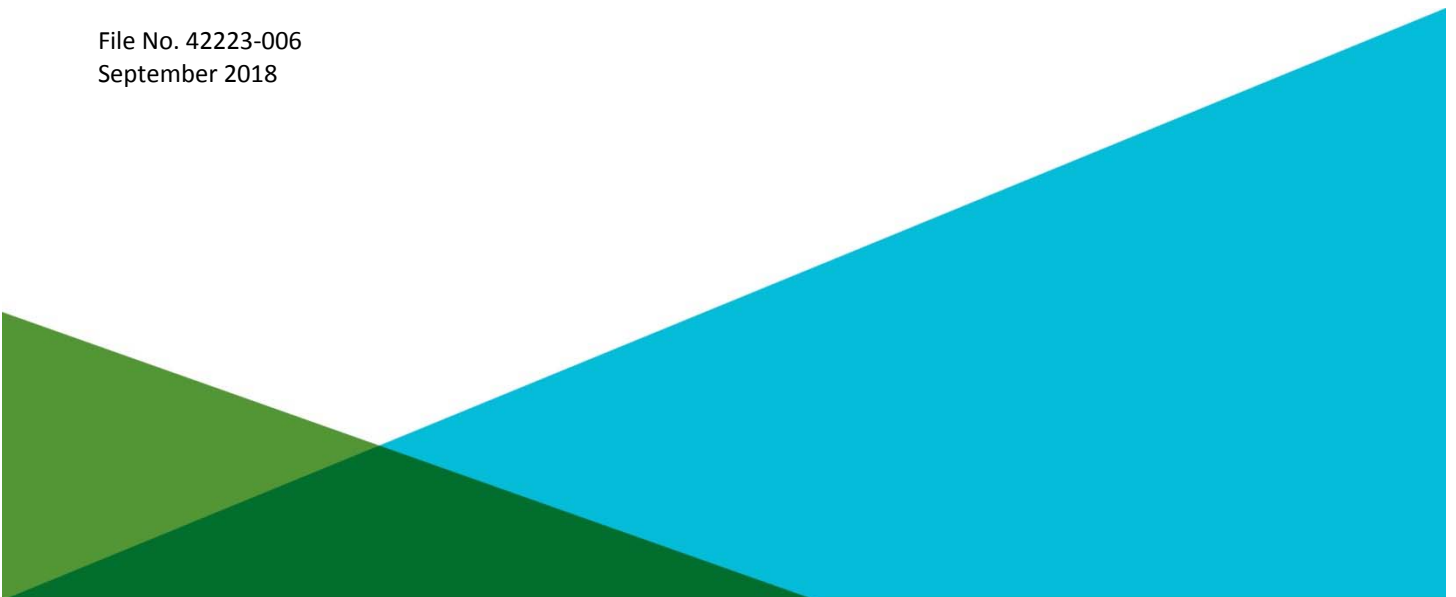


**GEOTECHNICAL DESIGN REPORT – 100% SUBMISSION
MULTI-USE TRAIL BRIDGE OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18736.10**

by Haley & Aldrich, Inc.
Portland, Maine

for Stantec
Scarborough, Maine

File No. 42223-006
September 2018





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

Subject: Geotechnical Design Report – 100% Submission
Multi-Use Trail Bridge over Cobbosseecontee Stream
Gardiner, Maine
MaineDOT WIN 18736.10

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 new bridge. 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 8 February 2018, and your subsequent authorization.

Project Understanding

A new bridge is proposed to carry the multi-use trail over the Cobbosseecontee Stream, as shown on Figure 1, Project Locus, and Figure 2, Boring Location Plan and Interpretive Subsurface Profile. The new bridge is proposed to be a 76.5-ft long single span structure, located approximately 40 ft west (upstream) of the existing Maine Avenue bridge.

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).

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. 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 primarily of schist with a lesser amount of pegmatitic granite. The above referenced map indicates a fault zone located south of the multi-use trail 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 superstructure.

Subsurface Explorations

PRELIMINARY DESIGN PHASE EXPLORATIONS BY

Haley & Aldrich conducted a preliminary phase geotechnical exploration program at the multi-use trail bridge site. A total of two test borings (BB-GCS-501 and BB-GCS-502) were completed to identify subsurface conditions along the proposed bridge alignment.

During an earlier phase of drilling at the adjacent Maine Avenue bridge, two test borings (BB-GCS-203 and BB-GCS-204) were performed for environmental purposes. Those test borings are located in the immediate vicinity of the multi-use trail bridge, and the test boring logs are 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, Route 24 over Cobbosseecontee Stream, Gardiner, Maine, MaineDOT WIN 18736.00, Bridge No. 3098," dated 8 June 2017 (Environmental Report).

Boring locations were laid out in the field by Haley & Aldrich by taping distances from existing site features. "As-drilled" locations and ground surface elevations at boring locations were surveyed by MaineDOT 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 track-mounted (BB-GCS-501 and BB-GCS-502) or truck-mounted Mobile B53 drill rig (BB-GCS-203 and BB-GCS-204). Test borings were drilled to depths ranging from 14.5 ft to 68 ft below ground surface (BGS).

Geotechnical test borings (BB-GCS-501 and BB-GCS-502) 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 drilling 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 geotechnical test borings were extended to bedrock and each sampled 9 to 10 ft of rock core.

Environmental test borings (BB-GCS-203 and BB-GCS-204) 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 below the groundwater table or in natural soil, whichever was shallower.

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 used for the rope and cathead/safety hammer system and a calculated hammer efficiency of 0.743 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.

Samples of the in-situ fill soils collected during both the geotechnical and environmental explorations were screened in the field with a photoionization detector (PID) to assess the presence of volatile organic compounds (VOCs). PID readings in the in-situ fill soils samples in borings BB-GCS-501 and BB-GCS-502 ranged from 0 to 0.7 ppm. No visual or olfactory signs of contamination/environmentally impacted soils were observed in these two borings. There were two instances of elevated PID readings in boring BB-GCS-203, specifically between 12 and 17 ft BGS. Refer to individual boring logs provided in Appendix A for actual PID readings. The elevated PID readings encountered in boring BB-GCS-203 are discussed further in the Environmental Report.

The two geotechnical borings sampled 9 to 10 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 and bedrock. Refer to Table I for a summary of the soil units and encountered thicknesses at each test boring location. Refer to Figure 2 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 DESCRIPTIONS

Unit	Approximate Range in Encountered Thickness (ft)	Generalized Description
Fill	13.9 to >18.0	Loose to very dense silty SAND, trace gravel (SM); SAND, trace to some silt and gravel (SP, SP-SM, SW-SM, SW); sandy GRAVEL, trace silt (GP, GW) Medium stiff to stiff fine sandy SILT, trace gravel (ML) <u>The fill contained frequent cobbles, boulders and wood</u> ; occasionally contained glass, asphalt, felt, sawdust, coal, ash, brick and concrete. Elevated PID readings were detected in BB-GCS-203 near/slightly above the groundwater level <i>(encountered in all test borings)</i>
Alluvial Deposit	13.0 to 19.5	Loose to very dense gravelly SAND or fine SAND, trace to little silt (SW, SP, SP-SM, SM); sandy GRAVEL (GW, GM) <u>The alluvial deposit contained cobbles and boulders</u> <i>(encountered in borings BB-GCS-501 and BB-GCS-502)</i>
Marine Deposit	>0.6 to 7.7	Loose silty fine SAND (SM); loose fine SAND, variable gravel, some medium sand, trace silt (SP); medium dense sandy GRAVEL, trace silt (GW) Very loose fine sandy SILT, trace clay, contained fine sand layers (ML) The full thickness of deposit was not penetrated in BB-GCS-204 <i>(encountered in borings BB-GCS-204, BB-GCS-501, BB-GCS-502)</i>

Glacial Till	6.5 to 15.5	Dense to very dense silty SAND, some to little gravel (SM); SAND, some to trace silt and gravel (SW, SW-SM) The glacial till was loosely bonded to well bonded and contained frequent cobbles and boulders <i>(encountered in borings BB-GCS-501 and BB-GCS-502)</i>
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BEDROCK

Bedrock was cored in boring BB-GCS-501 and BB-GCS-502. The top of the bedrock surface ranged from approximately 40.5 to 57.7 ft BGS (El. -45.1 to El. -26.1). A summary of top of bedrock elevations is provided below.

Substructure	Test Boring	Approx. Elev. Top of Bedrock (ft, NAVD 88)	Horizontal Distance between Test Borings (ft)	Approx. Slope of Rock Surface
Abutment 1	BB-GCS-501	-45.1	100	1V:5H
Abutment 2	BB-GCS-502	-26.1		

Bedrock encountered at the site consisted of moderately hard to very hard SCHIST, and hard pegmatitic GRANITE.

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. long divided by the total length of the core run. RQD values for bedrock encountered at the site ranged between 25 and 92 percent, this correlates to a Rock Mass Quality rating of very poor to excellent.

Substructure	Test Boring	Length of Rock Core Sampled (ft)	Range in RQD (%)	Average RQD (%) ¹
Abutment 1	BB-GCS-501	10.0	88 to 92	90
Abutment 2	BB-GCS-502	9.3	25 to 81	56

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:

Q1.1 (MLW): El. 1.9
Q1.1 (MHW): El. 3.9
Q50: El. 7.5
Q100: El. 8.2

Laboratory Testing Program

PRELIMINARY DESIGN PHASE TESTING PROGRAM

A preliminary laboratory testing program was undertaken on soil samples collected during the field investigation 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 R.W. Gillespie & Associates, Inc. of Saco, Maine. Geotechnical laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures.

The preliminary phase testing program included six grain size analyses (sieve only, no hydrometer). A summary of laboratory test results is provided below.

Test Boring No.	Soil Sample No.	Sample Depth (ft, BGS)	Strata	Percent Finer than #200 Sieve (%)	USCS Classification
BB-GCS-501	2D	4.0 to 6.0	Fill	13.0	SM
BB-GCS-501	8D	20.0 to 22.0	Alluvial Deposit	7.2	GW
BB-GCS-501	9D	25.0 to 27.0	Alluvial Deposit	19.2	SM
BB-GCS-502	4D	6.0 to 8.0	Fill	20.2	SM
BB-GCS-502	10D	20.0 to 22.0	Alluvial Deposit	12.6	GM
BB-GCS-502	12D	30.0 to 32.0	Marine Deposit	66.7	ML

Laboratory test results are provided in Appendix C.

FINAL DESIGN PHASE TESTING PROGRAM

Based on the extent and quality of the laboratory data collected during the preliminary design phase, Haley & Aldrich determined that additional, design phase laboratory testing was not needed to finalize geotechnical and foundation support requirements for the proposed bridge.

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 D.

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 was 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 D, Calculations, for supporting documentation.

APPROACH EMBANKMENTS

Settlement

Minimal (less than 1 ft) raises-in-grade are planned at the approach embankments. Given the proposed raises-in-grade, and the nature and consistency of the fill and naturally-deposited soils that are present at the site, we anticipate that the resulting magnitude of settlement will be minimal, will result from elastic compression of in-situ soils, and 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. 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 (longitudinal to the bridge) at both abutments. Evaluations were not performed transverse to the bridge since there is no approach embankment.

Soil and rock material and strength properties used in the global stability evaluations were based on the results of laboratory testing and our experience, and are summarized below.

	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
RipRap	140	45	0
Granular Borrow	125	32	0
Existing Fill	115	30	0
Alluvial Deposits	115	30	0
Marine Deposits	115	29	0
Glacial Till/Weathered Bedrock	130	36	0

The minimum calculated static factor of safety from our evaluations is 1.5. 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 minimum calculated factor of safety under pseudo-static earthquake loading from our evaluations is 1.3, 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) =	0.123
Horizontal Response Spectral Acceleration, S_{D5} (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 using a simplified liquefaction evaluation 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 5.98; see below) using the MSF (equal to 1.79, 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 5.98) 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 El. 3.9 (Q1.1), 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 1.1 to 7.3. 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 D for plots of factor of safety.

FOUNDATION SUPPORT ALTERNATIVES

As shown on the interpretive subsurface profile (Figure 2), the subsurface conditions based on the borings primarily consist of, in order of increasing depth below ground surface, man-placed fill, alluvial deposits, marine deposits, glacial till, and bedrock. The alluvial deposits, marine deposits, 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 most feasible:

- Spread footings bearing on alluvial deposits: The top of the alluvial deposit was encountered at El. -3 to El. -2, or approximately 15 to 17 ft below ground surface, at the proposed abutment locations. Therefore footings should bear at approximately El. -3 or lower, approximately 7 ft below the Q1.1 river level.
- Piles driven to refusal on bedrock: Piles would be approximately 40 to 60 ft long and driven through the overburden soils, including 15 to 17 ft of existing fill (see construction considerations below) and 6 to 16 ft of dense glacial till. Battered piles to resist lateral loads may be needed.
- Micropiles socketed into bedrock: A wide range of axial micropile capacities are achievable, depending on the micropile diameter (7 to 9 in. is typical) and rock socket length. A permanent steel casing is typically extended to bedrock (El. -45.1 to El. -26.1) and the rock socket length would be selected based on required axial and lateral resistance. Lateral resistance of plumb/vertical micropiles is typically relatively small, but rock sockets may slightly increase lateral load resistance. If additional lateral resistance is required, battered micropiles may be used to resist lateral loads.

Based on discussions with Stantec and the Department during the preliminary design phase, spread footings were selected to support the new bridge abutments. Geotechnical engineering evaluations and design recommendations for spread footings bearing on alluvial deposits are summarized in the following sections of this report. Additional evaluations for piles driven to refusal on bedrock and rock-socketed micropiles to support the bridge were not conducted during final design phase.

SPREAD FOOTINGS

Specific foundation design recommendations follow. The evaluations assume that the slope in front of the abutment is graded at 1.75 horizontal to 1 vertical (1.75H:1V), and that 4 ft of heavy riprap is placed on the face of the slope.

- Bearing Resistance
 - For the service limit state, footings should be designed such that footing contact pressures do not exceed 3.5 kips per square foot (ksf). At this pressure, it is estimated that settlement of footings bearing on alluvial deposits will be less than 1 in.

Note that the service limit bearing resistance was calculated as the lesser of the following: 1) the pressure that results in 1 in. of settlement; and 2) the pressure that results in a factor of safety

against global stability of 1.5. In this case, the global stability case controlled selection of the bearing resistance.

- Based on loading information provided by Stantec, for the strength limit state, minimum 12-ft wide footings should be designed for a factored bearing resistance of 5.0 ksf, using a resistance factor of 0.45.
- Based on loading information provided by Stantec, for the extreme event limit state, minimum 12-ft wide footings should be designed for a factored bearing resistance of 9.0 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 should be ignored to account for the presence of a slope in front of the abutment footings.
- Bearing resistance values stated above are based on the assumption that proposed footing subgrades will be prepared in-the-dry. The design resistances are also based on the assumption that all debris, topsoil, organic material, and existing fill (if encountered) will be removed from beneath the proposed footings and replaced with approved granular material or additional sub-footing concrete.

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.

In addition, walls should be designed for a live load surcharge equivalent to the earthfill height of 1 ft. 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 0.36 be used for design. Note that this soil pressure includes both the static and seismic lateral earth loads.

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

Design of temporary works, including temporary earth support system(s) is 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.

In order to minimize disturbance to the prepared footing subgrades, we recommend that the excavation support system (i.e., sheeting) remain in place after the substructures have been constructed. It is our opinion that removal of the systems could cause disturbance to the alluvial soils which could result in settlement of the footings.

SUBGRADES FOR BRIDGE FOOTINGS

Existing in-situ fill soils are anticipated to be present to a depth of 15 to 17 ft BGS (El. -3 to El. -2). These soils, if encountered below design footing bearing level, should be removed within the limits of the excavation support system and replaced with approved granular material or additional sub-footing concrete, or the bearing elevation of the footings should be lowered to El. 3 so that over-excavation/replacement is not required.

Mean High Water at Q1.1 (El. 3.9) is approximately 6 to 7 ft above the bottom of the existing fill (possible footing bearing level). Therefore dewatering inside of the temporary cofferdams will be

needed so that removal of existing fill, preparation of subgrade soils, construction of footings and backfilling activities can be completed in-the-dry.

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

- Make final excavations (e.g., within 5 ft of final footing bearing level) into bearing soils in-the-dry using smooth-bladed equipment to limit disturbance. Dewatering will be required within the excavation limits.
- During substructure construction, prevent water infiltration into the excavation to reduce the possibility of soil disturbance. All filling and concreting of sub-footings should be performed in-the-dry. Subgrades that become disturbed due to water infiltration should be over-excavated and stabilized.
- 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 until firm, as determined by a geotechnical engineer. Any soft areas revealed by proofrolling should be excavated and replaced with approved granular material or additional sub-footing concrete.
- Fill placed within the limits of the excavation support system should be placed in lifts not exceeding 12 in. in loose measure and compacted using self-propelled vibratory equipment. In confined areas, maximum loose layer should be reduced to 9 in., and compaction performed by hand-guided equipment. Cobbles or boulders having a size exceeding 2/3 of the loose lift thickness should be removed prior to compaction.
- Disturbance due to water infiltration and adverse weather could be reduced by maintaining footing excavations at least 12 in. above the final bearing level until immediately before placing sub-footing concrete.
- 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.

