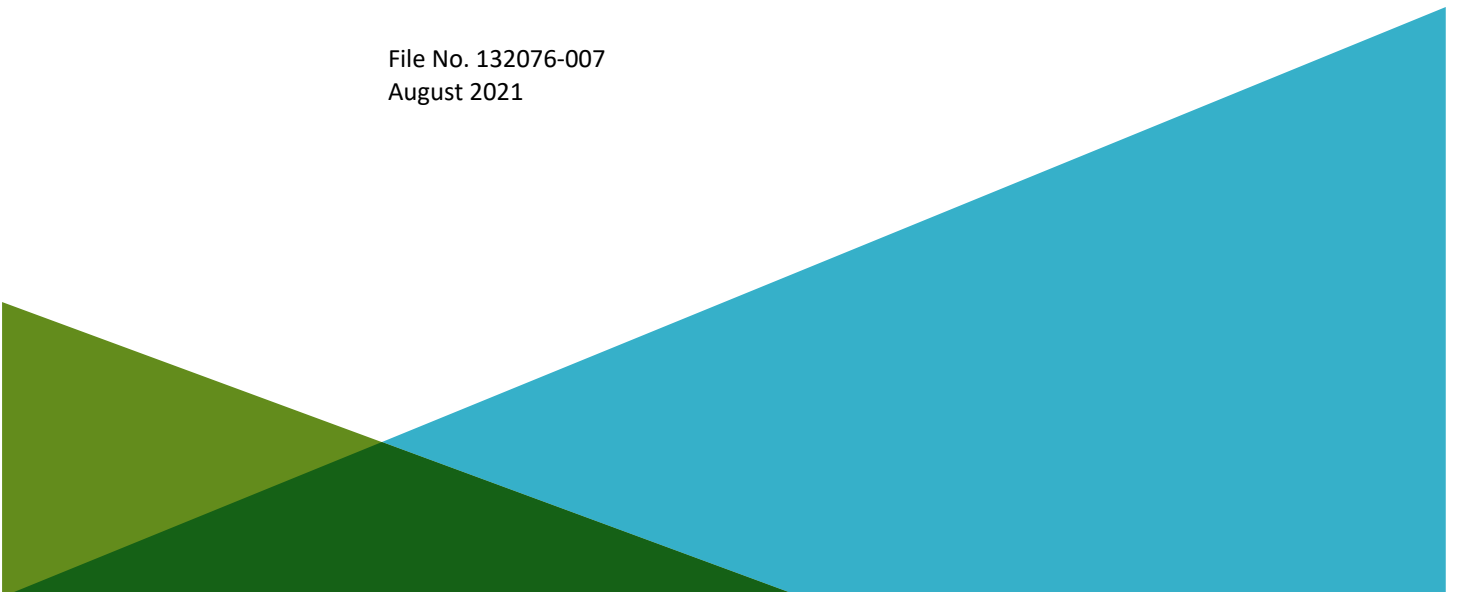


GEOTECHNICAL DESIGN REPORT
INTERSTATE 395/ROUTE 9 CONNECTOR
OVER LAMBERT ROAD, BRIDGE NO. 6647
MAINEDOT WIN 018915.00
EDDINGTON, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Maine Department of Transportation
Augusta, Maine

File No. 132076-007
August 2021





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31 August 2021
File No. 132076-007

Maine Department of Transportation
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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report
Interstate 395/Route 9 Connector over Lambert Road, Bridge No. 6647
MaineDOT WIN 018915.00
Eddington, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Geotechnical Design Report, Interstate 395/Route 9 Connector over Lambert Road, Bridge No. 6647 MaineDOT WIN 018915.00, Eddington, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposal, dated 22 January 2021 and executed by your Richard J. Crawford on 5 February 2021 in accordance with the provisions of our General Consultant Agreement (GCA) with the Maine Department of Transportation (MaineDOT), No. CT20150706000000000010.

Introduction

This GDR presents the results of preliminary (Phase I) and final design (Phase II) phase subsurface investigation and laboratory testing programs, technical evaluations, and geotechnical design recommendations completed by Haley & Aldrich, Inc. (Haley & Aldrich) on behalf of MaineDOT for the proposed bridge that will carry northbound (NB) and southbound (SB) vehicular traffic on the Interstate 395/Route 9 Connector (Connector Bridge) over Lambert Road in Eddington, Maine (see Figure 1, Project Locus).

Please note that geotechnical design recommendations and construction considerations for the Connector roadway (Connector) will be provided under separate cover.

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

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 Central Zone. The project elevation datum and elevations referenced herein are in feet and reference the North

American Vertical Datum of 1988 (NAVD 88). Two baselines were developed by MaineDOT for the proposed horizontal alignments as summarized below:

- Lambert Road: Sta. 6+00 to Sta. 11+00
- Interstate 395/Route 9 Connector: Sta. 196+00 to Sta. 203+00

PROJECT LOCATION AND EXISTING SITE CONDITIONS

The proposed Connector Bridge will carry NB and SB vehicular traffic over Lambert Road, which is generally oriented in an east-west direction, in Eddington, Maine. The project site is comprised of a residential property on the south side of the Lambert Road and a densely wooded area on the north side of road. An existing overhead electric transmission line is present east of the proposed Connector Bridge, which is generally oriented in a northwest-southeast direction. Existing site grades along the Connector range from El. 117 (Sta. 196+00) to El. 122 (Sta. 200+00).

PROPOSED BRIDGE STRUCTURE

During preliminary design, MaineDOT developed and evaluated multiple bridge alternatives considering several factors including but not limited to overall project cost, maintenance of traffic, and future bridge maintenance. The bridge replacement alternative recommended by MaineDOT in the Preliminary Design Report (PDR) consists of a 121-ft long, single-span bridge that is supported on two pile-supported, cast-in-place (CIP) concrete integral abutments at the stations and elevations summarized below.

Substructure	Approximate Station at Centerline of Connector Alignment (ft)	Approximate Proposed Bottom of Abutment Elevation (ft, NAVD 88)
Abutment No. 1	Sta. 197+57.8	El. 131
Abutment No. 2	Sta. 198+78.7	El. 134

The bridge superstructure will be constructed using metalized steel plate girders (five beam lines) running parallel to the long dimension of the bridge, with an 8-in. thick CIP concrete deck and 3-in. thick hot mix asphalt surface and ¼-in. thick high performance waterproofing membrane. The bridge superstructure will be approximately 40-ft wide (shoulder-to-shoulder) and will consist of two, 12-ft wide travel lanes and two, 8-ft wide shoulders. It is our understanding that the structure will be designed as a full integral abutment bridge with the bridge beams cast rigidly into an end diaphragm as described in MaineDOT BDG Section 5.4.2.

Based on our review of profile and cross section drawings developed by MaineDOT, it is our understanding that existing site grades along the Connector alignment will be raised by approximately 25 ft to meet proposed finish grades. The Connector Bridge approach embankment fills will be retained by mechanically-stabilized earth (MSE) walls. The top of the MSE walls are currently envisioned to be at El. 135 and El. 137.5 at Abutment No. 1 and Abutment No. 2, respectively.

Existing and proposed site conditions are shown on Figure 2, Site and Subsurface Exploration Location Plan.

Geologic Setting

Based on our review of the Maine Geological Survey's (MGS's) surficial geology map of the Veazie Quadrangle, Maine (2011), surficial geology in the immediate vicinity of Lambert Road consists of man-placed fill and/or naturally-deposited glacial till soils both of which were encountered in the Phase I and Phase II subsurface explorations completed at the site.

According to Bedrock Geology of the Veazie Quadrangle (2011), bedrock at the site is primarily mapped as siltstone and/or claystone slate of the Brewer Formation. Mapped subordinate rock types consist of fine-grained calcareous quartz-rich meta-arenite and noncalcareous feldspathic metawacke. Thin beds of dark grey to grey-black metalimestone may also be present. The Brewer Formation is Silurian to Ordovician in age. Rock core samples collected in the Phase I and Phase II subsurface explorations at the site generally consisted of metasiltstone.

Please refer to subsequent sections of this GDR for more specific information on the soil and bedrock conditions present at the site.

Subsurface Exploration Programs

PRELIMINARY PHASE (PHASE I) SUBSURFACE INVESTIGATION

Haley & Aldrich completed a preliminary design phase (Phase I) subsurface exploration program at the site in August 2018. The Phase I subsurface investigation consisted of two test borings, designated BB-ELAR-101 and BB-ELAR-102, which were drilled in the vicinity of each proposed Connector Bridge abutment.

The test boring locations were laid out in the field by Haley & Aldrich using global positioning system (GPS) survey equipment prior the start of drilling. The "as-drilled" plan locations of and ground surface elevations at test boring locations were determined in the field by MaineDOT upon the completion of drilling using GPS survey equipment and were provided to Haley & Aldrich. The plan locations of and existing ground surface elevations at Phase I test borings are summarized in Table I and are shown on Figure 2.

The test borings were drilled by Northern Test Boring, Inc. (NTB) of Gorham, Maine using a Diedrich D50 track-mounted drill rig. Test borings were advanced to depths ranging from approximately 16 to 17 ft below ground surface (BGS) using cased-washed drilling methods and a combination of solid-stem augers and 4 in. (HW-size) outside diameter (OD) steel casing.

Soil samples were generally collected continuously through the near-surface fill soils and at standard, 5-ft intervals thereafter, 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. The number of hammer blows required to

advance the sampler through each 6-in. interval was recorded and is provided on the logs. The uncorrected SPT N-value (N-uncorrected) 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 drill rig was equipped with a calibrated automatic hammer per MaineDOT requirements. The energy-corrected SPT N-value (N_{60}), which is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.907; 90.7 percent theoretical hammer efficiency) divided by 0.6, is also provided on the logs.

Each test boring was advanced approximately 10 ft into bedrock using a 2-in. (NQ-size) ID, diamond-tipped core barrel.

Soil and bedrock samples were collected and preserved in glass jars and wooden boxes, respectively. The samples that were not submitted for laboratory testing are available for review upon request. The available soil and bedrock samples are currently being stored at the Haley & Aldrich storage facility in Portland, Maine.

All Phase I drilling and sampling activities were performed in accordance with MaineDOT requirements.

DESIGN PHASE (PHASE II) SUBSURFACE INVESTIGATION

Haley & Aldrich completed a design phase (Phase II) subsurface exploration program at the site in December 2020 and January 2021. The Phase II subsurface investigation consisted of six test borings, designated BB-ELAR-201 through BB-ELAR-204 (including –201A and –203A), which were drilled at/near the proposed Connector Bridge abutments and proposed MSE wall ends. The plan locations of and existing ground surface elevations at the test borings are summarized in Table I and are shown on Figure 2.

The Phase II test borings were laid out in the field by MaineDOT using GPS survey equipment prior to the start of drilling. “As-drilled” test boring locations and ground surface elevations at test boring locations were determined in the field by MaineDOT upon the completion of drilling using GPS survey equipment.

The Phase II test borings were drilled by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile Drill B-53 track-mounted drill rig. Test borings were advanced to depths ranging from approximately 8 to 17 ft BGS using similar means and methods to those used to drill the Phase I test borings. The hammer efficiency factors for the automatic hammer used was 0.852 (85.2 percent theoretical hammer efficiency) as noted on the test boring logs.

Test borings were advanced approximately 6 to 12 ft into bedrock using a 2-in. (NQ-size) ID, diamond-tipped core barrel.

Soil and bedrock samples were collected and preserved in glass jars and wooden boxes, respectively. The samples that were not submitted for laboratory testing are available for review upon request. The available soil and bedrock samples are currently being stored at the Haley & Aldrich storage facility in Portland, Maine.

All Phase II drilling and sampling activities were performed in accordance with MaineDOT requirements.

Generalized Subsurface Conditions

The subsurface conditions present at the site generally consist of the man-placed fill soils overlying naturally-deposited marine soils, glacial till and bedrock. Refer to Table II for a summary of the soil units and encountered thicknesses at each test boring location. Detailed soil and bedrock descriptions are provided on the test boring logs included Appendix A. Refer to Figure 3, Interpretive Subsurface Profile, for a graphical representation of the subsurface conditions present along the proposed Connector Bridge alignment. A general description of each soil/bedrock unit is provided below.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Bituminous Concrete	0.5	A surficial layer of bituminous concrete was encountered in test borings completed within the limits of Lambert Road. <i>(encountered in test boring BB-ELAR-102)</i>
Topsoil/ Fill	0.2 to 0.6/ 4 to 5	Topsoil: very soft to soft SILT (ML) with variable amounts of sand and organic matter; Fill: medium dense to very dense, fine to coarse SAND (SW) with variable amounts of silt and gravel; medium stiff to very stiff SILT (ML) with variable amounts of sand and gravel. <i>(topsoil encountered in test borings BB-ELAR-201, -203 and -206; fill encountered in test borings BB-ELAR-101 and BB-ELAR-102)</i>
Marine Deposit	3	Medium stiff Silty CLAY with variable amounts of fine sand (CL). <i>(encountered in test boring BB-ELAR-204)</i>
Glacial Till	2 to 8	Very soft to hard SILT (ML) with variable amounts of sand and gravel; medium dense to very dense SAND (SM) with variable amounts of silt and gravel; medium dense GRAVEL (GP) with variable amounts of silt and sand. <i>(encountered in each test boring except BB-ELAR-101 and -203)</i>
Bedrock		Top of bedrock surface encountered at depths ranging from approximately 1 to 8 ft BGS (El. 118 to El. 113). <i>(encountered in each test boring)</i>

Please note that soil descriptions provided on the test boring logs do not represent actual field conditions other than at the specific test boring locations. The actual conditions encountered between test boring locations may vary from those described herein.

BEDROCK CONDITIONS

Approximately 6 to 12 ft of bedrock was cored in the Phase I and Phase II test borings. The sampled and recovered bedrock generally consisted of the following:

- Moderately hard, slightly to moderately weathered, grey, aphanitic to fine-grained, METASILTSTONE. Primary joints dip moderate to steep angles and are very close to closely spaced, tight to open, smooth to rough.

- Moderately hard to hard, fresh to slightly weathered, grey, aphanitic, SILTSTONE. Primary joints dip moderate to high angles and are moderate to widely spaced, tight to open, rough.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the SILTSTONE/METASILTSTONE encountered in the test borings drilled at the site ranged from 0 to 100 percent (average = 57 percent), indicating variable rock quality; from very poor to excellent as determined using the MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms Field Identification Information," dated January 2020.

A summary of bedrock core data and detailed descriptions are provided on Table III and on the boring logs in Appendix A, respectively. Photographs of the recovered bedrock core samples are also provided for reference in Appendix A.

GROUNDWATER CONDITIONS

Observation wells were not installed in any of the completed boreholes at the site. As a result, static water levels at the site were not determined. The following general observations were made relative to groundwater conditions during and/or immediately following drilling:

- collected soil samples in test borings BB-ELAR-201 and BB-ELAR-201A were visually observed to be "wet" at/below the ground surface (approximately El. 116 to El. 120), and
- water levels measured in each of the boreholes during or after drilling varied from approximately 1 to 7 ft BGS (El. 122 to El. 113; average = El. 118).

Please note that the visual observations made during drilling and water levels taken during or after drilling was completed may have been affected by drilling means/methods and may not be representative of actual static water levels at the site. In general, groundwater levels can be expected to fluctuate, subject to test boring drilling means/methods, seasonal variation, local soil conditions, topography and precipitation. Groundwater levels encountered during construction may differ from those observed in the test borings.

Laboratory Test Results

Phase I and Phase II laboratory testing programs were conducted by Haley & Aldrich on representative soil and rock samples collected during the preliminary and design phase subsurface exploration programs to aid in soil classification and determination of engineering soil and rock properties. Laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures by GeoTesting Express, Inc. of Acton, Massachusetts. A summary of the lab testing results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results ¹
Grain Size	ASTM D6913	Glacial Till	2	<u>AASHTO Classification:</u> A-4 <u>USCS Classification:</u> ML
Compressive Strength and Elastic Moduli of Rock	ASTM D7012	Bedrock	7	<u>Peak Compressive Stress:</u> 9,747 to 28,776 psi <u>Young's Modulus:</u> 2,770,000 to 14,200,000 psi <u>Poisson's Ratio:</u> 0.15 to 0.43

Notes:¹ psi = pounds per square inch.

Geotechnical Evaluations and Design Recommendations

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

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition, 2020, referred to herein as AASHTO LRFD, and
- MainedOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as Bridge Design Guide.

Engineering calculations that support the design recommendations outlined in this GDR are provided for reference in Appendix C.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 using Method B. In instances where SPT N-values were equal to 0 (i.e., weight of rod or weight of hammer), were greater than 100 blows per foot (bpf) or where bedrock was present, default values of 1 and 100 blows per foot (bpf) were used, respectively.

Based on the nature and thickness of the overburden soils and depth to bedrock at the site, as determined from the test borings, we recommend the site be considered "Site Class C." Spectral accelerations were determined based on the geographic site location and the recommended "Site Class C" designation using the United States Geological Survey (USGS) Seismic Design Web Service, which is based on the AASHTO recommended response spectra for a 7 percent probability of exceedance in 75 years (approx. 1,000-year return period). The recommended seismic design parameters are summarized below.

Design Parameter	Design Value
Site factor for short-period range of acceleration response spectrum, $F_a =$	1.2
Site factor for long-period range of acceleration response spectrum, $F_v =$	1.7
Site factor at zero-period on acceleration response spectrum, $F_{pga} =$	1.2
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, $S_s (g) =$	0.144
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, $S_1 (g) =$	0.043
Peak seismic ground acceleration coeff. on rock, $PGA (g) =$	0.067
Horizontal response spectral acceleration coeff. at 0.2-s period modified by $F_a, S_{D5} (g) =$	0.17
Horizontal response spectral acceleration coeff. at 1.0-s period modified by $F_v, S_{D1} (g) =$	0.07
Peak seismic ground acceleration coefficient modified by $F_{pga}, A_s (g) =$	0.08

In accordance with AASHTO LRFD Section 3.10.6, the site falls within Seismic Zone 1 based on the calculated value of S_{D1} (i.e., $S_{D1} < 0.15$ = Seismic Zone 1 from AASHTO LRFD Table 3.10.6.1).

In accordance with AASHTO LRFD Section 4.7.4.2, single-span bridges do not require seismic loads to be analyzed. However, we recommend that the requirements for superstructure connections and support lengths shall still apply in accordance with the requirements of AASHTO LRFD Sections 4.7.4.4 and 3.10.9.

Based on our review of the soil conditions encountered in the Phase I and Phase II test borings and the results of the laboratory testing, it is our opinion that the potential for saturated granular soils present at the site to liquefy during the design earthquake event is low.

APPROACH EMBANKMENTS

Subsurface soil conditions along the proposed Connector Bridge approach roadway alignment have the potential to affect the planning and design of the proposed embankments. As stated previously, approximately 25 ft of fill will be required to meet proposed finish grades. Global embankment stability and settlement analyses were conducted to assess the feasibility of constructing the approach embankments using normal weight earthfill over the subsurface conditions present at the site. The results of global embankment stability and settlement evaluations are discussed in the sections below.

Global Embankment Stability

Embankment construction using normal weight earthfill can cause excessive vertical and lateral strains that could potentially result in a shear failure of the foundation soil and subsequent failure of the embankments. A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 8.0 by Rocscience to evaluate the likelihood of global stability failures at the site.

Static and pseudo-static seismic (pseudo-static) global stability evaluations were conducted transverse to the proposed Connector Bridge baseline and behind proposed Abutments 1 and 2, where, based on our review of proposed grading plans, maximum raises in grade are anticipated. Static and pseudo-static evaluations were not conducted in the longitudinal direction because the proposed MSE walls will bear directly on bedrock.

Typical soil profiles were developed based on the subsurface conditions encountered in the Phase I and Phase II test borings completed at the site. Based on the nature and consistency of the collected soil samples and the SPT N-values measured during soil sampling, the following physical and strength properties were used to complete the global stability evaluations:

Material	Unit Weight (pcf)	Friction Angle (degrees)	Undrained Shear Strength (psf)
New Fill	125	32	0
Glacial Till	130	36	0
Bedrock	infinite strength		

In addition, a 250 psf live load surcharge (traffic) was assumed to act over the embankment width in both models (static and pseudo-static).

Please note that the factor of safety for the pseudo-static load case was calculated using a horizontal acceleration coefficient, k_h , of 0.04 g (i.e., one-half of the acceleration coefficient, A_s). A value of $A_s/2$ was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2. 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 calculated global stability factors of safety values are summarized below. The “shallow” failure surfaces were surficial surfaces that occurred when no limitations were put on the failure surface searches. The “deep” surfaces were failure surfaces that occurred when the failure surfaces were forced to extend all the way through the new embankment fill to the existing fill.

Type of Stability Analysis	Calculated Factor of Safety	
	Shallow Circular Failure Surface Through Proposed Fill	Deep Circular Failure Surface Through Glacial Till ¹
Static	1.24	1.62
Pseudo-Static	1.13	1.55

Notes:

¹ Failure searches included the existing glacial till and proposed fill. Lowest factors of safety were determined to be located at the approximate interface between the glacial till and proposed fill layers.

² Calculated factors of safety shown for Spencer method of analysis.

The slope stability software used for these analyses models the approach embankments as an infinitely wide embankment. Since the embankments have finite width, it is our opinion that the calculated factors of safety summarized above are conservative. Therefore, actual factors of safety for a three-dimensional model would be somewhat higher than those presented above.

The minimum required factor of safety as specified by both AASHTO LRFD, and the MaineDOT BDG is 1.3 for embankments under static conditions which do not support structures and is 1.5 for embankments under static conditions which do support structures (permanent condition). The minimum required factor of safety for embankments subjected to pseudo-static loading is 1.1 in accordance with FHWA GEC No. 3 (FHWA-NHI-11-032).

The results of transverse global embankment stability analyses summarized above indicate that the critical failure surface is a shallow failure surface through the slope of the proposed embankment fill, resulting in static factors of safety below 1.3. We modeled the proposed embankment slope at an inclination of 2H:1V. The critical failure surfaces are representative of surficial, sloughing failures, and not a deep-seated global instability. Despite the results, because the slope angle is less than the soils internal friction angle, and based on our experience on similar embankment projects, it is our opinion that 2H:1V embankment side slopes will be stable when constructed in accordance with the requirements of the MaineDOT Standard Specifications.

Furthermore, the results indicate that the proposed embankment has a critical, deep failure surface with a minimum static factor of safety of 1.62. This failure surface penetrates through the proposed fill. The factor of safety of this critical failure surface is greater than the minimum acceptable factor of safety of 1.3 (per MaineDOT BDG and AASHTO LRFD). In addition, the minimum calculated factor of safety for the proposed embankment subjected to seismic loading is 1.55, greater than the minimum acceptable factor of safety of 1.1 (per FHWA GEC No. 3).

We therefore conclude that proposed embankment construction using normal-weight earthfill will have acceptable global stability factors of safety.

Elastic and Consolidation Settlement

The existing fill and glacial till that underlie the proposed approach embankments will undergo elastic compression when embankment loads are applied. Elastic settlements due to construction of the embankments are anticipated to be small (less than 1 in.) and occur relatively quickly due to the absence of soft, compressible soils at the site.

Approximately 75 ft north of the Abutment No. 2 (at approx. Sta. 199+50), a raise in grade of approximately 26 ft is anticipated, and this fill height was modeled to compute settlement. Settlement due to the placement of normal weight earthfill for the approach embankments was evaluated using the computer software program Settle3D 4.0 by Rocscience. Based on our review of the Connector Bridge profile sheets, it is our opinion that this cross section represents the critical section at Lambert Road (i.e., maximum fill height). The results of the settlement analyses are summarized below.

Location	Elastic Settlement (in.)	Primary Consolidation Settlement (in.)	Secondary Consolidation Settlement (in.)	Total Settlement ¹ (in.)
Sta. 199+50	¾ to 1	0	0	¾ to 1

Notes:

¹Total settlement considers immediate, primary consolidation, and secondary consolidation settlement.

Based on the subsurface conditions present at the site, we anticipate that settlement will occur during embankment fill placement and prior to roadway paving. Furthermore, it is our opinion that the new bridge approach embankments will experience little to no post-construction (after paving of Connector) settlement.

Based on the results of the settlement evaluations summarized above, it is our opinion that constructing the proposed Connector Bridge approach embankments using normal weight earthfill is technically feasible without having to implement special embankment construction techniques.

BRIDGE ABUTMENT FOUNDATION SUPPORT

Based on our discussions with MaineDOT, it is our understanding that the preferred Connector Bridge abutment foundation support alternative consists of rock-socketed HP14x117 steel H-piles with a maximum factored axial compressive pile load (demand) equal to 408 kips (Strength Limit State). Rock sockets will be required to achieve a minimum pile resistance of 408 kips because of the limited pile length due to the presence of shallow bedrock and the need to provide fixity at or above the tips of the piles. We anticipate that the rock socket detail will consist of the following:

- A minimum 30-in. diameter, 5-ft long rock socket.
- Suspension of the piles above the bottom of the rock socket. Based on our discussions with you, it is our understanding that previous MaineDOT bedrock socket details have included a 3 to 12-in. gap between the bottom of the base plate (see next bullet) and the bottom of the bedrock socket. For this project, we recommend that the bottom of the base plate be located 1 ft above the bottom of the socket.
- A square base plate welded to the pile tips, with minimum plan dimensions equal to the pile depth and flange width (i.e., 14.2 in. x 14.9 in.) with accommodation for a tremie pipe that will extend through and slightly below the bottom of the base plate so that grout backfill can be reliably placed (using tremie methods) below and around the pile tips, which will promote uniform, full load transfer between the pile tips and the bedrock mass.
- A minimum 3-ft thickness of grout in the bottom of the rock socket (i.e., a minimum of 1 ft between the bottom of the base plate and the bottom of the rock socket and a minimum of 2 ft above the bottom of the base plate). Based on our discussions with you, it is our understanding that the location and thickness of grout within the rock sockets is controlled by the effective pile length needed to resist lateral loading, which was determined by MaineDOT.

- Aggregate for filling the annular space between the HP14x117 steel H-piles and bedrock (above the top of the grout zone) and the H-piles and the permanent casing should meet the requirements of Underdrain Backfill Material, Type C (MaineDOT 703.22).

