Footings for A1 Diner

Checked by JLL

D_f = footing embedment depth (ft)
 D_w = depth of water from ground surface (ft)
 B = footing width (ft)
 e_B = footing width eccentricity (ft) as specified in Section 10.6.3.3
 B' = effective footing width ($B - 2e$) (ft)
 L = footing length (ft)
 e_L = footing length eccentricity (ft) as specified in Section 10.6.3.3
 L' = effective footing length ($L - 2e$) (ft)
 C_{wq}, C_{wY} = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
 s_c, s_q, s_Y = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
 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 (dim).
 i_c, i_q, i_Y = load inclination factors

CALCULATION FOR STRENGTH LIMIT STATE

B = 3 ft
 e_B = 1.0 ft
 B' = 1.0 ft
 L = 3 ft
 e_L = 0 ft
 L' = 3 ft
 c = 0 ksf
 γ = 115 pcf
 ϕ = 30 degrees
 D_w = 0 ft
 D_f = 5.5 ft

$N_f = f(\phi)$ 3.00
 C_{wq} 0.5
 C_{wY} 0.5
 N_c 30.1
 S_c 1.20
 i_c 1
 N_{cm} 36.2
 N_q 18.4
 S_q 1.19
 d_q 1
 i_q 1
 N_{qm} 21.9
 N_Y 22.4
 S_Y 0.87
 i_Y 1
 N_{Ym} 19.4

q_n 7.5 ksf
 RF 0.45
 q_R 3.4 ksf

B = 4 ft
 e_B = 1.33 ft
 B' = 1.3 ft
 L = 4 ft
 e_L = 0 ft
 L' = 4 ft
 c = 0 ksf
 γ = 115 pcf
 ϕ = 30 degrees
 D_w = 0 ft
 D_f = 5.5 ft

$N_f = f(\phi)$ 3.00
 C_{wq} 0.5
 C_{wY} 0.5
 N_c 30.1
 S_c 1.20
 i_c 1
 N_{cm} 36.2
 N_q 18.4
 S_q 1.19
 d_q 1
 i_q 1
 N_{qm} 21.9
 N_Y 22.4
 S_Y 0.87
 i_Y 1
 N_{Ym} 19.4

q_n 7.7 ksf
 RF 0.45
 q_R 3.5 ksf

Client Maine Department of Transportation

Date 23-Jan-18

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by MMB

Subject Bearing Resistance of Replacement Footings for A1 Diner

Checked by JLL

PROCEDURE FOR SERVICE LIMIT STATE

10.6.2.4.2 - Settlement of Footings on Cohesionless Soils

$$S_e = \frac{q_0(1 - \nu^2)\sqrt{A'}}{144 E_s \beta_z} \quad \text{Equation 10.6.2.4.2-1}$$

where

 q_0 = applied vertical stress (ksf) A' = effective area of footing (ft^2) E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of insitu or laboratory tests (ksi) β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim) ν = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of ν are not available from the results of insitu or laboratory tests (dim) S_e = elastic settlement (ft)

CALCULATION FOR SERVICE LIMIT STATE

$$\begin{aligned} A' &= 9 \text{ ft}^2 \\ E_s &= 1 \text{ ksi, Table C10.4.6.3-1} \\ \nu &= 0.3 \\ \beta_z &= 1.08 \text{ Table 10.6.2.4.2-1} \\ S_e &= 0.042 \text{ ft} \end{aligned}$$

$$\begin{aligned} A' &= 16 \text{ ft}^2 \\ E_s &= 1 \text{ ksi, Table C10.4.6.3-1} \\ \nu &= 0.3 \\ \beta_z &= 1.08 \text{ Table 10.6.2.4.2-1} \\ S_e &= 0.042 \text{ ft} \end{aligned}$$

$$q_0 = 2.4 \text{ ksf}$$

$$q_0 = 1.8 \text{ ksf}$$

Note: the estimated E is about 0.5 ksi based on an average blow count of about 5 $(8+2+3+6)/4$, use 1 ksi for calculations.