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.

- Determination of over-excavation limits of unsuitable soils below footing bearing levels.
- Preparation of the footing bearing surfaces.
- Placement and compaction of compacted fills below footing bearing level.

Limitations and Closure

This report is prepared for the exclusive use of Stantec relative to the Multi-Use Trail 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.

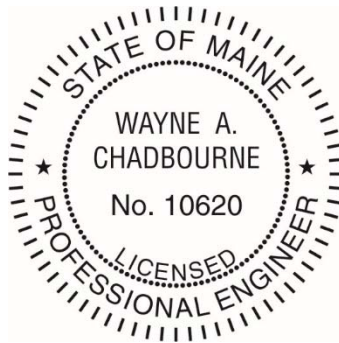
Sincerely yours,
HALEY & ALDRICH, INC.



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 and Interpretive Subsurface Profile
- Appendix A – Test Boring Logs
- Appendix B – Rock Core Photographs
- Appendix C – Laboratory Test Results
- Appendix D – Calculations

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References

1. Newburg, Donald W, *Bedrock Geology of the Gardiner 15' Quadrangle, Maine*, Maine Geological Survey, Department of Conservation, Augusta, Maine, 1984.
2. Thompson, Woodrow B, *Surficial Geology Gardiner Quadrangle, Maine*, Maine Geological Survey, Department of Conservation, Augusta, Maine, Open File Report No. 09-8, 2009.
3. Melo, Cristiano and Sharma, Sunil, *Seismic Coefficients for Pseudostatic Slope Analysis*, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6, 2004.

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TABLE I**Summary of Subsurface Explorations**

Multi-Use Trail Bridge over Cobbosseecontee Stream

Gardiner, Maine

MaineDOT WIN 018736.10

LOCATION DATA:

Test Boring No. ¹	Coordinates ²	
	Easting	Northing
BB-GCS-203	1,142,394	509,896
BB-GCS-204	1,142,454	509,782
BB-GCS-501	1,142,414	509,795
BB-GCS-502	1,142,397	509,893

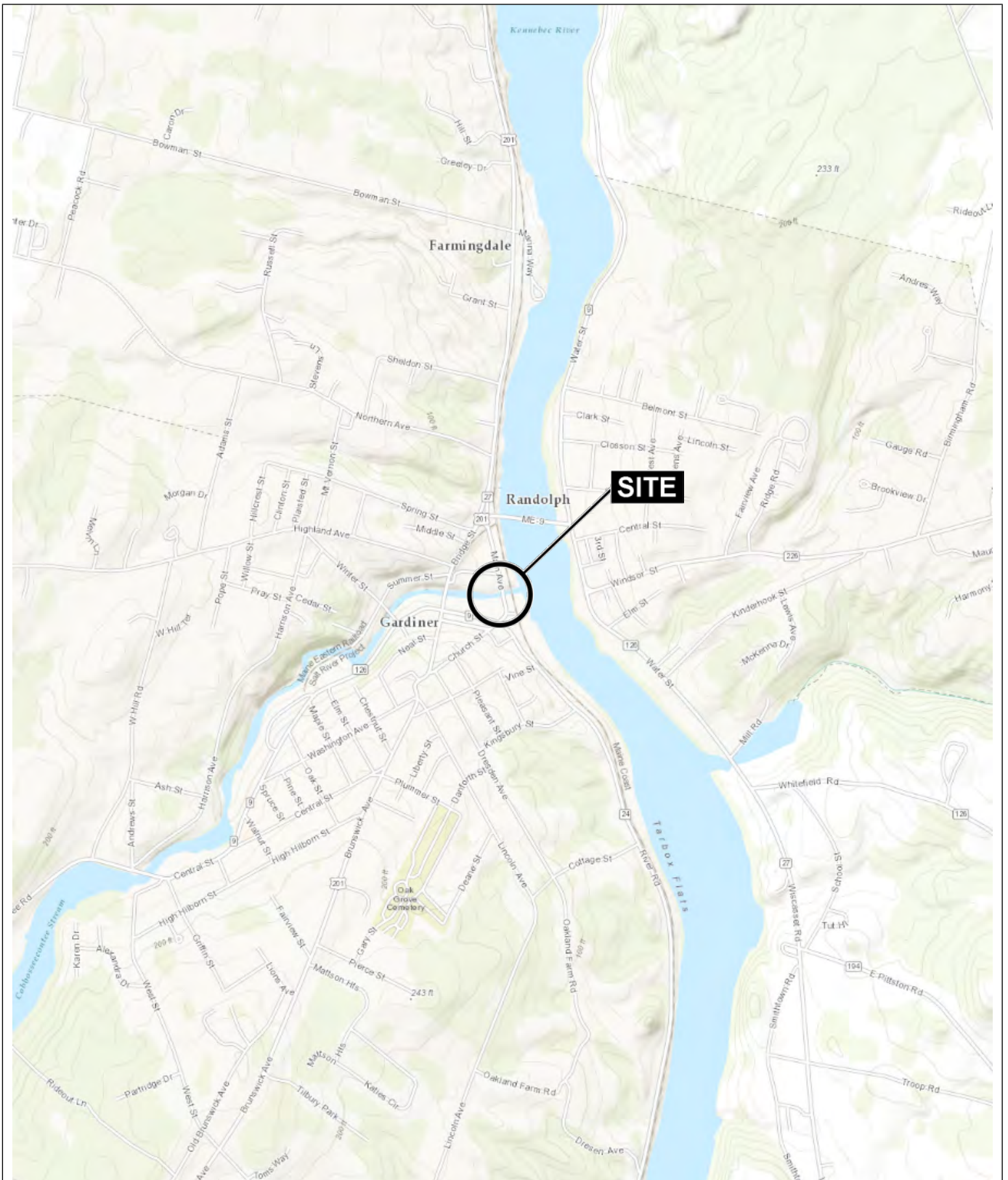
SUBSURFACE DATA:

Test Boring No. ¹	Ground Surface Elevation ^{3,4}	Approximate Strata Thickness ^{5,6,7} (ft)					Approximate Elevation of Top of Bedrock ^{3,4}	Approximate Elevation of Bottom of Exploration ^{3,4}
		Fill	Alluvial Deposit	Marine Silt	Marine Sand/Gravel	Glacial Till		
BB-GCS-203	14.3	>18.0	--	--	--	--	--	-3.7
BB-GCS-204	14.1	13.9	NE	NE	>0.6	--	--	-0.4
BB-GCS-501	12.6	15.0	19.5	NE	7.7	15.5	-45.1	-55.4
BB-GCS-502	14.4	17.0	13.0	4.0	NE	6.5	-26.1	-36.4

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 Central 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 strata.
7. BB-GCS-203 and BB-GCS-204 were performed for environmental purposes and were terminated in native soil or below the ground water surface.

Prepared By:	KAR	7/26/2017
Checked By:	EAF	7/27/2017
Reviewed By:	WAC	8/1/2017



MAP SOURCE: ESRI

SITE COORDINATES: 44°13'50"N, 69°46'17"W

**HALEY
ALDRICH**

MULTI-USE TRAIL BRIDGE OVER COBBOSSEECONTEE STREAM
GARDINER, MAINE
MAINEDOT WIN 18736.10



PROJECT LOCUS

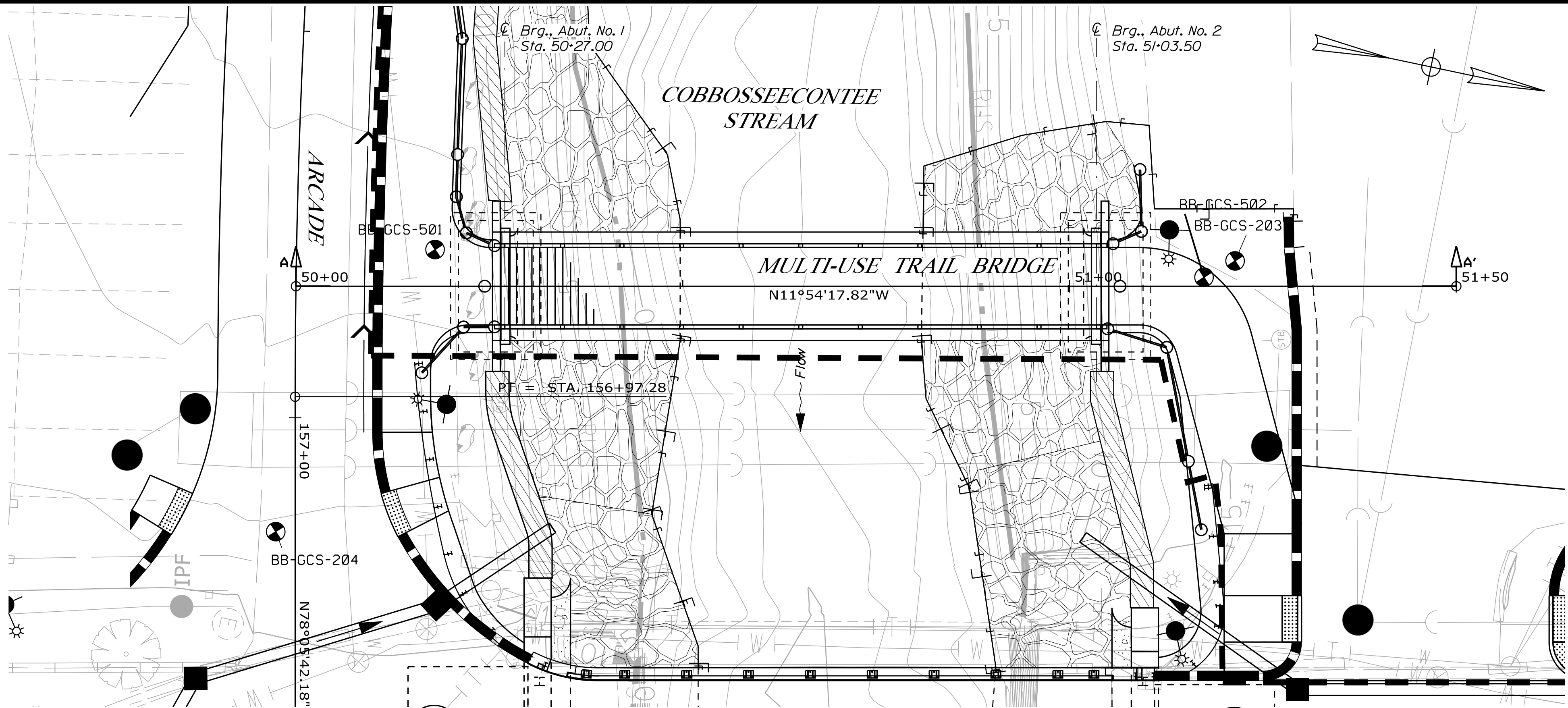
APPROXIMATE SCALE: 1 IN = 2000 FT
JULY 2018

FIGURE 1

Date: 6/7/2018

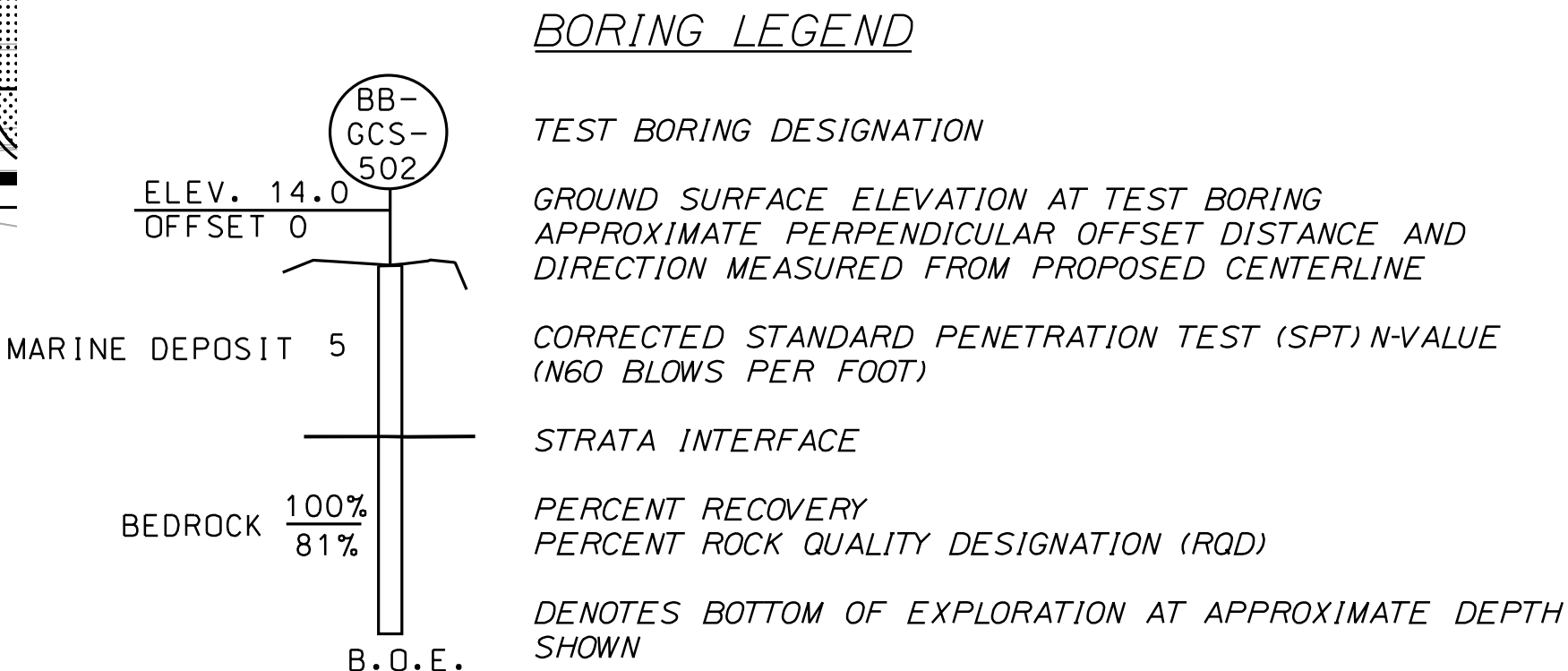
Username: dataylor

Filename: ... \OC8_Trail Bridge Plan and Profile.dgn Division: BRIDGE

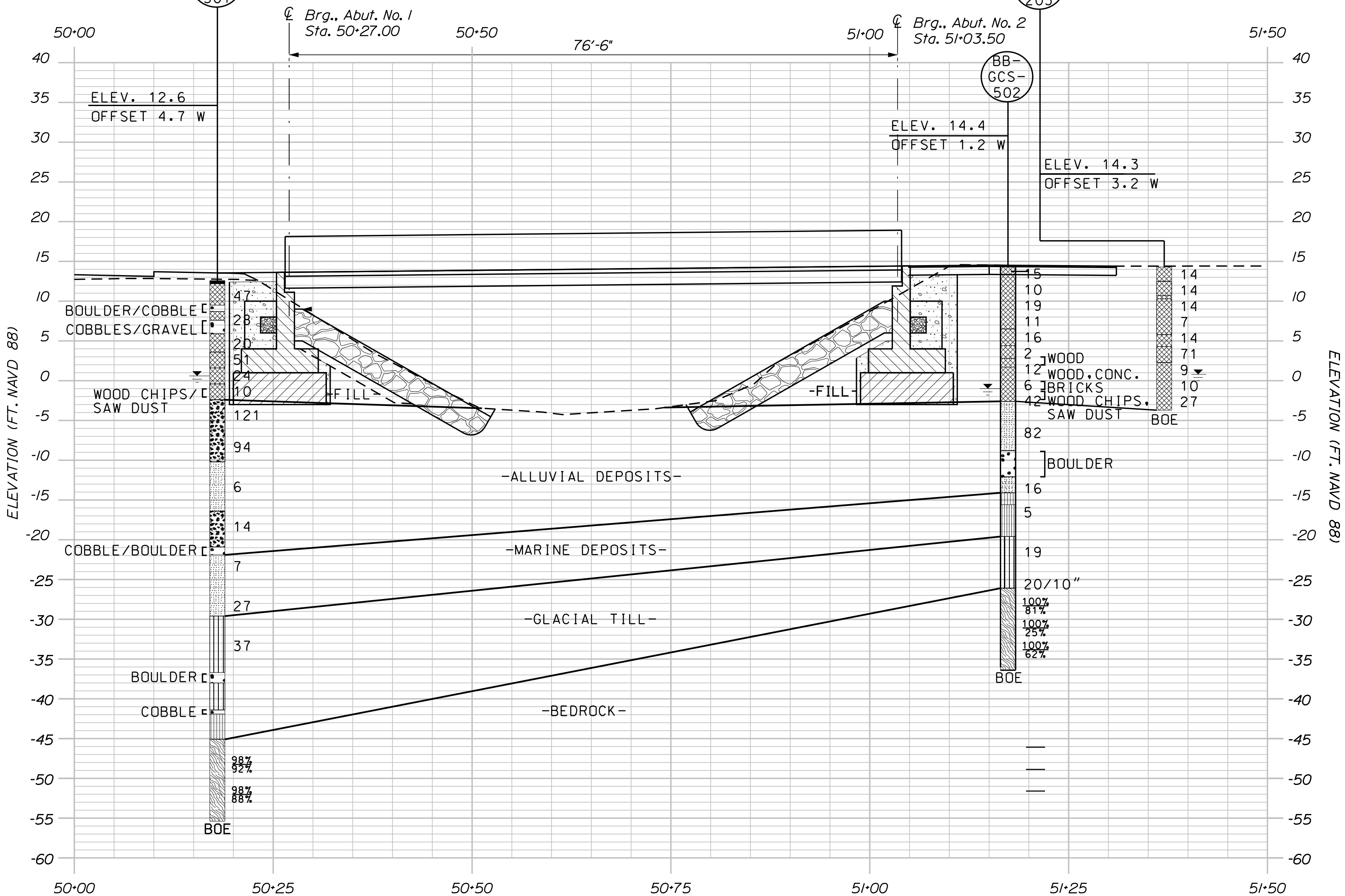


- LEGEND**
- BB-BGC-501 DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS FROM 5 TO 24 JULY 2017.
- BB-BGC-203 DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS ON 30 NOVEMBER 2015.
- LOCATION AND ORIENTATION OF SUBSURFACE PROFILE