Additional Connector Bridge abutment foundation support evaluations are recommendations are provided below.

Corrosion and Deterioration

The geotechnical engineering design of the proposed piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual observation of the soil samples and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosive potential. In addition, piles supporting the Connector Bridge abutments will be encased in permanent sleeves that will extend from the bottom of abutment level, through the MSE wall fill, to the top of bedrock. The portion of the piles within the rock socket will be encapsulated in grout. Therefore, the net factored pile resistance recommended below does not include a reduction in pile cross-sectional area for steel degradation.

Axial Compression Pile Resistance

As discussed previously, it is our opinion that based on the subsurface soil conditions present at the site and the magnitude of the maximum factored axial compression pile load provided by MaineDOT, the Connector Bridge abutment piles will be placed in rock sockets and resistance provided through end bearing resistance only.

Structural Resistance

In accordance with AASHTO LRFD, a distinction is made between piles driven to “soft” rock and piles driven to “hard rock.” Based on our experience on similar projects with similar bedrock conditions, we consider the bedrock at the site on which piles will bear to be “hard.” Therefore, the geotechnical resistance of the driven piles is controlled by their structural resistance, as discussed in AASHTO LRFD Section 10.7.3.2.3. The structural resistance factor (Section 6.5.4.2) for axial resistance of piles in compression that are not subject to damage due to severe driving (i.e., because the piles will be placed in rock sockets and will not be impact-driven) is 0.6. In addition, resistance factors for Service and Extreme Limit State loading are 1.0. Therefore, the nominal and factored structural resistance of steel H-pile sections (with $F_y = 50$ ksi) at the Service, Strength and Extreme Event Limit States for the proposed substructures (Abutment No. 1 and Abutment No. 2) is summarized below.

Steel H-pile Section	Nominal Structural Resistance (kips)	Factored Axial Compressive Structural Resistance (kips)		
		Service Limit State ($\phi=1.0$)	Strength Limit State ($\phi=0.6$)	Extreme Limit State ($\phi=1.0$)
HP14x117	1,722	1,722	1,033	1,722

Geotechnical Resistance

In accordance with AASHTO LRFD Section 10.7.3.2.3, the axial compressive geotechnical resistance may only be taken as the axial compressive structural resistance (for piles driven to “hard rock”) if the piles are driven with an impact hammer and the minimum required nominal resistance is confirmed with either a static load test or dynamic testing. Given that the Connector Bridge abutment piles will be placed (not impact-driven) into a bedrock socket, it is our opinion that performing dynamic testing to confirm that the minimum required nominal resistance has been achieved is not required. As a result, the factored axial compressive resistance of rock-socketed HP14x117 steel H-piles was determined based on the geotechnical end bearing resistance that is provided by the grout/bedrock at the base of the bedrock sockets.

The geotechnical end bearing resistance of rock-socketed HP14x117 steel H-piles was calculated based on two different assumptions for bedrock jointing consistent with guidance provided by AASHTO LRFD (i.e., equations 10.8.3.5.4c-1 and -2), which allows for maximum and minimum resistances to be calculated for “intact” and “jointed” rock mass conditions, respectively. Based on our review of the bedrock core descriptions on the test boring logs and Table III as well as the rock core photographs, it is our opinion that the geotechnical resistance should be calculated assuming “jointed” rock mass conditions. In addition, and as noted above, because the pile tips with minimum 14.2-in. x 14.9-in. end bearing plates will be located above the top of the bedrock socket, the axial compressive geotechnical resistance is controlled by the unconfined compressive strength of the grout (5,000 psi) since it is lower than the range of bedrock unconfined compressive strengths (9,747 to 28,776 psi). The estimated nominal and factored geotechnical resistances for rock-socketed HP14x117 steel H-piles at the Service, Strength and Extreme Event Limit States are summarized below.

Steel H-pile Section	Nominal Geotechnical Resistance (kips)	Factored Axial Compressive Geotechnical Resistance		
		Service Limit State ($\phi=1.0$)	Strength Limit State ($\phi=0.5$)	Extreme Limit State ($\phi=1.0$)
HP14x117	1,022	1,022	511	1,022

Drivability Resistance

As stated above, Connector Bridge Abutment piles will be placed in permanent sleeves and into pre-drilled bedrock sockets and will not be impact driven. Because of this, drivability evaluations are not required, and the factored axial compressive pile resistance of the Connector Bridge abutment piles is controlled by the lesser of the geotechnical and structural resistances.

Summary and Recommended Axial Compressive Pile Resistance

The Connector Bridge abutment maximum factored axial pile load as well as the factored resistances (structural and geotechnical) are summarized below.

Steel H-pile Section	Maximum Factored Strength I Limit State Axial Compressive Pile Load (Demand) (kips)	Strength Limit State Factored Axial Compressive Resistance (kips)		
		Factored Structural Resistance (kips)	Factored Geotechnical Resistance (kips)	Controlling Resistance (kips)
HP14x117	408	1,033	511	511

Please note that the calculated, controlling Strength Limit State axial compressive pile resistance value summarized above is greater than the maximum factored Strength I Limit State axial compressive pile load (demand) provided by MaineDOT. Therefore, we recommend that the Connector Bridge be designed based on a maximum factored axial compressive pile resistance of 511 kips.

We recommend that minimum pile spacing, clearance and embedment (into the pile cap) meet the requirements of AASHTO LRFD Section 10.7.1.2. A minimum 14.2 in. x 14.9 in. plate shall be welded directly to the pile tip web and flange. It is our opinion that cast steel drive shoes are not necessary for the piles. We also recommend that pile material conform to the requirements of ASTM A572 and have a minimum yield strength of 50 ksi.

Lateral Pile Resistance

At the request of MaineDOT, Haley & Aldrich developed soil profiles and soil and rock properties at Abutment No. 1 and completed a lateral pile resistance “check” evaluation using the computer program LPILE v. 2019.11.02 developed by Ensoft, Inc. The soil profiles and soil and rock properties used in the lateral pile resistance “check” evaluation at Abutment No. 1 are summarized below.

Stratum	Soil Model	Stratum Top Elevation	Stratum Thickness (ft)	Unit Weight (pcf)	Friction Angle (deg.)	Modulus (k, pci)	Unconfined Compressive Strength (psi)
Fill	Sand (Reese)	135	20	125	34	225	NA
Bedrock	Strong Rock (Vuggy Limestone)	115	5	150	NA	NA	6,000

For the “check” evaluation, the HP14x117 steel H-pile head boundary conditions included a factored axial compressive pile load of 408 kips, a pile head deflection corresponding to the thermal deflection of 0.6 inches (contraction case) in strong axis bending, and a fixed-head condition (zero slope condition). The results of the lateral pile resistance “check” evaluation were provided to MaineDOT for review and evaluation.

Estimated Pile Tip Elevations

As discussed previously, the piles are expected to develop their axial compressive resistance through end bearing and side friction in bedrock. We recommend the following pile tip elevations be used for estimating bid quantities at each substructure location:

Substructure	Approximate Bottom of Substructure Elevation ¹ (ft, NAVD 88)	Approximate Top of Pile Elevation ² (ft, NAVD 88)	Approximate Top of Bedrock Elevation ³ (ft, NAVD 88)	Approximate In-Place Pile Length ⁴ (ft)
Abutment No. 1	131.0	133.0	114.0	30
Abutment No. 2	134.0	136.0	116.0	

Notes:

¹ Estimated from Figure 3 and rounded up to the nearest 1 ft.

² Assumes an approximate 2-ft embedment of pile into the abutment.

³ Taken as the lowest top of bedrock elevation encountered at/near the abutments.

⁴ Assumes pile tip is placed 1 ft above the bottom of the 5-ft long rock socket.

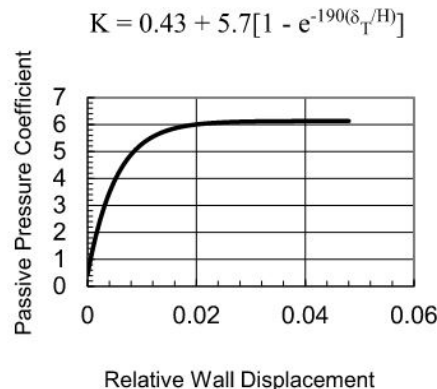
Please note that the recommended in-place pile length summarized above includes an additional 5 to 6 ft to account for variations in the top of bedrock surface across the abutments and weathering at/near the rock surface, which could increase rock socket lengths.

ABUTMENT DESIGN

As stated above, the Connector Bridge will be a fully integral structure. Because of that, thermal expansion of the structure will cause the abutments to move towards the backfill, which will result in lateral earth pressures that vary between at-rest and full passive conditions. The actual magnitude of lateral earth pressure developed by the backfill is dependent on the backfill material type, the in-place properties of the backfill material and the ratio of lateral abutment movement to the abutment height.

We recommend that the abutments be backfilled with a free-draining material that meets the requirements of Soil Type 4 (e.g., Granular Borrow) as specified in BDG Table 3-3. We also recommend that the abutment design include a drainage system in accordance with the requirements of BDG Sections 5.4.1.9 and 5.4.2.13 to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutment and/or MSE wall. Using a free-draining backfill material and providing adequate drainage will substantially reduce the potential for unbalanced hydrostatic pressures from developing.

In accordance with current MaineDOT Bridge Program standard practice, we recommend that the lateral earth pressure be calculated using a “movement-dependent” coefficient per AASHTO LRFD Article C3.11.5.4 and FHWA NHI-06-089. We recommend using the more stringent of the methodology developed by the Massachusetts Department of Transportation (MassDOT) as outlined below or FHWA HNI-06-089, Figure 10-4.



Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H (MassDOT).

It is our understanding that the above referenced methodology developed by MassDOT is based on results of full-scale wall tests completed by the University of Massachusetts. According to the MassDOT LRFD Bridge Manual, the results of the tests show “reasonable agreement between the predicted average passive earth pressure response of MassDOT’s standard compared gravel borrow and the curves of K versus δ_T/H for dense sand found” in other design manuals. Considering that AASHTO LRFD does not provide a load factor for passive earth pressure, we understand that the MaineDOT Bridge Program’s standard practice for integral abutment design considers a load factor equal to 1.5 to calculate factored passive lateral earth pressures.

Additional lateral earth pressures due to live load surcharge are required in accordance with AASHTO LRFD Section 3.11.6.4 and BDG Section 3.6.8 for abutments if an approach slab is not included. When an approach slab is specified, reduction, not elimination of the surcharge load is permitted in accordance with AASHTO LRFD Section 3.11.6.5. We recommend that the live load surcharge be estimated as a uniform horizontal earth pressure due to an equivalent height of soil that is related to the abutment and wingwall heights and the location and orientation of vehicular traffic relative to the back of the wall, as presented in BDG Table 3-4.

If determined applicable by MaineDOT, the abutments should be designed for a uniform lateral load to account for seismic soil loading in accordance with AASHTO LRFD Section A.11.3.1 (Mononobe-Okabe Method). Based on the seismic site class (Site Class “C”), we recommend using a seismic active earth pressure coefficient, K_{AE} , of 0.33 and 0.32 for design of Abutments 1 and 2, respectively. Please note that the soil pressure calculated using K_{AE} includes both the static and seismic lateral earth loads.

MECHANICALLY STABILIZED EARTH WALLS

Vendor-designed MSE walls will be used to retain the new Connector Bridge approach embankment fills. Based on the proposed grading, the exposed height of the proposed MSE walls is approximately 11 ft at Abutment No. 1 and Abutment No. 2, respectively. The ground surface in front of the walls will slope down an additional 6 to 7 ft to ditches that are adjacent to and parallel with Lambert Road as shown on Figure 3. For the purposes of MSE wall design and construction, we recommend the following:

- MSE wall design and construction should be completed in accordance with the requirements of Standard Specification 677 Mechanically Stabilized Earth Retaining Wall and AASHTO LRFD.
- The bottom of the proposed MSE walls should be located a minimum of 6 ft (frost depth; see the following section of this GDR) below proposed grades at the toe of the wall, or at the top of bedrock, whichever is shallower and should bear on a lean concrete leveling pad.
- The factored bearing resistance for an MSE wall bearing on bedrock should not exceed a 40 ksf at the Strength Limit State and 20 ksf and 55 ksf at the Service and Extreme Event Limit States, respectively.
- The factored bearing resistance at the Service Limit State for an MSE wall bearing on approach embankment fill (i.e., granular borrow) should not exceed 6 ksf, which is in the middle of the range of presumptive values for medium dense to dense, coarse to medium sand with little gravel presented in AASHTO LRFD Table C10.6.2.6.1-1, which were developed to limit settlement to less than 1 in. In addition, the nominal bearing resistance for an MSE wall bearing on approach embankment fill (i.e., granular borrow) should not exceed the value calculated using the following equation.

$$q_n = 6,624 \text{ psf} + 3,816 \text{ psf} \left(\frac{B'}{L'} \right) + 672 \text{ pcf} (B') - 270 \text{ pcf} \left(\frac{B'^2}{L'} \right)$$

Please note that B' and L' are equal to the effective reinforcement and effective MSE wall lengths, respectively. In addition, the equation shown above does not account for walls bearing on or near slopes. Because of this, the equation may need to be revised once final grading in front of the walls and wall bearing elevations are finalized. We recommend that a resistance factor equal to 0.65 be used to calculate the factored Strength Limit State bearing resistance, in accordance with AASHTO LRFD Table 11.5.7-1. A resistance factor equal to 0.9 should be used to calculate the Extreme Event Limit State bearing resistances for the seismic case, in accordance with AASHTO LRFD Section 11.5.8.

- For foundations bearing on soil or bedrock, the location of the resultant of the reaction forces should be within the middle two-thirds and middle nine-tenths of the base width in accordance with AASHTO LRFD Section 11.6.3.3.
- We recommend that sliding resistance of the MSE wall reinforced soil mass be evaluated in accordance with the requirements of AASHTO LRFD Section 11.10.5.3 and Table C3.11.5.3-1 and the resistance factors specified in Table 11.5.7-1. We also recommend the following material friction angles be used when evaluating sliding resistance:

– Bedrock =	40 degrees
– MSE wall reinforced soil mass (Gravel Borrow) =	36 degrees
– Proposed embankment fill (Granular Borrow) =	32 degrees
– Existing fill =	28 degrees

Where mixed conditions exist below the bottom of the MSE wall reinforced zone we recommend that the material that represents the lowest friction angle be used. For example, if existing fill and proposed embankment fill (Granular Borrow) are both present below the MSE

wall reinforced soil mass, evaluation of sliding resistance should be based the presence of existing fill soils using a friction angle of 28 degrees.

- We recommend that passive resistance provided by the soil slope in front of the MSE wall reinforced soil mass be neglected in sliding evaluations.
- Soil-pile-MSE wall interaction should be evaluated in accordance with the requirements of FHWA NHI-10-024. This reference document provides guidance on the interaction and loading for pile-supported stub abutments constructed behind MSE walls.
- The MSE wall reinforced zone should be constructed using Gravel Borrow (MaineDOT 703.20). This material should also meet the electrochemical and plasticity requirements of Standard Specification Section 677.048.
- Construction of the Connector Bridge approach embankments outside the limits of the MSE wall reinforced zone should consist of Granular Borrow (MaineDOT 703.19).

FROST PROTECTION

The minimum depth of embedment/cover for the Connector Bridge abutments and MSE wall leveling pads was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG. Based on a design freezing index equal to 1,650 freezing degree-days, we recommend that the abutments and MSE walls bear a minimum of 6.0 ft below the lowest adjacent ground surface exposed to freezing or on bedrock, whichever is shallower.

Limitations

This report is prepared for the exclusive use of MaineDOT relative to the subject project. There are no intended beneficiaries other than MaineDOT. 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 MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and Haley & Aldrich. Use of this report by such other person or entity without the written authorization of MaineDOT 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.

Use of this report by any person or entity, including by MaineDOT, for a purpose other than relative to the subject project is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

The information provided herein is 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.

It is our understanding that this report may be included as a reference document in the documents 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 documents and that the information contained in the documents takes precedence over the information provided in this report.

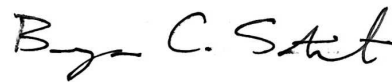
Closure

We appreciate the opportunity to continue to provide MaineDOT services on this project. Please do not hesitate to contact us if you have any questions or comments.

Sincerely yours,
HALEY & ALDRICH, INC.



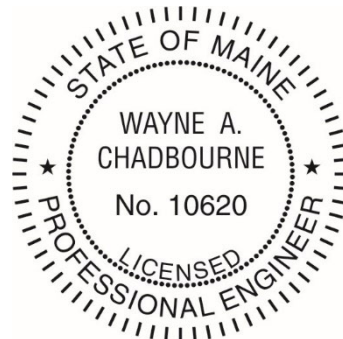
Justin A. DuBois, P.E.
Senior Geotechnical Engineer



Bryan C. Steinert, P.E.
Senior Project Manager



Wayne A. Chadbourne, P.E.
Principal | Lead Quality Control Engineer



Enclosures:

- Table I – Subsurface Exploration Location Data
- Table II – Subsurface Exploration Subsurface Data
- Table III – Subsurface Exploration Rock Core Data
- Figure 1 – Project Locus
- Figure 2 – Site and Subsurface Exploration Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Test Boring Logs and Rock Core Photographs
- Appendix B – Laboratory Test Results
- Appendix C – Geotechnical Design Calculations

TABLE I

Subsurface Exploration Location Data

Interstate 395/Route 9 Connector over Lambert Road, Bridge No. 6647

MaineDOT WIN 018915.00

Eddington, Maine

Haley & Aldrich, Inc. File No.: 132076-007

Test Boring No. ¹	Ground Surface Elevation (ft) ³	Station ⁴	Offset Distance (ft) & Direction ^{4,5}	Horizontal Coordinates ²	
				Northing (Y)	Easting (X)
BB-ELAR-101	119.8	197+80	19 LT	474,903	1,754,617
BB-ELAR-102	123.6	198+49	22 RT	474,948	1,754,684
BB-ELAR-201	116.0	197+08	48 LT	474,850	1,754,560
BB-ELAR-201A	119.8	197+83	15 LT	474,904	1,754,622
BB-ELAR-202	122.5	197+63	42 RT	474,861	1,754,665
BB-ELAR-203	119.0	198+77	35 LT	474,998	1,754,644
BB-ELAR-203A	121.5	198+69	1 LT	474,975	1,754,673
BB-ELAR-204	123.6	199+34	37 RT	475,018	1,754,734

Notes:¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.² As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet 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, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).⁴ Station and offset information shown is relative to the I-395/Route 9 Connector baseline and was determined by Haley & Aldrich based on information provided by MaineDOT.⁵ LT = offset distance toward left direction; RT = offset distance toward right direction; ft = feet.

	Individual	Date
Updated By:	SSM	2/8/2021
Checked By:	BCS	4/30/2021
Reviewed By:	WAC	8/31/2021

TABLE II
Subsurface Exploration Subsurface Data
Interstate 395/Route 9 Connector over Lambert Road, Bridge No. 6647
MaineDOT WIN 018915.00
Eddington, Maine

Haley & Aldrich, Inc. File No.: 132076-007

Test Boring No. ¹	Ground Surface Elevation ^{2,3} (ft)	Stratigraphy Data ^{2,3}												Approximate Bottom of Exploration Depth (ft)	Approximate Elevation of Bottom of Exploration ²
		Bituminous Concrete Thickness (ft)	Topsoil/Fill			Marine Deposit			Glacial Till			Bedrock			
			Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)		
BB-ELAR-101	119.8	NE	0.0	119.8	5.2	NE	NE	NE	NE	NE	NE	5.2	114.6	15.5	104.3
BB-ELAR-102	123.6	0.5	0.5	123.1	4.0	NE	NE	NE	4.5	119.1	2.1	6.6	117.0	16.5	107.1
BB-ELAR-201	116.0	NE	0.0	116.0	0.2	NE	NE	NE	0.2	115.8	2.7	2.9	113.1	8.4	107.6
BB-ELAR-201A	119.8	NE	NE	NE	NE	NE	NE	NE	0.0	119.8	4.4	4.4	115.4	14.7	105.1
BB-ELAR-202	122.5	NE	NE	NE	NE	NE	NE	NE	0.0	122.5	8.2	8.2	114.3	14.5	108.0
BB-ELAR-203	119.0	NE	0.0	119.0	0.6	NE	NE	NE	NE	NE	NE	0.6	118.4	6.8	112.2
BB-ELAR-203A	121.5	NE	NE	NE	NE	NE	NE	NE	0.0	121.5	5.4	5.4	116.1	17.0	104.5
BB-ELAR-204	123.6	NE	0.0	123.6	0.3	0.3	123.3	3.2	3.5	120.1	2.3	5.8	117.8	11.5	112.1

Notes:
¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
³ "NE" indicates stratum was not encountered in test boring.

	Individual	Date
Updated By:	SSM	2/8/2021
Checked By:	BCS	4/30/2021
Reviewed By:	WAC	8/31/2021

TABLE III
Subsurface Exploration Rock Core Data
Interstate 395/Route 9 Connector over Lambert Road, Bridge No. 6647
MaineDOT WIN 018915.00
Eddington, Maine

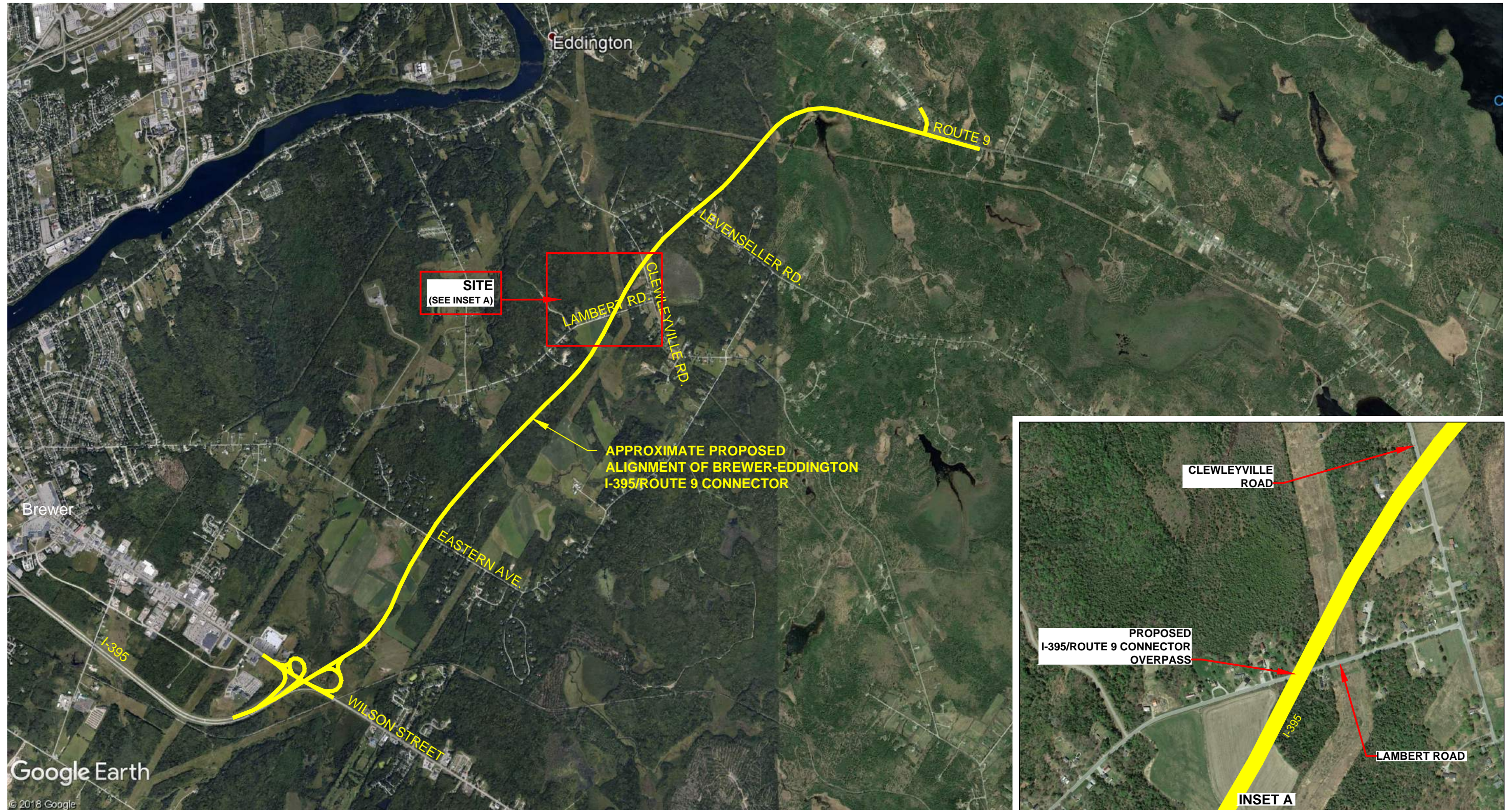
Haley & Aldrich, Inc. File No.: 132076-007

Test Boring No. ¹	Ground Surface Elevation ² (ft)	Core Diameter (in.)	Run					TCR ^{3,6}		RQD ^{4,5,6}			Physical Rock Parameters		Lithologic, Rock Mass and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	Length (ft)	%	Designation	Weathering	Estimated Field Strength	
				Top	Bottom	Midpoint									
BB-ELAR-101	119.8	NQ (1.875")	R1	5.5	7.0	6.3	1.5	1.5	100%	0.0	0%	Very Poor	Slight to Moderately	Mod. Hard	Grey, aphanitic to fine-grained, METASILTSTONE, discontinuities dipping at moderate to steep angles (35 to 85 degrees from axis), spacing very close to close (<2 in. to 12 in.), discontinuity apertures are tight to open. Discontinuity surfaces are planar to undulating, smooth to rough and oxidized. Highly fractured zones.
			R2	7.0	7.7	7.4	0.7	0.7	100%	0.0	0%	Very Poor	Slight to Moderately	Mod. Hard	
			R3	7.7	10.6	9.2	2.9	2.9	100%	1.1	37%	Poor	Slight	Mod. Hard	
			R4	10.6	11.6	11.1	1.0	0.9	92%	0.0	0%	Very Poor	Slight to Moderately	Mod. Hard	
			R5	11.6	15.5	13.6	3.9	3.8	98%	1.5	38%	Poor	Slight to Moderately	Mod. Hard	
BB-ELAR-102	123.6	NQ (1.875")	R1	6.6	9.6	8.1	3.0	3.0	100%	1.0	33%	Poor	Slight to Moderately	Mod. Hard	Grey, aphanitic to fine-grained, METASILTSTONE, discontinuities dipping at steep angles (55 to 85 degrees from horizontal), spacing very close to close (<2 in. to 12 in.), discontinuity apertures are open. Discontinuity surfaces are undulating and rough. Frequent thin calcite veins and occasional quartz veins.
			R2	9.6	12.4	11.0	2.8	2.8	100%	2.4	86%	Good	Slight to Moderately	Mod. Hard	
			R3	12.4	16.5	14.5	4.1	4.0	98%	1.7	41%	Poor	Slight to Moderately	Mod. Hard	
BB-ELAR-201	116.0	NQ (1.875")	R1	3.4	8.4	5.9	5.0	4.8	95%	4.8	95%	Excellent	Fresh to Slightly	Mod. Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at steep angles (55 to 85 degrees from horizontal), spacing moderate to wide (>36 in.), discontinuity apertures are open. Discontinuity surfaces are undulating and rough with calcite coatings on some joint surfaces. Frequent thin calcite veins and occasional quartz veins.
BB-ELAR-201A	119.8	NQ (1.875")	R1	4.7	9.7	7.2	5.0	4.8	95%	4.8	95%	Excellent	Fresh to Slightly	Mod. Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at steep to vertical angles (55 to 90 degrees from horizontal), spacing wide (>36 in.). Discontinuity surfaces are planar, tight and rough. Iron staining on some joint surfaces. Occasional calcite veins and banding present.
			R2	9.7	14.7	12.2	5.0	5.0	100%	4.5	90%	Good	Fresh to Moderately	Mod. Hard	
BB-ELAR-202	122.5	NQ (1.875")	R1	8.5	13.5	11.0	5.0	2.7	53%	2.1	42%	Poor	Fresh to Slightly	Mod. Hard to Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at steep angles (55 to 85 degrees from horizontal). Discontinuity surfaces are planar, tight and rough.
			R2	13.5	14.5	14.0	1.0	3.0	300%	1.0	100%	Excellent	Fresh	Mod. Hard to Hard	
BB-ELAR-203	119.0	NQ (1.875")	R1	1.5	3.8	2.7	2.3	2.2	94%	1.7	76%	Good	Slight	Mod. Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at steep angles (55 to 85 degrees from horizontal), spacing moderate (36 in.) Discontinuity surfaces are planar to undulating, tight to open and rough. Few calcite veins present, iron staining on joint surfaces.
			R2	3.8	6.8	5.3	3.0	2.7	89%	2.4	81%	Good	Fresh	Mod. Hard	
BB-ELAR-203A	121.5	NQ (1.875")	R1	6	11	8.5	5.0	4.0	80%	3.0	60%	Good	Fresh to Slightly	Mod. Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at low to steep angles (5 to 85 degrees from horizontal), spacing close to moderate (12 in. to 36 in.) Discontinuity surfaces are planar, tight to open and rough. Calcite veins present.
			R2	11	13	12.0	2.0	2.4	120%	0.9	46%	Poor	Slight	Mod. Hard	
			R3	13	17	15.0	4.0	3.8	94%	3.8	94%	Excellent	Fresh to Slightly	Mod. Hard	
BB-ELAR-204	123.6	NQ (1.875")	R1	6.5	11.5	9.0	5.0	4.9	97%	3.8	75%	Fair	Fresh to Slightly	Hard	Grey, aphanitic, SILTSTONE, discontinuities dipping at moderate to steep angles (55 to 85 degrees from horizontal), spacing close to moderate (12 in. to 36 in.) Discontinuity surfaces are planar, tight to open and smooth to rough. 0.05-in. thick silt infilled joint at 9.0 ft.