Compare to presumptive bearing resistances in AASHTO LRFD:

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance C10.6.2.6.1

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120-200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60-80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30-50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16-24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16-24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16-24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12-20	14
	Medium dense to dense	8-14	10
	Loose	4-12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8-12	8
	Medium dense to dense	4-8	6
	Loose	2-6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6-10	6
	Medium dense to dense	4-8	5
	Loose	2-4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6-10	6
	Medium dense to dense	4-8	5
	Loose	2-4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6-12	8
	Medium dense to dense	2-6	4
	Loose	1-2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4-8	6
	Medium stiff to stiff	2-6	3
	Soft	1-2	1

$$q_0 = 5 \text{ ksf}$$

Use lesser of service limit state bearing pressure (calculation vs. presumptive).

Client	Maine Department of Transportation	Date	23-Jan-18
Project	Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream	Computed by	MMB
Subject	Bearing Resistance of Replacement Footings for A1 Diner	Checked by	JLL

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The factored bearing resistance for the strength limit state is 3.4 ksf

Service Limit State

The factored bearing resistance for the service limit state is 1.8 ksf for a 1 in. settlement.

Site Retaining Wall Footing Bearing Resistance

Client Maine Department of Transportation

Date 22-May-18

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by MMB

Subject Bearing Resistance for Retaining Wall

Checked by EAF

PROBLEM STATEMENT & OBJECTIVE

Calculate the Strength and Service Limit State bearing resistance for the retaining wall.

EXECUTIVE SUMMARY

- | | | |
|--|-----|------|
| 1. The factored bearing resistance at the Strength Limit State is | 1.2 | ksf. |
| 2. The factored bearing resistance at the Service Limit State for 1 in. of settlement is | 1.0 | ksf. |

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

AVAILABLE INFORMATION

1. Plan set titled, "Gardiner Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream," dated August 2017.
2. Haley & Aldrich boring log BB-GCS-103 dated November 2015.

ASSUMPTIONS

1. The vertical load eccentricity only applies in one direction (i.e., overturning moment only in one direction).
The maximum eccentricity assumed is B/3 based on AASHTO Section 10.6.3.3.
2. Fully saturated soils beneath footing.
3. Wall begins at Station 2+24.11 and ties into existing wall at Station 2+63.11, wall length is 39 ft.
4. Subsurface conditions based on boring BB-GCS-103.
5. Bottom of footing is between about 2 and 2.3 ft below ground surface - assume 2 ft in the calculations.
6. Footing sizes considered: 2 ft (width) by 39 ft (length).
7. Footings will bear at El. 31.1 (below 4 in. concrete leveling pad) on fill material.
8. Soil properties for fill will be 115 pcf (unit weight) and 30 degrees (phi angle).

PROCEDURE FOR STRENGTH LIMIT STATE

AASHTO LRFD Equation 10.6.3.1.2a- Basic Formulation for Nominal Bearing Resistance

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma} \quad \text{Equation 10.6.3.1.2a-1}$$

$$N_{cm} = N_c s_c i_c \quad \text{Equation 10.6.3.1.2a-2}$$

$$N_{qm} = N_q s_q d_q i_q \quad \text{Equation 10.6.3.1.2a-3}$$

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma \quad \text{Equation 10.6.3.1.2a-4}$$

q_n = nominal strength limit state bearing resistance (ksf)

RF = resistance factor from Table 10.5.5.2.2-1

q_R = factored strength limit state bearing resistance (ksf) = RF x q_n

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

Client Maine Department of Transportation

Date 22-May-18

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by MMB

Subject Bearing Resistance for Retaining Wall

Checked by EAF

D_f = footing embedment depth (ft)
 D_w = depth of water from ground surface (ft)
 B = footing width (ft)
 e_B = footing width eccentricity (ft) as specified in Section 10.6.3.3
 B' = effective footing width ($B - 2e$) (ft)
 L = footing length (ft)
 e_L = footing length eccentricity (ft) as specified in Section 10.6.3.3
 L' = effective footing length ($L - 2e$) (ft)
 C_{wq}, C_{wY} = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)
 s_c, s_q, s_Y = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)
 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 (dim).
 i_c, i_q, i_Y = load inclination factors