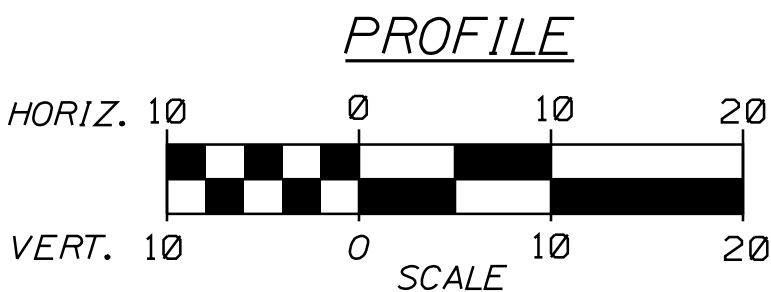
- NOTES**
- EXISTING AND PROPOSED SITE CONDITIONS AND CONTOURS OF EXISTING GROUND SURFACE ELEVATIONS ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY STANTEC ON 19 JULY 2017.
 - AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINE DOT USING GPS SURVEY EQUIPMENT.
 - SUBSURFACE EXPLORATIONS SHOWN WERE MONITORED BY HALEY & ALDRICH PERSONNEL.
 - ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).



- STRATA SYMBOLS**
- | | | |
|-----------------------------|---------------------------|---|
| BITUMINOUS CONCRETE | POORLY GRADED SAND | INTERLAYERED WELL POORLY GRADED SAND |
| TOPSOIL | WELL GRADED GRAVEL | POORLY GRADED SAND, LITTLE GRAVEL, TRACE SILT |
| FILL | SILTY SAND, LITTLE GRAVEL | SILT |
| COBBLES/BOULDERS | SILTY SAND | BEDROCK |
| WELL GRADED GRAVEL AND SAND | | |



PRELIMINARY
NOT FOR CONSTRUCTION



HALEY
ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
TAP-1873(610)
WIN
18736.10
HIGHWAY PLANS

PROJ. MANAGER	DATE	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
DESIGN-DETAILED	DEC 2017	EAF	DEC 2017			
CHECKED-REVIEWED		WAC				
DESIGN-DETAILED						
DESIGN-DETAILED						
REVISIONS 1						
REVISIONS 2						
REVISIONS 3						
REVISIONS 4						
FIELD CHANGES						

GARDNER
ARCADE / MAINE AVE
BORING LOCATION PLAN AND
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 2

APPENDIX A

Test Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-501 WIN: 18736.10			
Driller: New England Boring Contractors				Elevation (ft.): 12.6				Auger ID/OD: --			
Operator: T.Schaefer				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID			
Logged By: K.Russ				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS140#/30;HW&NW300#/18			
Date Start/Finish: 7-11-17/7-24-17				Drilling Method: HW Drive/NW Drive				Core Barrel: NQ-1.875 in. ID			
Boring Location: E1142414, N509795				Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID				Water Level*: 12.0 ft			
Hammer Efficiency Factor: 0.743				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample 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							PUSH	12.2		-BITUMINOUS CONCRETE-	G#14626a A-1-b SM
	1D	24/12	1.0 - 3.0	11/17/21/20	38	47				Brown, dry, dense gravelly SAND, well graded -FILL-(SW) PID=0.0 ppm	
5	2D	24/8	4.0 - 6.0	7/9/14/9	23	28	17	9.5		-BOULDER/COBBLE-	
								8.7		Brown, dry, medium dense gravelly SAND, little silt -FILL-(SM) PID=0.0 ppm	
								7.6		Note: Advanced HW to 6 ft, advanced rollerbit to 7 ft.	
										-COBBLES/GRAVEL-	
								5.9		Brown, dry, medium dense sandy GRAVEL, well graded -FILL-(GW) PID=0.0 ppm	
10	4D	24/6	9.0 - 11.0	4/5/36/7	41	51	7	3.6		Dark brown, wet, dense fine to medium SAND, little gravel, trace silt and coarse sand, poorly graded, contains brick, ash, and coal -FILL-(SP) PID=0.0 ppm	G#14626b A-1-a GW
15	6D	24/12	13.0 - 15.0	7/5/3/6	8	10	19	-0.4		Wood chips and saw dust -FILL- PID=0.3 ppm	
20	8D	24/9	20.0 - 22.0	25/40/36/24	76	94	109	-10.2		Gray-brown, wet, very dense sandy GRAVEL, well graded -ALLUVIAL DEPOSIT-(GW)	
25							14			Note: Drill wash water contains gray fine sand from 22.8 to 25.0 ft.	
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 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-501	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-501 WIN: 18736.10			
Driller: New England Boring Contractors				Elevation (ft.): 12.6				Auger ID/OD: --			
Operator: T.Schaefer				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID			
Logged By: K.Russ				Rig Type: Mobile B53 Track				Hammer Wt./Fall: SS140#/30;HW&NW300#/18			
Date Start/Finish: 7-11-17/7-24-17				Drilling Method: HW Drive/NW Drive				Core Barrel: NQ-1.875 in. ID			
Boring Location: E1142414, N509795				Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID				Water Level*: 12.0 ft			
Hammer Efficiency Factor: 0.743				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected			
								T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information							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	9D	24/11	25.0 - 27.0	1/1/4/6	5	6	17			Gray, wet, loose fine SAND, little silt, little gravel, trace medium to coarse sand -ALLUVIAL DEPOSIT-(SM)	G#14626c A-2-4(0) SM
							25				
							30				
							44				
							25				
30	10D	24/6	30.0 - 32.0	6/4/7/5	11	14	32			Gray, wet, medium dense sandy GRAVEL, trace silt, slight bonding of silty fine sand on gravel pieces, well graded -ALLUVIAL DEPOSIT-(GW)	
							45				
							55				
							94			Note: Advanced rollerbit through boulder from 33.5 to 34.5 ft.	
							22			-COBBLE/BOULDER-	
35	MD	24/0	35.0 - 37.0	3/2/4/4	6	7	28			No recovery, rock in spoon tip	
							29				
							31				
							46			Note: Drill wash water from 35 to 40 ft contains gray fine to medium sand, trace gravel at approximately 39 ft.	
							49				
40	11D	24/10	40.0 - 42.0	8/11/11/12	22	27	83			Gray, wet, medium dense SAND, poorly graded -MARINE DEPOSIT-(SP)	
							73				
							103				
							111			Note: Drill wash water from 40 to 45 ft contains gray fine sand, becomes coarser with depth. Gravel at approximately 42 ft.	
							101				
45	12D	24/9	45.0 - 47.0	16/13/17/23	30	37	22			Olive-gray, wet, dense silty SAND, some gravel, loosely to moderately bonded -GLACIAL TILL-(SM)	
							35				
							43				
							52			Note: Cored through boulder from 49.3 to 50.6 ft.	
							71				
50										-BOULDER-	
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-501	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-501 WIN: 18736.10		
Driller: New England Boring Contractors		Elevation (ft.): 12.6		Auger ID/OD: --				
Operator: T.Schaefer		Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID				
Logged By: K.Russ		Rig Type: Mobile B53 Track		Hammer Wt./Fall: SS140#/30;HW&NW300#/18				
Date Start/Finish: 7-11-17/7-24-17		Drilling Method: HW Drive/NW Drive		Core Barrel: NQ-1.875 in. ID				
Boring Location: E1142414, N509795		Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID		Water Level*: 12.0 ft				
Hammer Efficiency Factor: 0.743		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐						
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test								
Depth (ft.)	Sample Information							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							48	Visual Description and Remarks Note: Drill wash water from 50.6 to 54 ft contains sand, silt and gravel. Note: Encountered cobble from 54 to 54.5 ft. Note: Drill wash water contains coarse gravel from 54.5 to 57.7 ft. Top of Bedrock El. -45.1 Olive-gray to dark gray, aphanitic to fine-grained SCHIST. Very hard to moderately hard, fresh. Joints dipping low angle, parallel to foliation, very close to moderately close, tight, <1/16-in. silt infilling on joint surfaces, joint surfaces slightly weathered. Joint surfaces smooth to rough, undulating to stepped. Quartz/feldspar intrusion from 60.7 to 61.3 ft. Occasional calc-silicate zones Rock Mass Quality=Excellent Recovery=98% R1 Core Times (min:sec): 58.0-59.0' (2:55); 59.0-60.0' (2:39); 60.0-61.0' (3:15); 61.0-62.0' (2:11); 62.0-63.0' (2:46) Olive-gray to dark gray, aphanitic to fine-grained SCHIST. Hard to moderately hard, fresh to slightly weathered. Joints dipping at low to moderate angles, parallel to foliation, close to moderately close, open, 1/16 to 1/8-in. silt/clay infilling on joint surfaces, joints slightly weathered. Joint surfaces stepped to planar, rough. Rock Mass Quality=Good Recovery=98% R2 Core Times (min:sec): 63.0-64.0' (3:25); 64.0-65.0' (2:45); 65.0-66.0' (3:07); 66.0-67.0' (2:44); 67.0-68.0' (2:20) -VASSALBORO FORMATION- Bottom of Exploration at 68.0 feet below ground surface.
							29	
							49	
							73	
							20	
55							25	
							38	
							71	
	R1	60/59	58.0 - 63.0	RQD = 92%				
60								
	R2	60/59	63.0 - 68.0	RQD = 88%				
65								
70								
75								
Remarks:								
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.							Page 3 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-501	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-502 WIN: 18736.10	
Driller: New England Boring Contractors			Elevation (ft.): 14.4		Auger ID/OD: --		
Operator: M.Porter			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&NW-300#		
Date Start/Finish: 7-5-17/7-6-17			Drilling Method: HW Drive/NW Drive		Core Barrel: NQ-1.875 in. ID		
Boring Location: E1142397, N509893			Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID		Water Level*: 15.4 ft		
Hammer Efficiency Factor: 0.743			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐				
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt							
R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person							
S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected							
T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample 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	0.0 - 2.0	2/5/7/11	12	15	Spun	14.3		-TOPSOIL-	G#14626d A-1-b SM	
										0.1		Brown, dry, medium dense, fine SAND, little medium to coarse sand, little gravel, trace silt, poorly graded
	2D	24/6	2.0 - 4.0	7/5/3/2	8	10						-FILL-(SP) PID=0.1 ppm
												Brown, dry, loose, fine SAND, little medium to coarse sand, little gravel, trace silt, poorly graded
												-FILL-(SP) PID=0.2 ppm
5	3D	24/13	4.0 - 6.0	5/4/11/8	15	19						Brown, dry, medium dense, fine SAND, little medium to coarse sand, little gravel, contains trace concrete and trace wood, poorly graded
												-FILL-(SP) PID=0.0 ppm
	4D	24/14	6.0 - 8.0	6/5/4/7	9	11						Brown, dry, loose, fine to medium SAND, little coarse sand, little silt, trace fine gravel
												-FILL-(SM) PID=0.1 ppm
	5D	24/9	8.0 - 10.0	3/3/10/6	13	16				6.5		Olive-brown, wet, medium dense, fine SAND, little medium to coarse sand, little silt, trace gravel, contains black sand with creosote-like odor, poorly graded
												-FILL-(SP-SM) PID=0.2 ppm
10	6D	24/4	10.0 - 12.0	2/1/1/22	2	2				4.4		Very loose, sample contains trace brown fine sand. Contains mostly wood, concrete and brick chips.
												-FILL- PID=0.7 ppm
	7D	24/6	12.0 - 14.0	6/5/5/5	10	12				2.8		-WOOD- (based on drill wash)
										1.7		Dark brown, wet, loose, SAND, some gravel, trace silt, well graded
												-FILL-(SW) PID=0.6 ppm
15	8D	24/3	14.0 - 16.0	5/2/3/5	5	6	16					Wood and concrete pieces
												-FILL- PID=0.5 ppm
	9D	24/16	16.0 - 18.0	9/11/23/22	34	42	54			Wood chips and saw dust		
										-FILL-		
									-2.6	Olive-brown to gray-brown, wet, dense, gravelly SAND, well graded		
										-ALLUVIAL DEPOSIT-(SW)		
20	10D	24/12	20.0 - 22.0	19/39/27/20	66	82	83				Olive-brown to gray-brown, wet, very dense, sandy GRAVEL, little silt	
											-ALLUVIAL DEPOSIT-(GM)	
25										-8.8	Note: Casing refusal at 23.2 ft, advance rollerbit to 25 ft.	
											-BOULDER-(Granite)	

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-502

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-502					
WIN: 18736.10											
Driller: New England Boring Contractors		Elevation (ft.): 14.4		Auger ID/OD: --							
Operator: M.Porter		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&NW-300#							
Date Start/Finish: 7-5-17/7-6-17		Drilling Method: HW Drive/NW Drive		Core Barrel: NQ-1.875 in. ID							
Boring Location: E1142397, N509893		Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID		Water Level*: 15.4 ft							
Hammer Efficiency Factor: 0.743				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person							
				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q ₀ = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected							
				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							
Depth (ft.)	Sample Information							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	R1	24/22	25.0 - 27.0				-	-12.1		Recovery=92% R1 Core Times (min:sec): 25.0-26.0' (3:39); 26.0-27.0 (3:40)	G#14626f A-4(0) ML
							-				
	11D	24/9	27.0 - 29.0	7/7/6/10	13	16	-	-14.1		Gray, wet, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt -ALLUVIAL DEPOSIT-(SP-SM)	
							-				
							-	-15.6		Gray, wet, medium dense, fine to coarse SAND, some gravel, little silt, moderately bonded -ALLUVIAL DEPOSIT-(SM)	
30	12D	24/17	30.0 - 32.0	3/2/2/3	4	5	8	-19.6		Gray, wet, very loose, SILT, some sand, trace clay, few fine sand layers (3-in. thickness) -MARINE DEPOSIT-(ML)	
35	13D	24/7	35.0 - 37.0	8/5/10/10	15	19	25	-19.6		Note: Drill action indicates strata change at 34 ft. Gray, wet, medium dense, silty SAND, little gravel, well bonded -GLACIAL TILL-(SM)	
40	14D	6/4	40.0 - 40.5	44/20(0.0)			14/5	-26.1		Gray, wet, very dense, silty SAND, little gravel, well bonded -GLACIAL TILL-(SM)	
	R2	36/36	41.5 - 44.5	RQD = 81%						Top of Bedrock El.-26.1 Gray, fine-grained to aphanitic, granite SCHIST. Hard, fresh to slightly weathered. Joints dipping at low to steep angles, very close to moderately close, open, planar to undulating, rough. Pegmatitic intrusion at approximately 44 to 44.3 ft. Rock Mass Quality=Good Recovery=100%	
										R2 Core Times (min:sec): 41.5-42.5' (6:08); 42.5-43.5' (4:33); 43.5-44.5' (4:25)	
										Same R2, except joints very close to close, silt coatings on some joint surfaces, pegmatitic intrusion at approximately 44.8 to 45.1 ft. Rock Mass Quality=Very Poor Recovery=100%	
45	R3	16/16	44.5 - 45.8	RQD = 25%						R3 Core Times (min:sec): 44.5-45.8' (2:50)	
	R4	60/60	45.8 - 50.8	RQD = 62%						Gray, fine-grained granite SCHIST grading to coarse-grained pegmatitic GRANITE at approximately 46.5 ft. Hard, fresh to slightly weathered, joints dipping at low to moderate angles, very close to moderately close, tight to open, planar to undulating, rough, slight oxidation on some joint surfaces, occasional high angle secondary	
50											
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.											