Notes:

- ¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
- ² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
- ³ TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
- ⁴ RQD = rock quality designation. RQD is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
- ⁵ Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information.
- ⁶ BB-ELAR-202 R2 and BB-ELAR-203A R2 recovery and RQD include portion of R1 that was not initially recovered.

	Individual	Date
Prepared By:	SSM	4/16/2021
Checked By:	BCS	4/30/2021
Reviewed By:	WAC	8/31/2021



NOTES

1. IMAGE TAKEN FROM GOOGLE EARTH IMAGES, 2018.



**HALEY
ALDRICH**

INTERSTATE 395/ROUTE 9 CONNECTOR
OVER LAMBERT ROAD, BRIDGE NO. 6647
MAINEDOT WIN 018915.00
EDDINGTON, MAINE

PROJECT LOCUS

SCALE: AS SHOWN
AUGUST 2021

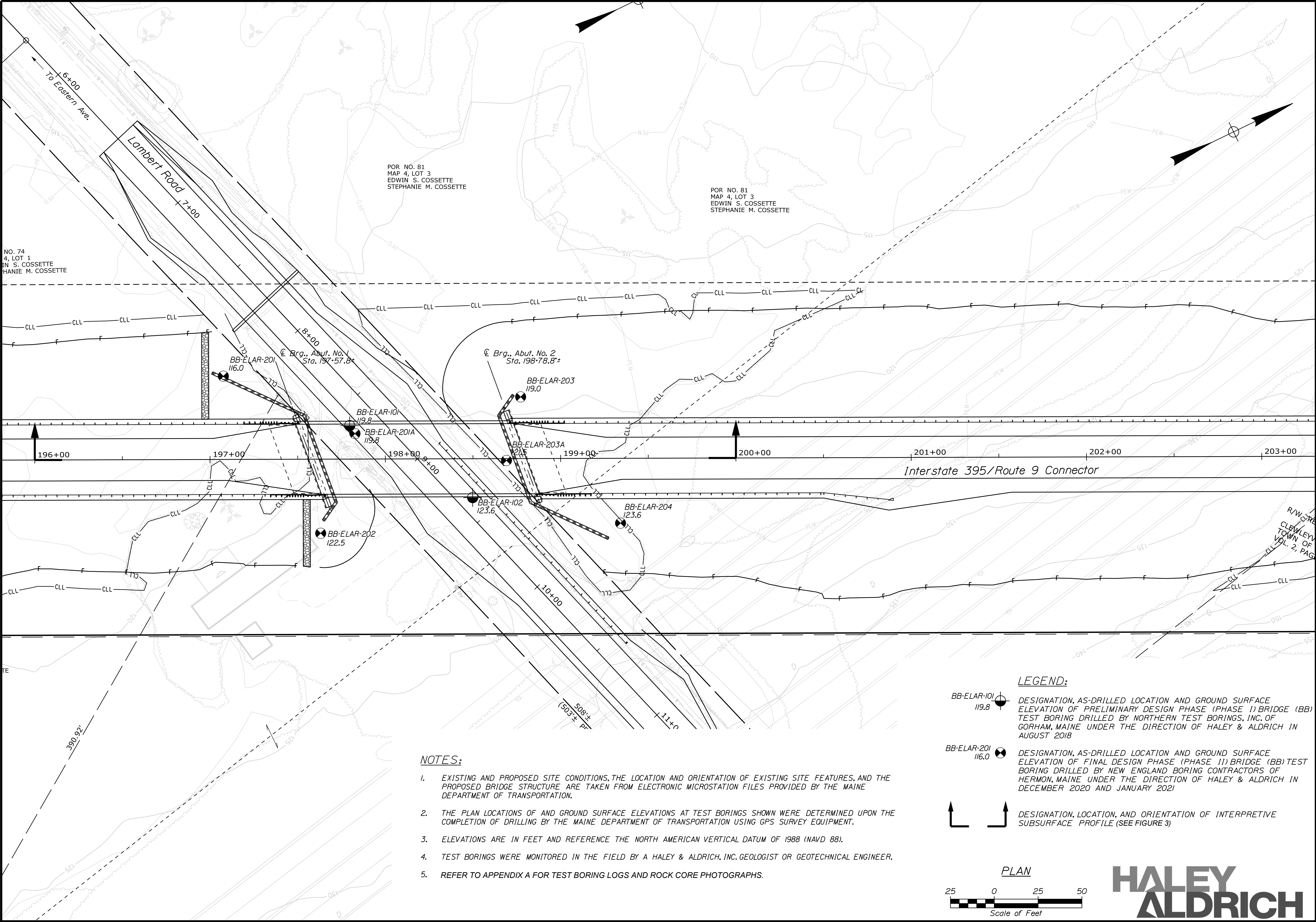
FIGURE 1

Date:9/3/2021

Username:

Division:

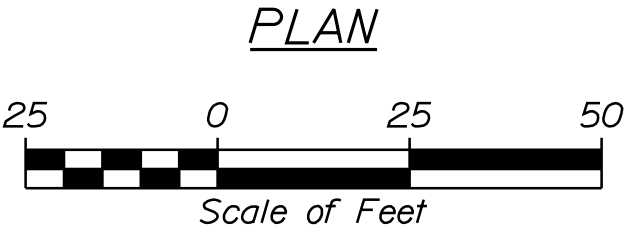
Filename: ... \2021\050_Plan_LambertRoad.dgn



NOTES:

- EXISTING AND PROPOSED SITE CONDITIONS, THE LOCATION AND ORIENTATION OF EXISTING SITE FEATURES, AND THE PROPOSED BRIDGE STRUCTURE ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY THE MAINE DEPARTMENT OF TRANSPORTATION.
- THE PLAN LOCATIONS OF AND GROUND SURFACE ELEVATIONS AT TEST BORINGS SHOWN WERE DETERMINED UPON THE COMPLETION OF DRILLING BY THE MAINE DEPARTMENT OF TRANSPORTATION USING GPS SURVEY EQUIPMENT.
- ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
- TEST BORINGS WERE MONITORED IN THE FIELD BY A HALEY & ALDRICH, INC. GEOLOGIST OR GEOTECHNICAL ENGINEER.
- REFER TO APPENDIX A FOR TEST BORING LOGS AND ROCK CORE PHOTOGRAPHS.

- LEGEND:**
- BB-ELAR-101 119.8
- DESIGNATION, AS-DRILLED LOCATION AND GROUND SURFACE ELEVATION OF PRELIMINARY DESIGN PHASE (PHASE I) BRIDGE (BB) TEST BORING DRILLED BY NORTHERN TEST BORINGS, INC. OF GORHAM, MAINE UNDER THE DIRECTION OF HALEY & ALDRICH IN AUGUST 2018
- BB-ELAR-201 116.0
- DESIGNATION, AS-DRILLED LOCATION AND GROUND SURFACE ELEVATION OF FINAL DESIGN PHASE (PHASE II) BRIDGE (BB) TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE UNDER THE DIRECTION OF HALEY & ALDRICH IN DECEMBER 2020 AND JANUARY 2021
-
- DESIGNATION, LOCATION, AND ORIENTATION OF INTERPRETIVE SUBSURFACE PROFILE (SEE FIGURE 3)



HALEY
ALDRICH

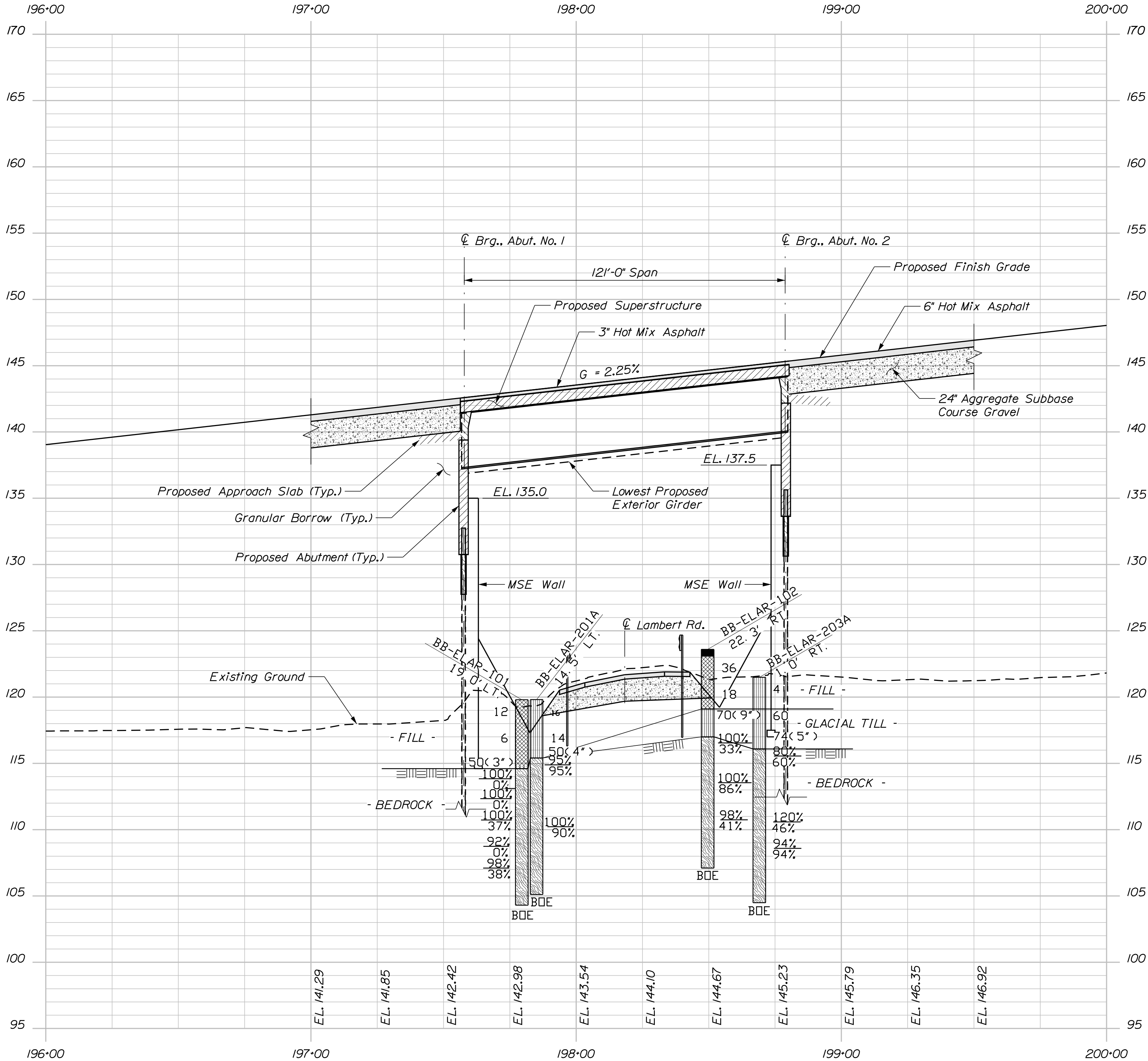
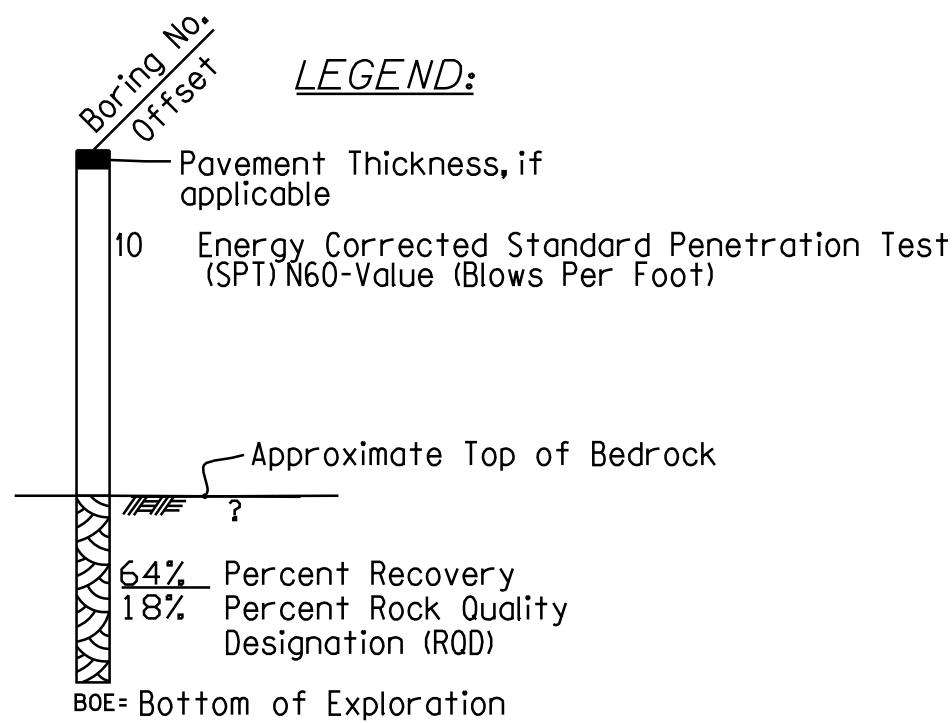
STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
STP-1891(500)		BRIDGE NO.	
WIN		018915.00	
BRIDGE PLANS			
I-395/ROUTE 9 CONNECTOR		PENOBSCOT COUNTY	
LAMBERT ROAD BRIDGE		EDDINGTON	
SITE AND SUBSURFACE		EXPLORATION LOCATION PLAN	
FIGURE		2	

Date:9/3/2021

Username:

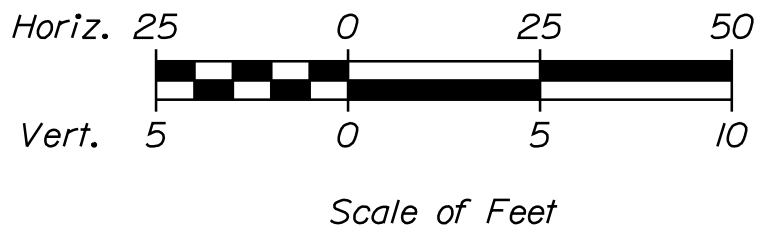
Division:

Filename: ... \051_Profile_LambertRoad.dgn



NOTES:

- BORING OFFSET BASED ON I-395/ROUTE 9 CONNECTOR BASELINE.
- THIS GENERALIZED INTERPRETIVE SUBSURFACE PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BY INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS AND SAMPLES. ACTUAL SOIL TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO THE BORING LOGS IN APPENDIX A.
- EXISTING AND PROPOSED SITE CONDITIONS, THE LOCATION AND ORIENTATION OF EXISTING SITE FEATURES, AND THE PROPOSED BRIDGE STRUCTURE ARE TAKEN FROM ELECTRONIC MICROSTATION FILES PROVIDED BY THE MAINE DEPARTMENT OF TRANSPORTATION.
- ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
- REFER TO APPENDIX A FOR TEST BORING LOGS AND ROCK CORE PHOTOGRAPHS.
- REFER TO FIGURE 2 FOR THE LOCATION AND ORIENTATION OF THE INTERPRETIVE SUBSURFACE PROFILE.



HALEY
ALDRICH

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
I-395/ROUTE 9 CONNECTOR		STP-1891(500)	
LAMBERT ROAD BRIDGE		WIN	
EDDINGTON		018915.00	
PENOBSCOT COUNTY		BRIDGE NO.	
INTERPRETIVE SUBSURFACE		BRIDGE PLANS	
PROFILE			
FIGURE			
3			
SIGNATURE		P.E. NUMBER	
DATE		DATE	
DESIGN-DETAILED		DESIGN-DETAILED	
CHECKED-REVIEWED		CHECKED-REVIEWED	
DESIGN-DETAILED		DESIGN-DETAILED	
REVISIONS 1		REVISIONS 1	
REVISIONS 2		REVISIONS 2	
REVISIONS 3		REVISIONS 3	
REVISIONS 4		REVISIONS 4	
FIELD CHANGES		FIELD CHANGES	

APPENDIX A

Test Boring Logs and Rock Core Photographs

UNIFIED SOIL CLASSIFICATION SYSTEM				
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, Gravelly sands, little or no fines
		(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.	
		CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.	
		OL	Organic silts and organic Silty clays of low plasticity.	
	SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.	
		CH	Inorganic clays of high plasticity, fat clays.	
		OH	Organic clays of medium to high plasticity, organic silts.	
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.		

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM			
<u>Descriptive Term</u>		<u>Portion of Total (%)</u>	
trace		0 - 10	
little		11 - 20	
some		21 - 35	
adjective (e.g. Sandy, Clayey)		36 - 50	

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.

<u>Consistency of Cohesive soils</u>	<u>SPT N-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^*}{\text{length of core advance}}$

*Minimum NQ rock core (1.88 in. OD of core)

Rock Quality Based on RQD	
Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

WIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	



**Maine Department of Transportation
Geotechnical Section**

Key to Soil and Rock Descriptions and Terms

Field Identification Information

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Maine Department of Transportation				Project: Route 9/1-395 Connector		Boring No.: BB-ELAR-101	
Soil/Rock Exploration Log US CUSTOMARY UNITS				Location: Brewer and Eddington, Maine		WIN: 18915.00	
Driller: Northern Test Borings, Inc.		Elevation (ft.): 119.8		Auger ID/OD: --			
Operator: M. Nadeau		Datum: NAVD 88		Sampler: Split-Spoon 1.375 in. ID			
Logged By: N. Klausmeyer		Rig Type: Diedrich D50 Track (Rig #377)		Hammer Wt./Fall: SS-140#/30; HW-140#/20			
Date Start/Finish: 08-1-18/08-1-18		Drilling Method: SSA/HW Drive		Core Barrel: NQ-2.0 in. ID			
Boring Location: Sta. 197+79.6, 19.0 LT		Casing ID/OD: HW-4.0 in. ID		Water Level*: 6.7 ft			
Hammer Efficiency Factor: 0.907				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>			
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	24/14	0.0 - 2.0	5/5/3/2	8	12	SSA	118.2		Brown, dry, medium dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW) (ROADWAY BASE/SUBBASE MATERIAL)	
	2D	24/14	2.0 - 4.0	3/2/2/3	4	6				Dark grey-brown, moist, loose, fine to coarse Sandy SILT, little gravel -FILL-(ML)	
5	3D	3/3	5.0 - 5.3	50(3")			26/5"	114.8		Brown, dry, very dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW)	qp=9,747 psi (8.1-9.3')
	R1	18/18	5.5 - 7.0	RQD = 0%			NQ	114.6			
	R2	8.4/8.4	7.0 - 7.7	RQD = 0%						Top of Bedrock at El. 114.6	
	R3	34.8/34.8	7.7 - 10.6	RQD = 37%						R1: Grey, aphanitic to fine-grained METASILTSTONE. Moderately hard, slightly to moderately weathered, highly fractured. Discernible joints dipping at steep angles, very close, open, discernible joints, planar to undulating, rough. Rock Quality=Very Poor Recovery=100%	
10	R4	12/11	10.6 - 11.6	RQD = 0%						-BREWER FORMATION- R1 Core Times (min:sec): 5.5-6.5' (1:55); 6.5-7.0' (2:20) R2: Similar to R1. Rock Quality=Very Poor Recovery=100%	
	R5	46.8/46	11.6 - 15.5	RQD = 38%						-BREWER FORMATION- R2 Core Times (min:sec): 7.0-7.7'(1:24) R3: Grey, apphanitic METASILTSTONE, moderately hard, slightly weathered. Joints dipping at moderate to steep angles, secondary near-vertical joint very close to close, tight to open, planar, smooth to rough. Slight oxidation on some joint surfaces. Rock Quality=Very Poor Recovery=100%	
15										-BREWER FORMATION- R3 Core Times (min:sec): 7.7-8.7' (3:47); 8.7-9.7' (1:39); 9.7-10.6' (2:44) R4: Grey, aphanitic to fine-grained METASILTSTONE, moderately hard, slightly to moderately weathered, highly fractured throughout, no discernible joints. Rock Quality=Very Poor Recovery=92%	
20										-BREWER FORMATION- R4 Core Times (min:sec): 10.6-11.6' (4:23) R5: Similar to R4, except joints dipping at low and steep angles, close. Rock Quality=Very Poor Recovery=98%	
										-BREWER FORMATION- R5 Core Times (min:sec): 11.6-12.6' (5:41); 12.6-13.6' (2:53); 13.6-14.6' (2:31); 14.6-15.5' (2:21)	
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-ELAR-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/1-395 Connector Location: Brewer and Eddington, Maine				Boring No.: BB-ELAR-102 WIN: 18915.00																							
Driller: Northern Test Borings, Inc.				Elevation (ft.): 123.6				Auger ID/OD: --																							
Operator: M. Nadeau				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in. ID																							
Logged By: N. Klausmeyer				Rig Type: Diedrich D50 Track (Rig #377)				Hammer Wt./Fall: SS-140#/30; HW-140#/20																							
Date Start/Finish: 08-1-18/08-1-18				Drilling Method: SSA/HW Drive				Core Barrel: NQ-2.0 in. ID																							
Boring Location: Sta. 198+49.4, 22.3 RT				Casing ID/OD: HW-4.0 in. ID				Water Level*: 4.8 ft																							
Hammer Efficiency Factor: 0.907				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected																							
				Tv = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																											
<table><tr><th colspan="8">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N60</th><th>Casing</th></tr></table>												Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing
Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																				
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing																								
0	1D	24/20	0.5 - 2.5	14/12/12/8	24	36	SSA	123.1		-BITUMINOUS CONCRETE-	qp=12,789 psi (10.7-11.7') qp=11,024 psi (14.3-15.3')																				
								121.1		Brown, dry, dense, fine to coarse SAND, some fine to coarse gravel, trace silt -FILL-(SW) (ROADWAY BASE/SUBBASE MATERIAL)																					
	2D	24/14	2.5 - 4.5	6/5/7/33	12	18		119.1		Dark brown-grey, dry to moist, very stiff, SILT, little fine to coarse sand, trace fine to coarse gravel, cobble in spoon tip -FILL-(ML)																					
5	3D	15/12	5.0 - 6.3	5/20/50(3")			97/7"	117.0		Grey-brown to rust-brown, mottled, very dense, fine Sandy SILT, little gravel, trace medium to coarse sand -GLACIAL TILL-(ML)																					
	R1	36/36	6.6 - 9.6	RQD = 33%			NQ CORE			Top of Bedrock at El. 117.0 R1: Grey, aphanitic to fine-grained METASILTSTONE. Moderately hard, slightly to moderately weathered. Joints dipping at steep angles, very close to close, open joints, undulating, rough. High angle foliation, frequent thin calcite veins, one 3-in. thick quartz vein. Rock Quality=Poor Recovery=100%																					
10										-BREWER FORMATION-																					
										R1 Core Times (min:sec): 6.6-7.6' (1:55); 7.6-8.6' (3:31); 8.6-9.6' (5:15)																					
	R2	33.6/34	9.6 - 12.4	RQD = 86%						R2: Similar to R1. Rock Quality=Good Recovery=100%																					
										-BREWER FORMATION-																					
15										R2 Core Times (min:sec): 9.6-10.6' (4:35); 10.6-11.6' (4:34); 11.6-12.4 (7:56)																					
	R3	49.2/48	12.4 - 16.5	RQD = 41%						R3: Similar to R1, except secondary low angle joints. Rock Quality=Poor Recovery=98%																					
										-BREWER FORMATION-																					
20									R3 Core Times (min:sec): 12.4-13.4' (8:21); 13.4-14.4' (8:30); 14.4-15.4' (6:52); 15.4-16.5' (7:28)																						
25										Bottom of Exploration at 16.5 feet below ground surface.																					
Remarks:																															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.																															
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.																															
Page 1 of 1																															
Boring No.: BB-ELAR-102																															