CALCULATION FOR STRENGTH LIMIT STATE

B = 2 ft
 e_B = 0.7 ft
 B' = 0.7 ft
 L = 39 ft
 e_L = 0 ft
 L' = 39 ft
 c = 0 ksf
 γ = 115 pcf
 ϕ = 30 degrees
 D_w = 0 ft
 D_f = 2 ft

 $N_f = f(\phi)$ 3.00

 C_{wq} 0.5

 C_{wY} 0.5

 N_c 30.1

 S_c 1.01

 i_c 1

 N_{cm} 30.4

 N_q 18.4

 S_q 1.01

 d_q 1

 i_q 1

 N_{qm} 18.6

 N_Y 22.4

 S_Y 0.99

 i_Y 1

 N_{Ym} 22.2

 q_n 2.6 ksf

 RF 0.45

 q_R 1.2 ksf

Client Maine Department of Transportation

Project Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Subject Bearing Resistance for Retaining Wall

SERVICE LIMIT STATE

Presumptive bearing resistances in AASHTO LRFD:

10.6.2.6—Bearing Resistance at the Service Limit State
10.6.2.6.1—Presumptive Values for Bearing Resistance C10.6.2.6.1

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

assume Service Limit State bearing resistance

$q_0 = 1$ ksf for Service Limit state bearing resistance based on possible fill samples:

- Loose coarse to medium sand and with little gravel (SW, SP)
- Loose fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)
- Loose fine sand, silty or clayey medium to fine sand (SP, SM, SC)
- Soft inorganic silt, sandy or clayey silt, varbed silt-clay-fine sand (ML, MH)

		CALCULATIONS		File No.	42223-004
				Sheet	4 of 4
Client	Maine Department of Transportation			Date	22-May-18
Project	Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream			Computed by	MMB
Subject	Bearing Resistance for Retaining Wall			Checked by	EAF
CONCLUSIONS AND RECOMMENDATIONS					
Strength Limit State					
The factored bearing resistance for the strength limit state is				1.2	ksf
Service Limit State					
The factored bearing resistance for the service limit state is				1.0	ksf for a 1 in. settlement.
G:\PROJECTS\42223 - gardiner bridges\004 - Bridge Street\Calculations\Retaining Wall\[2018-0522-HAI-Retaining Wall Bearing R v 1.0					

Footing Sliding Resistance

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, $\tan \delta$ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

 select
0.45

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f .

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Stair Foundation Resistance

File No.:	42223-004
Sheet:	1 of 2
Date:	20-Mar-2018
Computed by:	MMB
Checked by:	JLL

Client:	Maine Department of Transportation
Project:	Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream
Subject:	Estimated Geotechnical Axial Resistance for Sonotube-Formed Foundations

PROBLEM STATEMENT AND OBJECTIVE

Calculate the geotechnical axial resistance of the 2-ft diameter Sonotube-formed concrete piers proposed to support the A1 Diner stairway replacement.

AVAILABLE INFORMATION

1. Email from Stantec dated 14 March 2018 containing foundation sizes and service limit state load.
2. Plan set titled, "Gardiner Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream," dated August 2017

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

ASSUMPTIONS

1. Concrete piers formed using 2-ft diameter Sonotubes and embedded a minimum of 5 ft below ground surface.
2. Top of concrete pier will be at El. 12.5, final grade around the piers will be at El. 12.
3. Subsurface conditions based on the following borings closest to the pier locations: BB-GCS-401 and BB-GCS-401A.
4. Concrete piers will bear on granular (sand) fill with raw SPT N of 5 and 3 blows/ft.
5. Bearing soils are fully-saturated and have total unit weight of 120 pcf.
6. Side friction between Sonotube-formed concrete pier and soil is negligible, consider tip resistance only.
7. Geotechnical resistance calculated using drilled shaft design methods.
8. Vertical load eccentricity is assumed to be zero.