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Multi-Use Bridge over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-502 WIN: 18736.10									
Driller: New England Boring Contractors				Elevation (ft.): 14.4				Auger ID/OD: --									
Operator: M.Porter				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&NW-300#/									
Date Start/Finish: 7-5-17/7-6-17				Drilling Method: HW Drive/NW Drive				Core Barrel: NQ-1.875 in. ID									
Boring Location: E1142397, N509893				Casing ID/OD: HW-4.0 in. ID/NW-3.0 in. ID				Water Level*: 15.4 ft									
Hammer Efficiency Factor: 0.743				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐													
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test					
Sample Information												Graphic Log		Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)									
50									-36.4	joint, and silt coatings. Rock Mass Quality=Fair Recovery=100% R4 Core Times (min:sec): 45.8-46.8' (2:09); 46.8-47.8' (2:50); 47.8-48.8' (4:20); 48.8-49.8' (4:25); 49.8-50.8' (4:49) -VASSALBORO FORMATION- 50.8 Bottom of Exploration at 50.8 feet below ground surface.							
75																	
Remarks:																	
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.												Page 3 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-502					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine		Boring No.: BB-GCS-203 PIN: 18736					
Driller: New England Boring Contractors		Elevation (ft.): 14.3		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: 11-30-2015/11-30-2015		Drilling Method: Auger		Core Barrel: --							
Boring Location: E1142394; N509896		Casing ID/OD: --		Water Level*: 13.5							
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				
0	1D	24/12	0.0 - 2.0	5/6/8/6	14	14	HSA	12.5		Olive-brown, moist, medium dense, fine SAND, little silt, trace medium sand, trace fine gravel, poorly graded -FILL-(SP-SM) PID=0.1ppm	
	2D	24/13	2.0 - 4.0	3/8/6/4	14	14		10.7		Olive-brown, dry, medium dense, fine SAND, some medium to coarse sand and gravel, little silt -FILL-(SW-SM) PID=0.1ppm	
	3D	24/13	4.0 - 6.0	10/8/6/4	14	14		10.3		Olive-brown, dry, stiff, fine sandy SILT -FILL-(ML) PID=0.1ppm	
5	4D	24/2	6.0 - 8.0	5/4/3/5	7	7				Olive-brown, dry, medium dense, silty fine SAND, little medium to coarse sand, trace fine gravel -FILL-(SM) PID=0.1ppm	
	5D	24/9	8.0 - 10.0	9/11/3/3	14	14		5.8		Olive-brown, dry, loose, silty fine SAND, little medium sand, trace coarse sand, trace fine gravel -FILL-(SM) PID=0.4ppm	
10	6D	24/6	10.0 - 12.0	1/2/69/43	71	71		4.3		Olive-brown, dry, medium dense, silty fine SAND, little medium to coarse sand, trace coarse gravel -FILL-(SM) PID=0.5ppm	
	MD	24/0	12.0 - 14.0	4/4/5/4	9	9		2.3		Olive-brown, dry, medium dense, fine to coarse sandy GRAVEL, trace silt -FILL-(GW) PID=2.4ppm	
15	7D	24/16	14.0 - 16.0	3/5/5/6	10	10				Black, moist, very dense, gravelly fine to coarse SAND, little silt, contains cardboard fabric -FILL-(SW-SM) PID=0.1ppm	
	8D	24/10	16.0 - 18.0	11/17/10/33	27	27		-3.7		No recovery, wet(split spoon sampler), wood fragment in spoon tip -FILL- Recovered 16 in. wood, strong gasoline-like odor PID=382.0ppm Similar to above PID=8.0ppm	
20										Bottom of Exploration at 18.0 feet below ground surface.	
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-203	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Bridges Over Cobbosseecontee Stream Location: Gardiner, Maine				Boring No.: BB-GCS-204 PIN: 18736			
Driller: New England Boring Contractors				Elevation (ft.): 14.1				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: 11-30-2015/11-30-2015				Drilling Method: Auger				Core Barrel: --			
Boring Location: E1142454; N509782				Casing ID/OD: --				Water Level*: 12.0			
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	24/12	0.5 - 2.5	17/13/15/10	28	28	HSA		13.9		-BITUMINOUS CONCRETE- Brown, dry, medium dense, fine to coarse SAND, trace silt, little gravel, well graded -FILL-(SW) PID=0.0ppm Similar to 1D, except contains bituminous concrete pieces PID=0.3ppm Brown, dry, medium dense, fine to coarse SAND, trace silt, trace fine gravel, well graded -FILL-(SW) PID=0.2ppm Brown, dry, medium dense, sandy GRAVEL, trace silt, poorly graded, rock piece in tip of split spoon -FILL-(GP) PID=0.1ppm Light brown, dry, very dense, sandy GRAVEL, strong cement-like odor, poorly graded, contains concrete -FILL-(GP) PID=0.0ppm Dark olive-brown, wet, medium stiff, SILT, little fine sand, trace gravel, contains roots, wood, glass, slight organic odor -FILL-(ML) PID=0.0ppm Dark olive-brown, wet, stiff, fine sandy SILT, contains wood and organics -FILL-(ML) PID=0.0ppm Gray, wet, loose, silty fine SAND -MARINE DEPOSIT-(SM) PID=0.0ppm Bottom of Exploration at 14.5 feet below ground surface.	
	2D	24/7	2.5 - 4.5	11/11/7/3	18	18						
	3D	24/2	4.5 - 6.5	4/2/2/2	4	4						
5	4D	24/3	6.5 - 8.5	2/3/11/16	14	14			7.6			
	5D	24/9	8.5 - 10.5	64/92/44/32	136	136						
	6D	24/16	10.5 - 12.5	8/4/4/4	8	8			3.6			
10	7D	24/24	12.5 - 14.5	5/5/4/5	9	9						
									0.2			
									-0.4			
15												
20												
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-204

APPENDIX B

Rock Core Photographs



BORING ID	CORE ID	CORE RUN DEPTH (FT)	CORE RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2, 3}	
				IN.	%	IN.	%
BB-GCS-501	Boulder	49.3 to 50.6	-36.7 to -38.0	11	71	NA	NA
BB-GCS-501	R1	58.0 to 63.0	-45.4 to -50.4	59	98	55	92
BB-GCS-501	R2	63.0 to 68.0	-50.4 to -55.4	59	98	53	88

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).
3. "NA" INDICATES NOT APPLICABLE DUE TO THE ROCK CORED BEING A BOULDER

**HALEY
ALDRICH**

UNDERGROUND
ENGINEERING &
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MULTIUSE TRAIL BRIDGE
GARDINER MAINE

**PHOTOGRAPH OF
BEDROCK CORE
BORINGS: BB-GCS-501**

FILE NO. 42223-003

August 2017



BORING ID	CORE ID	CORE RUN DEPTH (FT)	CORE RUN EL. ^{NOTE 1}	RECOVERY		RQD ^{NOTE 2, 3}	
				IN.	%	IN.	%
BB-GCS-502	R1	25.0 to 27.0	-10.6 to -12.6	22	92	NA	NA
BB-GCS-502	R2	41.5 to 44.5	-27.1 to -30.1	36	100	29	81
BB-GCS-502	R3	44.5 to 45.8	-30.1 to -31.4	16	100	4	25
BB-GCS-502	R4	45.8 to 50.8	-31.4 to -36.4	60	100	37	62

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).
3. "NA" INDICATES NOT APPLICABLE DUE TO THE ROCK CORED BEING A BOULDER



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GARDINER, MAINE

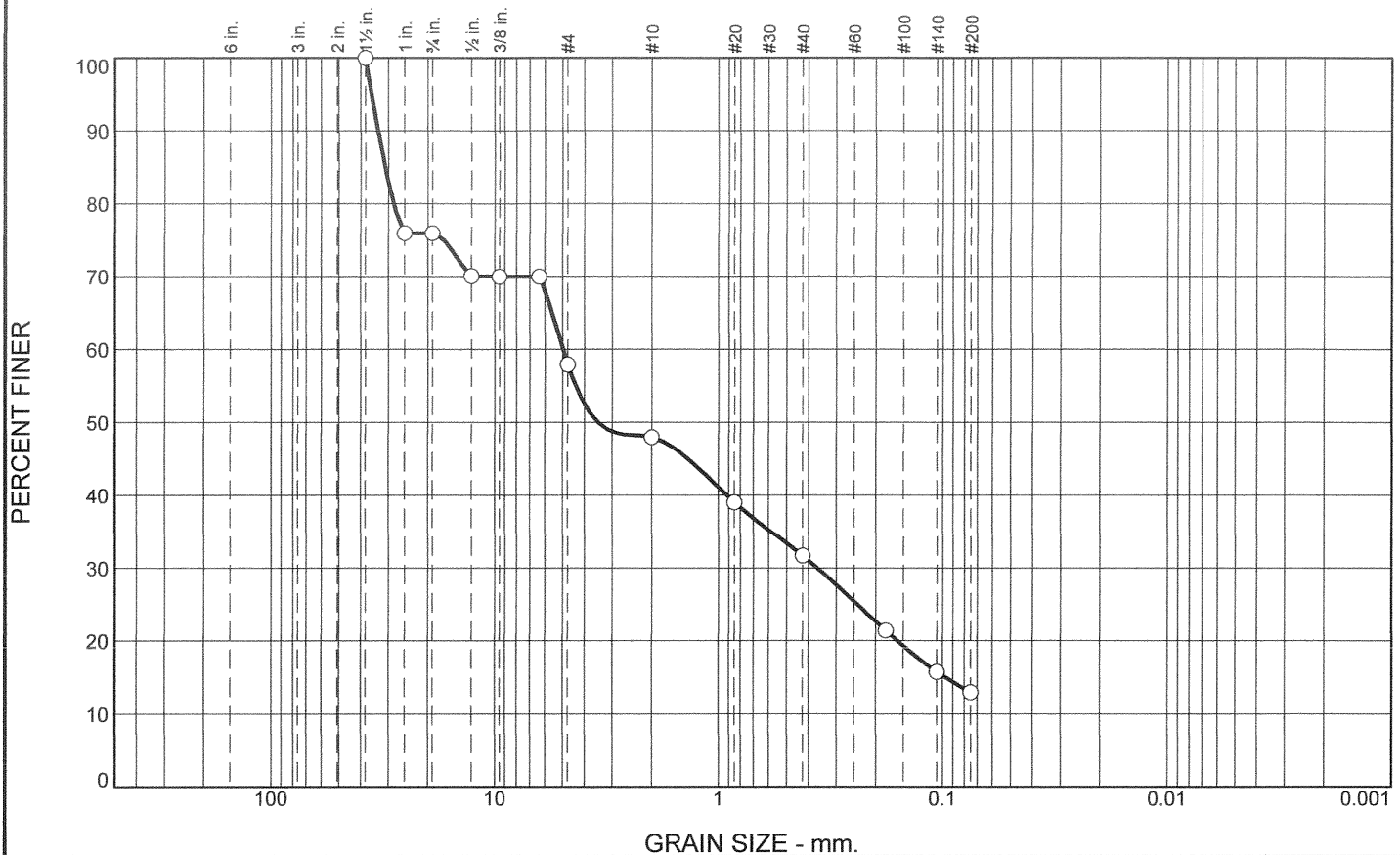
**PHOTOGRAPH OF
BEDROCK CORE
BORING: BB-GCS-502**

FILE NO. 42223-003

August 2017

Appendix C
Laboratory Test Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.1	18.0	10.0	16.2	18.7	13.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	75.9		
3/4"	75.9		
1/2"	70.1		
3/8"	70.0		
1/4"	70.0		
#4	57.9		
#10	47.9		
#20	39.0		
#40	31.7		
#80	21.4		
#140	15.8		
#200	13.0		

* (no specification provided)

<u>Soil Description</u>		
Silty Sand with Gravel		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₉₀ = 33.4497	D ₈₅ = 31.1057	D ₆₀ = 4.9849
D ₅₀ = 3.4811	D ₃₀ = 0.3654	D ₁₅ = 0.0970
D ₁₀ =	C _u =	C _c =
<u>Classification</u>		
USCS= SM	AASHTO= A-1-b	
<u>Remarks</u>		
Moisture Content: 3.3%		

Location: BB-GCS-501, Gardiner, ME
Sample Number: 2D Depth: 4'-6'

Date: 8/1/17

**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

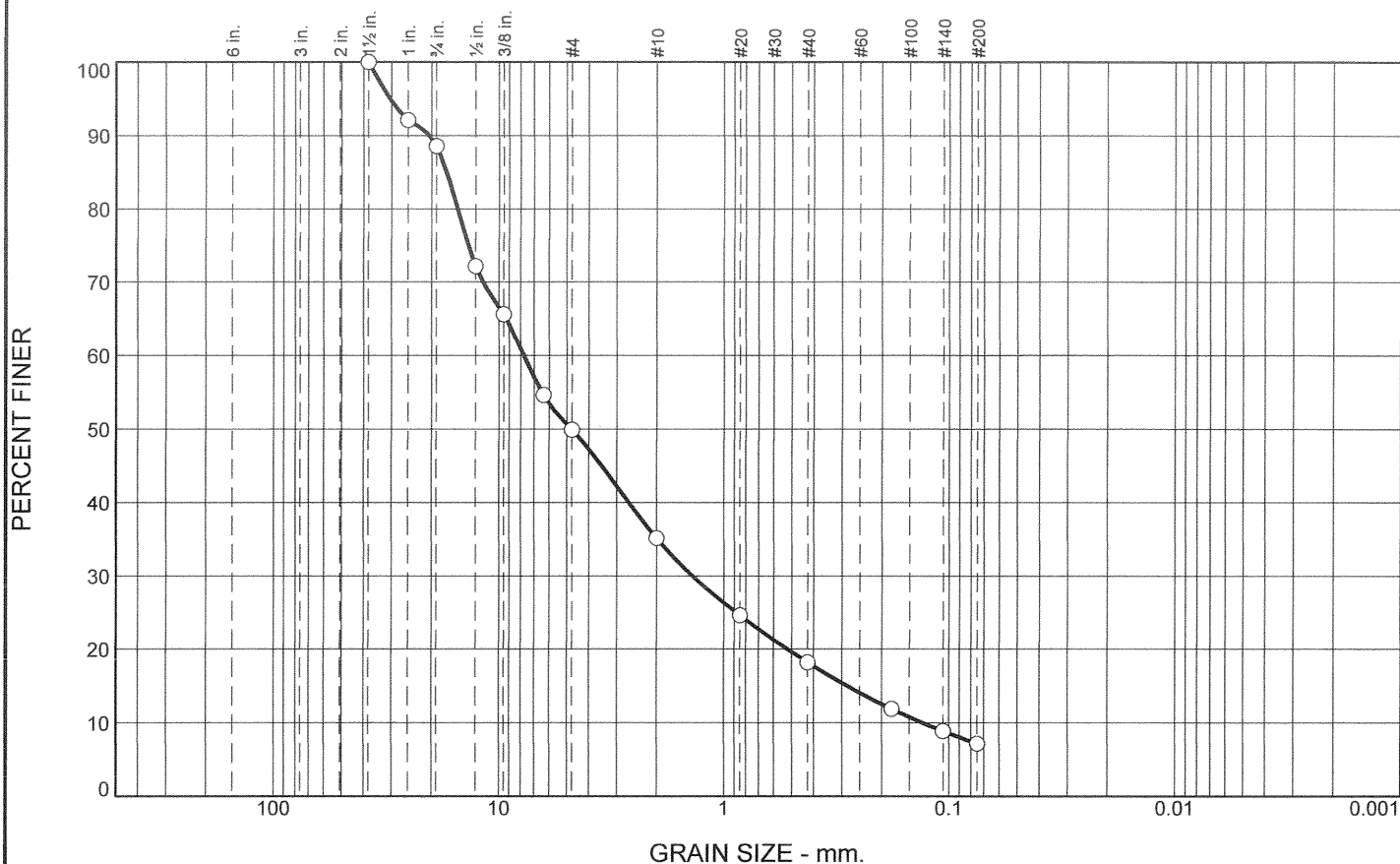
Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013 Lab No. 14626a

Tested By: JJB

Checked By: MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	11.5	38.6	14.8	16.9	11.0	7.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	92.1		
3/4"	88.5		
1/2"	72.2		
3/8"	65.6		
1/4"	54.6		
#4	49.9		
#10	35.1		
#20	24.7		
#40	18.2		
#80	11.9		
#140	8.9		
#200	7.2		

* (no specification provided)

<u>Soil Description</u>		
Well-graded Gravel with Silt and Sand		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₉₀ = 20.4476	D ₈₅ = 17.1632	D ₆₀ = 7.7434
D ₅₀ = 4.7829	D ₃₀ = 1.3846	D ₁₅ = 0.2838
D ₁₀ = 0.1308	C _u = 59.19	C _c = 1.89
<u>Classification</u>		
USCS= GW-GM	AASHTO= A-1-a	
<u>Remarks</u>		
Moisture Content: 10.9%		

Location: BB-GCS-501, Gardiner, ME
Sample Number: 8D Depth: 20'-22'