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine		Boring No.: BB-ELAR-201 WIN: 18915.00				
Driller: New England Boring Contractors		Elevation (ft.): 116.0		Auger ID/OD: --						
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID						
Logged By: J. Fletcher		Rig Type: Mobile B-53 Track		Hammer Wt./Fall: SS-140#/30; HW-300#/16						
Date Start/Finish: 1-04-2021/1-04-2021		Drilling Method: SSA/HW Drive		Core Barrel: NQ-2.0 in. ID						
Boring Location: Sta. 197+07.7, 47.5 LT		Casing ID/OD: HW-4.0 in. ID		Water Level*: 1.3 ft						
Hammer Efficiency Factor: 0.852		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>								
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							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	24/7	0.0 - 2.0	WOH/WOH/1/3			SSA	115.8		
								114.0		
	2D	10.8/8	2.0 - 2.9	WOH/13(5")			✓	113.1		
	R1	60/57	3.4 - 8.4	RQD = 95%			RC NQ CORE			
5								107.6		
10										
15										
20										
25										
Bottom of Exploration at 8.4 feet below ground surface.										
Remarks:										

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine		Boring No.: BB-ELAR-201A WIN: 18915.00		
Driller: New England Boring Contractors			Elevation (ft.): 119.8		Auger ID/OD: --			
Operator: M. Porter			Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID			
Logged By: J. Fletcher			Rig Type: Mobile B-53 Track		Hammer Wt./Fall: SS-140#/30; HW-300#/16			
Date Start/Finish: 1-05-2021/1-05-2021			Drilling Method: SSA/HW Drive		Core Barrel: NQ-2.0 in. ID			
Boring Location: Sta. 197+82.7, 14.5 LT			Casing ID/OD: HW-4.0 in. ID		Water Level*: 1.8 ft			
Hammer Efficiency Factor: 0.852			Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>					
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							
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)
0	1D	24/8	0.0 - 2.0	6/6/5/2	11	16	SSA	
	2D	24/7	2.0 - 4.0	1/4/6/14	10	14		
	3D	5/4	4.0 - 4.4	50(4")		71		
5	R1	60/57	4.7 - 9.7	RQD = 95%				
10	R2	60/60	9.7 - 14.7	RQD = 90%				
15								Note: Moved boring approximately 10 ft towards road away from power lines. Brown, wet, very stiff, fine to medium Sandy SILT, trace gravel, loosely bonded -GLACIAL TILL-(ML) Brown, wet, stiff, SILT, some fine gravel, little fine to medium sand, trace coarse sand and gravel, moderately bonded -GLACIAL TILL-(ML) Brown, wet, hard, SILT, some sand, little gravel, loosely bonded -GLACIAL TILL-(ML) Top of Bedrock El. 115.4 R1: Grey, aphanitic, SILTSTONE, moderately hard, fresh to slightly weathered. Vertical joints, wide spacing, tight, rough, planar joints. Occasional calcite veins and banding, Rock Quality=Excellent Recovery=95% -BREWER FORMATION- R1 Core Times (min:sec): 4.7-5.7' (2:43); 5.7-6.7' (2:52); 6.7-7.7' (1:58); 7.7-8.7' (2:17); 8.7-9.7' (2:23) R2: Grey, aphanitic, SILTSTONE, moderately hard, fresh to moderately weathered. Steep angle joints, moderate spacing, tight to open, rough joints, iron staining on some joints. Occasional calcite veins. Rock Quality=Good Recovery=100% -BREWER FORMATION- R2 Core Times (min:sec): 9.7-10.7' (2:22); 10.7-11.7' (2:00); 11.7-12.7' (2:02); 12.7-13.7' (2:14); 13.7-14.7' (1:53) Bottom of Exploration at 14.7 feet below ground surface.
25								
Remarks:								
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.								Page 1 of 1 Boring No.: BB-ELAR-201A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine				Boring No.: BB-ELAR-202 WIN: 18915.00																																																																																																																																																																																																																					
Driller: New England Boring Contractors				Elevation (ft.): 122.5				Auger ID/OD: --																																																																																																																																																																																																																					
Operator: M. Porter				Datum: NAVD 88				Sampler: Split Spoon 1.375 in. ID																																																																																																																																																																																																																					
Logged By: J. Fletcher				Rig Type: Mobile B-53 Track				Hammer Wt./Fall: SS-140#/30; HW-300#/16																																																																																																																																																																																																																					
Date Start/Finish: 1-05-2021/1-05-2021				Drilling Method: SSA/HW Drive				Core Barrel: NQ-2.0 in. ID																																																																																																																																																																																																																					
Boring Location: Sta. 197+62.9, 42.3 RT				Casing ID/OD: HW-4.0 in. ID				Water Level*: 3.8 ft																																																																																																																																																																																																																					
Hammer Efficiency Factor: 0.852				Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																																																																																																																																																																																																																									
<div>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</div> <div>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</div> <div>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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected</div> <div>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</div>																																																																																																																																																																																																																													
<table><tr><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows / (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>0</td><td>1D</td><td>24/9</td><td>0.0 - 2.0</td><td>5/6/5/5</td><td>11</td><td>16</td><td>SSA</td><td></td><td rowspan="3">118.5</td><td>Brown, moist, medium dense, fine to medium SAND, some silt, trace gravel, loosely bonded -GLACIAL TLL-(SM)</td><td rowspan="10">qp=28,776 psi (10.1'-11.0')</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>No Recovery</td></tr><tr><td></td><td>2D</td><td>24/0</td><td>2.0 - 4.0</td><td>6/4/4/7</td><td>8</td><td>11</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="3">114.3</td><td>Grey-brown, hard, SILT, little gravel, little fine sand, moderately bonded -GLACIAL TILL-(ML)</td></tr><tr><td>5</td><td>3D</td><td>24/15</td><td>4.0 - 6.0</td><td>7/10/16/18</td><td>26</td><td>37</td><td>↘ 37</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>HW</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>↘</td><td></td><td></td><td rowspan="3">8.2</td><td>Top of Bedrock El. 114.3 R1: Grey, aphanitic, SILTSTONE, moderately hard to hard, fresh to slightly weathered. Steep angle joints, tight, planar, rough. Rock Quality=Poor Recovery=53% -BREWER FORMATION- R1 Core Times (min:sec): 8.5-9.5' (1:58); 9.5-10.5' (1:49); 10.5-11.5' (1:42); 11.5-12.5' (1:36); 12.5-13.5' (1:51)</td></tr><tr><td>10</td><td>R1</td><td>60/32</td><td>8.5 - 13.5</td><td>RQD = 42%</td><td></td><td></td><td>RC NQ CORE</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="3">108.0</td><td>R2: Grey, aphanitic, SILTSTONE, moderately hard to hard, fresh. No joints. Rock Quality=Excellent Recovery=300% Note: R2 recovery and RQD includes 24 in. from R1 that was not initially recovered. -BREWER FORMATION- R2 Core Times (min:sec): 13.5-14.5' (1:57)</td></tr><tr><td>15</td><td>R2</td><td>12/36</td><td>13.5 - 14.5</td><td>RQD = 100%</td><td></td><td></td><td>↘</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td rowspan="5">14.5</td><td>Bottom of Exploration at 14.5 feet below ground surface.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>20</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></table>												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.)	0	1D	24/9	0.0 - 2.0	5/6/5/5	11	16	SSA		118.5	Brown, moist, medium dense, fine to medium SAND, some silt, trace gravel, loosely bonded -GLACIAL TLL-(SM)	qp=28,776 psi (10.1'-11.0')										No Recovery		2D	24/0	2.0 - 4.0	6/4/4/7	8	11													114.3	Grey-brown, hard, SILT, little gravel, little fine sand, moderately bonded -GLACIAL TILL-(ML)	5	3D	24/15	4.0 - 6.0	7/10/16/18	26	37	↘ 37										HW										↘			8.2	Top of Bedrock El. 114.3 R1: Grey, aphanitic, SILTSTONE, moderately hard to hard, fresh to slightly weathered. Steep angle joints, tight, planar, rough. Rock Quality=Poor Recovery=53% -BREWER FORMATION- R1 Core Times (min:sec): 8.5-9.5' (1:58); 9.5-10.5' (1:49); 10.5-11.5' (1:42); 11.5-12.5' (1:36); 12.5-13.5' (1:51)	10	R1	60/32	8.5 - 13.5	RQD = 42%			RC NQ CORE																						108.0	R2: Grey, aphanitic, SILTSTONE, moderately hard to hard, fresh. No joints. Rock Quality=Excellent Recovery=300% Note: R2 recovery and RQD includes 24 in. from R1 that was not initially recovered. -BREWER FORMATION- R2 Core Times (min:sec): 13.5-14.5' (1:57)	15	R2	12/36	13.5 - 14.5	RQD = 100%			↘																							14.5	Bottom of Exploration at 14.5 feet below ground surface.											20																														25											
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* 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-ELAR-202																																																																																																																																																																																																																			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine		Boring No.: BB-ELAR-203 WIN: 18915.00			
Driller: New England Boring Contractors			Elevation (ft.): 119.0			Auger ID/OD: --			
Operator: M. Porter			Datum: NAVD 88			Sampler: Split Spoon 1.375 in. ID			
Logged By: J. Fletcher			Rig Type: Mobile B-53 Track			Hammer Wt./Fall: SS-140#/30; HW-300#/16			
Date Start/Finish: 12-23-2020/12-23-2020			Drilling Method: SSA/HW Drive			Core Barrel: NQ-2.0 in. ID			
Boring Location: Sta. 198+77.3, 35.2 LT			Casing ID/OD: HW-4.0 in. ID			Water Level*: 0.8 ft			
Hammer Efficiency Factor: 0.852			Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>						
<div style="font-size: small;"> Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) D = Split Spoon Sample SSA = Solid Stem Auger S_u(lab) = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index V = Field Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test </div>									
Depth (ft.)	Sample Information							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	7/3	0.0 - 0.6	2/50(1")			RC	118.7 118.4	
	R1	27.6/26	1.5 - 3.8	RQD = 76%			NQ		
							CORE		
	R2	36/32	3.8 - 6.8	RQD = 81%					
5								112.2	
10									
15									
20									
25									
Remarks:									
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.								Page 1 of 1 Boring No.: BB-ELAR-203	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector		Boring No.: BB-ELAR-203A				
				Location: Brewer and Eddington, Maine		WIN: 18915.00				
Driller: New England Boring Contractors		Elevation (ft.): 121.5		Auger ID/OD: --						
Operator: M. Porter		Datum: NAVD 88		Sampler: Split Spoon 1.375 in. ID						
Logged By: J. Fletcher		Rig Type: Mobile B-53 Track		Hammer Wt./Fall: SS-140#/30; HW-300#/16						
Date Start/Finish: 12-23-2020/12-23-2020		Drilling Method: SSA/HW Drive		Core Barrel: NQ-2.0 in. ID						
Boring Location: Sta. 198+69.1, 1.0 LT		Casing ID/OD: HW-4.0 in. ID		Water Level*: 2.4 ft						
Hammer Efficiency Factor: 0.852		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>								
<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <div> 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 </div> <div> 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 </div> <div> 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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected </div> <div> 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 </div> </div>										
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			
0	1D	24/9	0.0 - 2.0	2/2/1/4	3	4	SSA	Elevation (ft.)	Graphic Log	Visual Description and Remarks Grey-brown, moist, soft, fine Sandy SILT -GLACIAL TILL-(ML) Brown-grey, moist, hard, SILT, little fine sand, trace gravel, moderately bonded -GLACIAL TILL-(ML) Note: Gravel from 3.9 to 4.0 ft. Brown-grey, moist, hard, Sandy SILT, little fine gravel, moderately bonded -GLACIAL TILL-(ML) Top of Bedrock El. 116.1 R1: Grey, aphanitic, SILTSTONE, moderately hard, fresh to slightly weathered. Joints dipping at low and steep angles, close to moderate spacing, tight, planar, rough, calcite veins. Rock Quality=Good Recovery=80% -BREWER FORMATION- R1 Core Times (min:sec): 6.0-7.0' (0:58); 7.0-8.0' (1:15); 8.0-9.0' (1:16); 9.0-10.0' (1:19); 10.0-11.0' (1:24) R2: Grey, aphanitic, SILTSTONE, moderately hard, slightly weathered. Dipping at steep angles, moderate spacing, tight to open, rough, calcite veins. Rock Quality=Poor Recovery=120% Note: R2 recovery and RQD includes 5 in. from R1 that was not initially recovered. -BREWER FORMATION- R2 Core Times (min:sec): 11.0-12.0' (1:43); 12.0-13.0' (1:31) R3: Grey, aphanitic, SILTSTONE, moderately hard, fresh to slight weathering. Steep angle joint, wide spacing, tight, rough, calcite veins. Rock Quality=Excellent Recovery=94% -BREWER FORMATION- R3 Core Times (min:sec): 13.0-14.0' (1:12); 14.0-15.0' (1:21); 15.0-16.0' (1:25); 16.0-17.0' (1:22) Bottom of Exploration at 17.0 feet below ground surface.
5	2D	24/14	2.0 - 4.0	2/10/32/27	42	60	3			
							4			
	3D	16.8/15	4.0 - 5.4	16/24/50(5")			46			
							RC			
	R1	60/48	6.0 - 11.0	RQD = 60%			NQ CORE			
10										
	R2	24/29	11.0 - 13.0	RQD = 46%						
	R3	48/45	13.0 - 17.0	RQD = 94%						
15										
20										
25										
Remarks: Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										
Page 1 of 1 Boring No.: BB-ELAR-203A										

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine		Boring No.: BB-ELAR-204 WIN: 18915.00					
Driller: New England Boring Contractors			Elevation (ft.): 123.6			Auger ID/OD: --					
Operator: M. Porter			Datum: NAVD 88			Sampler: Split Spoon 1.375 in. ID					
Logged By: J. Fletcher			Rig Type: Mobile B-53 Track			Hammer Wt./Fall: SS-140#/30; HW-300#/16					
Date Start/Finish: 12-22-2020/12-22-2020			Drilling Method: SSA/HW Drive			Core Barrel: NQ-2.0 in. ID					
Boring Location: Sta. 199+34.0, 37.0 RT			Casing ID/OD: HW-4.0 in. ID			Water Level*: 2.1 ft					
Hammer Efficiency Factor: 0.852			Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>								
<div style="display: flex; justify-content: space-between; font-size: 0.8em;"> <div> 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 </div> <div> 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 </div> <div> 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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected </div> <div> 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 </div> </div>											
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	24/7	0.0 - 2.0	2/2/2	4	6	SSA	123.3		Brown, moist, soft, SILT, trace fine sand, organics -TOPSOIL-(OL) 0.3 Brown-grey, moist, medium stiff, Silty CLAY, trace fine sand, moderate plasticity -MARINE DEPOSIT-(CL) Similar to 1D, except stiff -MARINE DEPOSIT-(CL) 3.5 Grey, moist, medium dense, GRAVEL, trace fine sand, trace silt, loosely bonded -GLACIAL TILL-(GP) 4.0 Brown-grey, moist, hard, SILT, some fine sand, little gravel, loosely bonded -GLACIAL TILL-(ML) Note: Cobble from 4.2 to 4.4 ft. 5.8 Top of Bedrock at El. 117.8 R1: Grey, aphanitic, SILTSTONE, hard, fresh to slightly weathered. Joints dipping at moderate to steep angles, close to moderately close, planar, smooth to rough, tight to open, 0.05-in. thick silt infilled joint at 9.0 ft. Rock Quality=Fair Recovery=97% -BREWER FORMATION- R1 Core Times (min:sec): 6.5-7.5' (2:33); 7.5-8.5' (2:11); 8.5-9.5' (1:41); 9.5-10.5' (1:48); 10.5-11.5' (1:55) 11.5 Bottom of Exploration at 11.5 feet below ground surface.	
	2D/A	24/20	2.0 - 4.0	5/8/7/20	15	21		120.1			
								119.6			
5	3D	22/14	4.0 - 5.8	32/23/17/50(4")	40	57	RC	117.8			
	R1	60/58	6.5 - 11.5	RQD = 75%			NQ				
							CORE				
								112.1			
10											
15											
20											
25											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 1 Boring No.: BB-ELAR-204	

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



- Top Row:** BB-ELAR-102: Run No. R1 6.6' (left) to 9.6' (middle); Run No. R2 9.6' (middle) to 12.4' (right)
- Top Middle Row:** BB-ELAR-102: Run No. R2 continued 9.6' (left) to 12.4' (middle); Run No. R3 12.4' (left) to 16.5' (right)
- Bottom Middle Row:** BB-ELAR-101: Run No. R1 5.5' (left) to 7.0' (middle-left); Run No. R2 7.0' (middle-left) to 7.7' (middle-right); Run No. R3 7.7' (middle-right) to 10.6' (right)
- Bottom Row:** BB-ELAR-101: Run No. R3 continued 7.7' (left) to 10.6' (middle-left); Run No. R4 10.6' (middle-left) to 11.6' (middle-right); Run No. R5 11.6' (middle-right) to 15.5' (right)

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



Bottom Row: BB-ELAR-101; Run No. R5 11.6' (middle-right) to 15.5' (right)

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



Top Row: BB-ELAR-201: Run No. R1 3.4' (left) to 8.4' (right)
Top Middle Row: BB-ELAR-201A: Run No. R1 4.7' (left) to 9.7' (right)
Bottom Middle Row: BB-ELAR-201A: Run No. R2 9.7' (left) to 14.7' (right)
Bottom Row: BB-ELAR-202: Run No. R1 8.5' (left) to 13.5' (right)

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



Top Row: BB-ELAR-202: Run No. R2 13.5' (left) to 14.5' (right)

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



Top Row: BB-ELAR-203: Run No. R1 1.5' (left) to 3.8' (middle); Run No. R2 3.8' (middle) to 6.8' (right)

Top Middle Row: BB-ELAR-203: Run No. R2 continued 3.8' (left) to 6.8' (middle-left); BB-ELAR-203A: Run No. R1 6.0' (middle-left) to 11.0' (right)

Bottom Middle Row: BB-ELAR-203A: Run No. R2 11.0' (left) to 13.0' (right)

Bottom Row: BB-ELAR-203A: Run No. R3 13.0' (left) to 17.0' (right)

**ROCK CORE PHOTOGRAPHS
INTERSTATE 395/ROUTE 9 CONNECTOR BRIDGE OVER LAMBERT ROAD
MAINEDOT WIN 018915.00
EDDINGTON, MAINE**



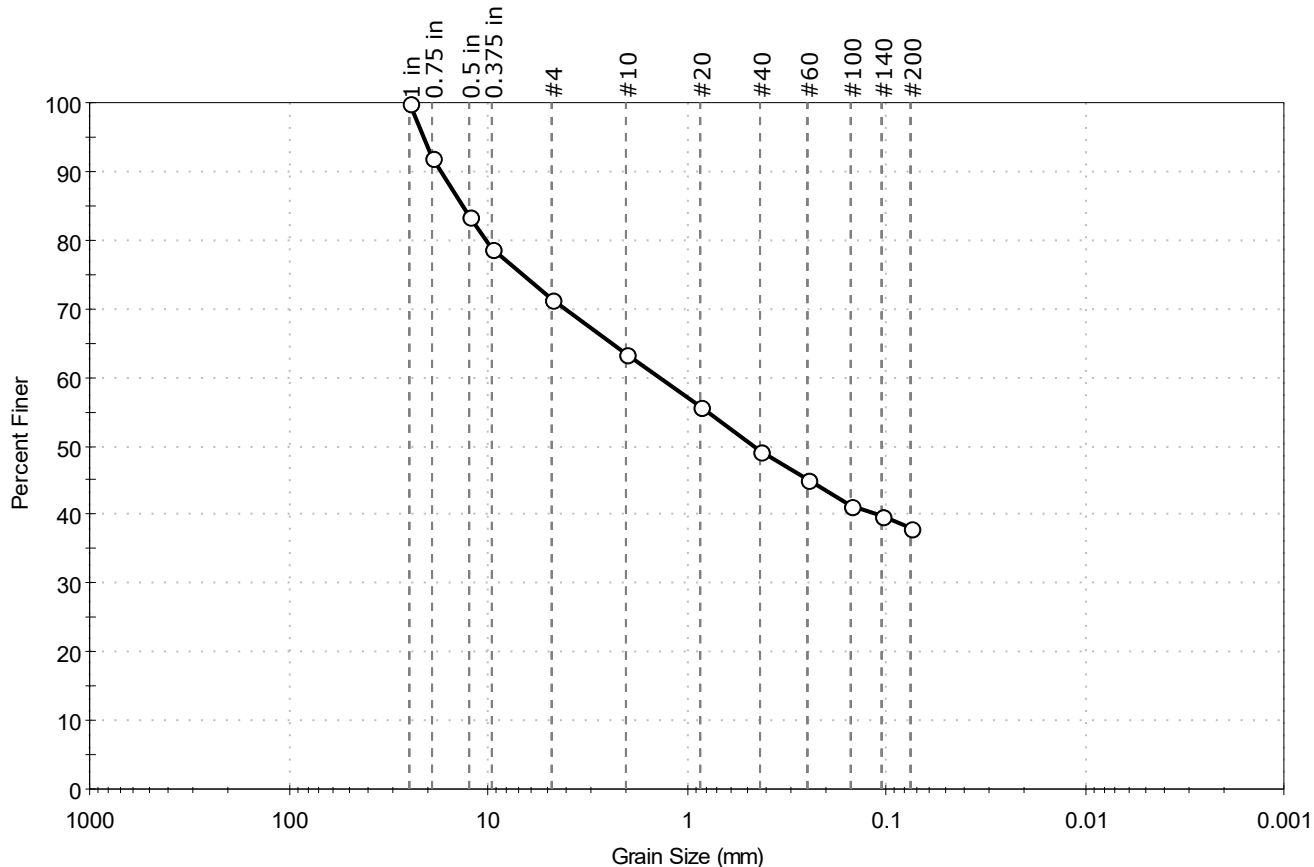
Bottom Middle Row: BB-ELAR-204: Run No. R1 6.5' (middle-right) to 11.5' (right)
Bottom Row: BB-ELAR-204: Run No. R1 continued 6.5' (left) to 11.5' (right)

APPENDIX B

Laboratory Test Results

Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Bridge (Lambert Rd)		
Location:	Eddington, ME	Project No:	GTX-313323
Boring ID:	BB-ELAR-201A	Sample Type:	jar
Sample ID:	2D	Test Date:	03/19/21
Depth :	2-4	Test Id:	613355
Test Comment:	---		
Visual Description:	Moist, dark brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	28.6	33.5	37.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	92		
0.5 in	12.50	83		
0.375 in	9.50	79		
#4	4.75	71		
#10	2.00	63		
#20	0.85	56		
#40	0.42	49		
#60	0.25	45		
#100	0.15	41		
#140	0.11	40		
#200	0.075	38		

Coefficients

$D_{85} = 13.4687 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = 1.3581 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.4577 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

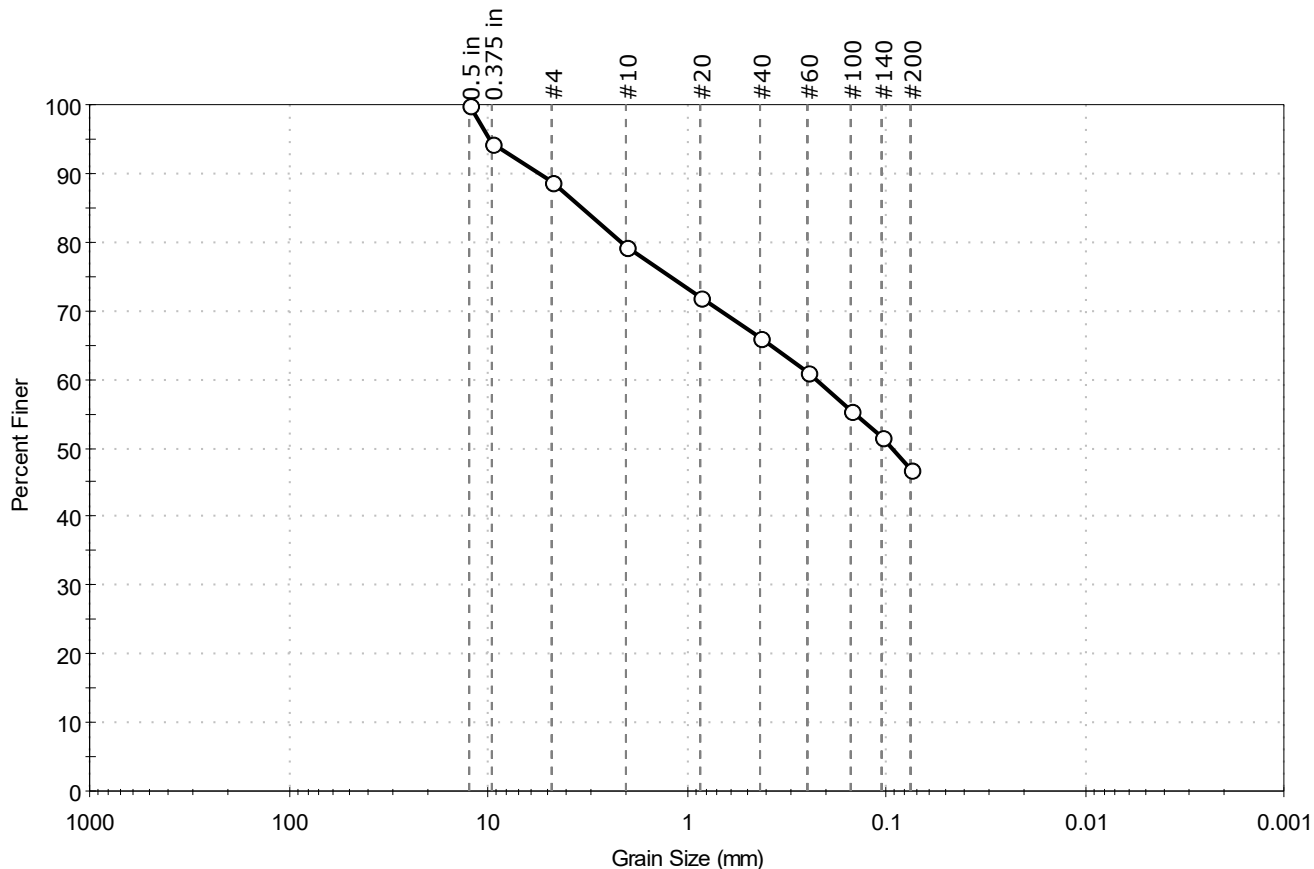
AASHTO Silty Soils (A-4 (0))