CALCULATIONS

10.8.3.5 - Nominal Axial Compression Resistance of Single Drilled Shaft

The factored resistance of drilled shafts, R_R , shall be taken as

$$R_R = \phi R_n = \phi_{qp} R_p \quad 10.8.3.5-1$$

in which

$$R_p = q_p A_p \quad 10.5.3.5-2$$

where

R_p = nominal shaft tip resistance (kips)

ϕ_{qp} = resistance factor for tip resistance specified in Table 10.5.5.2.4-1

q_p = unit tip resistance (ksf)

A_p = area of shaft tip (ft^2)

Client:	Maine Department of Transportation
Project:	Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream
Subject:	Estimated Geotechnical Axial Resistance for Sonotube-Formed Foundations

10.8.3.5.2c - Tip Resistance

The nominal unit tip resistance q_p , in ksf, for drilled shafts in cohesionless soils by the method described in Brown et al. (2010) shall be taken as:

$$\text{If } N_{60} \leq 50, \text{ then } q_p = 1.2N_{60} \quad 10.8.3.5.2c-1$$

where

N_{60} = average SPT blow count (corrected only for hammer efficiency) in the design zone under consideration (blows/ft).

The value of q_p in Equation 10.8.3.5.2c-1 should be limited to 60 ksf, unless greater values can be justified through load test data.

$N_{60} =$	4.5	blows/ft
$q_p =$	5.40	ksf
$A_p =$	3.14	ft ²
$R_p =$	17.0	kips
$\phi_{qp} =$	0.5	from Table 10.5.5.2.4-1 in AASHTO LRFD

$$R_R = \phi R_n = \phi_{qp} R_p$$

$$\phi_{qp} R_p = 8.5 \quad \text{kips}$$

$$R_R = 8.5 \quad \text{kips}$$

CONCLUSIONS

The factored resistance for the concrete pier is estimated to be about 8.5 kips.

Extend the sonotube to minimum embedment of 6 ft.

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Sheet:	1 of 3
Date:	20-Apr-2018
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Client:	Maine Department of Transportation
Project:	Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream
Subject:	Geotechnical Service Limit State for Sonotube-Formed Concrete Pier Foundations

PROBLEM STATEMENT AND OBJECTIVE

Estimate the settlement the 2-ft diameter Sonotube-formed concrete support piers proposed to support the A1 Diner stairway replacement under the service limit state load

AVAILABLE INFORMATION

1. Email from Stantec dated 14 March 2018 containing concrete pier dimensions and service load.
2. Plan set titled, "Gardiner Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream," dated August 2017 created by Maine Department of Transportation.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.

ASSUMPTIONS

1. Concrete piers formed using 2-ft diameter Sonotubes and embedded a minimum of 5 ft below ground surface.
2. Top of concrete piers will be at El. 12.5, ground surface is at El. 12. Tip elevation of the sonotube is El. 7.
3. Subsurface conditions based on the following borings closest to the pier locations: BB-GCS-401 and BB-GCS-401A.
4. Fully saturated soil with unit weight of 120 pcf.
5. Assume soils below sonotube tip are loose to medium dense (say raw SPT N value is 10).
6. Contact pressure is 8 kips / [(PI)*(2ft)^2*(1/4)] = 2.546 ksf which includes weight of 5 ft long concrete pier.
7. Foundation length is 2 ft and width is 2 ft (for Hough method calculation on second page).
8. For the elastic half space method calculation on first page, use A' as the area of both piers due to the close proximity of the two proposed concrete piers.
9. Compression of Glacial deposits below the fill is assumed negligible.

CALCULATIONS

10.6.2.4.2 - Settlement of Footings on Cohesionless Soils

$$S_e = \frac{q_0 (1 - \nu^2) \sqrt{A'}}{144 E_s \beta_z} \quad \text{Equation 10.6.2.4.2-1}$$

where

- q_0 = applied vertical stress (ksf)
 A' = effective area of footing (ft²)
 E_s = Young's Modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of insitu or laboratory tests (ksi)
 β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)
 ν = Poisson's Ratio, taken as specified in Article 10.4.6.2 if direct measurements of ν are not available from the results of insitu or laboratory tests (dim)
 S_e = elastic settlement (ft)