Date: 8/1/17

**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

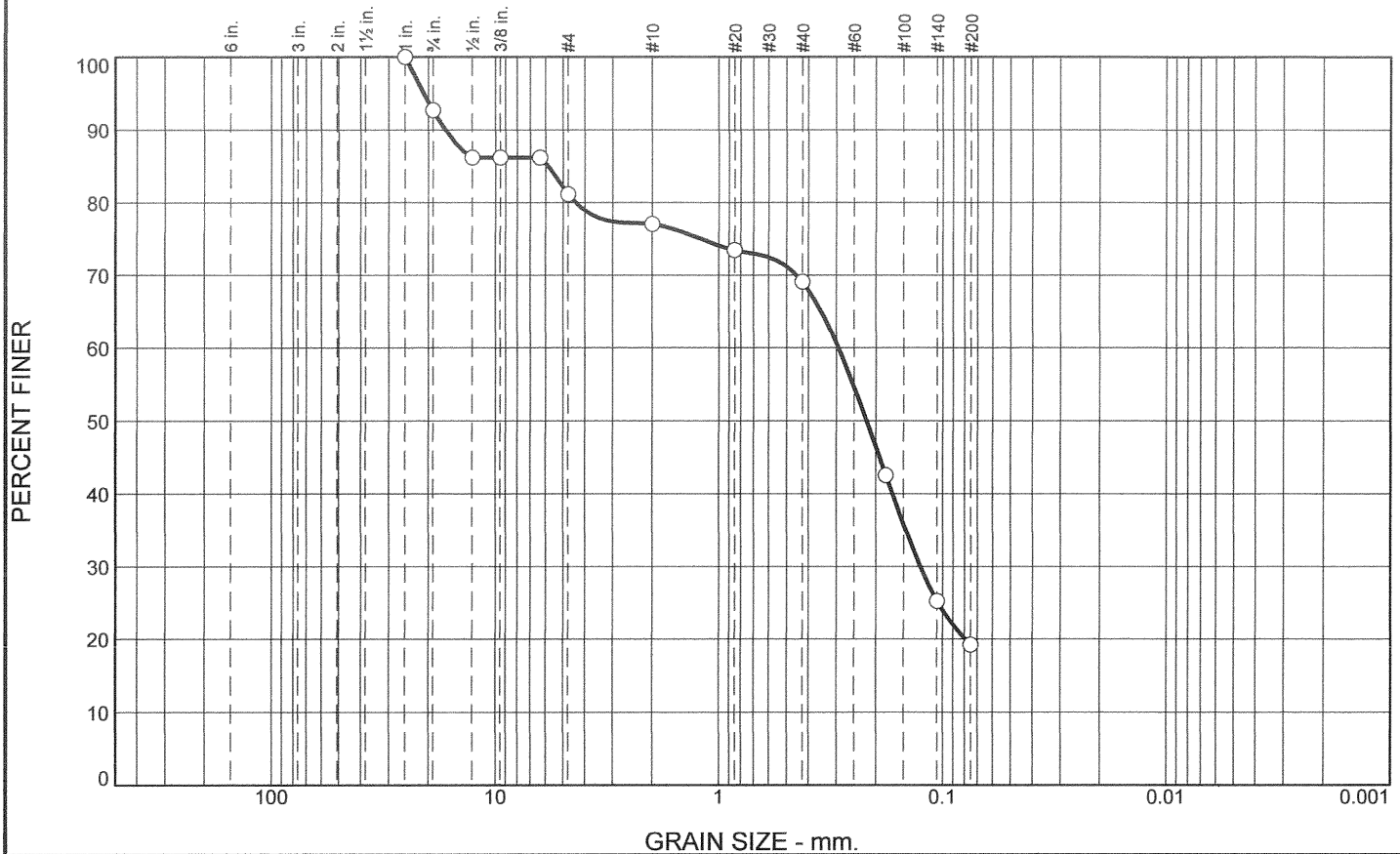
Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013 Lab No. 14626b

Tested By: JJB

Checked By: MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.3	11.5	4.1	8.0	49.9	19.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	92.7		
1/2"	86.2		
3/8"	86.2		
1/4"	86.2		
#4	81.2		
#10	77.1		
#20	73.4		
#40	69.1		
#80	42.5		
#140	25.3		
#200	19.2		

* (no specification provided)

<u>Soil Description</u>		
Silty Sand with Gravel		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₉₀ = 16.8024	D ₈₅ = 5.8436	D ₆₀ = 0.2938
D ₅₀ = 0.2197	D ₃₀ = 0.1261	D ₁₅ =
D ₁₀ =	C _u =	C _c =
<u>Classification</u>		
USCS= SM	AASHTO= A-2-4(0)	
<u>Remarks</u>		
Moisture Content: 14.7%		

Location: BB-GCS-501, Gardiner, ME
Sample Number: 9D Depth: 25'-27'

Date: 8/1/17

**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

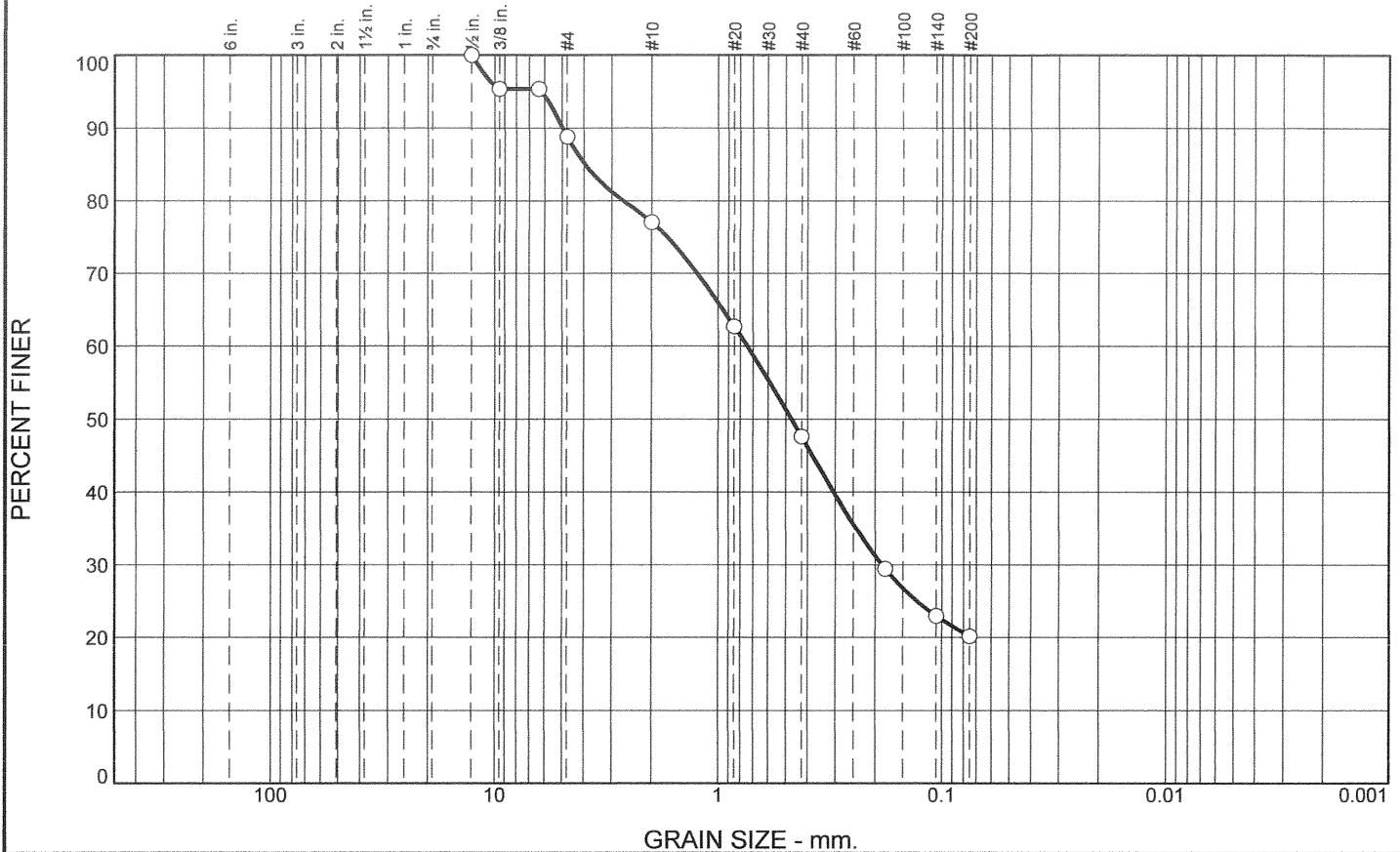
Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013 Lab No. 14626c

Tested By: JJB

Checked By: MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.2	11.7	29.5	27.4	20.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	95.4		
1/4"	95.4		
#4	88.8		
#10	77.1		
#20	62.7		
#40	47.6		
#80	29.4		
#140	23.0		
#200	20.2		

* (no specification provided)

<u>Soil Description</u>		
Silty Sand		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₉₀ = 4.9764	D ₈₅ = 3.9608	D ₆₀ = 0.7450
D ₅₀ = 0.4718	D ₃₀ = 0.1866	D ₁₅ =
D ₁₀ =	C _u =	C _c =
<u>Classification</u>		
USCS= SM	AASHTO= A-1-b	
<u>Remarks</u>		
Moisture Content: 7.1%		

Location: BB-GCS-502, Gardiner, ME
Sample Number: 4D Depth: 6'-8'

Date: 8/1/17

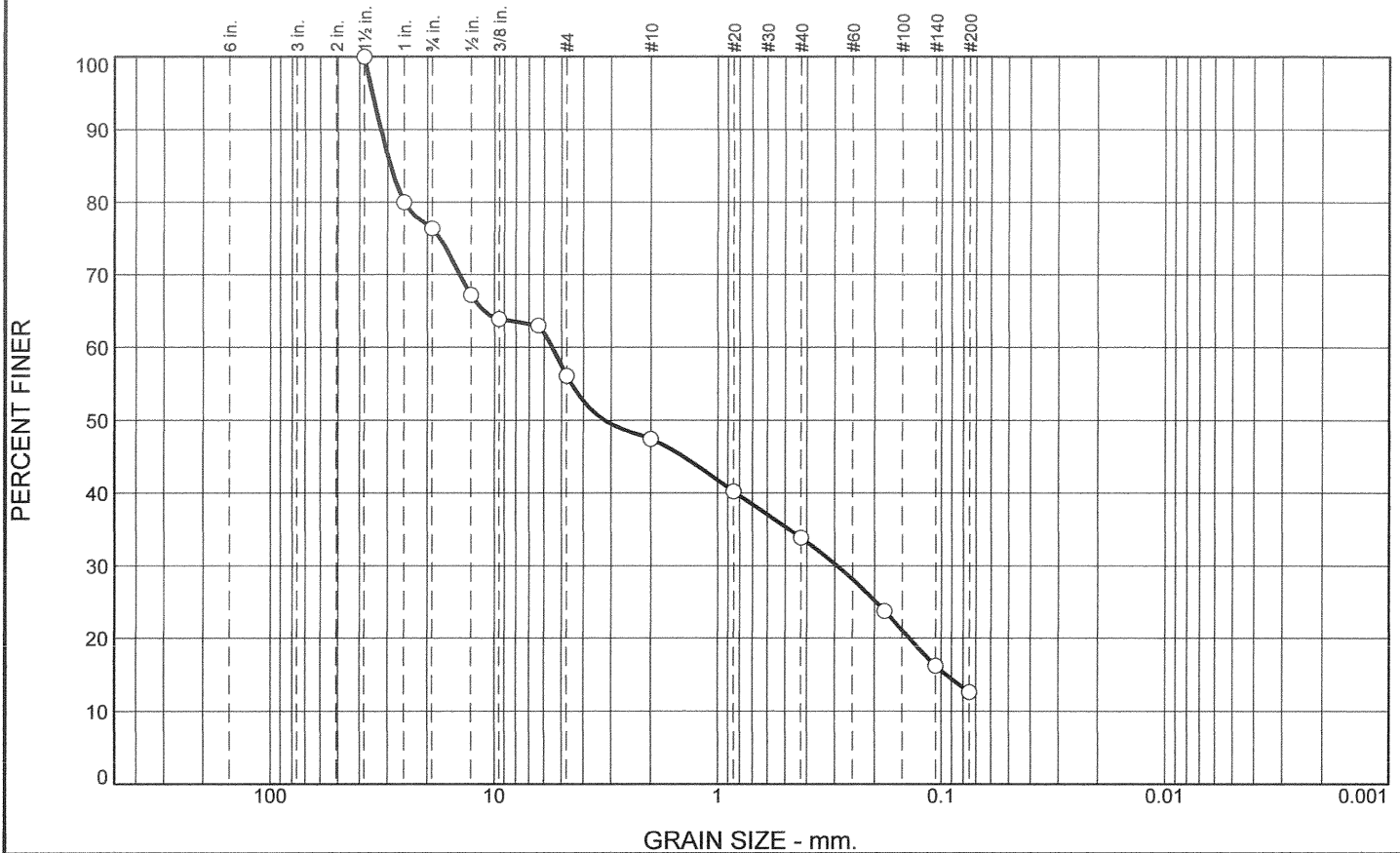
**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013
Lab No. 14626d

Tested By: JJB

Checked By: MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	23.6	20.3	8.7	13.6	21.2	12.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	80.0		
3/4"	76.4		
1/2"	67.2		
3/8"	63.9		
1/4"	63.0		
#4	56.1		
#10	47.4		
#20	40.2		
#40	33.8		
#80	23.8		
#140	16.3		
#200	12.6		

* (no specification provided)

<u>Soil Description</u>		
Silty Gravel with Sand		
<u>Atterberg Limits</u>		
PL=	LL=	PI=
<u>Coefficients</u>		
D ₉₀ = 32.2082	D ₈₅ = 29.1968	D ₆₀ = 5.5088
D ₅₀ = 3.2085	D ₃₀ = 0.2934	D ₁₅ = 0.0950
D ₁₀ =	C _u =	C _c =
<u>Classification</u>		
USCS= GM	AASHTO= A-1-b	
<u>Remarks</u>		
Moisture Content: 10.3%		

Location: BB-GCS-502, Gardiner, ME
Sample Number: 10D Depth: 20'-22'

Date: 8/1/17

**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

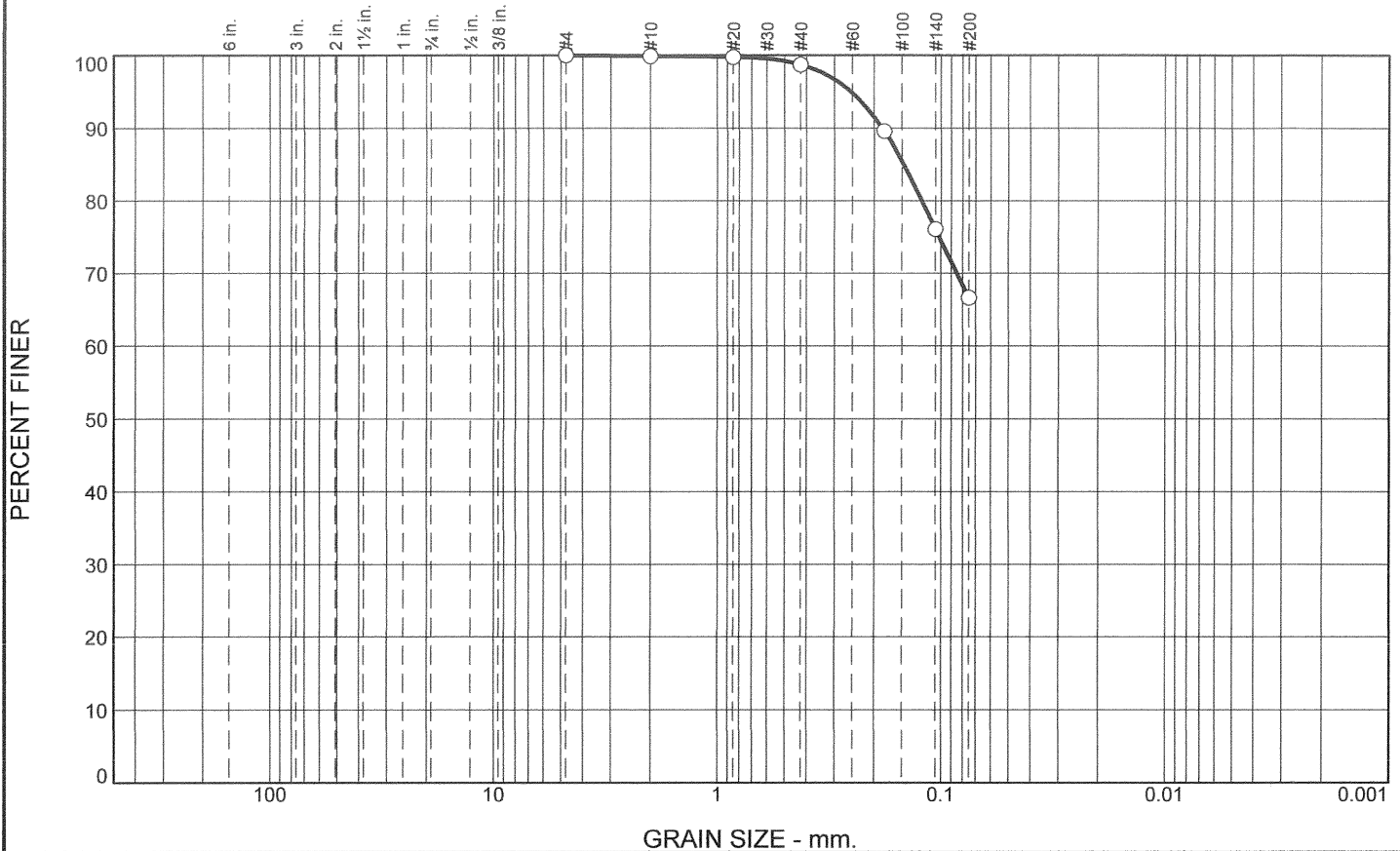
Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013
Lab No. 14626e