Sample/Test Description

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

Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Bridge (Lambert Rd)		
Location:	Eddington, ME	Project No:	GTX-313323
Boring ID:	BB-ELAR-203A	Sample Type:	jar
Sample ID:	3D	Test Date:	03/22/21
Depth :	4-5.5	Test Id:	613356
Test Comment:	---		
Visual Description:	Moist, olive sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.3	41.8	46.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	95		
#4	4.75	89		
#10	2.00	79		
#20	0.85	72		
#40	0.42	66		
#60	0.25	61		
#100	0.15	56		
#140	0.11	52		
#200	0.075	47		

Coefficients

$D_{85} = 3.3596 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = 0.2274 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.0945 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

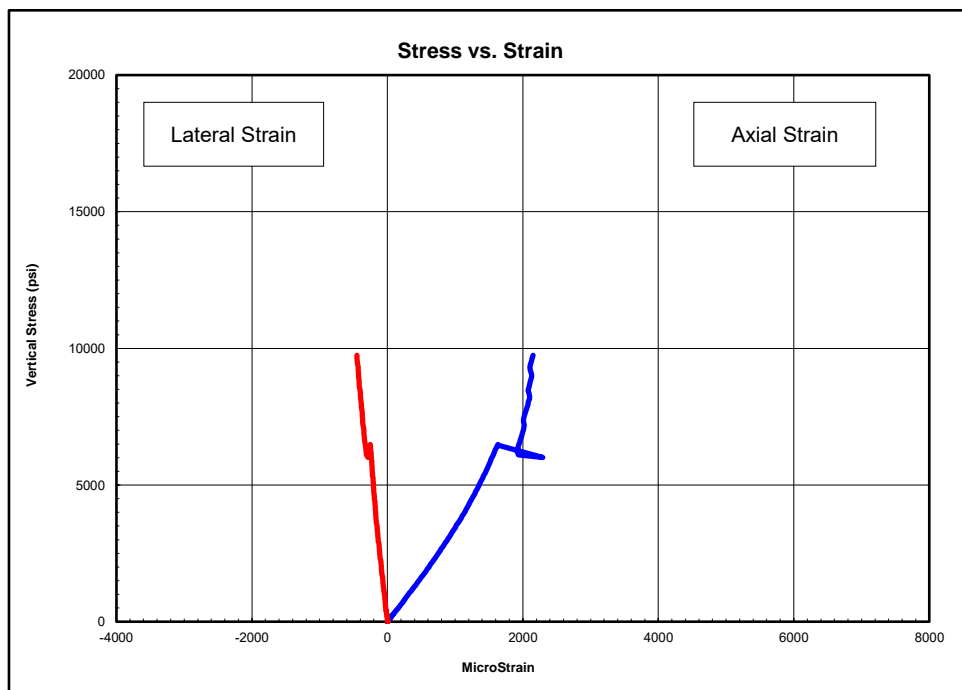
Sample/Test Description

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



Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-ELAR-101
Sample ID:	R3
Depth, ft:	8.1-9.3
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 9,747 psi

The axial strain gauges picked up an initial failure within the specimen and then continued reading until total failure occurred.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1000-3500	3,530,000	0.16
3500-6100	4,740,000	0.15
6100-8700	2,770,000	0.20

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

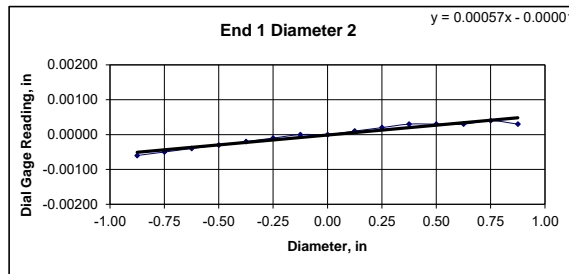
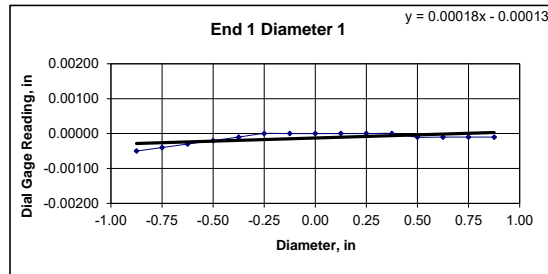


Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/1-395 Lambert Rd Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308856		
Boring ID:	BB-ELAR-101		
Sample ID:	R3		
Depth:	8.1-9.3 ft		
Visual Description:	See photographs		

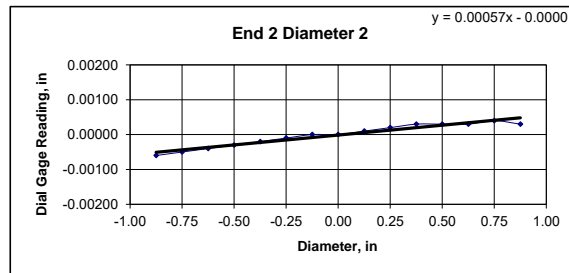
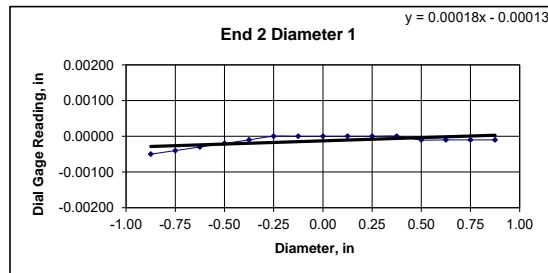
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.30	4.30	4.30	Maximum difference must be < 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	586.57				
Bulk Density, lb/ft ³ :	168				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES	Length to Diameter Ratio Tolerance Met? YES		

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00030	0.00040	0.00030
Difference between max and min readings, in: 0° = 0.00050 90° = 0.00100															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00030	0.00040	0.00030
Difference between max and min readings, in: 0° = 0.0005 90° = 0.001 Maximum difference must be < 0.0020 in. Difference = ± 0.00050 Flatness Tolerance Met? YES															



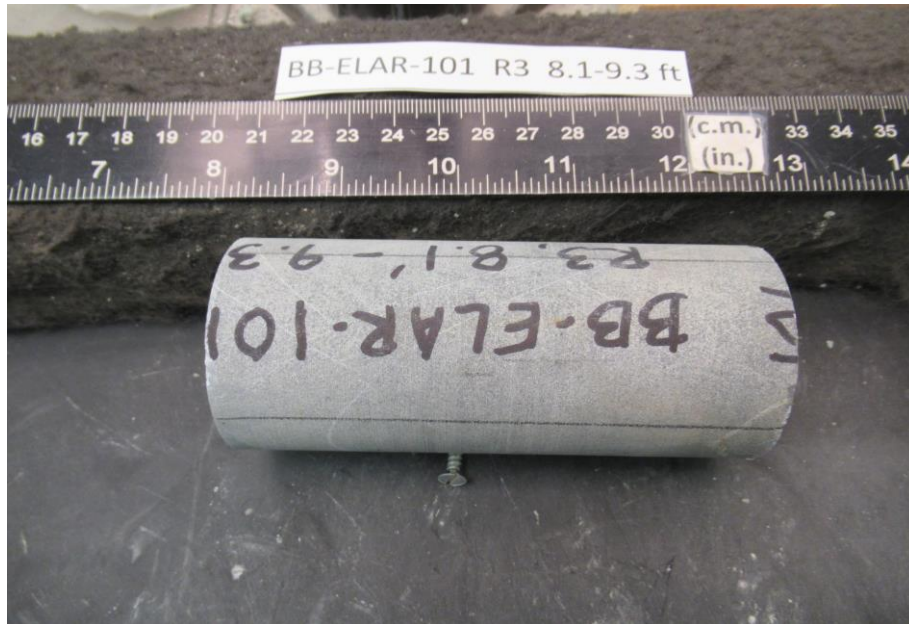
DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01031
End 2:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01031
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00057
Angle of Best Fit Line:	0.03241
End 2:	
Slope of Best Fit Line	0.00057
Angle of Best Fit Line:	0.03241
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met? Spherically Seated	YES

PERPENDICULARITY (Procedure P1)					(Calculated from End Flatness and Parallelism measurements above)	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq 0.25^\circ$
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES	Perpendicularity Tolerance Met? YES
Diameter 2, in (rotated 90°)	0.00100	1.980	0.00051	0.029	YES	
END 2						
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES	
Diameter 2, in (rotated 90°)	0.00100	1.980	0.00051	0.029	YES	

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-101
Sample ID:	R3
Depth, ft:	8.1-9.3



After cutting and grinding

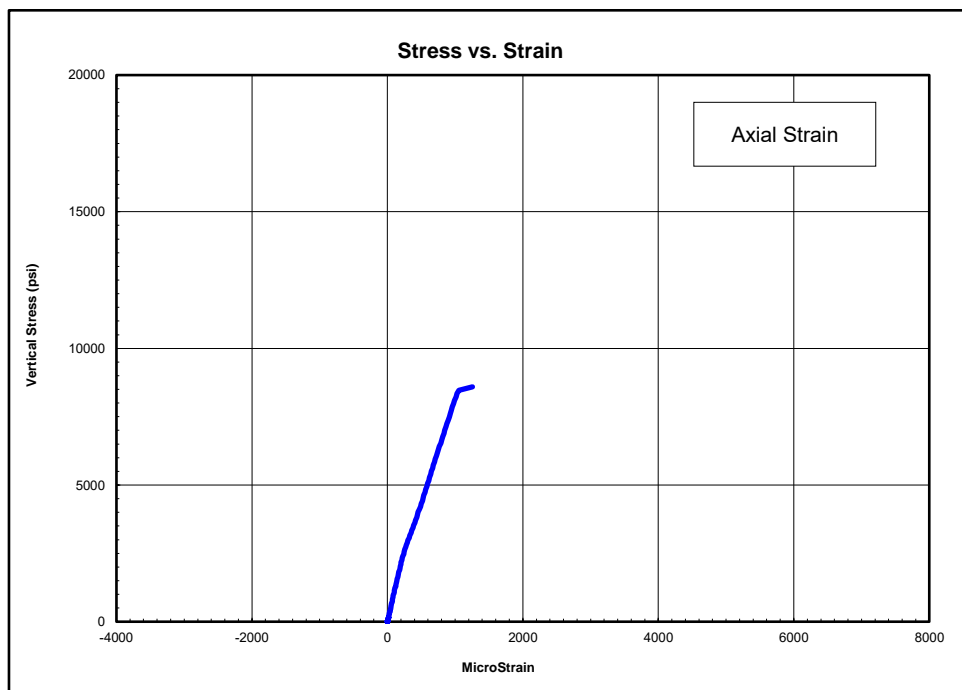


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-ELAR-102
Sample ID:	R2
Depth, ft:	10.7-11.7
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 12,789 psi

Both lateral strain gauge failed to record meaningful data. Poisson's Ratio could not be determined. The axial strain gauges failed before the peak value was attained.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1300-4700	7,720,000	---
4700-8100	7,660,000	---
8100-11500	---	---

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

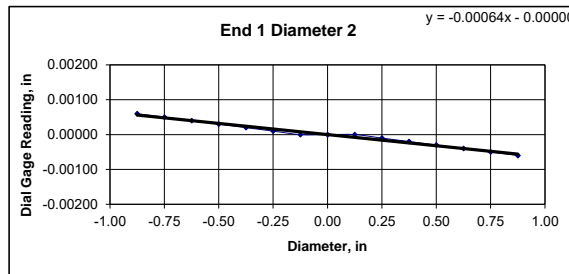
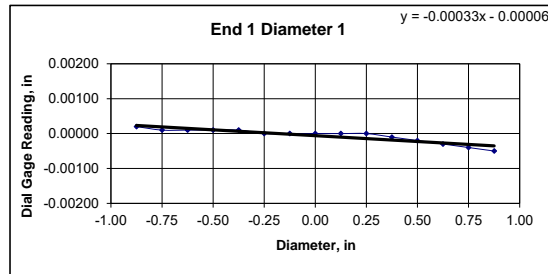


Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/1-395 Lambert Rd Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308856		
Boring ID:	BB-ELAR-102		
Sample ID:	R2		
Depth:	10.7-11.7 ft		
Visual Description:	See photographs		

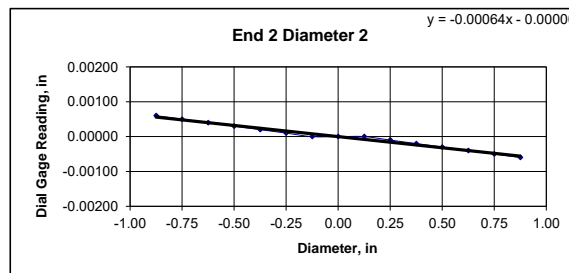
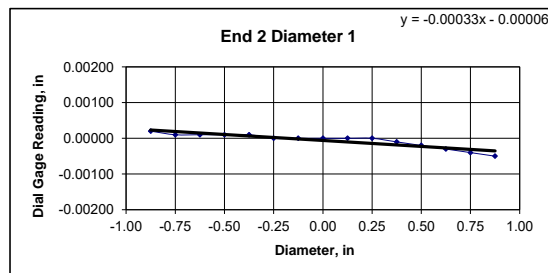
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.34	4.34	4.34	Maximum difference must be $<$ 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	593.34				
Bulk Density, lb/ft ³ :	169				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES	Length to Diameter Ratio Tolerance Met? YES		

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060
Difference between max and min readings, in: 0° = 0.00070 90° = 0.00120															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060
Difference between max and min readings, in: 0° = 0.0007 90° = 0.0012 Maximum difference must be < 0.0020 in. Difference = ± 0.00060 Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00033
Angle of Best Fit Line:	0.01915
End 2:	
Slope of Best Fit Line	0.00033
Angle of Best Fit Line:	0.01915
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00064
Angle of Best Fit Line:	0.03667
End 2:	
Slope of Best Fit Line	0.00064
Angle of Best Fit Line:	0.03667
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met? Spherically Seated	YES

PERPENDICULARITY (Procedure P1)					(Calculated from End Flatness and Parallelism measurements above)	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be \leq 0.25°
Diameter 1, in	0.00070	1.980	0.00035	0.020	YES	Perpendicularity Tolerance Met? YES
Diameter 2, in (rotated 90°)	0.00120	1.980	0.00061	0.035	YES	
END 2						
Diameter 1, in	0.00070	1.980	0.00035	0.020	YES	
Diameter 2, in (rotated 90°)	0.00120	1.980	0.00061	0.035	YES	

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-102
Sample ID:	R2
Depth, ft:	10.7-11.7



After cutting and grinding

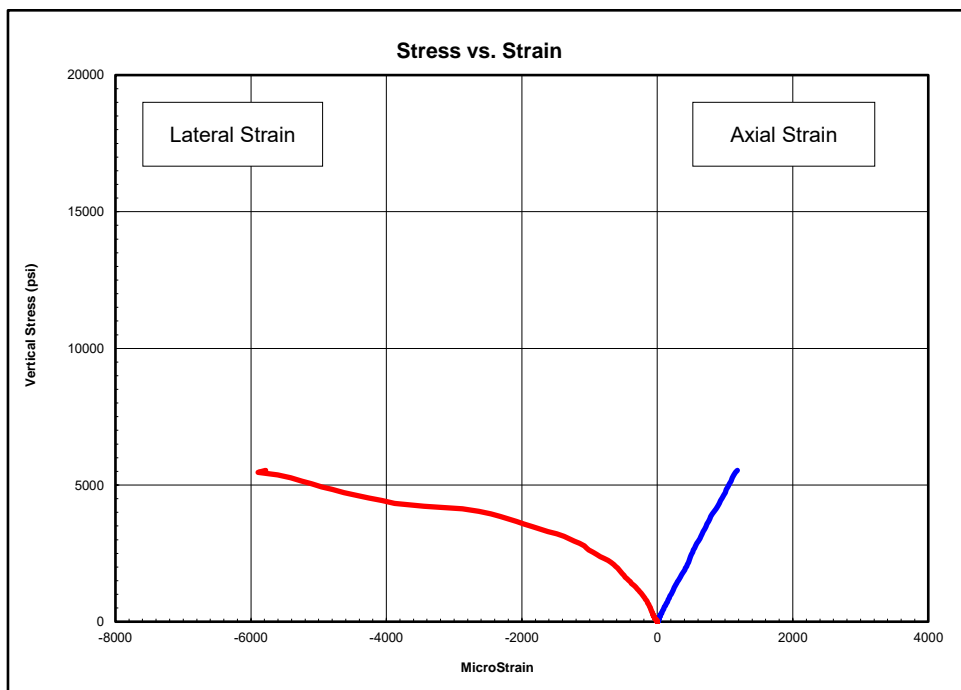


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-ELAR-102
Sample ID:	R3
Depth, ft:	14.3-15.3
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 11,024 psi

The strain values recorded for this test produce values of Poisson's Ratio that exceed maximum values found in rocks. The strain gauges failed before the peak value was attained.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-4000	4,870,000	---
4000-5500	4,670,000	---
5500-10000	---	---

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

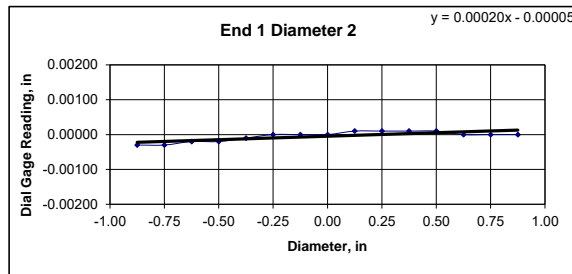
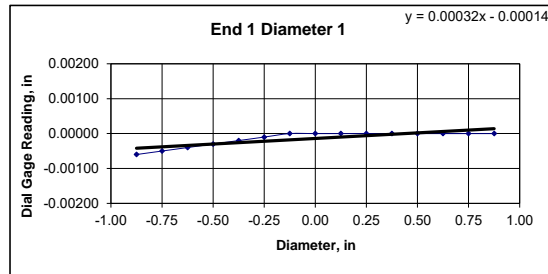


Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/1-395 Lambert Rd Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308856		
Boring ID:	BB-ELAR-102		
Sample ID:	R3		
Depth:	14.3-15.3 ft		
Visual Description:	See photographs		

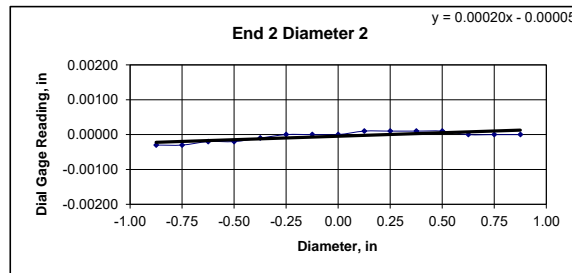
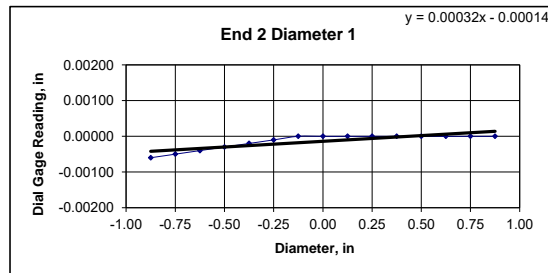
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.23	4.22	4.23	Maximum difference must be $<$ 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	572.84				
Bulk Density, lb/ft ³	167				
Length to Diameter Ratio:	2.1				
		Minimum Diameter Tolerance Met?	YES		
		Length to Diameter Ratio Tolerance Met?	YES		

END FLATNESS AND PARALLELISM (Procedure FP1)														
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000
Difference between max and min readings, in: 0° = 0.00060 90° = 0.00040														
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000
Difference between max and min readings, in: 0° = 0.0006 90° = 0.0004 Maximum difference must be $<$ 0.0020 in. Difference = \pm 0.00030 Flatness Tolerance Met? YES														



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00032
Angle of Best Fit Line:	0.01833
End 2:	
Slope of Best Fit Line	0.00032
Angle of Best Fit Line:	0.01833
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met?	YES
Spherically Seated	



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01146
End 2:	
Slope of Best Fit Line	0.00020
Angle of Best Fit Line:	0.01146
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met?	YES
Spherically Seated	

PERPENDICULARITY (Procedure P1)						Maximum angle of departure must be \leq 0.25°	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00060	1.980	0.00030	0.017	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES		
END 2							
Diameter 1, in	0.00060	1.980	0.00030	0.017	YES		
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Lambert Rd Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308856
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-102
Sample ID:	R3
Depth, ft:	14.3-15.3



After cutting and grinding

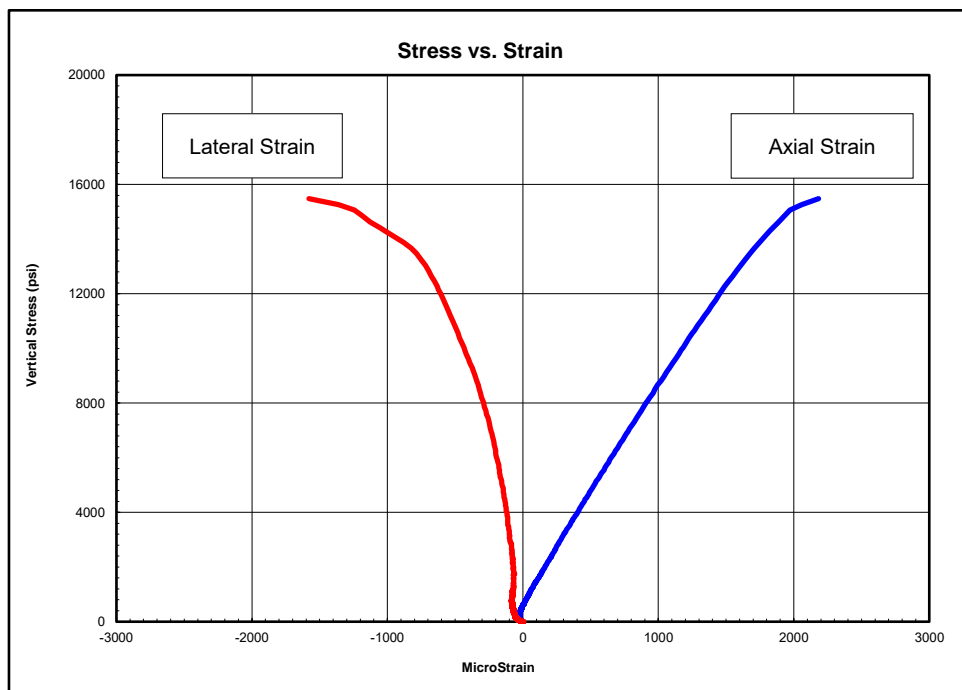


After break



Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-201A
Sample ID:	R2
Depth, ft:	13.2-14.2
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 15,479 psi

The strain values recorded within the third stress range for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1500-5700	8,270,000	0.22
5700-9800	7,640,000	0.43
9800-13900	6,970,000	---

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

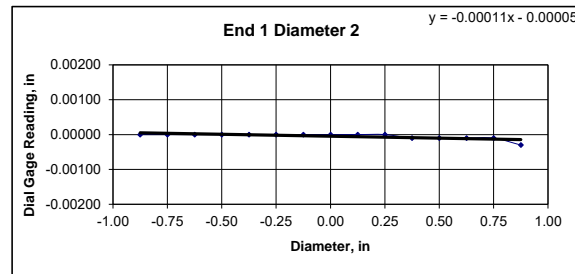
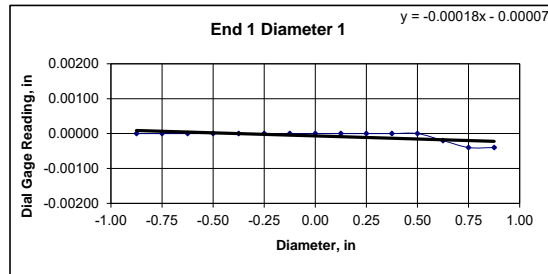


Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)	Tested By:	cmh
Project Location:	Eddington, ME	Checked By:	smd
GTx #:	313323		
Boring ID:	BB-ELAR-201A		
Sample ID:	R2		
Depth:	13.2-14.2 ft		
Visual Description:	See photographs		

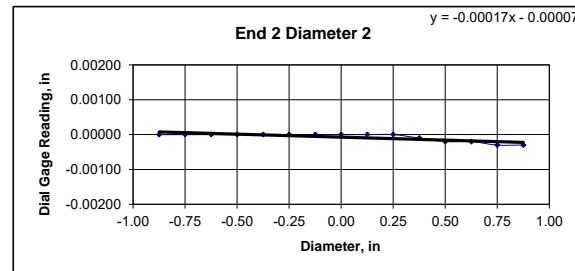
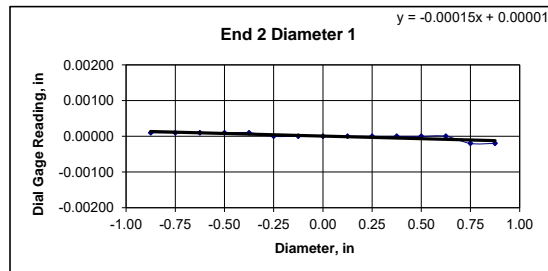
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.41	4.41	4.41	Maximum difference must be < 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	605.35				
Bulk Density, lb/ft ³ :	168				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