- A' = 6.28 ft² (total area of the two piers that are close together)
 E_s = 0.8 ksi, Table C10.4.6.3-1, based on (0.097N₁)₆₀
 ν = 0.3
 β_z = 1.13 Table 10.6.2.4.2-1
 q_0 = 2.55 ksf
 S_e = 0.043 ft
 0.5 in

Client: Maine Department of Transportation

Date: 20-Apr-2018

Project: Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by: MMB

Subject: Geotechnical Service Limit State for Sonotube-Formed Concrete Pier Foundations

Checked by: JLL

AASHTO Empirical Method (per 10.6.2.4.2 - Hough Method)
for COHESIONLESS SOIL

$$S = \sum_{i=1}^n \Delta H_i \quad \text{where} \quad \Delta H_i = H_c \frac{1}{C'} \log \frac{\sigma_o' + \Delta \sigma_v}{\sigma_o'}$$

n = number of soil layers within zone of stress influence of the footing. Divide subsurface profile into layers to a depth of about 3B below footing. The maximum layer spreadsheet should be about 10 ft.

ΔH_i = elastic settlement of layer i (ft)

H_c = initial height of layer i (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1 (below);

note - use CORRECTED $(N_1)_{60}$

σ_o' = initial vertical effective stress at the midpoint of layer i (ksf)

$\Delta \sigma_v$ = increase in vertical stress at the midpoint of layer i (ksf)

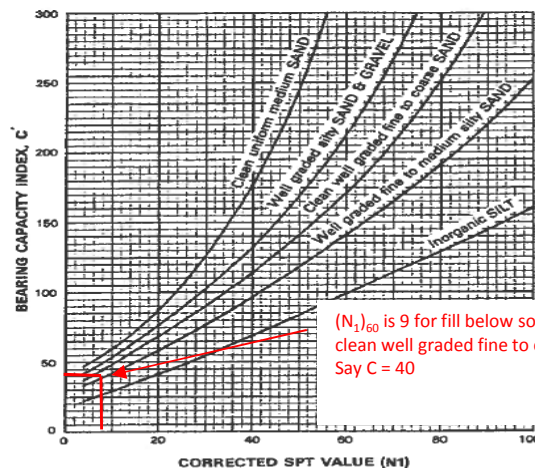
Total Unit Weight of Soil, γ_t =	120	pcf
Unit Weight of Water, γ_{H2O} =	62.4	pcf
Footing Depth BGS, D_f =	5	ft
Groundwater Depth BGS, D_{H2O} =	0	ft
Footing Length, L =	2.0	ft
Footing Width, B =	2.0	ft
Contact Pressure, q =	2,546	psf

Calc. tot. stress at base of ftg. =	600	psf
Calc. water pressure at base of ftg. =	312	psf
Calc. eff. stress at base of ftg., P_o =	288	psf
Calc. net fnd. pressure, D_p =	2,258	psf

Sublayer					N-value (bpf)	C'	σ_o'	$\Delta \sigma_v^*$	ΔH_i
Top	Bottom	Midpoint (below footing)	Midpoint (bgs)	Thickness					
0	5.5	2.8	7.8	6	10	40	446	451	0.042

* $\Delta \sigma_v$ is based on 2V:1H load distribution

Settlement (ft) =	0.04
Settlement (in) =	0.50



$(N_1)_{60}$ is 9 for fill below sonotube. Assume clean well graded fine to coarse sand. Say $C = 40$

Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

Figure 10.6.2.4.2-1—Bearing Capacity Index versus Corrected SPT (modified from Cheney and Chassie, 2000, after Hough, 1959)



CALCULATIONS

File No.: 42223-004

Sheet: 3 of 3

Client: Maine Department of Transportation

Date: 20-Apr-2018

Project: Gardiner, Kennebec County, Bridge Street Bridge over Cobbosseecontee Stream

Computed by: MMB

Subject: Geotechnical Service Limit State for Sonotube-Formed Foundations

Checked by: JLL

CONCLUSIONS

The 2 ft diameter concrete piers are estimated to settle less than 0.5 in. under the 8 kips per pier service limit state load.