Tested By: JJB

Checked By: MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.2	32.0	66.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.8		
#40	98.7		
#80	89.5		
#140	76.1		
#200	66.7		

* (no specification provided)

Soil Description
Sandy Silt

Atterberg Limits
PL= LL= PI=

Coefficients
D₉₀= 0.1841 D₈₅= 0.1478 D₆₀=
D₅₀= D₃₀= D₁₅=
D₁₀= C_u= C_c=

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

Remarks
Moisture Content: 21.3%

Location: BB-GCS-502, Gardiner, ME
Sample Number: 12D Depth: 30'-32'

Date: 8/1/17

**R.W. Gillespie
& Associates, Inc.
Saco, Maine**

Client: Haley & Aldrich
Project: Multi-use Trail Bridge Over Cobbosseecontee Stream H & A File No.: 42223-003
Project No: 0956-013 Lab No. 14626f

Tested By: JJB

Checked By: MTG

MTG

APPENDIX D

Calculations

Table of Contents

- Frost Protection
- Approach Embankment Global Stability
- Seismic Site Class
- Liquefaction Analysis
- Footing Bearing Resistance
- Footing Sliding Resistance
- Lateral Soil Pressures

Frost Protection

Client:	MAINEDOT
Project:	MULTI-USE TRAIL 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	MOISTURE CONTENT
ABUTMENT NO. 1	BB-GCS-501	El. 12.6	El. 0.6	2D El. 8.6 to 6.6	A-1-b/ SM	13.0	3.3
				8D El. -7.4 to -9.4	A-1-a/ GW	7.2	10.9
ABUTMENT NO. 2	BB-GCS-502	El. 14.4	El. -1.0	4D El. 8.4 to 6.4	A-1-b/ SM	20.2	7.1
				10D El. -5.6 to -7.6	A-1-b/ GM	12.6	10.3

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 from tidal influences and throughout the year due to seasonal variations and precipitation events.

2. Water levels at the site are as follows:

Q1.1 (MLW) El. 1.9
 Q1.1 (MHW) El. 3.9
 Q50 El. 7.5
 Q100 El. 8.2

3. Proposed bottom of abutment elevations are as follows:

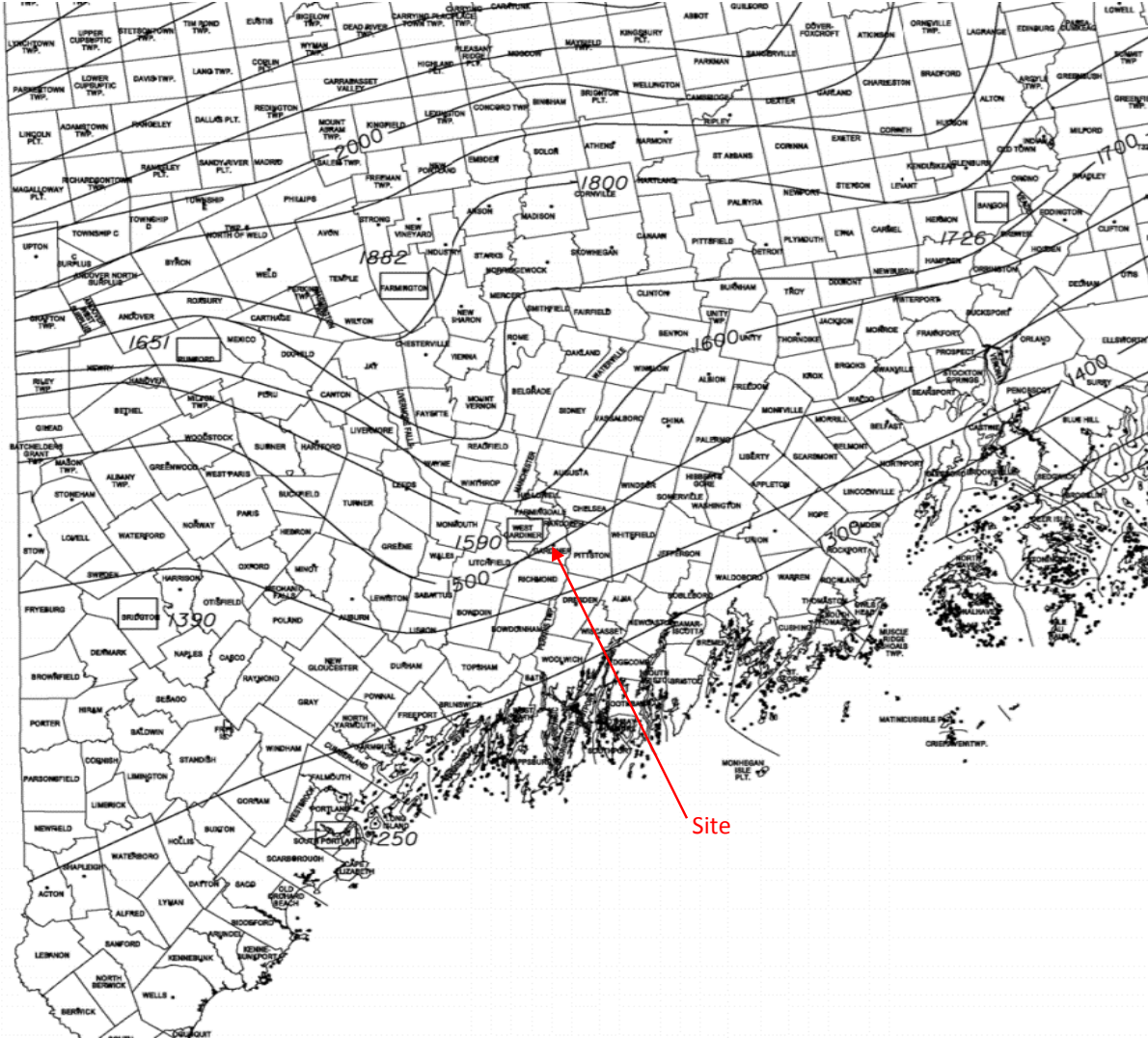
Abutment No. 1 El. -3.0
 Abutment No. 2 El. -3.0

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.

From Table 5-1, frost penetration depths vary between approximately 5 ft (w=30%), 5.5 ft (20%) and 7 ft (w=10%) for coarse-grained soils. Frost penetration depths vary between approximately 4 ft (w=30%) and 5 ft (w=10%).

Recommend 5.5 ft


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 Global Stability

File No.:	42223-006
Sheet:	1 of 1
Date:	27-Jun-2018
Computed by:	KAR
Checked by:	EAF

Client:	Maine Department of Transportation
Project:	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine
Subject:	Global Stability

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for the proposed bridge structure.

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, Maine Avenue Bridge over Cobbosseecontee Stream," dated August 2017.

ASSUMPTIONS

1. Water level will be modeled at Q1.1 at El. 3.9 (MHW).
2. Seismic cases will have a seismic force of $A_s/2$ ($0.193g/2$) = 0.0965 g based on the seismic site class calculations .
3. A 250 psf traffic surcharge will be modeled.
4. A 3,500 psf service limit bearing pressure will be modeled at each abutment footing.

SOIL PROPERTIES

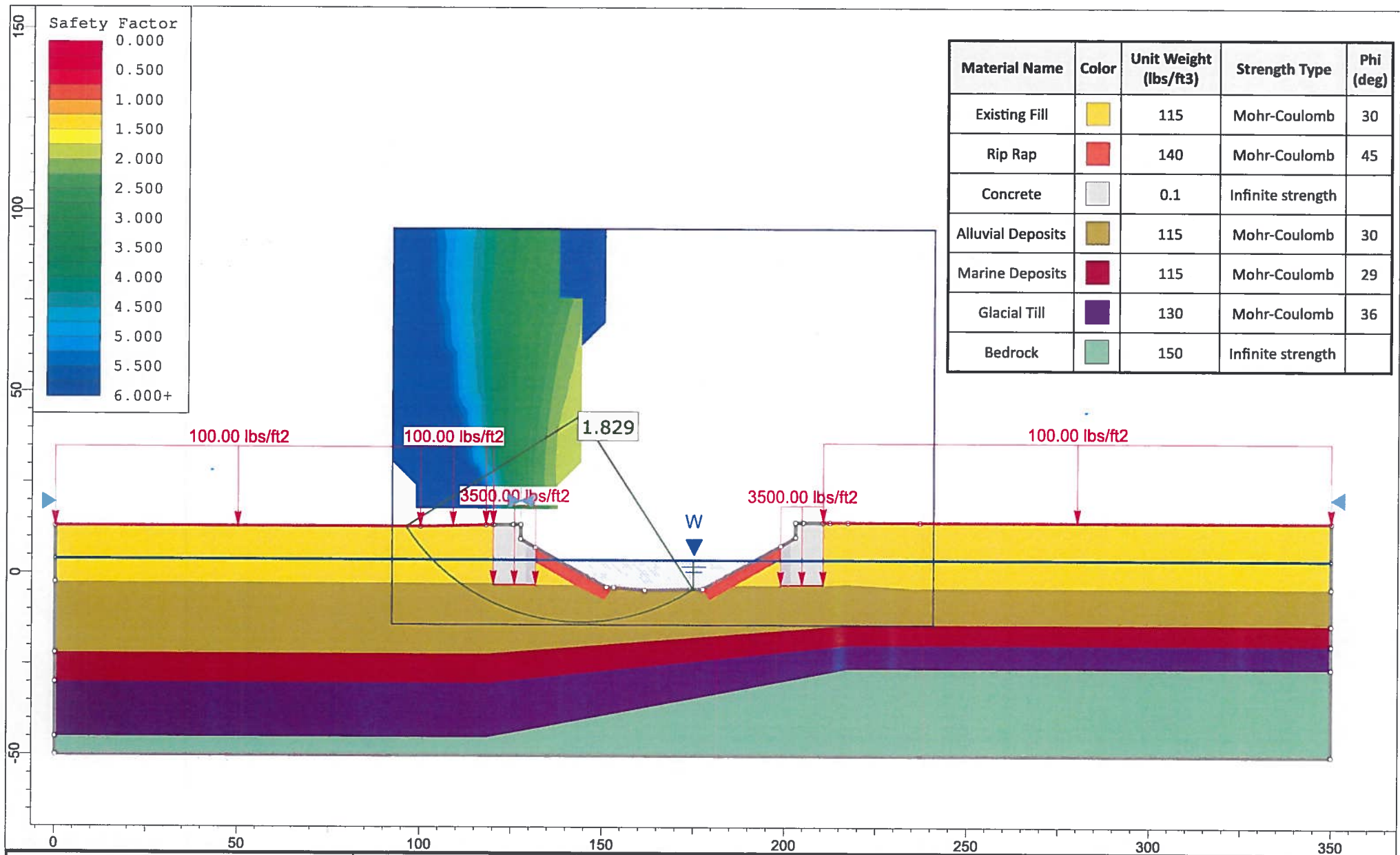
Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
Existing Fill	115	30	0
Granular Borrow	125	32	0
Rip Rap	140	45	0
Alluvial Deposit	115	30	0
Marine Deposit - Granular	115	29	0
Glacial Till/Weathered Bedrock	130	36	0
Bedrock and Abutments	infinite strength		

RESULTS AND CONCLUSIONS

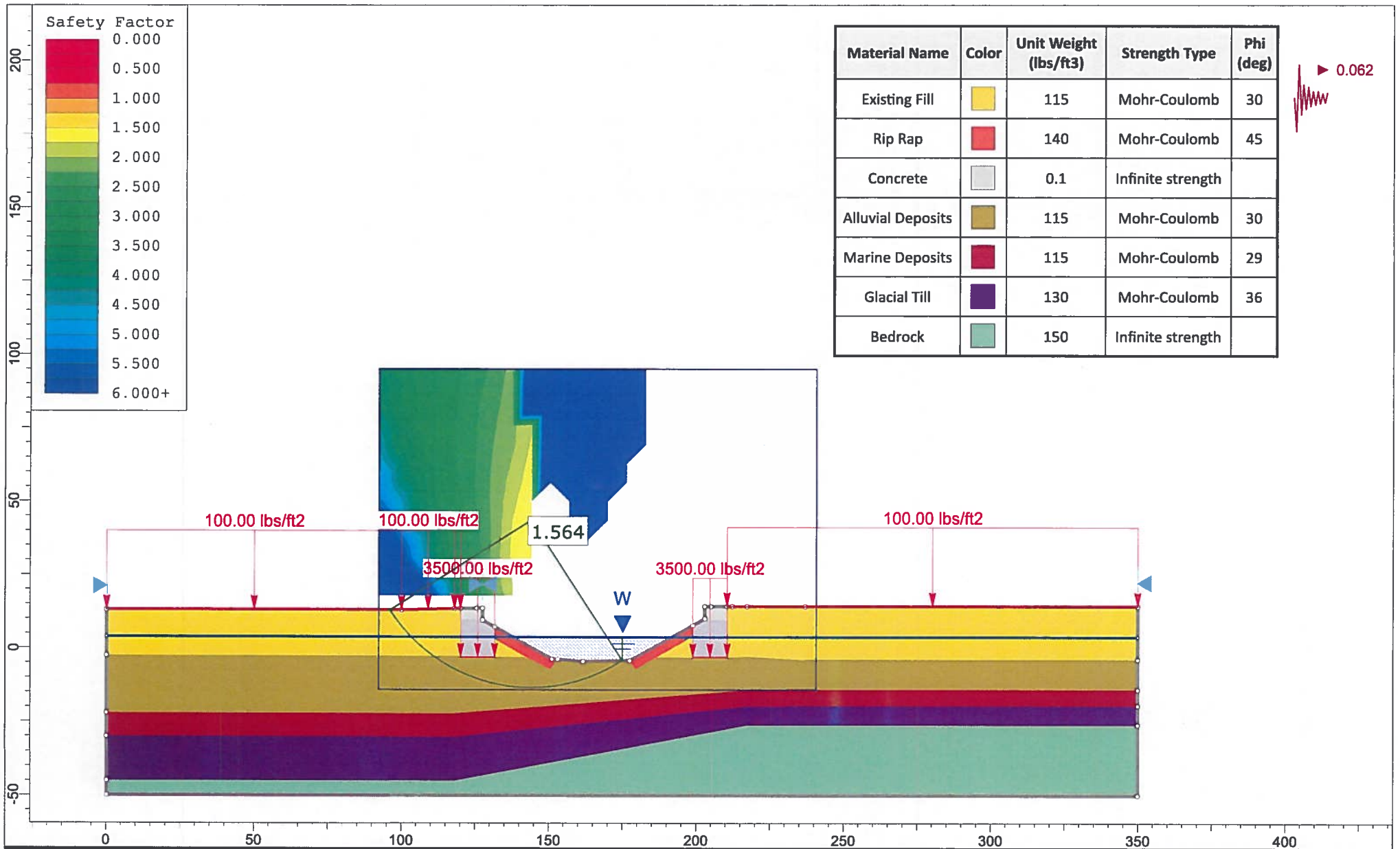
	F.S.	
	Static	Seismic
Abutment 1	1.83	1.56
Abutment 2	1.53	1.32

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. = $1/0.65 = 1.5$).