END FLATNESS AND PARALLELISM (Procedure FP1)																
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875	
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00040	-0.00040	
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00030	
Difference between max and min readings, in:																
0° =												0.00040		90° =		0.00030
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875	
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00020	
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00030	
Difference between max and min readings, in:																
0° =												0.0003		90° =		0.0003
Maximum difference must be < 0.0020 in. Difference = ± 0.00020																
Flatness Tolerance Met? YES																



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01015
End 2:	
Slope of Best Fit Line	0.00015
Angle of Best Fit Line:	0.00835
Maximum Angular Difference:	0.00180
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00638
End 2:	
Slope of Best Fit Line	0.00017
Angle of Best Fit Line:	0.00982
Maximum Angular Difference:	0.00344
Parallelism Tolerance Met? Spherically Seated	YES

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

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-ELAR-201A
Sample ID:	R2
Depth, ft:	13.2-14.2



After cutting and grinding

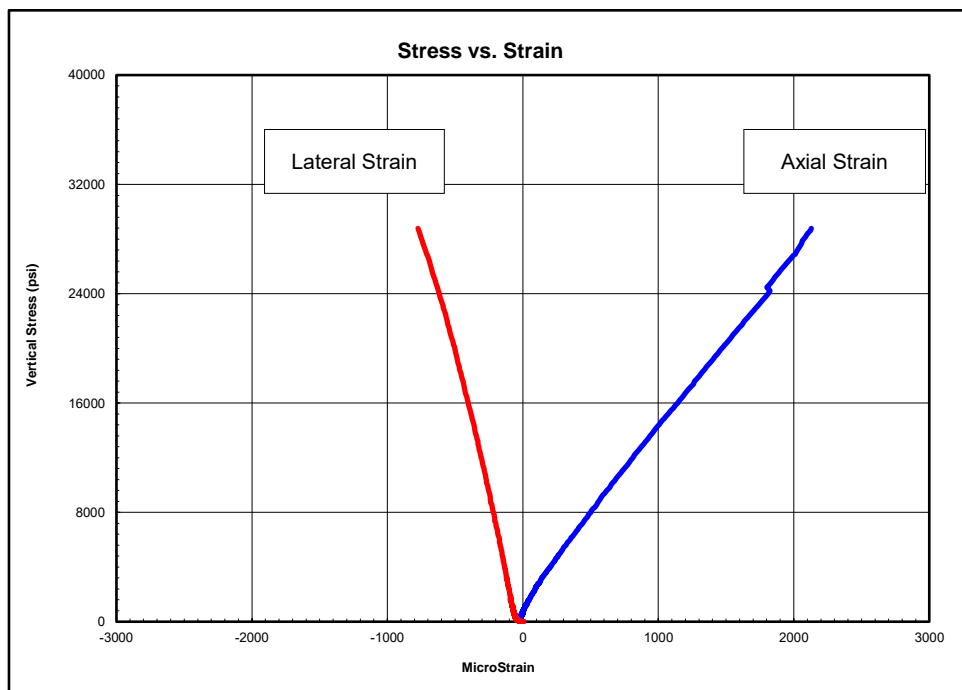


After break



Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-202
Sample ID:	R1
Depth, ft:	10.1-11.0
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 28,776 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2900-10600	13,400,000	0.28
10600-18200	12,200,000	0.30
18200-25900	12,500,000	0.35

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

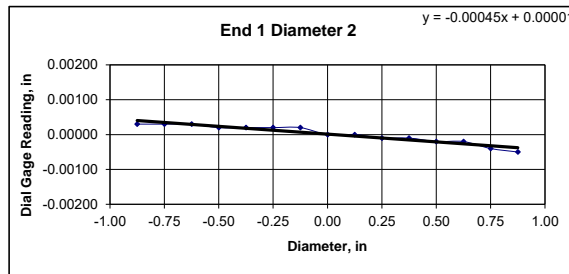
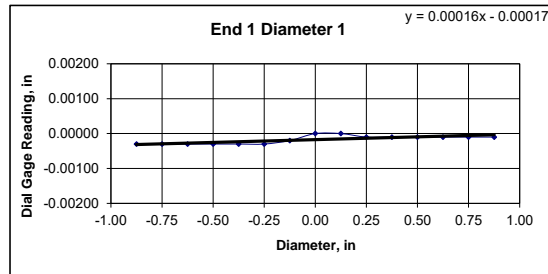


Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)	Tested By:	cmh
Project Location:	Eddington, ME	Checked By:	smd
GT#:	313323		
Boring ID:	BB-ELAR-202		
Sample ID:	R1		
Depth:	10.1-11.0 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.41	4.41	4.41	Maximum difference must be $<$ 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	607.23				
Bulk Density, lb/ft ³	168				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES	Length to Diameter Ratio Tolerance Met? YES		

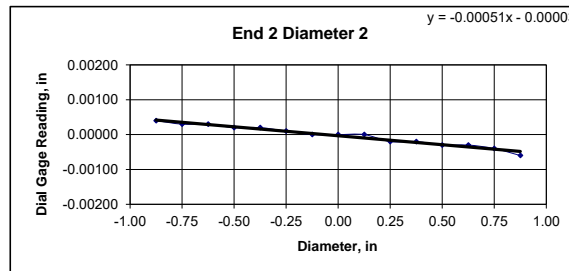
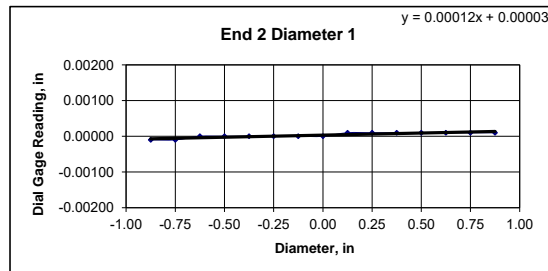
END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00020	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00020	0.00020	0.00020	0.00020	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00040	-0.00050
Difference between max and min readings, in: 0° = 0.00030 90° = 0.00080															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	-0.00020	-0.00020	-0.00030	-0.00030	-0.00040	-0.00060
Difference between max and min readings, in: 0° = 0.0002 90° = 0.001 Maximum difference must be < 0.0020 in. Difference = ± 0.00050 Flatness Tolerance Met? YES															



DIAMETER 1

End 1:	Slope of Best Fit Line	0.00016
	Angle of Best Fit Line:	0.00917
End 2:	Slope of Best Fit Line	0.00012
	Angle of Best Fit Line:	0.00671
Maximum Angular Difference:		0.00246

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:	Slope of Best Fit Line	0.00045
	Angle of Best Fit Line:	0.02554
End 2:	Slope of Best Fit Line	0.00051
	Angle of Best Fit Line:	0.02947
Maximum Angular Difference:		0.00393

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)					(Calculated from End Flatness and Parallelism measurements above)	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be \leq 0.25°
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES	
END 2						
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES	
Diameter 2, in (rotated 90°)	0.00100	1.990	0.00050	0.029	YES	

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-ELAR-202
Sample ID:	R1
Depth, ft:	10.1-11.0



After cutting and grinding

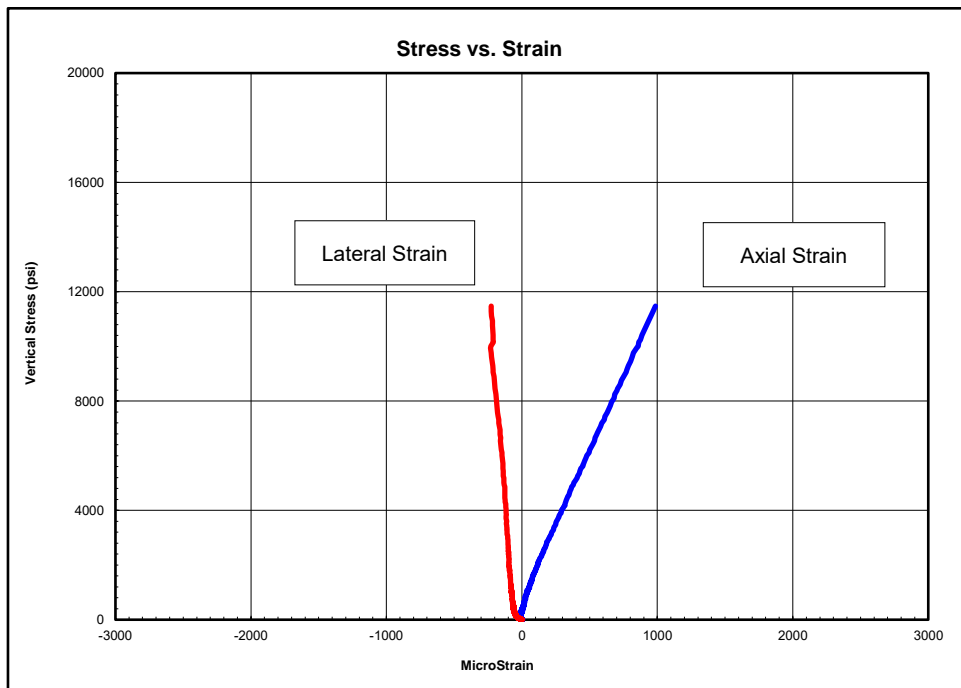


After break



Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-203
Sample ID:	R2
Depth, ft:	5.1-5.6
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 11,830 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1200-4300	11,500,000	0.16
4300-7500	10,600,000	0.18
7500-10600	10,900,000	0.22

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

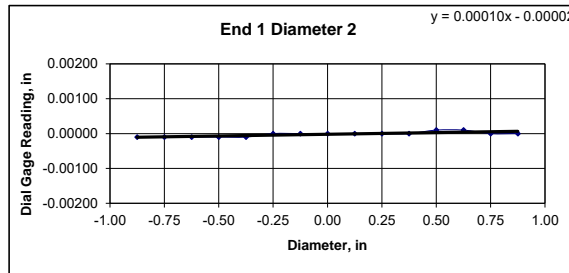
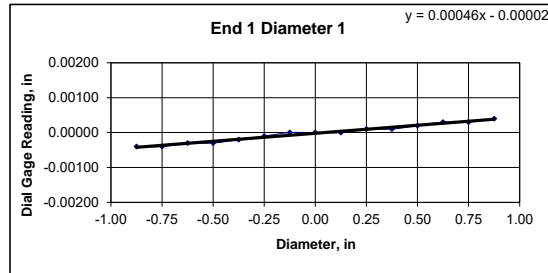


Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)	Tested By:	cmh
Project Location:	Eddington, ME	Checked By:	smd
GTX #:	313323		
Boring ID:	BB-ELAR-203		
Sample ID:	R2		
Depth:	5.1-5.6 ft		
Visual Description:	See photographs		

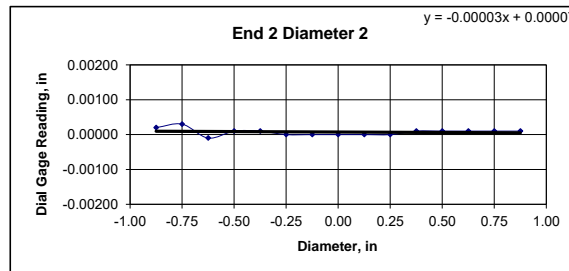
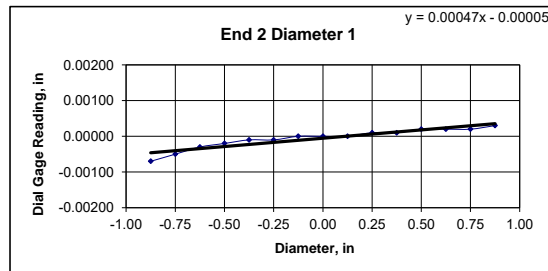
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.? YES	
Specimen Length, in:	4.50	4.50	4.50	Maximum difference must be < 0.020 in.	
Specimen Diameter, in:	1.99	1.98	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	618.18				
Bulk Density, lb/ft ³ :	169				
Length to Diameter Ratio:	2.3				
		Minimum Diameter Tolerance Met?	YES		
		Length to Diameter Ratio Tolerance Met?	YES		

END FLATNESS AND PARALLELISM (Procedure FP1)													
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625
Diameter 1, in	-0.00040	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000
Difference between max and min readings, in:													
0° = 0.00080 90° = 0.00020													
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625
Diameter 1, in	-0.00070	-0.00050	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030
Diameter 2, in (rotated 90°)	0.00020	0.00030	-0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Difference between max and min readings, in:													
0° = 0.001 90° = 0.0004													
Maximum difference must be < 0.0020 in. Difference = ± 0.00050													
Flatness Tolerance Met? YES													



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00046
Angle of Best Fit Line:	0.02636
End 2:	
Slope of Best Fit Line	0.00047
Angle of Best Fit Line:	0.02668
Maximum Angular Difference:	0.00033
Parallelism Tolerance Met?	YES
Spherically Seated	



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00010
Angle of Best Fit Line:	0.00557
End 2:	
Slope of Best Fit Line	0.00003
Angle of Best Fit Line:	0.00147
Maximum Angular Difference:	0.00409
Parallelism Tolerance Met?	YES
Spherically Seated	

PERPENDICULARITY (Procedure P1)						Maximum angle of departure must be $\leq 0.25^\circ$	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00080	1.985	0.00040	0.023	YES		
Diameter 2, in (rotated 90°)	0.00020	1.985	0.00010	0.006	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00100	1.985	0.00050	0.029	YES		
Diameter 2, in (rotated 90°)	0.00040	1.985	0.00020	0.012	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-ELAR-203
Sample ID:	R2
Depth, ft:	5.1-5.6



After cutting and grinding

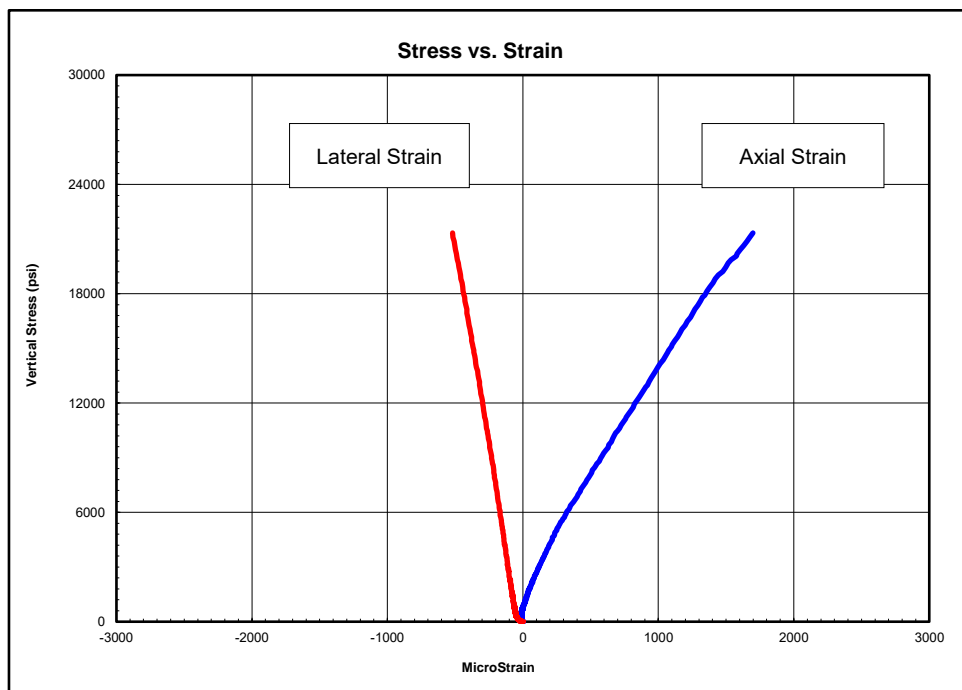


After break



Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-ELAR-203A
Sample ID:	R3
Depth, ft:	14.7-15.7
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 21,335 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2100-7800	14,200,000	0.28
7800-13500	11,800,000	0.26
13500-19200	11,500,000	0.27

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

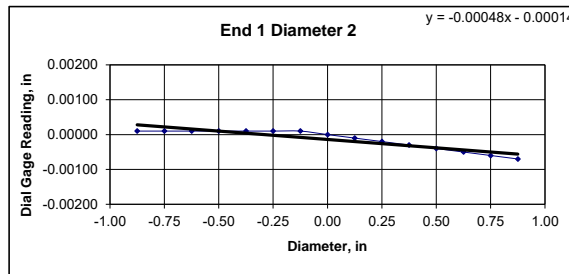
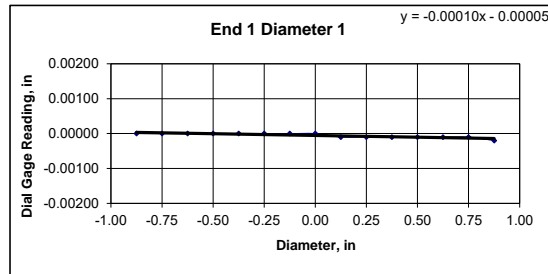


Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)	Tested By:	cmh
Project Location:	Eddington, ME	Checked By:	smd
GTX #:	313323		
Boring ID:	BB-ELAR-203A		
Sample ID:	R3		
Depth:	14.7-15.7 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap \leq 0.02 in.?	
Specimen Length, in:	4.45	4.46	4.46	YES	
Specimen Diameter, in:	1.99	1.99	1.99	Maximum difference must be < 0.020 in.	
Specimen Mass, g:	614.29			Straightness Tolerance Met?	
Bulk Density, lb/ft ³ :	169			YES	
Length to Diameter Ratio:	2.2			Minimum Diameter Tolerance Met?	
				YES	
				Length to Diameter Ratio Tolerance Met?	
				YES	

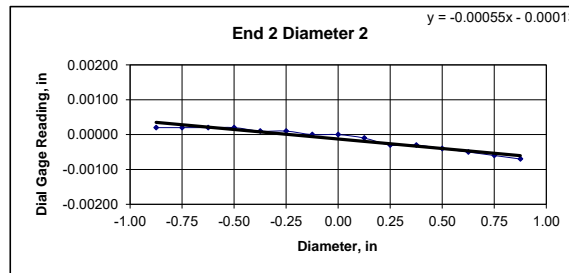
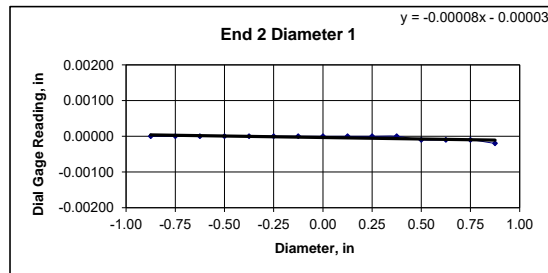
END FLATNESS AND PARALLELISM (Procedure FP1)																	
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875		
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00020		
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00070		
Difference between max and min readings, in:																	
	0° =						0.00020								90° =		0.00080
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875		
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020		
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00030	-0.00030	-0.00040	-0.00050	-0.00060	-0.00070		
Difference between max and min readings, in:																	
	0° =						0.0002								90° =		0.0009
Maximum difference must be < 0.0020 in.																	
Difference = ± 0.00045																	
Flatness Tolerance Met?																	
YES																	



DIAMETER 1

End 1:	Slope of Best Fit Line	0.00010
	Angle of Best Fit Line:	0.00573
End 2:	Slope of Best Fit Line	0.00008
	Angle of Best Fit Line:	0.00475
Maximum Angular Difference:		0.00098

Parallelism Tolerance Met? YES
Spherically Seated



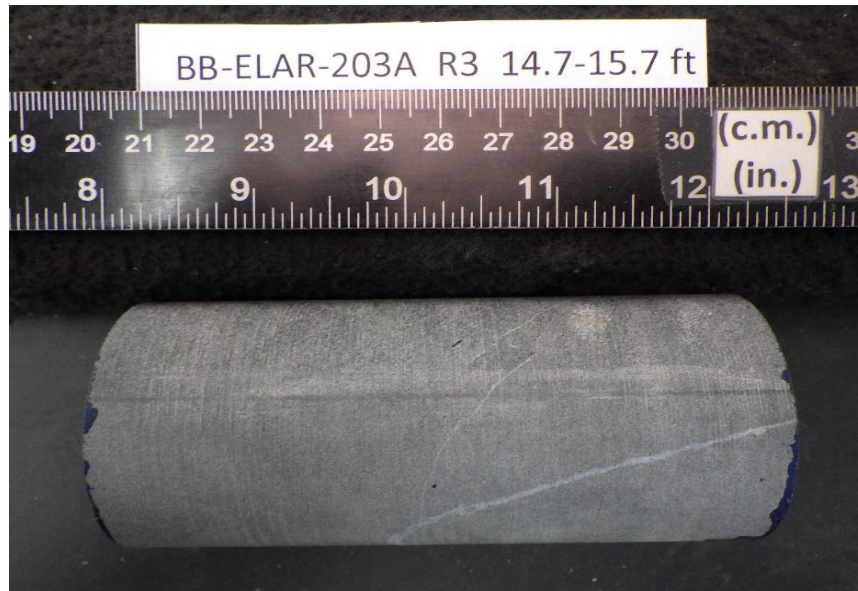
DIAMETER 2

End 1:	Slope of Best Fit Line	0.00048
	Angle of Best Fit Line:	0.02750
End 2:	Slope of Best Fit Line	0.00055
	Angle of Best Fit Line:	0.03127
Maximum Angular Difference:		0.00377

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						Maximum angle of departure must be $\leq 0.25^\circ$	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00080	1.990	0.00040	0.023	YES	Perpendicularity Tolerance Met?	
						YES	
END 2							
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00090	1.990	0.00045	0.026	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Lambert Rd)
Project Location:	Eddington, ME
GTX #:	313323
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-ELAR-203A
Sample ID:	R3
Depth, ft:	14.7-15.7



After cutting and grinding

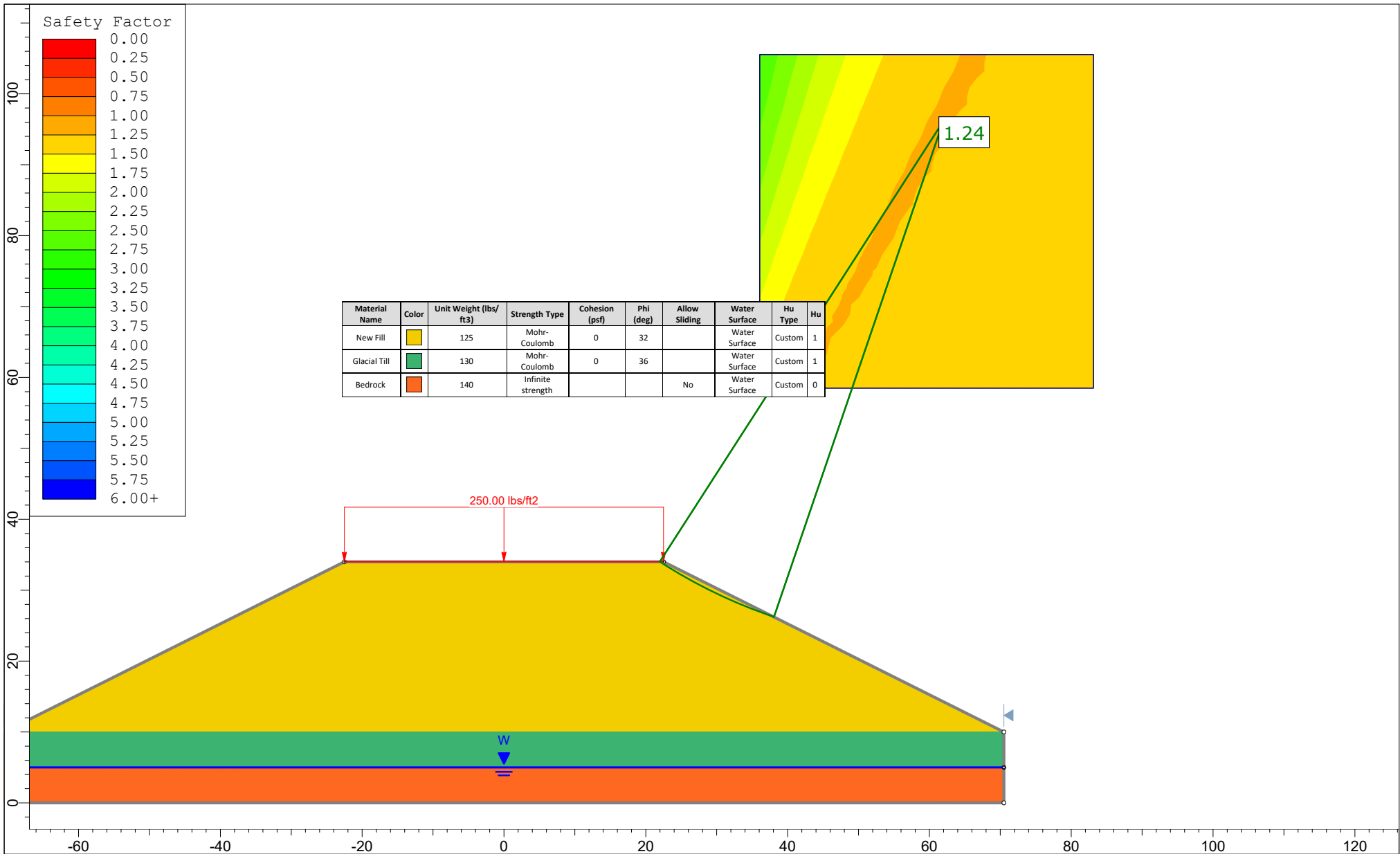



After break

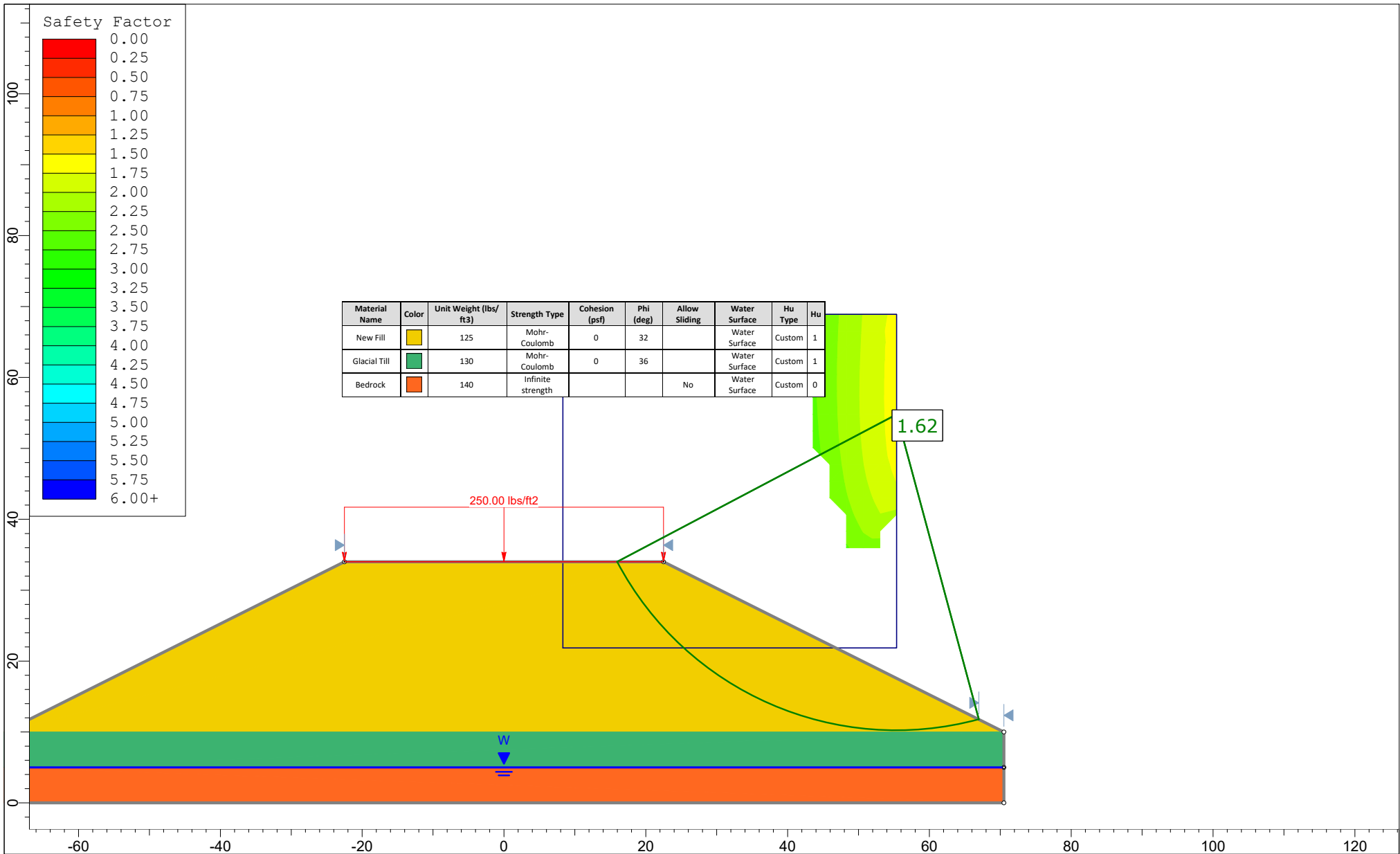
APPENDIX C


Geotechnical Design Calculations

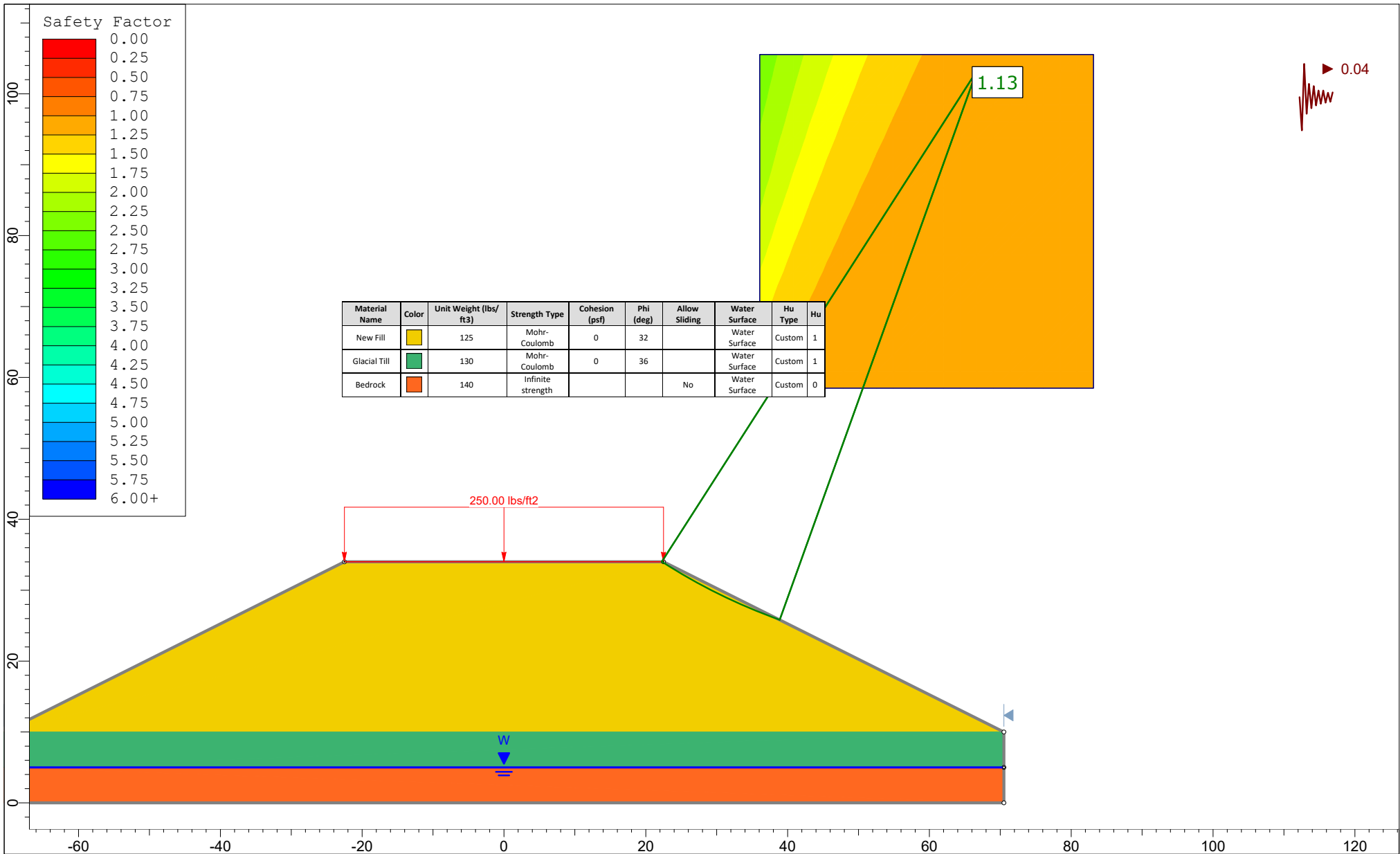
Global Stability



 <small>SLIDEINTERPRET 9.004</small>	<i>Project</i> SLIDE - An Interactive Slope Stability Program	
	<i>Group</i> 2020-0520-HAI-Lambert Road Stability-D1.slim	<i>Scenario</i> 2020-0520-HAI-Lambert Road Stability-D1.slim
	<i>Drawn By</i>	<i>Company</i>
	<i>Date</i> 5/20/2020, 11:04:24 AM	<i>File Name</i> 2020-0520-HAI-Lambert Road Stability-D1.slim



	Project			SLIDE - An Interactive Slope Stability Program		
	Group	2020-0520-HAI-Lambert Road Stability-D1.slim			Scenario	2020-0520-HAI-Lambert Road Stability-D1.slim
	Drawn By				Company	
	Date	5/20/2020, 11:04:24 AM			File Name	2020-0520-HAI-Lambert Road Stability-D1.slim



Project

SLIDE - An Interactive Slope Stability Program

Group

2020-0520-HAI-Lambert Road Stability-D1.slim

Scenario

2020-0520-HAI-Lambert Road Stability-D1.slim

Drawn By

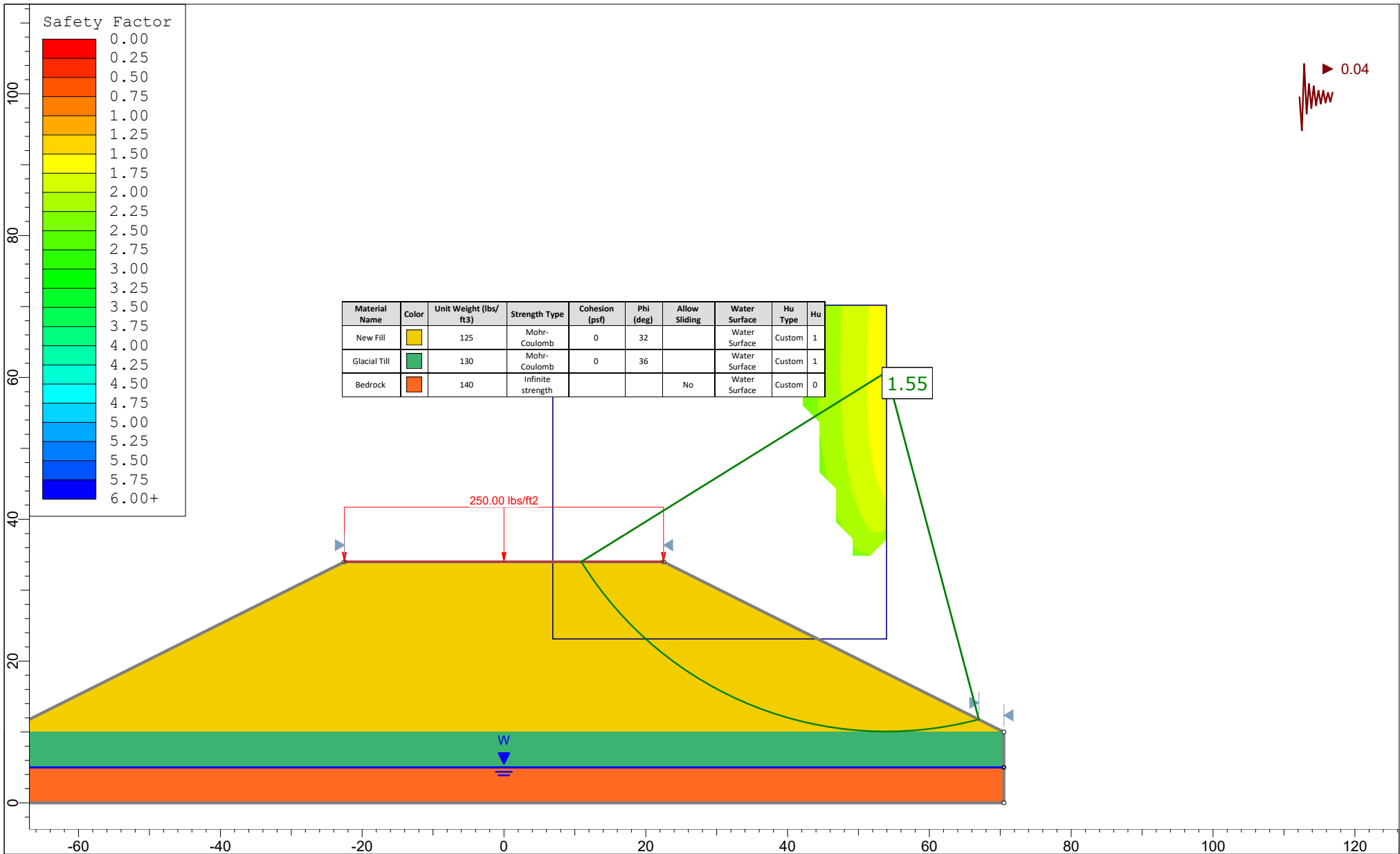
Company

Date

5/20/2020, 11:04:24 AM

File Name

2020-0520-HAI-Lambert Road Stability-D1.slim



	Project		
	SLIDE - An Interactive Slope Stability Program		
	Group	2020-0520-HAI-Lambert Road Stability-D1.slim	Scenario 2020-0520-HAI-Lambert Road Stability-D1.slim
	Drawn By		Company
	Date	5/20/2020, 11:04:24 AM	File Name 2020-0520-HAI-Lambert Road Stability-D1.slim

Axial Compression Pile Resistance

Client: Maine Department of Transportation

Date: 28-Jan-2021

Project: I-395/Route 9 Connector Bridge over Lambert Road - WIN 018915.00

Computed by: NAS

Subject: Geotechnical and Structural Resistance of Steel H-Piles

Checked by: BCS

ASSUMPTIONS

- Currently, three different steel H-pile sections are being considered (HP12x74, HP14x89, and HP14x117).
- Steel plates will be welded across the pile tips to increase the nominal geotechnical axial pile resistance and the piles will be installed (suspended) in bedrock sockets (length and diameter to TBD), which will be backfilled with grout up to the top of the bedrock surface. Grout, having an unconfined compressive strength of 5,000 psi, will be present between the bottom of the steel plate and the bottom of the rock socket.
- Estimate the nominal geotechnical axial pile resistances based on two different assumptions for bedrock jointing consistent with guidance provided in AASHTO LRFD eqns. 10.8.3.5.4c-1 and -2, which allows for maximum and minimum resistances to be calculated for "intact" and "jointed" rock mass conditions, respectively.
- Plate dimensions for HP12x74, HP14x89 and HP14x117 sections are assumed to match the depth and flange widths of the H-piles.
- The quality and description of the bedrock within 2B below the bottom of the assumed 5-ft-long rock socket

Assuming the bedrock below the base of the bedrock socket is either intact or tightly jointed (up to 2B below the bottom of the socket), the nominal unit tip resistance is as follows:

Uniaxial compressive strength of grout, q_u = 5.0 ksi (per MaineDOT)

Note: Because grout is present between the bottom of the plate and the bottom of the rock socket, the tip resistance will be controlled by the lesser of the grout uniaxial compressive strength (UCS) and the bedrock UCS. Since bedrock UCS ranges from 9,747 psi to 28,776 psi, which is greater than the UCS for grout, the UCS for grout controls. Per MaineDOT, the UCS for grout is typically no greater than 6,000 psi in accordance with SP 501 Rock Socketed H-Piles

Uniaxial compressive strength of grout, q_u = 720.0 ksf

Nominal Geotechnical Unit Tip Resistance, $q_{p,intact}$ = 1,800.0 ksf (per LRFD Eq. 10.8.3.5.4c-1)

Assuming the bedrock below the base of the bedrock socket is jointed (up to 2B below the bottom of the socket), the nominal unit tip resistance is as follows:

Estimate Hoek-Brown strength parameters determined from Article 10.4.6.4

Geological strength index, GSI = 70.0 (per LRFD Figure 10.4.6.4-1; bedrock core generally described as slightly to moderately weathered, smooth to rough based on Phase I/II logs and core photographs; see Abutment sections)

Disturbance factor, D = 0.0 (per Hoek-Brown 2002; quality controlled socket, min. disturb.)

Constant m_i = 7.0 (per LRFD Table 10.4.6.4-1, fine siltstone)

Parameter, m_b = 2.4 (per LRFD Eq. 10.4.6.4-4)

Parameter, s = 0.04 (per LRFD Eq. 10.4.6.4-2)

Parameter, a = 0.50 (per LRFD Eq. 10.4.6.4-3)

Estimate the vertical effective stress at the socket bearing elevation (tip elevation)

Assume the following to calculate the vertical effective stress:

- Bottom of rock socket is 5 feet below top of bedrock elevation at both Abutments. The actual rock socket length will be determined from lateral pile evaluations and will be equal to or greater than the min. socket length required to provide fixity.
- Vertical effective stress includes the proposed fill height (from MSE wall) or MSE wall applied pressure.
- Engineering properties of existing fill and bedrock based on H&A PGDR report dated 8/28/20 and all lab test results.
- Groundwater is at El. 113 (based on test borings at Abutment No. 1)

Total unit weight of soil (fill, glacial till), γ_t = 125.0 pcf

Total unit weight of bedrock, $\gamma_{t,rock}$ = 168.0 pcf (average of lab test results)

Thickness of existing fill/till, $h_{fill/till}$ = 20.0 ft (avg. from top of rock to top of MSE wall)

Socket length/rock thickness, h_{rock} = 5.0 ft

Groundwater depth = 2.0 ft

Vertical effective stress at socket tip, σ'_{vb} = 3.2 ksf

Calculate A parameter

A parameter = 157.2 ksf (per LRFD Eq. 10.8.3.5.4c-3)

Calculate Nominal Geotechnical Unit Tip Resistance, $q_{p,jointed}$ = 695.2 ksf (per LRFD Eq. 10.8.3.5.4c-2)

Client:	Maine Department of Transportation
Project:	I-395/Route 9 Connector Bridge over Lambert Road - WIN 018915.00
Subject:	Geotechnical and Structural Resistance of Steel H-Piles

CALCULATE NOMINAL AND FACTORED AXIAL GEOTECHNICAL PILE RESISTANCES

Pile Section	Pile Tip Area ¹ (ft ²)	Nominal Geotechnical Unit Tip Resistance (ksf)		Nominal Axial Comp. Geotechnical Resistance (kips)		Factored Axial Geotechnical Resistance (kips)		
		Min. (Jtd. Rock)	Max. (Intact Rock)	Min. (Jtd. Rock)	Max. (Intact Rock)	Service Limit State ($\phi=1.0$)	Strength Limit State ($\phi=0.5$)	Extreme Limit State ($\phi=1.0$)
HP 12x74	1.03	695	1,800	713	1,845	713	356	713
HP 14x89	1.41	695	1,800	979	2,536	979	490	979
HP 14x117	1.47	695	1,800	1,022	2,645	1,022	511	1,022

Notes:

¹ - Pile tip area taken as the depth multiplied by the flange width of the section noted.

Client: Maine Department of Transportation

Project: I-395/Route 9 Connector Bridge over Lambert Road - WIN 018915.00

Computed by: NAS

Subject: Geotechnical and Structural Resistance of Steel H-Piles

Checked by: BCS

STRUCTURAL RESISTANCE ASSUMPTIONS

1. H-Piles are installed in bedrock sockets and the structural axial compression resistance of the pile is limited by the geotechnical axial resistance of the pile sections (see Sheet 2).
2. Since the H-piles won't be driven (installed in bedrock sockets instead), resistance factor is 0.6, due to "good" driving conditions for H-piles in compression (Section 6.5.4.2 AASHTO LRFD 2020)
3. No corrosion loss is assumed.
4. H-Piles are completely embedded in soil and rock and there is no reduction in axial resistance due to buckling.
5. This calculation only addresses axial resistance, if the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated.
6. The pile section as $F_y = 50$ ksi.
7. Equations used herien are from AASHTO LRFD 2020.

CALCULATIONS P_n shall be determined as follows:

- If $\frac{P_e}{P_o} \geq 0.44$, then:

$$P_n = \left[0.658 \left(\frac{P_o}{P_e} \right) \right] P_o \quad (6.9.4.1.1-1)$$

- If $\frac{P_e}{P_o} < 0.44$, then:

$$P_n = 0.877 P_e \quad (6.9.4.1.1-2)$$

where:

 A_g = gross cross-sectional area of the member (in.²) F_y = specified minimum yield strength (ksi) P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural-torsional buckling, as applicable (kips) P_o = equivalent nominal yield resistance = $Q F_y A_g$ (kips) Q = slender element reduction factor determined as specified in Article 6.9.4.2. Q shall be taken equal to 1.0 for bearing stiffeners.**6.9.4.1.2—Elastic Flexural Buckling Resistance**The elastic critical buckling resistance, P_e , based on flexural buckling shall be taken as:

$$P_e = \frac{\pi^2 E}{\left(\frac{K \ell}{r_s} \right)^2} A_g \quad (6.9.4.1.2-1)$$

where:

 A_g = gross cross-sectional area of the member (in.²) K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5 ℓ = unbraced length in the plane of buckling (in.) r_s = radius of gyration about the axis normal to the plane of buckling (in.)

Section	Corrosion (in)	Effective A_s (in)	f_y (ksi)	Q	$P_o = Q \cdot f_y \cdot A_s$ (kip)	P_e (kip)	P_o/P_e	P_n (kip)	ϕP_n (kip)
HP 12x74	0.000	21.8	50	1.0	1,090	∞	0	1,090	654
HP 14x89	0.000	25.9	50	1.0	1,295	∞	0	1,295	777
HP 14x117	0.000	34.4	50	1.0	1,722	∞	0	1,722	1,033

Notes:

1. $Q \cdot f_y \cdot A_s$ from Section 6.9.4.1.1 of AASHTO LRFD, $Q=1$ for nonslender elements. Pile is considered nonslender because the unbraced length is zero (i.e., completely embedded in soil or pile sleeve backfilled with stone).
2. P_e from Eq. 6.9.4.1.2-1 (Elastic Flexural Buckling Resistance). With unbraced length = 0, $P_e = \infty$.
3. P_n from Eq. 6.9.4.1.1-1.
4. Since the H-piles won't be driven (installed in bedrock sockets instead), resistance factor is 0.6, due to "good" driving conditions for H-piles in compression (Section 6.5.4.2 AASHTO LRFD 2020).

Client:	Maine Department of Transportation
Project:	I-395/Route 9 Connector Bridge over Lambert Road - WIN 018915.00
Subject:	Geotechnical and Structural Resistance of Steel H-Piles

SUMMARY AND RECOMMENDED FACTORED AXIAL COMPRESSIVE PILE RESISTANCES (STRENGTH LIMIT STATE)

Pile Section	Factored Axial Structural Resistance (kips)	Factored Axial Geotech. Resistance (kips)	Controlling Factored Axial Compressive Resistance (kips)
HP 12x74	654	356	356
HP 14x89	777	490	490
HP 14x117	1,033	511	511

Seismic Site Class

File No.	132076-007
Sheet	1 of 7
Date	12-Feb-21
Computed by	JAD
Checked by	BWC

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values and assumed S_u values from test borings drilled approximately near the proposed substructures.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the eight test borings near the proposed substructures (BB-ELAR-101, BB-ELAR-102 and BB-ELAR-201 through BB-ELAR-204), recommend a **Seismic Site Class C**.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Maine DOT Bridge Design Guide, August 2003

AVAILABLE INFORMATION

1. Boring logs dated 1 August 2018 drilled by Northern Test Borings, Inc.
2. Draft boring logs dated December 2020 and January 2021 drilled by New England Boring Contractors
3. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. WOH/WOR = SPT N-value of 1.
2. Used Method B and SPT N-values per boring logs.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.10.3.1-1), 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 .

File No.	132076-007
Sheet	2 of 7
Date	12-Feb-21
Computed by	JAD
Checked by	BWC

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

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 A/B/C will be used for the seismic Site Class evaluation.

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

SITE CLASS DEFINITIONS

(Table from AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5,000$ ft/s
B	Rock with $2,500$ ft/sec $< \bar{v}_s < 5,000$ ft/s
C	Very dense soil and soil rock with $1,200$ ft/sec $< \bar{v}_s < 2,500$ ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s $< \bar{v}_s < 1,200$ ft/s, or with either $15 < \bar{N} < 50$ blows/ft, or $1.0 < \bar{s}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s < 600$ ft/s or with either $\bar{N} < 15$ blows/ft or $\bar{s}_u < 1.0$ ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u < 0.5$ ksf
F	Soils requiring site-specific evaluations, such as: <ul style="list-style-type: none"> Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) Very high plasticity clays ($H > 25.0$ 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 classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

\bar{v}_s	=	average shear wave velocity for the upper 100 ft of the soil profile
\bar{N}	=	average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
\bar{s}_u	=	average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
PI	=	plasticity index (ASTM D4318)
w	=	moisture content (ASTM D2216)

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-ELAR-101
Ground Surface El.: 119.8

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0.8	119	SAND (Fill)	1.6	8	0.200
2D	3	116.8	SILT (Fill)	3.6	4	0.900
R1-R3	5.2	114.6	BEDROCK	94.8	100	0.948
Totals =				100.0		2.048

N-bar (blows/ft) = 49
Site Class = D

Exploration ID: BB-ELAR-102
Ground Surface El.: 123.6

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1.5	122.1	SAND (Fill)	2.5	24	0.104
2D	3.5	120.1	SILT (Fill)	2.0	12	0.167
3D	5.6	118	SAND (Fill)	2.1	50	0.042
R1-R3	6.6	117	BEDROCK	93.4	100	0.934
Totals =				100.0		1.247

N-bar (blows/ft) = 80
Site Class = C

CALCULATIONS

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Computed by	JAD
Checked by	

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-ELAR-201
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	SILT (Glacial Till)	2.0	1	2.000
2D	2	-2	SAND (Glacial Till)	0.9	1	0.900
-	2.9	-2.9	BEDROCK	97.1	100	0.971
Totals =				100.0		3.871

N-bar (blows/ft) = 26
Site Class = D

Exploration ID: BB-ELAR-201A
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	SILT (Glacial Till)	2.0	11	0.182
2D	2	-2	SILT (Glacial Till)	2.0	10	0.200
3D	4	-4	SILT (Glacial Till)	0.4	100	0.004
-	4.4	-4.4	BEDROCK	95.6	100	0.956
Totals =				100.0		1.342

N-bar (blows/ft) = 75
Site Class = C

**CALCULATIONS**

File No.	132076-007
Sheet	6 of 7
Date	12-Feb-21
Computed by	JAD
Checked by	BWC

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-ELAR-203
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	SILT	0.6	100	0.006
-	0.6	-0.6	BEDROCK	99.4	100	0.994

Totals = 100.0 1.000

N-bar (blows/ft) = 100
Site Class = C

Exploration ID: BB-ELAR-203A
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	SILT (Glacial Till)	2.0	3	0.667
2D	2	-2	SILT (Glacial Till)	2.0	42	0.048
3D	4	-4	SILT (Glacial Till)	1.4	100	0.014
-	5.4	-5.4	BEDROCK	94.6	100	0.946

Totals = 100.0 1.674

N-bar (blows/ft) = 60
Site Class = C

CALCULATIONS

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Date	12-Feb-21
Computed by	JAD
Checked by	BWC

Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road - WIN No. 18915.00
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-ELAR-202
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	SAND (Glacial Till)	2.0	11	0.182
2D	2	-2	SAND (Glacial Till)	2.0	8	0.250
3D	4	-4	SILT (Glacial Till)	4.2	26	0.162
-	8.2	-8.2	BEDROCK	91.8	100	0.918
Totals =				100.0		1.511

N-bar (blows/ft) = 66
Site Class = C

Exploration ID: BB-ELAR-204
Ground Surface El.:

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	0	CLAY	2.0	4	0.500
2D	2	-2	CLAY	1.5	15	0.100
3D	4	-4	SILT (Glacial Till)	4.3	40	0.108
-	5.8	-5.8	BEDROCK	92.2	100	0.922
Totals =				100.0		1.630

N-bar (blows/ft) = 61
Site Class = C

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Frost Evaluation

Client: Maine Department of Transportation

Project: I-395/Route 9 Connector Bridge over Lambert Road Bridge - WIN 18915.00

Subject: Frost Penetration Depth Evaluation

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. Haley & Aldrich Phase I and Phase II test boring logs.
3. Plan set titled "I-395/Route 9 Connector, Profile - Route 9 Over Lambert Road" prepared by MaineDOT dated 9 January 2020.