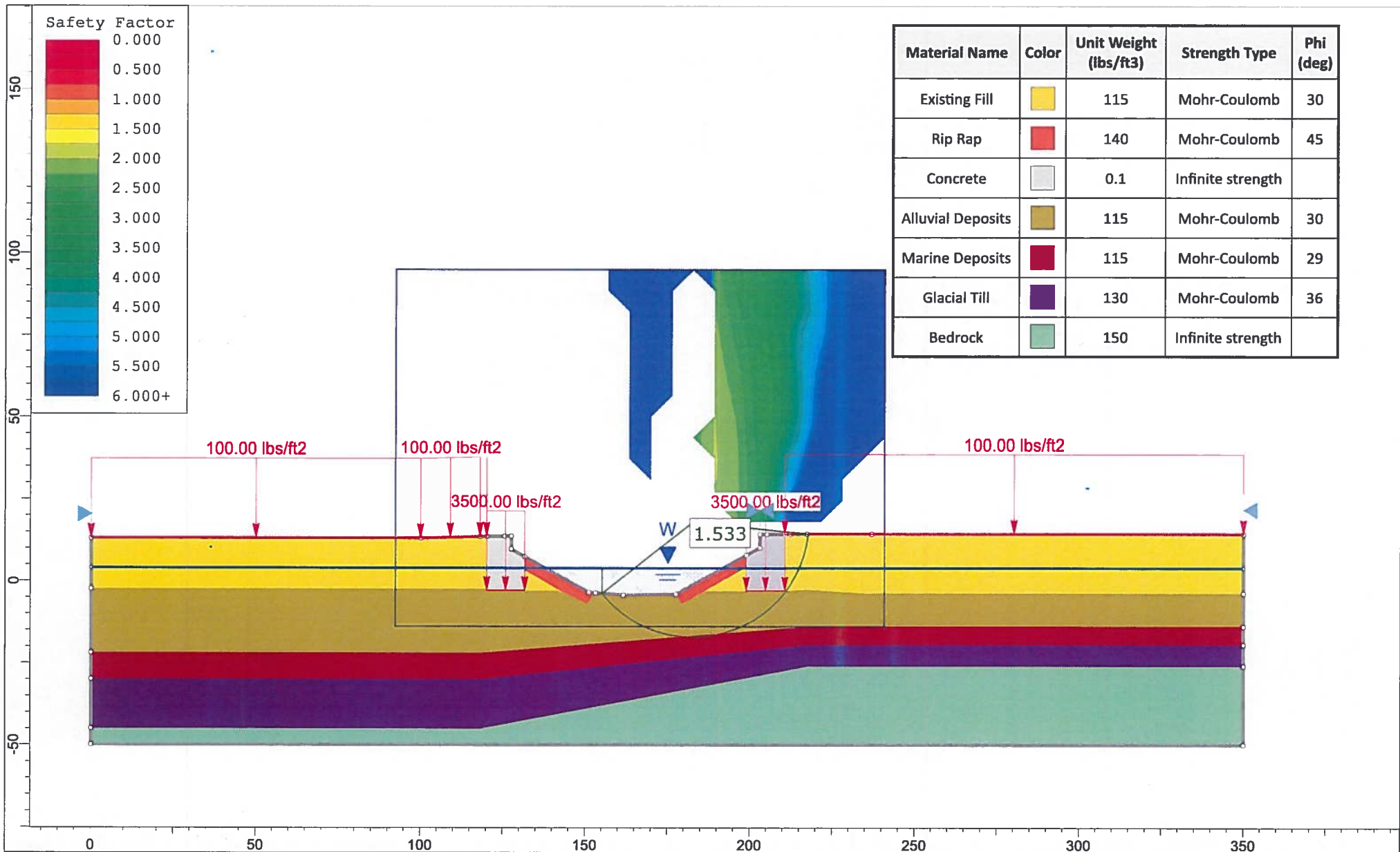
Based on Maine DOT Bridge Design Guide Section 5.9.4, a minimum seismic factor of safety of 1.0 is acceptable for slope stability.



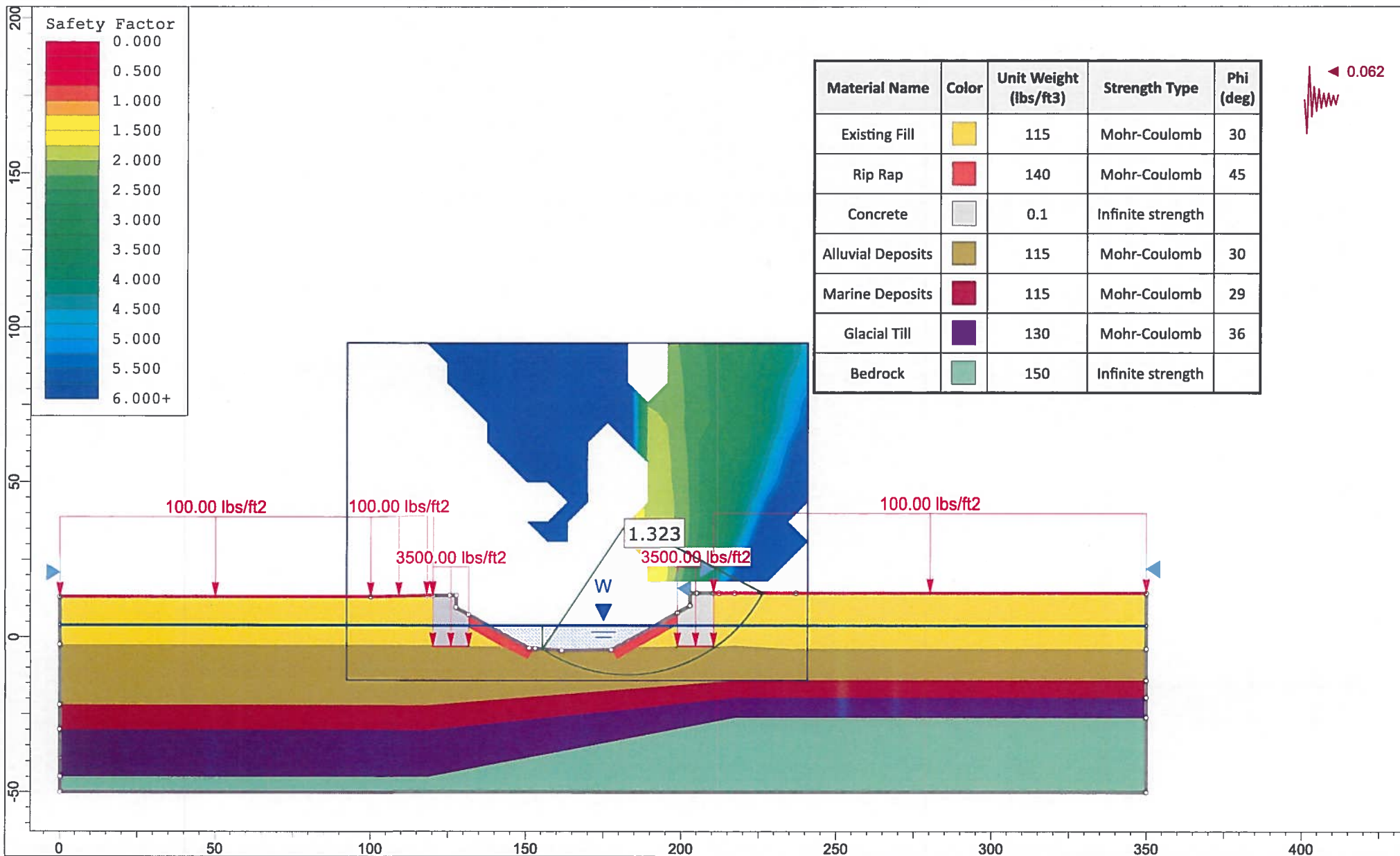
	Project			
	Multi-Use Trail Bridge over Cobbosseecontee Stream - Abutment No. 1			
	Analysis Description			
	Global Stability - Static			
	Drawn By	Kevin A. Russ	Scale	1:436
	Company	Haley & Aldrich, Inc.		
	Date	6/12/2018, 8:56:30 AM		File Name
				2018_0612_Global Stability_Abutment 1_Static.slim



	Project			
	Multi-Use Trail Bridge over Cobbosseecontee Stream - Abutment No. 1			
	Analysis Description			
	Global Stability - Seismic			
	Drawn By	Kevin A. Russ	Scale	1:538
	Date	6/12/2018, 8:56:30 AM	Company	Haley & Aldrich, Inc.
SLIDEINTERPRET 7.018			File Name	2018_0612_Global Stability_Abutment 1_Static.slm



	Project			
	Multi-Use Trail Bridge over Cobbosseecontee Stream - Abutment No. 2			
	Analysis Description			
	Global Stability - Static			
SLIDEINTERPRET 7.018	Drawn By	Kevin A. Russ	Scale	1:485
	Date	6/12/2018, 8:56:30 AM	Company	Haley & Aldrich, Inc.
			File Name	2018_0612_Global Stability_Abutment 1_Static.slim



roc science

SLIDEINTERPRET 7.018

Project				
Multi-Use Trail Bridge over Cobbosseecontee Stream - Abutment No. 2				
Analysis Description				
Global Stability - Seismic				
Drawn By	Kevin A. Russ	Scale	1:515	Company
				Haley & Aldrich, Inc.
Date	6/12/2018, 8:56:30 AM			File Name
				2018_0612_Global Stability_Abutment 1_Static.slim

Seismic Site Class

Client	Stantec
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine
Subject	Seismic Site Class Evaluation

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using available subsurface, SPT N information.

EXECUTIVE SUMMARY

A Seismic Site Class D is recommended.

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
3. United States Geological Survey (USGS) earthquake hazards website.

AVAILABLE INFORMATION

1. Boring logs BB-GCS-501 and BB-GCS-502 dated July 2017.
2. Other: e.g., Vs or geophysical data report, lab data, geotech reports, etc.

ASSUMPTIONS

1. Where SPT N data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N for the extended profile was then assumed based on the available information.
2. Earthquake event return period of approximately 1,000 years.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A

Average shear wave velocity for the upper 100 ft of the soil profile:

$$\bar{V}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of i th soil (ft/s).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Client Stantec

Date 25-Jul-17

Project Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine

Computed by MMH

Subject Seismic Site Class Evaluation

Checked by JLL

PROCEDURE**Method B**

Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}}$$

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th soil layer not to exceed 100 ft (blows/ft).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Method C

Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th cohesionless soil layer (blows/ft).

d_i = thickness of i th cohesionless soil layer (ft).

m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and m .

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

$$\bar{s}_u = \frac{\sum_{i=1}^k d_i}{\sum_{i=1}^k \frac{d_i}{s_{ui}}}$$

where

s_{ui} = undrained shear strength of i th cohesive soil layer (psf), not to exceed 5000 psf

d_i = thickness of i th cohesive soil layer (ft).

k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and k .

Based on the available information, Method B will be used for the seismic Site Class evaluation.

Client Stantec

Date 25-Jul-17

Project Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine

Computed by MMH

Subject Seismic Site Class Evaluation

Checked by JLL

SITE CLASS DEFINITIONS

(Table from AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (with 2012 Interim Revisions)).

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with $2500 \text{ ft/sec} < \bar{v}_s < 5000 \text{ ft/sec}$
C	Very dense soil and soil rock with $1200 \text{ ft/sec} < \bar{v}_s < 2500 \text{ ft/sec}$, or with either $\bar{N} > 50$ blows/ft or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with $600 \text{ ft/sec} < \bar{v}_s < 1200 \text{ ft/sec}$, or with either $15 \text{ blows/ft} < \bar{N} < 50 \text{ blows/ft}$ or $1.0 \text{ ksf} < \bar{s}_u < 2.0 \text{ ksf}$
E	Soil profile with $\bar{v}_s < 600 \text{ ft/sec}$, or with either $\bar{N} < 15 \text{ blows/ft}$ or $\bar{s}_u < 1.0 \text{ ksf}$, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40\%$, and $\bar{s}_u < 0.5 \text{ ksf}$
F	Soils requiring site-specific ground motion response evaluations, such as: - Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay, where H = thickness of soil) - Very high plasticity clays ($H > 25$ ft with $PI > 75$) - Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions:

Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.

where:
 $\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

 $\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

 $\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

 PI = plasticity index (ASTM D 4318)

 w = moisture content (ASTM D 2216)

Client	Stantec
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-GCS-501
 Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2	-2	Fill	3.5	38	0.092
2D	5	-5	Fill	3.0	23	0.130
3D	8	-8	Fill	2.5	16	0.156
4D	10	-10	Fill	2.0	41	0.049
5D	12	-12	Fill	2.0	19	0.105
6D	14	-14	Fill	2.0	8	0.250
7D	17	-17	Alluvial	4.0	98	0.041
8D	21	-21	Alluvial	3.8	76	0.050
9D	26	-26	Marine Sand	5.7	5	1.140
10D	31	-31	Marine Sand	5.0	11	0.455
MD	36	-36	Marine Sand	5.0	6	0.833
11D	41	-41	Marine Sand	3.7	22	0.168
12D	46	-46	Glacial Till	15.5	30	0.517
	57.7	-57.7	Top of Rock	42.3	100	0.423
Totals =				100.0		4.409

N-bar (blows/ft) = 22.7
 Site Class = D

CALCULATIONS

File No.	42223-003
Sheet	5 of 6
Date	25-Jul-17
Computed by	MMH
Checked by	JLL

Client	Stantec
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-GCS-502
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1	-1	Fill	2.0	12	0.167
2D	3	-3	Fill	2.0	8	0.250
3D	5	-5	Fill	2.0	15	0.133
4D	7	-7	Fill	2.0	9	0.222
5D	9	-9	Fill	2.0	13	0.154
6D	11	-11	Fill	2.0	2	1.000
7D	13	-13	Fill	2.0	10	0.200
8D	15	-15	Fill	2	5	0.400
9D	17	-17	Fill	1.0	34	0.029
10D	21	-21	Alluvial Deposit	7.5	66	0.114
11D	28	-28	Alluvial Deposit	5.5	13	0.423
12D	31	-31	Marine Deposit	4.0	4	1.000
13D	36	-36	Glacial Till	6.5	15	0.433
	40.5	-40.5	Top of Rock	59.5	100	0.595
Totals =				100.0		5.121

N-bar (blows/ft) = 19.5
Site Class = D

Client Stantec

Project Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine

Subject Seismic Site Class Evaluation

RESULTS SUMMARY

Boring Number	Parameter	Average Value	Site Class
BB-GCS-501	SPT-N	22.7	D
BB-GCS-502	SPT-N	19.5	D

CONCLUSIONS & RECOMMENDATIONS

Latitude: 44.230595

Longitude: -69.770976

Return Period: 1000 years

PGA = 0.077 g

S_s = 0.160 g

S_1 = 0.045 g

F_{PGA} = 1.6

F_a = 1.6

F_v = 2.4

A_s = 0.123 g

S_{DS} = 0.256 g

S_{D1} = 0.108 g

In accordance with AASHTO, the seismic design category is A.

Article 3.5 - Selection of Seismic Design Category (SDC)

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

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

For $S_{D1} = 0.108 g$, Seismic Design Category = A

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

Liquefaction Analysis

	CALCULATIONS												File No.	42223-003
													Sheet	1 of 3
Client	Stantec												Date	25-Jul-17
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine												Computed by	MMH
Subject	Liquefaction Analyses for Abutments												Checked by	JLL

PROBLEM STATEMENT AND OBJECTIVE

-Evaluate the liquefaction resistance using the Youd and Idriss (2001) method for calculating factor of safety (FS) against liquefaction.

EXECUTIVE SUMMARY

The factor of safety against liquefaction at the site is above 1.1 for the soil samples evaluated.

REFERENCES

1. Youd and Idriss, "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils," 2001.

AVAILABLE INFORMATION

- 1. Haley and Aldrich borings BB-GCS-501 and BB-GCS-502 dated July 2017.
- 2. De-aggregation plot from the United State Geological Survey (USGS) Earthquake Hazards website.
- 3. As of 0.123 g from seismic site class calculations.
- 4. Q1.1 water is at El. 3.8, which is a depth of 9.2 ft below ground surface for boring BB-GCS-501 and a depth of 10.2 ft below ground surface for boring BB-GCS-502.

ASSUMPTIONS

- 1. Total unit weights were estimated as follows: Fill 115 pcf, Alluvial Deposit 120 pcf, Marine Deposit 120 pcf, and Glacial Till 135 pcf.
- 2. Ground water depth assumed to correspond to Q1.1 water level for liquefaction evaluations.
- 3. Fines content percentages were estimated in the field by the field geologist.
- 4. Magnitude M=5.98 is based on the mean M from the USGS deaggregation plot.