EVALUATION:

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

SUB-STRUCTURE	SUBSTRUCTURE BEARING ELEVATION	TEST BORING NO./GS EL.	GROUND WATER	SAMPLE NO. AND ELEVATION	LAB USCS	MOISTURE CONTENT
ABUTMENT NO. 1	El. 131	BB-ELAR-101 El. 119.8	113.1	No index laboratory testing available.		
		BB-ELAR-201 El. 116.0	114.7			
		BB-ELAR-201A El. 119.8	118.0			
		BB-ELAR-202 El. 122.5	118.7			
ABUTMENT NO. 2	El. 134	BB-ELAR-102 El. 123.6	118.8			
		BB-ELAR-203 El. 119.0	118.2			
		BB-ELAR-203A El. 121.5	119.1			
		BB-ELAR-204 El. 123.6	121.5			

2. Portions of the MSE wall footings will bear in new embankment fill. Assume the new embankment fill consists of granular borrow.

3. From MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1650 °F - days

4. Estimate range in frost penetration depth using MaineDOT Bridge Design Guide Table 5-1 and the design freezing index above.

5. For coarse grained soil with water contents ranging from 10 to 30 percent, from Table 5-1, maximum depth of frost penetration ranges from approximately 5 to 7 ft.

Recommend MSE wall footings be founded at least 6 ft below the lowest adjacent ground surface exposed to freezing.

Client: Maine Department of Transportation

Date: 19-May-2020

Project: I-395/Route 9 Connector Bridge over Lambert Road Bridge - WIN 18915.00

Computed by: EMS

Subject: Frost Penetration Depth Evaluation

Updated by: SSM

Checked by: BCS

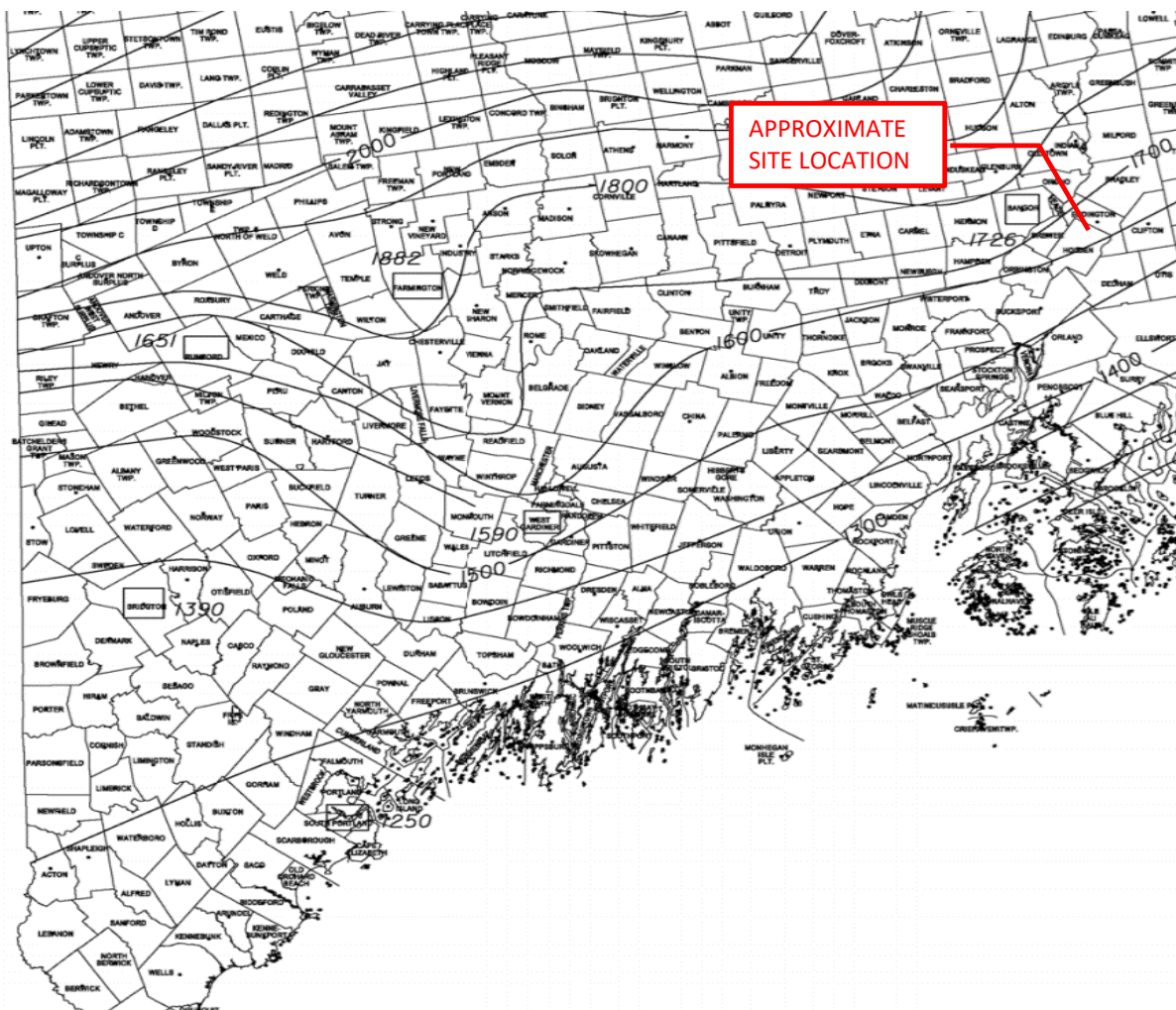


Table 5-1 Depth of Frost Penetration

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

Bearing Resistance

File No.	132076-007
Sheet	1 of 8
Date	29-Mar-21
Computed by	JAD
Updated by	SSM
Checked by	BCS

Client Maine Department of Transportation

Project I-395/Route 9 Connector Bridge over Lambert Road

Subject Bearing Resistance of Bedrock

PROBLEM STATEMENT & OBJECTIVE

Calculate the factored bearing resistance at the service, strength and extreme limit states for the proposed MSE Wall reinforced soil mass.

EXECUTIVE SUMMARY

A factored bearing resistance of	40	ksf for the strength limit state is recommended.
A factored bearing resistance of	20	ksf for the service limit state for 0.5 in. settlement is recommended.
A factored bearing resistance of	55	ksf for the extreme event limit state is recommended.

AVAILABLE INFORMATION

1. Phase I (-100 series) test boring logs dated 1 and 2 August 2018 drilled by Northern Test Borings, Inc.
2. Phase II (-200 series) test boring logs dated 5 through 10 November 2021 drilled by New England Boring Contractors.
3. Bottom of abutment elevations provided by MaineDOT on 15 March 2021 (see below).
4. Phase I (preliminary design) and Phase II (final design) laboratory test results.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. NCHRP Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, 2010.

ELEVATION DATUM

Elevations reference the North American Vertical Datum of 1988 (NAVD88).

ASSUMPTIONS

1. Bottom of footing will bear on bedrock at the following elevations:
2. The peak compressive strength of bedrock is based on laboratory test data (see page 4 for a summary of laboratory test results).

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Client	Maine Department of Transportation
Project	I-395/Route 9 Connector Bridge over Lambert Road
Subject	Bearing Resistance of Bedrock

PROCEDURE FOR STRENGTH LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

10.6.3.2 - Bearing Resistance of Rock

10.6.3.2.1 - General

The methods used for design of footings on rock shall consider the presence, orientation, and condition of discontinuities, weathering profile, and other similar profiles as they apply at a particular site. For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. For footings on less competent rock, more detailed investigations and analyses shall be performed to account for the effects of weathering and the presence and condition of discontinuities.

The designer shall judge the competency of a rock mass by taking into consideration both the nature of the intact rock, and the orientation and condition of the discontinuities of the overall rock mass. Where engineering judgment does not verify the presence of competent rock, the competency of the rock mass should be verified using the procedures for RMR rating.

10.6.3.2.2 Semiempirical Procedures

The nominal bearing resistance of rock should be determined using empirical correlation with the Geometrics Rock Mass Rating system. Local experience shall be considered in the use of these semi-empirical procedures. The factored bearing stress of the foundation shall not be taken to be greater than the factored compressive resistance of the footing concrete.

C10.6.3.2.2

The bearing resistance of jointed or broken rock may be estimated using the semi-empirical procedure developed by Carter and Kulhawy (1988). This procedure is based on the unconfined compressive strength of the intact rock core sample. Depending on the rock mass quality measured in terms of RMR system, the nominal bearing resistance of a rock mass varies from small fraction to six times the unconfined compressive strength of intact rock core samples.

2. See the nominal bearing resistance equation based on Carter and Kulhawy (1988) From NCHRP Report 651:

$$q_{ult} = q_u(\sqrt{s} + (m\sqrt{s} + s)^{0.5}) \quad \text{Equation 82b} \quad \text{An errata to Carter and Kulhawy 1988}$$

3. Determine the Rock Mass Ratio (RMR) and strength parameters s and m from NCHRP Report 651 to be used in Equation 82b:

RMR from Table 15 and Table 16

m and s from Hoek-Brown Failure Criterion

4. Apply resistance factor of 0.65 from Table 11.5.7-1 in AASHTO LRFD 2020 for bearing resistance of MSE Wall reinforced soil mass.

Client Maine Department of Transportation

Date 29-Mar-21

Project I-395/Route 9 Connector Bridge over Lambert Road

Computed by JAD

Subject Bearing Resistance of Bedrock

Updated by SSM

Checked by BCS

PROCEDURE FOR SERVICE LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

10.6.2.6 - Bearing Resistance at the Service Limit State

10.6.2.6.1 - Presumptive Values for Bearing Resistance

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

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

2. Use AASHTO LRFD 2017 presumptive bearing resistance for service limit state for settlement stated.

PROCEDURE FOR EXTREME EVENT LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2020:

11.5.8 - Resistance Factors for Extreme Event Limit state

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ , of 0.9 shall be used. For bearing resistance, a resistance factor of 0.9 for MSE Walls.

2. Use nominal resistance calculated for the Strength Limit State and apply a resistance factor of 0.9 from AASHTO LRFD 2020 Section 11.5.8 to obtain the factored resistance.

CALCULATIONS

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Client Maine Department of Transportation
 Project I-395/Route 9 Connector Bridge over Lambert Road
 Subject Bearing Resistance of Bedrock

AVAILABLE LABORATORY TEST DATA

Abutment No.	Test Boring No.	Ground Surface Elevation	Rock Core No.	Avg. Rock Specimen Depth BGS (ft)	Avg. Specimen Elevation	Depth of Specimen Below Ftg. Bearing Level (ft)	Peak Compressive Strength (psi)	Failure Type
1	ELAR-101	119.8	R3	8.7	111.1	--	9,747	intact
	ELAR-201A	119.8	R2	13.7	106.1	--	15,479	intact
	ELAR-202	122.5	R1	10.6	111.9	--	28,776	intact
2	ELAR-102	123.6	R2	11.2	112.4	--	12,789	intact
			R3	14.8	108.8	--	11,024	intact
	ELAR-203	119.0	R2	5.4	113.6	--	11,830	intact
	ELAR-203A	121.5	R3	15.2	106.3	--	21,335	intact

SUMMARY OF BEDROCK DATA AT SITE

Abutment No.	Test Boring No.	Ground Surface Elevation	Rock Core No.	Avg. Rock Core Depth (BGS)	Avg. Rock Core Elevation	Depth of Rock Core Below Ftg. Bearing Level (ft)	Rock Core Run Recovery (%)	Rock Quality Designation (RQD, %)
1	ELAR-101	119.8	R1	6.3	113.5	--	100	0
			R2	7.4	112.4	--	100	0
			R3	9.2	110.6	--	100	37
			R4	11.1	108.7	--	92	0
			R5	13.6	106.2	--	98	38
	ELAR-201	116.0	R1	5.9	110.1	--	95	95
	ELAR-201A	119.8	R1	7.2	112.6	--	95	95
			R2	12.2	107.6	--	100	90
2	ELAR-202	122.5	R1	11.0	111.5	--	--	--
			R2	14.0	108.5	--	--	--
			R3	14.5	109.1	--	98	41
	ELAR-102	123.6	R1	8.1	115.5	--	100	33
			R2	11.0	112.6	--	100	86
			R3	14.5	109.1	--	98	41
	ELAR-203	119.0	R1	2.7	116.3	--	94	76
			R2	5.3	113.7	--	89	81
			R3	15.0	106.5	--	94	94
	ELAR-203A	121.5	R1	8.5	113.0	--	80	60
	ELAR-204	123.6	R2	12.0	109.5	--	--	--
			R3	15.0	106.5	--	94	94

PARAMETERS FOR CALCULATIONS

Average RQD from borings at proposed bridge abutments: 56 %
 Average RQD from Abutment 1 test borings: 44 %
 Average RQD from Abutment 2 test borings: 68 %

Average peak compressive strength at proposed bridge abutments: 15,854 psi
 Average peak compressive strength at Abutment 1: 18,001 psi
 Average peak compressive strength at Abutment 2: 14,245 psi

For conservatism use lowest average RQD and lowest average unconfined compressive strength to calculate one bearing resistance: 44 %
 14,245 psi
 2,051 ksf

Client Maine Department of Transportation

Project I-395/Route 9 Connector Bridge over Lambert Road

Subject Bearing Resistance of Bedrock

Strength Limit State

Determine RMR

Table 15 from NCHRP Report 651:

PARAMETER			RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, unconfined compressive test is preferred		
		Unconfined compressive strength	>4,320 ksf	2,160–4,320 ksf	1,080–2,160 ksf	520–1,080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating	15	12	7	4	2	1	0	
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%		<25%	
	Relative Rating		20	17	13	8		3	
3	Spacing of joints		>10 ft	3–10 ft	1–3 ft	2 in–1 ft		<2 in	
	Relative Rating		30	25	20	10		5	
4	Condition of joints		<ul style="list-style-type: none">• Very rough surfaces• Not continuous• No separation• Hard joint wall rock	<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in• Hard joint wall rock	<ul style="list-style-type: none">• Slightly rough surfaces• Separation <0.05 in• Soft joint wall rock	<ul style="list-style-type: none">• Slitten-sided surfaces or• Gouge <0.2 in thick or• Joints open 0.05–0.2 in• Continuous joints	<ul style="list-style-type: none">• Soft gouge >0.2 in thick or• Joints open >0.2 in• Continuous joints		
	Relative Rating		25	20	12	6		0	
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal/hr	400–2,000 gal/hr		>2,000 gal/hr		
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5		>0.5		
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure		Severe water problems		
	Relative Rating		10	7	4		0		

Table 16 from NCHRP Report 651:

Strike and dip orientations of joints		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Total RMR Value

Parameter	Design Value	Value Based on Table 15 (above)	Relative Rating
Intact Rock Strength	2,051 ksf	1080 - 2160 ksf	7
RQD	44%	25% to 50%	8
Joint Spacing	2 in to 1 ft (observed in photos)	2 in. to 1 ft	10
Joint Condition	Slightly rough surfaces separation <0.05 in (observed in photos)	Slightly rough surfaces Separation <0.05 in soft joint wall rock	12
Groundwater Condition	Moist only (interstitial water)	Moist only (interstitial water)	7
Joint Strike and Dip	Fair	Fair	-7
Total Rating =			37

Client Maine Department of Transportation
 Project I-395/Route 9 Connector Bridge over Lambert Road
 Subject Bearing Resistance of Bedrock

Strength Limit State Continued

Determine s and m
 Assume the rock type B

Table 17 from NCHRP Report 651:

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

Table 19 from NCHRP Report 651:

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

Values of m and s from Hoek-Brown 1988:

$$\frac{m}{m_1} = e^{\left(\frac{RMR-100}{14}\right)} \quad \text{Equation 18}$$

m₁ is the value of m for *intact* rock

$$s = e^{\left(\frac{RMR-100}{6}\right)} \quad \text{Equation 19}$$

Rock Quality	Rock Type	RMR	m ₁	m	s
Poor Rock	B	37	10.00	1.11E-01	2.75E-05

Client Maine Department of Transportation
 Project I-395/Route 9 Connector Bridge over Lambert Road
 Subject Bearing Resistance of Bedrock

Strength Limit State Continued

Semi-empirical method by Carter and Kulhawy 1988:

$$\begin{aligned}
 q_u &= 14,245 \text{ psi} \\
 m &= 0.111 \\
 s &= 2.75E-05 \\
 q_{ult} &= 61.4 \text{ ksf} && \text{Equation 82b} \\
 \phi &= 0.65 && \text{from Table 10.5.5.2.2-1} \\
 q_R &= 40 \text{ ksf} && \text{Equation 82b}
 \end{aligned}$$

Service Limit State

Based on Table C10.6.2.6.1-1 the service limit state for bearing resistance on weathered or broken rock is recommended at 20 ksf for settlements of 1 in.

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

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

Extreme Event Limit State

From the Strength Limit State calculations, the nominal bearing resistance is the following:

$$q_{ult} = 61.4 \text{ ksf}$$

Using a resistance factor of 0.9 from Section 11.5.8, the factored bearing resistance is the following:

$$q_R = 55 \text{ ksf}$$



CALCULATIONS

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Date	29-Mar-21
Computed by	JAD
Updated by	SSM
Checked by	BCS

Client Maine Department of Transportation

Project I-395/Route 9 Connector Bridge over Lambert Road

Subject Bearing Resistance of Bedrock

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The recommended factored bearing resistance for the strength limit state is 40 ksf

Service Limit State

The recommended presumptive value for weathered bedrock is 20 ksf for the service limit state for a settlement up to 0.5 in.

Extreme Event Limit State

The recommended factored bearing resistance for the extreme event limit state is 55 ksf

Client MAINEDOT

Project I-395/ROUTE 9 CONNECTOR OVER LAMBERT ROAD

Subject MSE WALL BEARING RESISTANCE

File No. 132076-007
 Sheet 1 of 3
 Date 5/28/21
 Computed By BCS
 Checked By EAF

OBJECTIVE: CALCULATE NOMINAL AND FACTORED BEARING RESISTANCES FOR USE IN VENDOR DESIGNED MSE WALLS.

REFERENCES:

1. AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 9TH ED., 2020.
2. MAINEDOT BRIDGE DESIGN GUIDE, AUGUST 2003 WITH INTERIM REV'S. THROUGH JUNE 2018.
3. MAINEDOT STANDARD SPECIFICATION SECTION 677-MECHANICALLY STABILIZED EARTH RETAINING WALL.

ASSUMPTIONS:

1. PORTIONS OF MSE WALL WILL BEAR ON GRANULAR BORROW. PER MAINEDOT BDG SECTION 3.6.1, $\gamma_f = 125 \text{ pcf}$ AND $\phi = 32^\circ$. BECAUSE STANDARD SPECIFICATION SECTION 703.19 ALLOWS UP TO 20 PERCENT PASSING THE NO. 200 SIEVE, USE MORE CONSERVATIVE SOIL PROPERTIES TO ACCOUNT FOR POTENTIAL MATERIAL VARIABILITY. ASSUME $\gamma_f = 120 \text{ pcf}$ AND $\phi = 30^\circ$.
2. BOTTOM OF CONCRETE LEVELLING PAD IS 6 FT (MIN.) BELOW GROUND SURFACE FOR FROST PROTECTION ($D_f = 6 \text{ FT}$).
3. GROUNDWATER LEVEL IS AT THE GROUND SURFACE ($D_w = 0 \text{ FT}$); CONSERVATIVE.
4. LOAD INCLINATION FACTORS (i_c, i_g, i_y) TAKEN AS 1.0 DUE TO MODEST EMBEDMENT (AASHTO LRFD C10.6.3.1.2a).
5. DEPTH CORRECTION FACTOR (d_g) TAKEN AS 1.0; CONSERVATIVE.
6. RESISTANCE FACTORS:

SERVICE LIMIT STATE = 1.0 (AASHTO LRFD SECTION 11.5.7)

STRENGTH LIMIT STATE = 0.65 (AASHTO LRFD TABLE 11.5.7-1)

EXTREME EVENT LIMIT STATE = 1.0 (AASHTO LRFD SECTION 11.5.8)

EVALUATION:

$$Q_n = C N_{cm} + \gamma D_f N_{gm} C_{wg} + 0.5 \gamma B N_{gm} C_{wy} \quad (\text{AASHTO LRFD EQN. 10.6.3.1.2a-1})$$

ASSUME GRANULAR BORROW BEARING MATERIAL IS COHESIONLESS, $C = 0 \text{ PSF}$.

$$Q_n = \gamma D_f N_{gm} C_{wg} + 0.5 \gamma B N_{gm} C_{wy}$$

Client MAINEDOT
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EVALUATION: (CONT.)

DETERMINE BEARING CAPACITY FACTORS FROM AASHTO LRFD TABLE 10.6.3.1.2a-1....

FOR GRANULAR BORROW WITH $\phi = 30^\circ$: $N_g = 18.4$ AND $N_\gamma = 22.4$

DETERMINE FOOTING SHAPE CORRECTION FACTORS FROM AASHTO LRFD TABLE 10.6.3.1.2a-3

FOR GRANULAR BORROW WITH $\phi = 30^\circ$: $s_g = 1 - 0.4(B/L)$ AND $s_\gamma = 1 + (B/L \tan \phi)$

RECALL DEPTH CORRECTION AND LOAD INCLINATION FACTORS TAKEN AS 1.0

DETERMINE FACTORS N_{gm} AND $N_{\gamma m}$

$$N_{gm} = N_g s_g d_g i_g \quad (\text{AASHTO LRFD EQN. 10.6.3.1.2a-3})$$

$$= (18.4) (1 + (B/L \tan 30^\circ)) (1.0) (1.0)$$

$$= 18.4 + 10.6 (B/L)$$

$$N_{\gamma m} = N_\gamma s_\gamma i_\gamma \quad (\text{AASHTO LRFD EQN. 10.6.3.1.2a-4})$$

$$= (22.4) (1 - 0.4(B/L)) (1.0)$$

$$= 22.4 - 9.0 (B/L)$$

BECAUSE GROUNDWATER ASSUMED $\textcircled{3}$ GROUND SURFACE ($D_w = 0$), GROUNDWATER CORRECTION FACTORS (C_{wq} AND $C_{w\gamma}$) TAKEN AS 0.5 (AASHTO LRFD TABLE 10.6.3.1.2a-2)

SUBSTITUTING THE ABOVE INFORMATION INTO AASHTO LRFD EQN. 10.6.3.1.2a-1 YIELDS THE FOLLOWING:

$$q_n = (120 \text{ PCF})(6 \text{ FT})(18.4 + 10.6(B/L))(0.5) + (0.5)(120 \text{ PCF})(B) [22.4 - 9.0(B/L)] (0.5)$$

$$= 6,624 \text{ PSF} + 3,816 \text{ PSF} (B/L) + 672 \text{ PCF} (B) - 270 \text{ PCF} (B^2/L)$$

IN ACCORDANCE WITH AASHTO LRFD SECTION 10.6.3.1.1 EFFECTIVE FOOTING DIMENSIONS $B' = B - 2e_B$ AND $L' = L - 2e_L$ (AASHTO LRFD SECTION 10.6.1.3) SHALL BE USED IN LIEU OF OVERALL DIMENSIONS B AND L WHERE LOADS ARE ECCENTRIC.

SUBSTITUTE THE ABOVE INFORMATION INTO AASHTO LRFD EQN. 10.6.3.1.2a-1....

IN ACCORDANCE WITH AASHTO LRFD SECTION 10.6.3.1.1 EFFECTIVE FOOTING DIMENSIONS $B' = B - 2e_B$ AND $L' = L - 2e_L$ (AASHTO LRFD SECTION 10.6.1.3) SHALL BE USED IN LIEU OF OVERALL DIMENSIONS B AND L WHERE LOADS ARE ECCENTRIC.

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EVALUATION : (CONT.)

$$q_n = 6624 \text{ psf} + 3,816 \text{ psf} \left(\frac{B'}{L'} \right) + 672 \text{ pcf} (B) - 270 \text{ pcf} \left(\frac{B'^2}{L'} \right)$$

WHERE B' AND L' ARE EQUAL TO THE EFFECTIVE REINFORCEMENT AND EFFECTIVE MSE WALL LENGTHS, RESPECTIVELY.

- USE THE ABOVE EQUATION TO CALCULATE NOMINAL BEARING RESISTANCE FOR AN MSE WALL BEARING ON GRANULAR BORROW EMBANKMENT FILL.

FACTORED BEARING RESISTANCES MAY BE CALCULATED USING THE AASHTO LRFD SPECIFIED RESISTANCE FACTORS ON SHEET 1 OF THIS CALCULATION PACKAGE.

NOTE THAT THE EGN. ABOVE DOES NOT ACCOUNT FOR WALLS BEARING ON OR NEAR SLOPES. BECAUSE OF THIS, THE FORMULA MAY NEED TO BE REVISED ONCE GRADING IN FRONT OF THE WALLS IS FINALIZED AS WELL AS THE WALL BEARING ELEVATIONS.