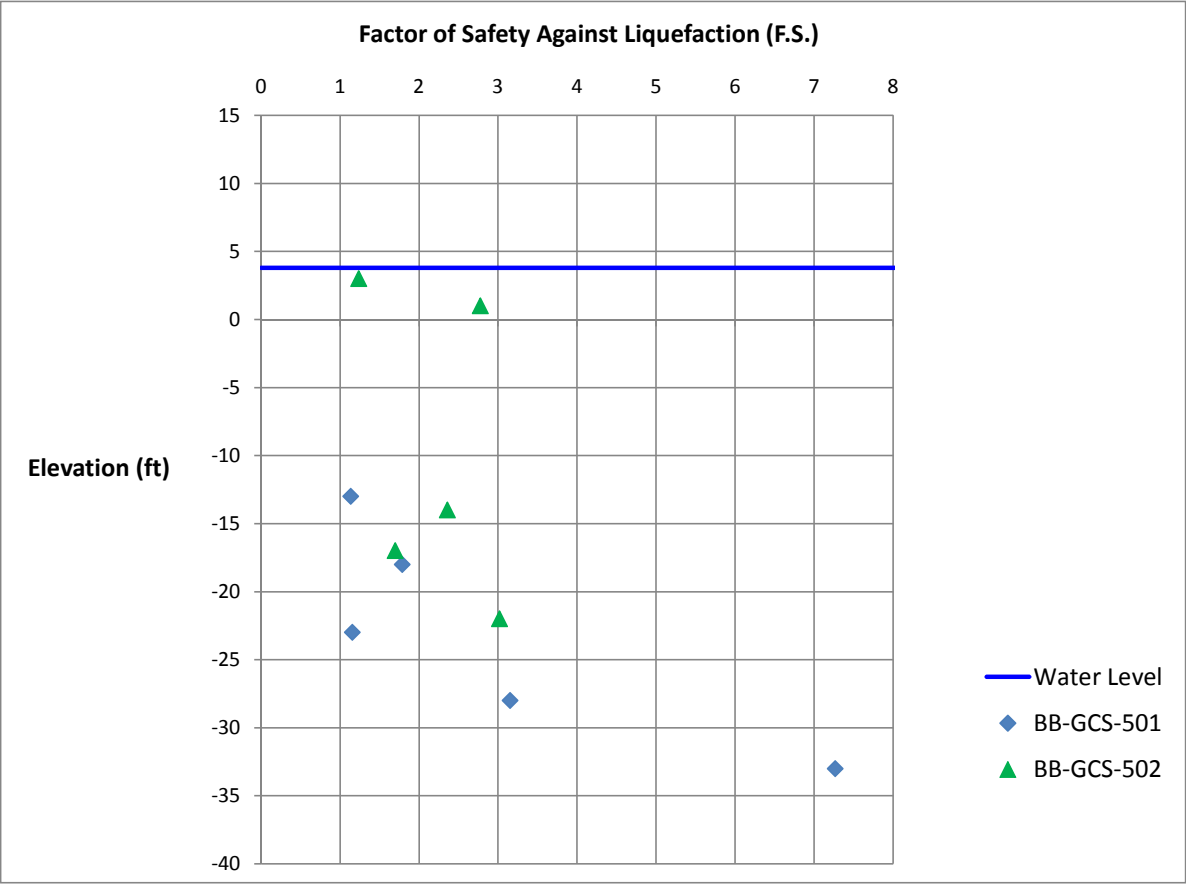
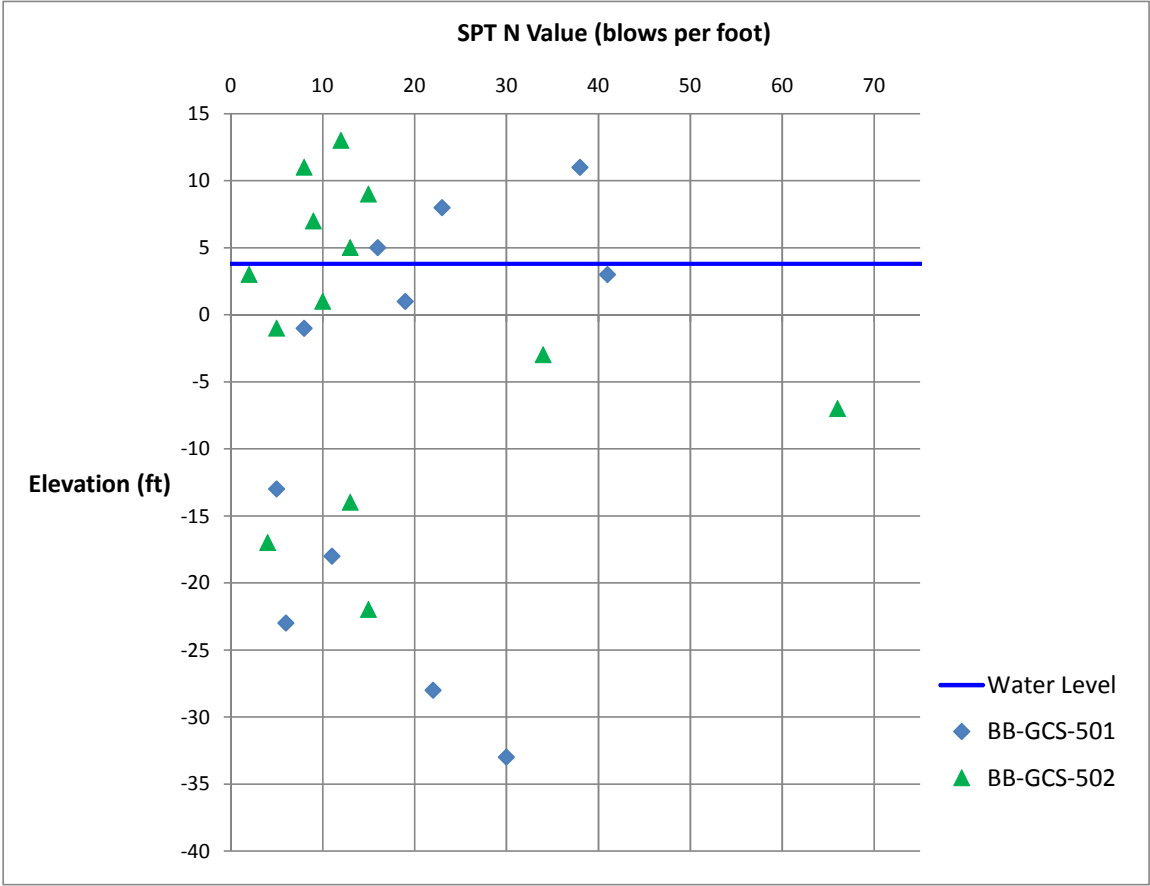
CALCULATIONS

Boring No.	BB-GCS-501
Approximate Ground Surface Elevation	13 ft
Depth to Water Table, d _w	9.2 ft
Unit weight of water, Y _w	62.4 pcf
Site Specific PGA, A _s	0.123 g
Magnitude, M	5.98 from deaggregation plot
Magnitude Scaling Factor, MSF	1.79

Depth (ft)	Elevation	Description	SPT N	Yt (pcf)	σv (psf)	σw (psf)	σ'v (psf)	Cn	(N1) ₆₀	Fines Content (%)	α	β	(N1) ₆₀ CS	CRR 7.5	r _d	CSR	FS=(CRR 7.5/CSR)*MSF
2	11	Fill	38	115	230	0	230	1.70	65	0	0.00	1.00	65	NL	1.00	0.080	#N/A
5	8	Fill	23	115	575	0	575	1.70	39	0	0.00	1.00	39	NL	0.99	0.079	#N/A
8	5	Fill	16	115	920	0	920	1.51	24	0	0.00	1.00	24	0.275	0.98	0.078	#N/A
10	3	Fill	41	115	1,150	50	1,100	1.38	56	10	0.87	1.02	59	NL	0.98	0.082	#N/A
12	1	Fill	19	115	1,380	175	1,205	1.32	25	25	4.29	1.12	32	NL	0.97	0.089	#N/A
14	-1	Fill	8	115	1,610	300	1,310	1.26	10	0	0.00	1.00	10	NL	0.97	0.095	#N/A
17	-4	Alluvial	98	120	1,970	487	1,483	1.19	116	0	0	1	116	NL	0.96	0.102	#N/A
21	-8	Alluvial	76	120	2,450	736	1,714	1.10	84	5	0	1	84	NL	0.95	0.109	#N/A
26	-13	Marine Deposit	5	120	3,050	1,048	2,002	1.02	5	5	0	1	5	0.073	0.94	0.114	1.1
31	-18	Marine Deposit	11	120	3,650	1,360	2,290	0.95	11	5	0	1	11	0.118	0.92	0.117	1.8
36	-23	Marine Deposit	6	120	4,250	1,672	2,578	0.90	5	0	0	1	5	0.075	0.88	0.116	1.2
41	-28	Marine Deposit	22	120	4,850	1,984	2,866	0.85	19	0	0	1	19	0.201	0.84	0.114	3.2
46	-33	Glacial Till	30	135	5,525	2,296	3,229	0.80	24	20	3.61	1.08	30	0.445	0.80	0.109	7.3

sawdust, wood chips

HALEYALDRICH Client Project Subject	CALCULATIONS		File No.	42223-003
			Sheet	3 of 3
			Date	25-Jul-17
			Computed by	MMH
	Stantec		Checked by	JLL
	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine			
	Liquefaction Analyses for Abutments			



- Notes:
1. Non-liquefiable samples ("NL") are not plotted above.
 2. Q1.1 water elevation = 3.8 shown on plots, all samples above water level designated as NL.

CONCLUSIONS

The factor of safety against liquefaction of the soil samples evaluated is over 1.0.

Footing Bearing Resistance

	CALCULATIONS	File No.	42223-003
		Sheet	1 of 3
Client	Stantec	Date	27-Jul-17
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine	Computed by	MMH
Subject	Bearing Resistance for Abutment Footings	Checked by	JLL

PROBLEM STATEMENT AND OBJECTIVE

Calculate the bearing resistance and settlement for the abutment footings for the bridge.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2014 (AASHTO).

AVAILABLE INFORMATION

1. Haley & Aldrich borings BB-GCS-501 and BB-GCS-502 dated July 2017.
2. Email from Stantec dated 18 July 2017 regarding footing dimensions and loading package and 28 July 2017 email regarding pressures and eccentricity for 12-ft wide footing.
3. Plan sheet 6 dated July 2017 with profile of proposed bridge.

ASSUMPTIONS

1. Strength limit state eccentricity (2.3 ft) considered for effective width.
2. Footing size will be 12 ft (width) by 20 ft (length).
3. Slope in front of footing is approximately 30 degrees.
4. Elastic Modulus and Poisson's Ratio of bearing material for settlement calculation from Table C10.4.6.3-1 in AASHTO.
5. Footing depth is 12 ft from ground surface, (i.e., top of slope in front of wall).
6. Footing will be located at the top of the slope. The bottom graph in Figure 10.6.3.1.2c-2 in AASHTO was used to determine N_{γ} .

CALCULATIONS

Strength Limit State Calculations

Article 10.6.3.1.2a- Basic Formulation for Nominal Bearing Resistance

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

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

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

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

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)

D_f = footing embedment depth (ft)

B = footing width (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_{γ} = 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_{γ} = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

***Based on 10.6.3.1.2c Considerations for Footings on Slopes, N_c and N_{γ} from Equation 10.6.3.1.2a-1 shall be replaced with N_{cq} and $N_{\gamma q}$ respectively, from Figures 10.6.3.1.2c-1 and 10.6.3.1.2c-2 for footings bearing on or near slopes.

HALEY ALDRICH	CALCULATIONS	File No.	42223-003
Client	Stantec	Sheet	2 of 3
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine	Date	27-Jul-17
Subject	Bearing Resistance for Abutment Footings	Computed by	MMH
		Checked by	JLL

Since the footing is founded on granular material and $c=0$ psf, ignore first term of equation.

Distance from slope to footing edge, b	0.1	ft		
Footing Length, L	20	ft	From Plot:	
Unit Weight, γ	120	pcf	D_f/B	$\phi=34$ deg., $i=30$ deg.
Bearing Friction, ϕ	34	degrees	0	8
$N_{\gamma q}$	59		1	40
Shape Factor, S_γ	0.85	from Figure 10.6.3.1.2c-2	1.6	59.2
$N_{\gamma m}$	50.27	Table 10.6.3.1.2a-3	1.9	68.8
Depth to Water, D_w	9.2	Equation 10.6.3.1.2a-4	2.75	96
Footing Depth, D_f	12			
$C_w \gamma$	0.5	Table 10.6.3.1.2a-2		
i_γ	1	Equations 10.6.3.1.2a-5 through 10.6.3.1.2a-9 ; estimated at 1		
RF	0.45	Table 10.5.5.2.2-1		

Footing Width, B (ft)	Eccentricity, e (ft)	Effective Footing Width, B' (ft)	D_f/B'	b/B'	Nominal Bearing Resistance, q_n (ksf)	Factored Bearing Resistance, q_R (ksf)
12	2.30	7.40	1.6	0.0	11.2	5.0

Client	Stantec
Project	Multi-Use Trail Bridge over Cobbosseecontee Stream, Gardiner, Maine
Subject	Bearing Resistance for Abutment Footings

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

Service Limit State Calculations

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

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)

Unit Weight of Fill =	120	pcf
Unit Weight of Water, γ_{H2O} =	62.4	pcf
Footing Depth BGS, Df =	12	ft
Groundwater Depth BGS, DH2O =	9.2	ft
Footing Length, L =	20	ft
Footing Width, B =	12	ft
Contact Pressure, q =	3,500	psf

Calc. tot. stress at base of ftg. =	1440	psf
Calc. water pressure at base of ftg. =	174.72	psf
Calc. eff. stress at base of ftg., P_o =	1265.28	psf
Calc. net fnd. pressure, D_p =	2,235	psf

Boring BB-GCS-501

Sublayer					Unit Weight (pcf)	N-value (bpf)	C'	σ_o'	$\Delta \sigma_v^*$	ΔH_i
Top	Bottom	Midpoint (below footing)	Midpoint (bgs)	Thickness						
0	7.8	3.9	15.9	7.8	120	87	300	1490	2210	0.010
7.8	27.2	17.5	29.5	19.4	120	11	55	2273	759	0.044
27.2	39.5	33.4	45.4	12.3	135	30	75	3866	347	0.006

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

Settlement (ft) =	0.06
Settlement (in) =	0.73

Boring BB-GCS-502

Sublayer					Unit Weight (pcf)	N-value (bpf)	C'	σ_o'	$\Delta \sigma_v^*$	ΔH_i
Top	Bottom	Midpoint (below footing)	Midpoint (bgs)	Thickness						
0	13	6.5	18.5	13.0	120	39.5	115	1640	1713	0.035
13	17	15.0	27.0	4.0	120	4	45	2129	889	0.013
17	23.5	20.3	32.3	6.5	135	15	45	2915	647	0.013

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

Settlement (ft) =	0.06
Settlement (in) =	0.73

For use in Figure 10.6.2.4.2-1

Alluvial Deposits considered as well graded silty sand and gravel

Marine Deposits considered as sand and gravel

Glacial Till considered as silty sand

CONCLUSIONS

The factored bearing resistance for the strength limit state is	5.0	ksf
The elastic settlement at the 3.50 ksf service limit state is about	0.73	in

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.

Lateral Soil Pressures

Client: Stantec

Date: 26-Sep-2018

Project: Multi-Use Trail 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).

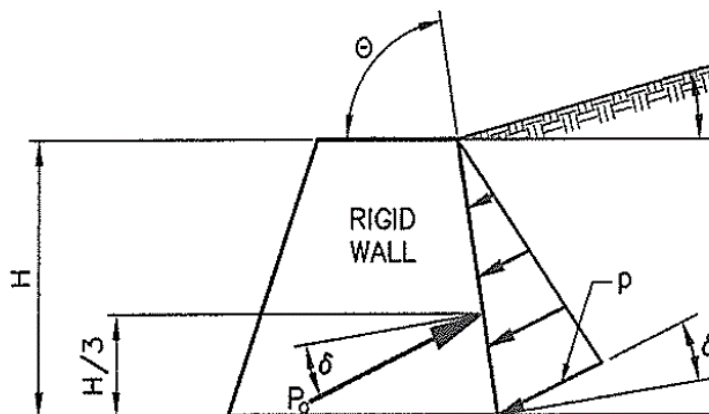


Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure

Client: Stantec

Date: 26-Sep-2018

Project: Multi-Use Trail 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

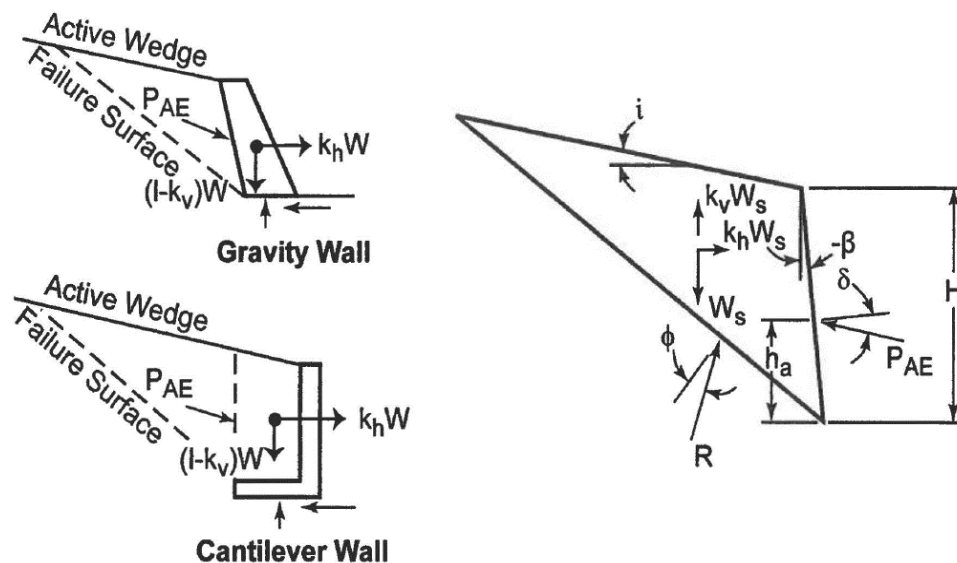


Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams