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GEOTECHNICAL DESIGN REPORT
EASTERN AVENUE BRIDGE OVER
INTERSTATE 395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Maine Department of Transportation
Augusta, Maine

File No. 132076-007
June 2021

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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Geotechnical Design Report
Eastern Avenue Bridge over Interstate 395/Route 9 Connector
MaineDOT WIN 018915.00
Brewer, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Geotechnical Design Report, Eastern Avenue over Interstate 395/Route 9 Connector, MaineDOT WIN 018915.00, Brewer, 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 and 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 eastbound (EB) and westbound (WB) vehicular traffic on Eastern Avenue over the proposed Interstate 395/Route 9 Connector (Connector) in Brewer, Maine (see Figure 1, Project Locus).

Please note that geotechnical design recommendations and construction considerations for the Connector will be provided under separate cover at a later date.

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:

- Eastern Avenue: Sta. 3+00 to Sta. 10+00
- Interstate 395/Route 9 Connector: Sta. 107+00 to Sta. 110+00

PROJECT LOCATION AND EXISTING SITE CONDITIONS

The proposed Eastern Avenue bridge will carry EB and WB vehicular traffic over the north-south oriented Connector in Brewer, Maine. The project site consists of an open field with light vegetation south of Eastern Avenue and a densely wooded area north of Eastern Avenue. Existing site grades along Eastern Avenue are relatively flat, ranging between approximately El. 138 and El. 140 between Sta. 3+00 and Sta. 10+00. Existing grades along the proposed Connector generally slope up from approximately El. 135 at Sta. 107+00 (south) to El. 144 at Sta. 110+00 (north).

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 an 88-ft long, single-span bridge that is supported on two cast-in-place (CIP) concrete, full-height, cantilever abutments at the stations and elevations summarized below.

Substructure	Station at Centerline of Eastern Avenue Alignment (ft)	Proposed Footing Bearing Elevation (ft, NAVD 88)
Abutment 1	Sta. 5+56	El. 111 to El. 113
Abutment 2	Sta. 6+44	El. 116.9

The bridge superstructures will be constructed using metalized steel plate girders (five beam lines) running parallel to the long dimension of the bridge, with a 9-in. thick CIP concrete deck. The bridge structure will be approximately 28-ft wide (shoulder-to-shoulder) and will consist of two, 11-ft wide travel lanes and two, 4-ft wide shoulders. It is our understanding that the bridge will be designed as a semi-integral structure.

Based on our review of profile and cross section drawings developed by MaineDOT for the recommended bridge and Connector alternatives, we anticipate that the existing site grades along Eastern Avenue will be raised by approximately 1 to 2 ft to meet proposed finish grades. Construction of the Connector in the immediate vicinity of Eastern Avenue will require approximately 20 to 30 ft of excavation to meet proposed finish grades.

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 mapped in the vicinity of Eastern Avenue consist 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 MGS's Bedrock Geology Map of the Veazie Quadrangle (2011), bedrock within 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 gray to gray-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 completed at the site generally consisted of Siltstone with moderate to steeply dipping beds.

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-BEA-101 and BB-BEA-102, which were drilled in the vicinity of the proposed bridge abutments.

The test boring locations were laid out in the field by Haley & Aldrich using global positioning system (GPS) survey equipment prior to the start of drilling. "As-drilled" test boring locations and ground surface elevations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The Phase I test boring locations and ground surface elevations 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 32 to 35 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 existing 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 to 11 ft into bedrock using a 2.0-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 laboratory facility in Portland, Maine.

One observation well was installed in a borehole on the south side of Eastern Avenue to provide information on the static groundwater levels at the site. The observation well consisted of 2-in. ID, machine-slotted PVC pipe and solid PVC riser pipe extending to approximately 2 ft above existing ground surface. The observation well was outfitted with a steel guard pipe and steel lock/cap assembly. The observation well installation and groundwater monitoring reports are provided in Appendix B.

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 November 2020. The Phase II subsurface investigation consisted of four bridge test borings, designated BB-BEA-201 through BB-BEA-204, and two highway test borings, designated HB-BE-217 and HB-BE-218, which were drilled at/near the ends of the abutment wingwalls (bridge test borings) and along the Connector, south and north of Eastern Avenue (highway test borings).

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 were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The Phase II test boring locations and ground surface elevations are summarized in Table I and are shown on Figure 2.

The Phase II test borings were drilled by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile Drill B-53 truck or track-mounted drill rig. Test borings were advanced to depths ranging from approximately 30 to 35 ft BGS using similar means and methods to those used to drill the Phase I test borings. The hammer efficiency factors for the automatic hammers used were either 0.852 or 0.867 (85.2 or 86.7 percent theoretical hammer efficiency) as shown on the test boring logs.

Test borings were advanced approximately 4 to 13 ft into bedrock using a 2.0-in. (NQ-size) ID, diamond-tipped core barrel.

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

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. A general description of each soil/bedrock unit is provided separately, below. 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 bridge alignment.

Geologic Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Bituminous Concrete	0.4 to 0.5	A surficial layer of bituminous concrete was encountered in test borings completed within the limits of Eastern Avenue (*encountered in test borings BB-BEA-101 and -102)*
Fill	0 to 6	Medium dense to very dense SAND with varying amounts of silt and gravel (SM, SW); very stiff SILT with varying amounts of sand and gravel (ML) *(encountered in test borings BB-BEA-101, -102 and -201)*
Marine Deposit	4	Soft SILT (ML) *(encountered in test boring BB-BEA-202)*
Glacial Till	15 to 25	Very dense GRAVEL with varying amounts of sand and silt (GP, GM); loose to very dense SAND with varying amounts of silt and gravel (SM, SP); soft to hard SILT with varying amounts of sand and gravel (ML) *(encountered in each test boring)*
Bedrock	Top of bedrock surface encountered at depths ranging from approximately 18 to 31 ft BGS (El. 106 to El. 118) and slopes up from south to north and from west to east. *(encountered in each test boring)*	

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

As stated previously, approximately 4 to 13 ft of bedrock was cored in the test borings. The sampled and recovered bedrock generally consisted of the following:

- Moderately hard to hard, fresh to highly weathered, grey, aphanitic, SILTSTONE to METASILTSTONE. Primary joints dipping at low to steep angles and are very close to closely spaced, tight to open, smooth to rough.
- An approximate 6-ft thick layer of weathered bedrock, consisting of very dense silty GRAVEL with varying amounts of sand was encountered in test boring BB-BEA-201.

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 92 percent (average = 50 percent), indicating variable rock quality; from very poor to excellent.

Detailed bedrock core data and descriptions are provided on Table III and on the logs in Appendix A. In addition, photographs of the recovered bedrock core samples are provided for reference in Appendix A.

GROUNDWATER CONDITIONS

As discussed previously, an observation well was installed in a completed borehole on the south side of Eastern Avenue to provide information on the static groundwater levels at the site. The measured water levels during the period 6 August 2018 to 24 March 2021 ranged from approximately 2 to 15 ft BGS (El. 124 to El. 137). Please note that the invert of the proposed ditches adjacent to the new Connector are planned to be constructed at approximately El. 113 and the finished Connector roadway grade is planned to be approximately El. 117.

In general, water levels may fluctuate with season, precipitation, local soil/bedrock conditions, and excavation means and methods. Therefore, water levels may vary from those summarized above, provided on the testing boring logs included in Appendix A and shown on the groundwater monitoring report included in Appendix B.

Geotechnical Laboratory Testing Programs

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. (GTX) 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]
Moisture Content	ASTM D2216	Glacial Till	5	WC = 7.6 to 11.3%
Grain Size	ASTM D422	Glacial Till	13	AASHTO Classification: A-2-4, A-4, A-1-b USCS Classification: SM, ML
Direct Shear	ASTM D3080	Glacial Till	1	Cohesion = 803 psf Friction Angle = 32.7°
Compressive Strength and Elastic Moduli of Rock	ASTM D7012	Bedrock	8	Peak Compressive Stress: 3,818 to 20,742 psi Young's Modulus: 1,250,000 to 58,200,000 psi Poisson's Ratio: 0.05 to 0.41

[1] WC = Moisture Content; psi = pounds per square in.; psf = pounds per square foot

All laboratory test results are shown on the test boring logs included in Appendix A and complete results are provided in Appendix C.

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,
- 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 report are provided for reference in Appendix D.

APPROACH EMBANKMENTS

As stated above, existing site grades along Eastern Avenue will be raised by approximately 1 to 2 ft to meet proposed finish grades. Because of the limited amount of raise in grade along Eastern Avenue and based on the subsurface conditions encountered in the Phase I and Phase II test borings drilled at the site, we anticipate that post-construction settlement of the new approach roadways will be negligible.

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 in excess of 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 (approximately 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 =	1.200
Site factor for long-period range of acceleration response spectrum, F_v =	1.700
Site factor at zero-period on acceleration response spectrum, F_{pga} =	1.200
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, S_S (g) =	0.143
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) =	0.066
Horizontal response spectral acceleration coeff. at 0.2-s period modified by F_a, S_{DS} (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}, As (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).

Based on our review of the soil conditions encountered in the 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 very low.

BRIDGE ABUTMENT AND WINGWALL FOUNDATION SUPPORT

As shown on Figure 3 and as discussed herein, the subsurface conditions present at the site generally consist of naturally-deposited glacial till overlying bedrock and the proposed Connector will require excavation of approximately 20 to 30 ft below Eastern Avenue to construct. Based on our discussions with you, it is our understanding that mass concrete spread footings bearing on intact bedrock is the preferred alternative to support the proposed bridge abutments and wingwalls. Current proposed bottom of footing elevations range from El. 111 to El. 113 for the Abutment 1 breastwall and wingwalls to El. 116.9 for the Abutment 2 breastwall and wingwalls, which are generally coincident with or slightly below the top of bedrock surface encountered in the subsurface explorations completed at the site. Additional foundation recommendations are summarized below. Please note that the design recommendations presented below assume foundation subgrade preparation is completed in accordance with the guidance provided in the Construction Considerations section of this report as well as the requirements of the Contract Documents (CDs; drawings, standard specifications and special provisions).

- Bearing Resistance:
  - For the service limit state, mass concrete footings should be designed such that footing contact pressures do not exceed 20 kips per square foot (ksf). At this pressure, it is estimated that elastic settlement of footings bearing on “fair to very good” bedrock will generally be less than ½ in. per LRFD Section 10.6.2.4.4.
  - For the strength limit state, footings should be designed for a factored bearing resistance of 52 ksf, using a resistance factor of 0.45.
  - For the extreme event limit state, footings should be designed for a factored bearing resistance of 93 ksf, using a resistance factor of 0.8.

- Load Distribution and Eccentricity:
  - Application of permanent and transient loads is specified in AASHTO LRFD Section 11.5.6. We recommend the stress distribution at the base of the footings be assumed to be a triangular or trapezoidal distribution over the effective footing base as shown in AASHTO LRFD Figure 11.6.3.2-2.
  - The eccentricity of loading at the Strength Limit State, based on factored loads, should not exceed 0.45 of the spread footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the base width and length.

- Sliding Resistance:
  - In accordance with AASHTO LRFD Tables C3.11.5.3-1 and 10.5.5.2.2-1, we recommend that sliding resistance of abutment and wingwall footings be calculated using the design parameters presented below.

Subgrade Saturation Condition During Construction	**Coefficient of Friction (tan δ)**	**Interface Friction Angle (δ, deg.)**	**Strength Limit State Resistance Factor for Sliding (φ_τ)**	**Service/Extreme Limit State Resistance Factor for Sliding (φ_τ)**
Prepared in-the-dry	0.7	35	0.8	1.0

  - Lateral passive soil resistance in front of the footings, if present, should be neglected in accordance with requirements of the BDG. “Anchorage” of footings to bedrock (e.g., rock dowels) may be required to provide additional sliding resistance. If additional lateral load resistance is needed between the footings and bedrock, as determined by the bridge designer, we will provide additional geotechnical recommendations for rock dowels.

ABUTMENT AND WINGWALL DESIGN

- Drainage:
  - The abutment and wingwall design should include a drainage system to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutment and wingwall spread footings. We recommend that drainage be provided in accordance with BDG Section 5.4.2.13.

- Lateral Earth Pressures:
  - Recommendations summarized in the table below are based on the following:
    - Abutments and wingwalls are backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3; total unit weight = 125 pcf; internal angle of friction = 32 degrees).
    - The abutment and wingwall backwalls are vertical.
    - Adequate drainage is provided, as recommended herein and in accordance with the requirements of the BDG, to eliminate the potential for unbalanced hydrostatic pressures to develop.
    - A 0.6 degree backfill surface at Abutment 1 breastwall and a -0.6 degree backfill surface at Abutment 2 breastwall (based on 1 percent grade of proposed vertical profile of Eastern Avenue).
    - A 19 degree backfill surface at Abutment 1 and Abutment 2 wingwalls (maximum slope angle indicated by MaineDOT).

Substructure	Active Lateral Earth Pressure Coefficient (K_a, dim.)		At-Rest Lateral Earth Pressure Coefficient (K_o)	Passive Lateral Earth Pressure Coefficients (K_p, dim.)	
	Rankine	Coulomb		Rankine	Coulomb
Abutment 1 Breastwall	0.31	0.28	0.47	3.25	8.70
Abutment 2 Breastwall		0.27			8.07
Abutment 1 and 2 Wingwalls	0.37			NA	

  - The Coulomb active earth pressure coefficients apply to wall designs that are "gravity-shaped" or short-heeled, cantilever-types where the top of the stem wall interferes with the shear zone. For long-heeled cantilever-type walls we recommend the use of Rankine active earth pressure coefficients. In addition, alternative Rankine active earth pressure coefficients, based on a range of backfill surface inclinations, for use in designing the wingwalls are provided in the calculations in Appendix D.

- In accordance with BDG Section 5.4.3, semi-integral abutments should be designed for Rankine active earth pressures over the rigid abutment height and a uniform pressure distribution due to the height of soil behind the superstructure/end diaphragm. We recommend that the superstructure backwall (end diaphragm) be designed for full passive pressure only.
- Additional lateral earth pressures due to live load surcharge are required in accordance with 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, as presented to BDG Table 3-4.
- If determined applicable by MaineDOT, the abutment breastwalls and wingwalls should be designed for a uniform lateral load to account for seismic soil loading in accordance with LRFD Section A.11.3.1 (Mononobe-Okabe Method). Based on the seismic site class (Site Class “C”), we recommend a seismic active earth pressure coefficient, K_{AE}, of 0.33 be used for design of Abutment 1 and 0.32 be used for design of Abutment 2. In addition, a seismic active earth pressure coefficient equal to 0.47, which is based on a 19 degree backfill slope inclination (maximum anticipated by MaineDOT), should be used for the design of the abutment wingwalls. Please note that the soil pressure calculated using K_{AE} includes both the static and seismic lateral earth loads.

FROST PROTECTION

The abutment and wingwall spread footings will bear directly on bedrock. It is our opinion that the potential for frost-induced heave for foundations bearing on bedrock is low and therefore, a minimum footing embedment depth requirement is not considered necessary.

Construction Considerations

The purpose of this section of the report is to provide comments and recommendations on items related to excavation, earthwork, and other geotechnical aspects of the proposed construction. Since it identifies potential construction issues related to foundations and earthwork, it will aid personnel who monitor the construction activities. Prospective Contractors for this project should evaluate construction issues based on their own knowledge and experience in the Brewer, Maine, area taking into consideration their proposed construction means, methods and procedures.

EXCAVATION

Soil

As stated above, construction of the bridge and the Connector will require approximately 20 to 30 ft of excavation to meet proposed finish grades.

We anticipate that excavation of the in-situ fill, marine deposits and glacial till can be accomplished using normal earth-excavating equipment (i.e., hydraulic backhoes and excavators). In our opinion, temporary cut slopes in glacial till should, typically, be stable if constructed no steeper than about 1.5H:1V. Some sloughing and raveling should be anticipated in all temporary earth slopes. All temporary excavations should be made in accordance with all OSHA and other applicable regulatory agency requirements. The Contractor should be responsible for the design, stability, and safety of all temporary excavations.

As noted on the test boring logs, the naturally-deposited glacial till soils may contain cobbles and possibly some large boulders. We recommend that the Contract Documents require the Contractor to include provisions for cobble/boulder removal in their bid.

Bedrock

Based on the elevation of the top of bedrock surface at test boring locations and the proposed footing elevations presented herein, we anticipate that up to approximately 4 ft of bedrock will need to be removed to construct the abutment footings.

It is our opinion that because of the limited depth to which bedrock needs to be removed and our experience with similar bedrock types, the most practical method of bedrock excavation is using conventional earth moving equipment, such as a large excavator outfitted with a hoe-ram. If the actual top of bedrock surface level is higher than those indicated on the test boring logs, removal of rock using mechanical methods (i.e., hoe-ramming) alone may be difficult and an alternative method of rock removal may be needed (e.g., controlled blasting).

Based on the proposed cut slope angles currently shown on the plans, we do not anticipate that perimeter control methods such as pre-splitting (line drilling) will be needed to minimize over-break.

CONSTRUCTION DEWATERING

Groundwater was measured in the observation well at depths ranging from approximately 2 to 15 ft BGS (El. 124 to El. 137) during the period 6 August 2018 to 24 March 2021. Based on the measured water levels and considering that the proposed bottom of abutment and wingwall footings vary between El. 111 and El. 117, we anticipate groundwater will be encountered in excavations during construction since the bottom of excavation will be approximately 7 to 26 ft below the measured water levels depending on the time of year that excavation is performed. We anticipate that temporary dewatering will be required and could likely be accomplished by passively pumping from open sumps and temporary ditches located at the base of the excavations. Sumps should be provided with filters suitable to prevent pumping of fine-grained soil particles.

The Contractor should be responsible for controlling all surface runoff, infiltration and water from other sources at all times during excavation. Rainwater or snowmelt should be directed away from exposed foundation bearing surfaces. Dewatering should be performed as required to maintain the undisturbed nature of soil surfaces and enable all final excavation, foundation construction and backfilling to be completed "in-the-dry."

Dewatering should be performed in accordance with all applicable regulations. Dewatering effluent should be treated as required by applicable state and local regulations.

BEDROCK SUBGRADE PREPARATION

The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the abutments and wingwalls are completed. Construction activities that disturb the bedrock below the abutment footings should not be permitted. We recommend that the bedrock surface be cleared of all loose, fractured or weathered/decomposed bedrock and soil (i.e., "unsuitable material") prior to concrete placement. If localized areas of "unsuitable material" are found to extend below the bearing level of the footings, we recommend that Class S concrete be placed from the bottom of excavation up to the proposed footing bearing level, after the unsuitable material has been removed and the surface has been examined in the field by the Resident and/or project Geotechnical Engineer, as discussed below. In cases where more significant excavation of "unsuitable material" is required we recommend that Class A concrete be placed from the bottom of the excavation up to the proposed footing bearing level.

- Foundation bearing areas should be level. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. Smooth bedrock, if present, should be roughened or serrated prior to placing concrete to enhance sliding stability.
- In-the-dry or underwater excavation of steeply sloping and/or loose, fractured bedrock may be done using conventional excavation methods. Based on the proposed bearing level of the abutment and wingwall footings and the top of bedrock surface encountered in the test borings we do not anticipate the need for bedrock removal using controlled blasting techniques.
- Prior to placing concrete for the sub-footings (if over-excavation is required) or abutment and wingwall footings we recommend that the exposed bedrock surface be washed with high pressure water and air to remove loose, fractured and/or decomposed rock fragments and other debris. We also recommend that the prepared bedrock surface be approved by the Resident and/or project Geotechnical Engineer prior to concrete placement.
- It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps so that subgrade preparation and foundation construction is completed in-the-dry. We recommend that dewatering effluent be managed in accordance with all local, state and/or federal regulations.

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 will 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 CDs and that the information contained in the CDs takes precedence over the information provided in this report.

Please note that geotechnical design recommendations and construction considerations for the Connector will be provided under separate cover at a later date.

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.

Shahad Mattloob
Staff Geotechnical Engineer

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

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

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 – Observation Well Installation and Groundwater Monitoring Reports
- Appendix C – Laboratory Test Results
- Appendix D –Geotechnical Design Calculations

TABLE I
Subsurface Exploration Location Data
Eastern Avenue Bridge over Interstate 395/Route 9 Connector
MaineDOT WIN 018915.00
Brewer, 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-BEA-101	139.5	107+93.1	38.9 LT	467,986	1,748,947
BB-BEA-102	139.3	107+81.2	38.4 RT	467,935	1,749,007
BB-BEA-201	136.5	107+43.4	67.1 LT	467,958	1,748,897
BB-BEA-202	137.1	108+28.1	70.1 LT	468,032	1,748,939
BB-BEA-203	135.7	107+47.9	57.4 RT	467,897	1,749,006
BB-BEA-204	138.7	108+23.8	63.5 RT	467,959	1,749,051
HB-BE-217	135.1	106+96.5	2.9 LT	467,884	1,748,928
HB-BE-218	138.7	109+01.6	4.9 LT	468,061	1,749,032

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
Prepared By:	SSM	2/8/2021
Checked By:	BCS	3/9/2021
Reviewed By:	WAC	4/30/2021

TABLE II
Subsurface Exploration Subsurface Data
Eastern Avenue Bridge over Interstate 395/Route 9 Connector
MaineDOT WIN 018915.00
Brewer, Maine

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

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Stratigraphy Data[2,3]																Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration[2]
		Bituminous Concrete Thickness (ft)	Topsoil/Fill			Marine Deposit			Glacial Till			Weathered Bedrock			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)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)			
BB-BEA-101	139.5	0.4	0.4	139.1	4.6	NE	NE	NE	5.0	134.5	19.5	NE	NE	NE	24.5	115.0		35.0	104.5
BB-BEA-102	139.3	0.5	0.5	138.8	6.0	NE	NE	NE	6.5	132.8	14.6	NE	NE	NE	21.1	118.2		32.0	107.3
BB-BEA-201	136.5	NE	0.0	136.5	2.0	NE	NE	NE	2.0	134.5	23.0	25.0	111.5	6.0	31.0	105.5		35.0	101.5
BB-BEA-202	137.1	NE	NE	NE	NE	0.0	137.1	4.2	4.2	132.9	16.0	NE	NE	NE	20.2	116.9		29.7	107.4
BB-BEA-203	135.7	NE	0.0	135.7	2.0	NE	NE	NE	2.0	133.7	15.5	NE	NE	NE	17.5	118.2		30.0	105.7
BB-BEA-204	138.7	NE	0.0	138.7	0.2	NE	NE	NE	0.2	138.5	21.6	NE	NE	NE	21.8	116.9		31.1	107.6
HB-BE-217	135.1	NE	0.0	135.1	2.0	NE	NE	NE	2.0	133.1	24.5	NE	NE	NE	26.5	108.6		33.5	101.6
HB-BE-218	138.7	NE	0.0	138.7	0.4	NE	NE	NE	0.4	138.3	21.7	NE	NE	NE	22.1	116.6		30.2	108.5

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
Prepared By:	SSM	2/8/2021
Checked By:	BCS	3/9/2021
Reviewed By:	WAC	4/30/2021

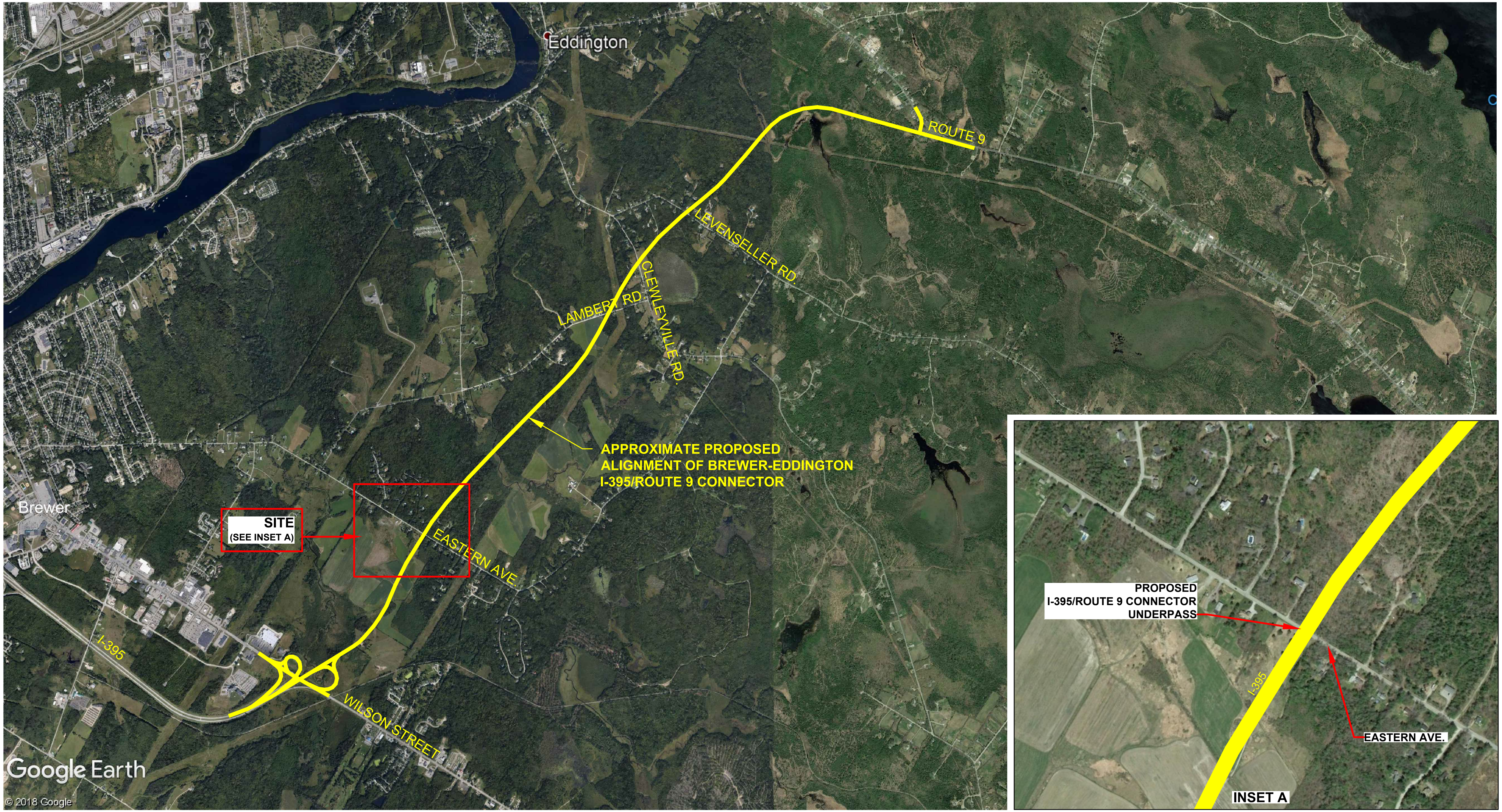
TABLE III
Subsurface Exploration Bedrock Core Data
Eastern Avenue Bridge over Interstate 395/Route 9 Connector
MaineDOT WIN 018915.00
Brewer, Maine

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

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery[3,6]		Rock Quality Designation[4,5,6]			Physical Rock Parameters		Lithologic, Rock Mass and Discontinuity Description
			No.	Depth Below Ground Surface (ft) Top	Bottom	Midpoint	Total Length (ft)	Recovered Length (ft)	%	Length (ft)	%	Designation	Weathering	Estimated Field Strength	
BB-BEA-101	139.5	NQ (1.875")	R1	25.0	30.0	27.5	5.0	4.2	83%	2.7	53%	Fair	Fresh to Slightly	Hard	Grey, aphanitic, SILTSTONE. Discontinuities dipping at moderate to steep angles (35 to 85 degrees from horizontal axis), spacing very close to close (<2 in. to 12 in.), discontinuity aperatures are tight to open, discontinuity surfaces are planar, undulating and rough. Frequent calcite stringers.
			R2	30.0	32.2	31.1	2.2	2.2	100%	1.4	62%	Fair	Fresh to Highly	Hard	
			R3	32.2	35.0	33.6	2.8	2.6	94%	2.0	71%	Fair	Fresh	Hard	
BB-BEA-102	139.3	NQ (1.875")	R1	22.0	27.0	24.5	5.0	4.9	97%	4.6	92%	Excellent	Fresh	Hard	Grey, aphanitic, SILTSTONE. Discontinuities dipping at low to moderate angles (5 to 55 degrees from horizontal axis), spacing very close to moderately close (<2 in. to 3 ft), discontinuity aperatures are tight. Occasional quartz veins and frequent calcite veins.
			R2	27.0	32.0	29.5	5.0	4.4	87%	2.8	55%	Fair	Slight to Moderately	Hard	
BB-BEA-201	136.5	NQ (1.875")	R1	32.0	35.0	33.5	3.0	2.7	89%	0.8	25%	Very poor	Slight	Hard	Grey, aphanitic, METASILTSTONE. Discontinuities dipping at low to moderate angles (5 to 55 degrees from horizontal axis), spacing very close (<2 in.), discontinuity aperatures are open, discontinuity surfaces are planar and rough.
BB-BEA-202	137.1	NQ (1.875")	R1	20.2	25.2	22.7	5.0	3.9	77%	2.4	48%	Poor	Fresh to Slightly	Mod. Hard to Hard	Grey, aphanitic, SILTSTONE. Discontinuities dipping at low to steep angles (5 to 85 degrees from horizontal axis), spacing very close to close (<2 in. to 12 in.), discontinuity aperatures are tight to open, discontinuity surfaces are planar, undulating and rough. Highly fractured zones.
			R2	25.2	27.4	26.3	2.2	2.0	91%	1.0	46%	Poor	Slight	Mod. Hard	
			R3	27.4	29.7	28.55	2.3	1.8	80%	1.2	54%	Fair	Slight	Mod. Hard to Hard	
BB-BEA-203	135.7	NQ (1.875")	R1	20.0	25.0	22.5	5.0	5.0	100%	1.9	37%	Poor	Fresh	Hard	Grey, aphanitic, METASILTSTONE. Discontinuities dipping at low to steep angles (5 to 85 degrees from horizontal axis), spacing very close to moderately close (<2 in. to 3ft), discontinuity aperatures are tight to open, discontinuity surfaces are planar, undulating and rough, and occasionally oxidized. R2 has fractured zones up to 10-in. thick with occasional silt infilling up to 2-in. thick
			R2	25.0	30.0	27.5	5.0	4.2	83%	0.0	0%	Very poor	Fresh	Hard	
BB-BEA-204	138.7	NQ (1.875")	R1	21.9	26.9	24.4	5.0	4.4	88%	4.4	87%	Good	Fresh to Slightly	Hard	Grey, aphanitic, SILTSTONE. Discontinuities dipping at moderate to steep angles (35 to 85 degrees from horizontal axis), spacing close to moderately close (12 in. to 3ft), discontinuity aperatures are tight to open, discontinuity surfaces are planar and rough. Frequent calcite/quartz veins up to 1-in. thick.
			R2	26.9	31.1	29.0	4.2	4.4	105%	3.1	74%	Fair	Fresh to Slightly	Hard	
HB-BE-217	135.1	NQ (1.875")	R1	28.5	33.5	31.0	5.0	4.9	97%	1.6	32%	Poor	Slight	Hard	Grey, aphanitic, METASILTSTONE. Discontinuities dipping at low to vertical angles (5 to 90 degrees from horizontal axis), spacing very close to close (<2 in. to 12 in.), discontinuity aperatures are tight to open, discontinuity surfaces are planar and rough and occasionally are infilled with silt. Frequent quartz veins up to 3-in. thick.
HB-BE-218	138.7	NQ (1.875")	R1	22.7	27.0	24.9	4.3	3.5	81%	2.8	64%	Fair	Fresh to Slightly	Hard	Grey, aphanitic, SILTSTONE. Discontinuities dipping at low to vertical angles (5 to 90 degrees from horizontal axis), spacing close to moderately close (12 in. to 3ft), discontinuity aperatures are tight to open, highly fractured zones.
			R2	27.0	30.2	28.6	3.2	1.8	55%	0.0	0%	Very poor	Moderate	Mod. Hard	

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-BEA-204 R2 recovery and RQD include portion of R1 that was not initially recovered.

	Individual	Date
Prepared By:	SSM	4/5/2021
Checked By:	BCS	4/22/2021
Reviewed By:	WAC	4/30/2021



NOTES

1. IMAGE TAKEN FROM GOOGLE EARTH IMAGES, 2018.

N

0 2950 5900

APPROX. SCALE IN FEET

HALEY ALDRICH

EASTERN AVENUE BRIDGE OVER
INTERSTATE 395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER, MAINE

PROJECT LOCUS

SCALE: AS SHOWN
JUNE 2021

FIGURE 1

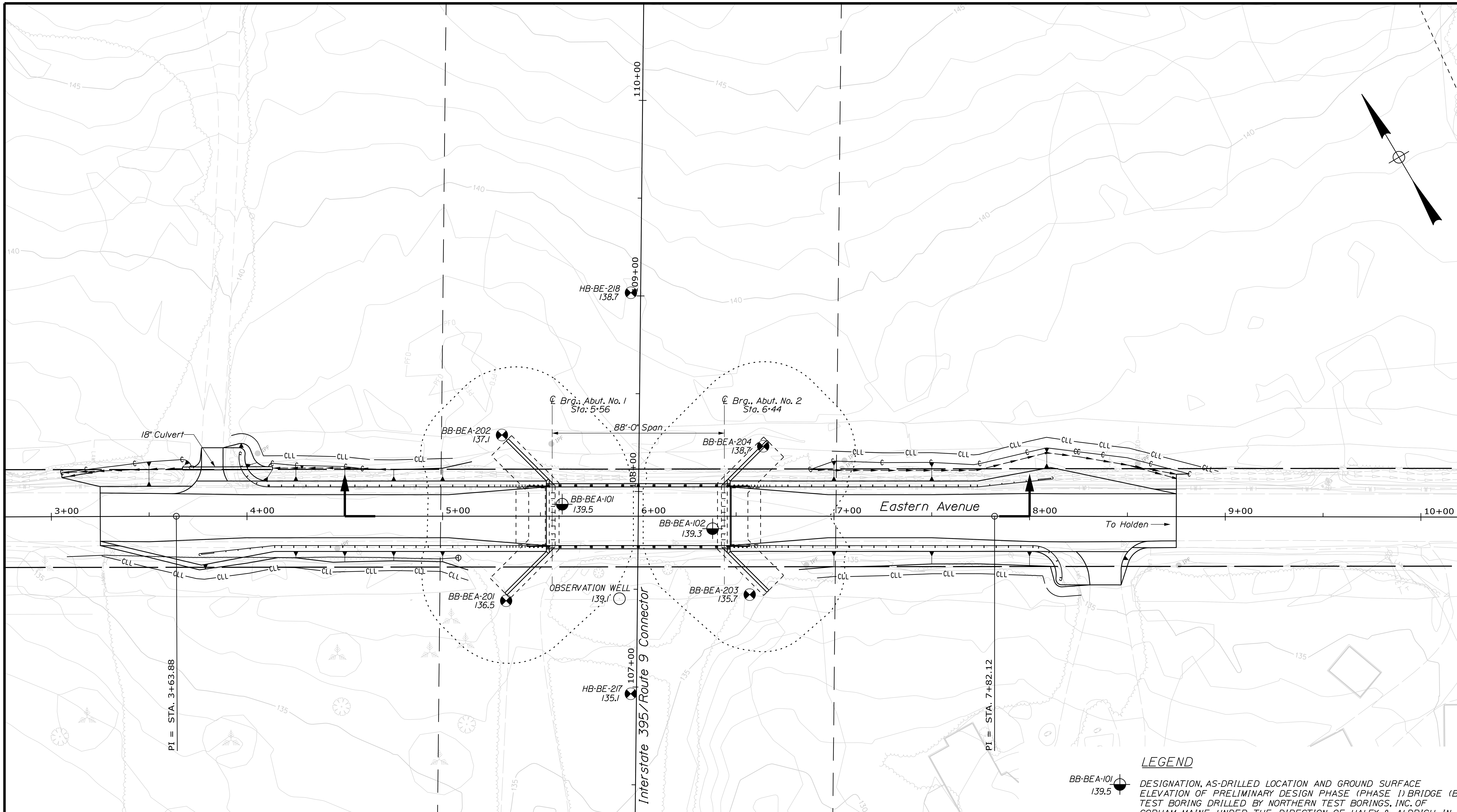
Saved by: KPOSTOLOWSKI Printed: 5/14/2020 10:45 AM Sheet: FIG 1
Z:\POR_COMMON\PROJECTS\132076 - BREWER EDDINGTON\CAD\GEOLOGY\FIGURES\132076_004_0011_PROJECT LOCUS EASTERN AVE.DWG

Date:4/9/2021

Username:

Division:

Filename: ...\046_Plan_EasternAvenue.dgn



NOTES:

1. 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.
2. 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.
3. ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88).
4. TEST BORINGS WERE MONITORED IN THE FIELD BY A HALEY & ALDRICH, INC. GEOLOGIST OR GEOTECHNICAL ENGINEER.
5. REFER TO APPENDIX A FOR TEST BORING LOGS AND ROCK CORE PHOTOGRAPHS AND APPENDIX B FOR OBSERVATION WELL INSTALLATION AND GROUNDWATER MONITORING REPORTS.

LEGEND

BB-BEA-101 139.5 — 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-BEA-201 136.5 / HB-BE-217 135.1 — DESIGNATION, AS-DRILLED LOCATION AND GROUND SURFACE ELEVATION OF FINAL DESIGN PHASE (PHASE II) BRIDGE (BB) OR HIGHWAY (HB) TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE UNDER THE DIRECTION OF HALEY & ALDRICH IN NOVEMBER 2020

OBSERVATION WELL 139.1 — DESIGNATION, AS-INSTALLED LOCATION, AND TOP ELEVATION OF OBSERVATION WELL

DESIGNATION, LOCATION, AND ORIENTATION OF INTERPRETIVE SUBSURFACE PROFILE (SEE FIGURE 3)

PLAN

25 0 25 50

Scale of Feet

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(500)

BRIDGE NO. — WIN 018915.00 — BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	B. STEINERT	K. POST	5-9-20
CHECKED-REVIEWED	B. STEINERT	W. CHADBOURNE	4-16-21
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

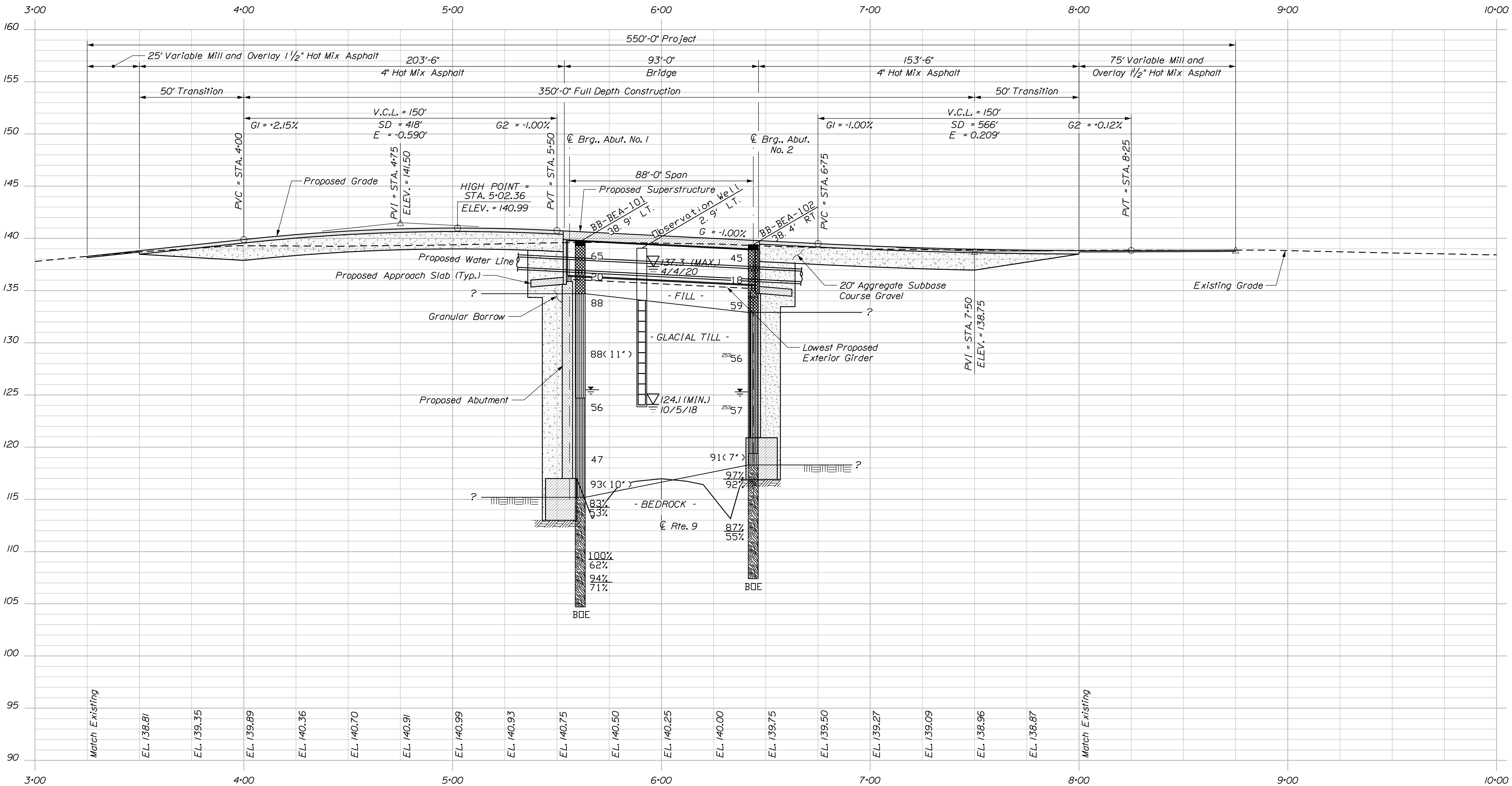
P.E. NUMBER

DATE

I-395 - ROUTE 9 CONNECTOR
EASTERN AVENUE BRIDGE
BREWER-EDDINGTON PENOBSCOT COUNTY

SITE AND SUBSURFACE
EXPLORATION LOCATION PLAN

FIGURE 2 OF 3



PROFILE

LEGEND

- Boring No. / Offset
- Pavement Thickness, if applicable
- 10 Energy Corrected Standard Penetration Test (SPT) N60-Value (Blows Per Foot)
- Water Level Measured During Drilling
- 4/4/20 Water Level measured in Observation Well on Date Indicated
- Approximate Top of Bedrock
- 64% Percent Recovery / 18% Percent Rock Quality Designation (RQD)
- BOE = Bottom of Exploration

NOTES:

1. Boring offset based on proposed Connector baseline.
2. 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.
3. 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.
4. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
5. Refer to Appendix A for test boring logs and rock core photographs and Appendix B for observation well installation and groundwater monitoring reports.

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1891(500)

WIN 018915.00

BRIDGE PLANS

BRIDGE NO.

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	E. FORCE	K. POST	5-9-20
CHECKED-REVIEWED	B. STEINERT	W. CHADBOURNE	5-21-21
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

I-395 - ROUTE 9 CONNECTOR
ROUTE 9
BREWER-EDDINGTON PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 3

Filename: ...\047_Profile_EasternAvenue.dgn Division: Username: Date:6/1/2021

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 (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, 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.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, Gravelly sands, 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
			SC	Clayey sands, sand-clay 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

Descriptive Term	Portion of Total (%)
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).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
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.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
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):

$$RQD\ (\%) = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*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))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine
Boring No.: BB-BEA-101
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	139.5	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	08-1-18/08-2-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+93.1, 38.9 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.5 - 2.5	14/25/18/11	43	65	SSA	139.1		-BITUMINOUS CONCRETE- 0.4	
										Brown, dry, very dense, Gravelly fine to coarse SAND, trace silt -FILL-(SW)(ROADWAY BASE/SUBBASE MATERIAL)	
	2D	24/14	2.5 - 4.5	15/9/4/5	13	20		136.5		3.0	
										Brown, moist, very stiff, SILT, little fine sand, trace medium to coarse sand, trace gravel -FILL-(ML)	
5	3D	24/12	5.0 - 7.0	10/24/34/49	58	88		134.5		5.0 Brown to rust-brown, moist to dry, very dense, Silty fine to coarse SAND, some fine to coarse gravel, well bonded -GLACIAL TILL-(SM)	G#474412 A-4(0), SM WC=7.6
							HW				
							66				
							101				
10	4D	17/12	10.0 - 11.4	19/38/50(5")			122			Olive-brown, wet, very dense, fine to coarse SAND, some silt and fine to coarse gravel, well bonded -GLACIAL TILL-(SM) Note: Washed ahead of casing from 10.0 to 15.0 ft.	G#474413 A-2-4(0), SM WC=8.5
							77				
							117				
							86				
							70				
15	5D	24/16	15.0 - 17.0	17/18/19/42	37	56	92	124.5		15.0 Olive-brown, wet, hard, SILT, some fine to coarse sand and fine to coarse gravel, well bonded -GLACIAL TILL-(ML) Note: Washed ahead of casing from 15.0 to 20.0 ft.	G#474414 A-4(0), ML WC=11.3
							98				
							55				
							55				
							63				
20	6D	24/2	20.0 - 22.0	14/16/15/15	31	47	41	119.5		20.0 Brown, wet, dense, fine to coarse SAND, little silt, little gravel, well bonded -GLACIAL TILL-(SM)	
							57				
							57				
	7D	16/8	23.0 - 24.3	25/43/50(4")			56			Brown, wet, very dense, fine to coarse SAND, some silt, little gravel, well bonded -GLACIAL TILL-(SM)	
25							94/6"	115.0		24.5	

Remarks:

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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-BEA-101
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	139.5	**Auger ID/OD:**	--
Operator:	M. Nadeau	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon 1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Diedrich D50 Track (Rig #377)	**Hammer Wt./Fall:**	SS-140#/30; HW-140#/20
Date Start/Finish:	08-1-18/08-2-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+93.1, 38.9 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R1	60/50	25.0 - 30.0	RQD = 53%			NQ CORE			Top of Bedrock at El. 115.0 R1: Grey, aphanitic SILTSTONE, hard, fresh to slightly weathered. Joints dipping at moderate to steep angles, very close to close, tight to open, planar to undulating, rough. Frequent veins 0.25 to 0.5-in. thick, frequent calcite stringers. Rock Quality=Fair Recovery=83% -BREWER FORMATION- R1 Core Times (min:sec): 25.0-26.0' (3:37); 26.0-27.0' (3:22); 27.0-28.0' (4:07); 28.0-29.0' (3:46); 29.0-30.0' (4:48)	qp=3,818 psi (28.7-29.6')
30	R2	26/26	30.0 - 32.2	RQD = 62%						R2: Similar to R1, except fresh to highly weathered, occasional 2 to 4-in. thick moderate to highly weathered zones with moderately weathered joint surfaces, few calcite veins. Rock Quality=Fair Recovery=100% -BREWER FORMATION- R2 Core Times (min:sec): 30.0-31.0' (4:37); 31.0-32.0' (3:22); 32.0-32.2' (1:11)	qp=6,520 psi (31.6-32.2')
	R3	34/32	32.2 - 35.0	RQD = 71%						R3: Similar to R1, except fresh, one 0.25-in. thick band of pyrite mineralization at approximately 34.8 ft. Rock Quality=Fair Recovery=94% -BREWER FORMATION- R3 Core Times (min:sec): 32.2-33.3' (3:30); 33.3-34.3' (3:18); 34.3-35.0' (2:44)	
35								104.5		35.0 **Bottom of Exploration at 35.0 feet below ground surface.**	
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector		**Boring No.:**	BB-BEA-102
Location: Brewer and Eddington, Maine		**WIN:**	18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	139.3	**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:	08-2-18/08-2-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+81.2, 38.4 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.4 - 2.4	12/16/14/14	30	45	SSA	138.8		-BITUMINOUS CONCRETE- 0.5 Brown, dry, dense, fine to coarse SAND, little gravel, trace silt -FILL-(SW)(ROADWAY BASE/SUBBASE MATERIAL)	
	2D	24/8	2.5 - 4.5	10/6/6/7	12	18		136.8		2.5 Brown to rust-brown, dry, medium dense, fine Sandy SILT, little medium to fine sand, little gravel, reworked naturally-deposited soils -FILL-(ML)	
5	3D	22/20	5.0 - 6.8	18/17/22/50(4")	39	59	16	134.3		5.0 Brown to rust-brown, dry, very dense, fine to coarse SAND, some silt, little gravel, reworked naturally-deposited soil -FILL-(SM)	
							25	132.8		6.5	
							129				
							90				
							68				
10	4D	24/20	10.0 - 12.0	12/18/19/22	37	56	78			Olive-brown, moist, very hard, SILT, some fine to coarse sand, little fine gravel, well bonded -GLACIAL TILL-(ML) Note: Washed ahead of casing from 10.0 to 15.0 ft.	G#474415 A=4(0), ML WC=10.9
							73				
							71				
							67				
							64				
15	5D	24/20	15.0 - 17.0	14/19/19/20	38	57	69			Olive-brown, moist to wet, hard, SILT, some fine to coarse gravel, little fine to coarse sand, well bonded -GLACIAL TILL-(ML)	G#474416 A=4(0), ML WC=9.7
							71				
							63				
							68				
							65				
20	6D	13/8	20.0 - 21.1	12/41/50(1")			57 25/1"	119.3		20.0 Olive-brown, wet, very dense, fine to coarse SAND, some silt, little gravel, well bonded -GLACIAL TILL-(SM)	
							RC	118.2		21.1	
	R1	60/58	22.0 - 27.0	RQD = 92%			NQ CORE			Top of Bedrock at El. 118.2 R1: Grey, aphanitic, SILTSTONE. Hard, fresh, joints dipping at low to moderate angles, very close to moderately close, tight, quartz veins approximately 0.1 to 2-in. thick throughout core run. Frequent calcite veins. Rock Quality=Excellent	qp=7,261 psi (22.8-23.2')
25											

Remarks:

1. Observaton well installed in completed borehole. See observation well installation and groundwater monitoring reports for details.

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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-BEA-102
WIN: 18915.00

Driller:	Northern Test Borings, Inc.	**Elevation (ft.)**	139.3	**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:	08-2-18/08-2-18	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+81.2, 38.4 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor:	0.907	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ CORE			Recovery=97% -BREWER FORMATION- R1 Core Times (min:sec): 22.0-23.0' (4:39); 23.0-24.0' (5:47); 24.0-25.0' (7:08); 25.0-26.0' (7:16); 26.0-27.0' (6:50)	
	R2	60/52	27.0 - 32.0	RQD = 55%						R2: Similar to R1, except slight to moderate weathering from 27.5 to 27.9 ft, and from 31.0 to 31.5 ft, tight to open. Rock Quality=Fair Recovery=87% -BREWER FORMATION- R2 Core Times (min:sec): 27.0-28.0' (2:45); 28.0-29.0' (3:22); 29.0-30.0' (4:26); 30.0-31.0' (1:56); 31.0-32.0' (2:19) Note: Lost approximately 250 gallons of water at approximately 30.0 ft depth.	qp=20,635 psi (29.3-30.0')
30											
								107.3		32.0 **Bottom of Exploration at 32.0 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

1. Observaton well installed in completed borehole. See observation well installation and groundwater monitoring reports for details.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 9/I-395 Connector			**Boring No.:**	BB-BEA-201
Location:	Brewer and Eddington, Maine			**WIN:**	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	136.5	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW-300#/16
Date Start/Finish:	11-10-2020/11-10-2020	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+43.4, 67.1 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
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G = Grain Size Analysis
C = Consolidation Test

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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.0 - 2.0	6/8/11/11	19	27	HSA			Brown, moist, medium dense, Silty fine SAND, little medium sand, trace coarse sand, trace fine to coarse gravel -FILL-(SM)	
								134.5		2.0	
5	2D	19/15	5.0 - 6.6	27/35/31/50(1")	66	95	6			Grey-brown, dry, very dense, fine SAND, some silt, little medium sand, trace gravel, cobbles likely present -GLACIAL TILL-(SM) Note: Advanced HSA to 10.0 ft prior to use of casing. Frequent cobbles from 6.5 to 10.0 ft.	
							6				
							7				
							25				
							97				
10	3D	24/18	10.0 - 12.0	30/29/38/37	67	97	98			Similar to 2D -GLACIAL TILL-(SM)	
							130				
							80			Note: Used 140-lb auto-hammer to drive HW casing from 5.0 to 15.0 ft. Washed ahead of casing from 10.0 to 15.0 ft.	
							75				
							132				
15	4D	24/16	15.0 - 17.0	12/15/17/17	32	46	110	121.5		15.0 Olive-brown, wet, hard, SILT, little fine to medium sand, trace gravel, well bonded -GLACIAL TILL-(ML)	
							72				
							60			Note: Washed ahead of casing from 15.0 to 20.0 ft. Frequent cobbles from 15.0 to 20.0 ft.	
							75				
							67				
20	MD	24/0	20.0 - 22.0	20/27/30/30	57	82	HW			No Recovery Note: Drove HW casing to 20.0 ft. Washed ahead of casing from 20.0 to 25.0 ft. Very few cobbles. Borehole stayed open to 25.0 ft.	
25											

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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-BEA-201
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	136.5	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW-300#/
Date Start/Finish:	11-10-2020/11-10-2020	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+43.4, 67.1 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	Not Measured
Hammer Efficiency Factor: 0.867		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
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WOR/C = Weight of Rods or Casing
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	12/10	25.0 - 26.0	10/89/50(0")			HW	111.5		25.0 Top of Weathered Bedrock El. 111.5 Grey to grey-brown, moist, very dense, Silty GRAVEL, some coarse sand, little medium sand, consisting of highly decomposed bedrock fragments exhibiting distinct rock fabric -WEATHERED BEDROCK-(GM) Note: Washed ahead with rollerbit through severely decomposed bedrock. Drill action and cuttings indicate decomposed bedrock from 25.0 to 30.0 ft.	
30	6D	12/12	30.0 - 31.0	13/50/50(0")						Similar to 5D, except no little medium sand -WEATHERED BEDROCK-(GM)	
							RC	105.5		31.0 Top of Bedrock El. 105.5	
	R1	36/32	32.0 - 35.0	RQD = 25%			NQ CORE			R1: Grey, aphanitic, METASILTSTONE, hard, slightly weathered. Joints low angle to moderately dipping, very close, planar, rough, open. Note: Core jam at 35.0 ft. Rock Quality=Very Poor Recovery=89% -BREWER FORMATION- R1 Core Times (min:sec): 32.0-33.0' (2:56); 33.0-34.0' (1:48); 34.0-35.0' (1:57)	qp=16,538 psi (33'-33.7')
35								101.5		35.0	
										Bottom of Exploration at 35.0 feet below ground surface.	
40											
45											
50											

Remarks:

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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-BEA-202
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	137.1	**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:	11-5-2020/11-6-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 108+28.1, 70.1 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	7.1 ft
Hammer Efficiency Factor:	0.852	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	WOH/1/1/4	2	3	SSA			Brown to rust-brown mottled, moist, soft, SILT -MARINE DEPOSIT-(ML)	
								132.9		4.2 Note: Boulder from 4.2 to 6.0 ft.	
5											
								131.1		6.0	
	2D	24/21	6.0 - 8.0	23/42/49/69	91	129	41			Brown, moist, hard, SILT, little fine sand, trace gravel -GLACIAL TILL-(ML) Note: Cobble layer from 6.7 to 6.9 ft.	
							175				
							248				
							223				
10								127.1		10.0	
	3D	24/15	10.0 - 12.0	60/52/42/38	94	133	29			Brown, wet, very dense, fine to medium SAND, some silt, little gravel, moderately bonded -GLACIAL TILL-(SM)	
							29				
							22				
							22				
							24				
15								122.1		15.0	
	4D	24/14	15.0 - 17.0	13/18/15/15	33	47	56			Brown, wet, hard, SILT, some fine to medium sand, trace gravel, moderately bonded -GLACIAL TILL-(ML)	
							30				
							107				
							69				
							92				
20								117.1		20.0	
	5D	2/1	20.0 - 20.2	70			NQ	116.9		Grey, wet, very dense, GRAVEL, little medium to coarse sand, loosely bonded -GLACIAL TILL-(GP)	
	R1	60/46	20.2 - 25.2	RQD = 48%			CORE			20.2	
										Top of Bedrock El. 116.9 R1: Grey, aphanitic, SILTSTONE, moderately hard to hard, fresh to slightly weathered, silt seam at beginning of rock core. Joints dipping at low to moderate angles, very close to close spacing, tight, rough, planar, frequent calcite stringers/veins (up to 0.25 in. thickness). Secondary steeply dipping joint, planar, rough, tight.	qp=20,742 psi (22.2'-22.9')
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.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector				**Boring No.:**	BB-BEA-202
Location: Brewer and Eddington, Maine				**WIN:**	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	137.1	**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#/
Date Start/Finish:	11-5-2020/11-6-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 108+28.1, 70.1 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	7.1 ft
Hammer Efficiency Factor: 0.852		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	26.4/24	25.2 - 27.4	RQD = 46%			NQ CORE			Rock Quality=Poor Recovery=77% -BREWER FORMATION- R1 Core Times (min:sec): 20.2-21.2' (5:18); 21.2-22.2' (3:45); 22.2-23.2' (3:50); 23.2-24.2' (2:07); 24.2-25.2' (3:18) R2: Grey, aphanitic, SILTSTONE, moderately hard, slightly weathered. Joints dipping at steep angles, close spacing, tight to open, rough, planar, frequent thin calcite veins. Highly fractured zone from approximately 25.2 to 26.2 ft. Rock Quality=Poor Recovery=91% -BREWER FORMATION- R2 Core Times (min:sec): 25.2-26.2' (2:25); 26.2-27.4' (3:38) R3: Grey, aphanitic, SILTSTONE, hard to moderately hard, slightly weathered. Joints dipping at steep angles, close spacing, tight to open, rough, planar. Highly fractured zone from approximately 27.4 to 28.2 ft. Low angle to moderately dipping. Secondary joints planar, rough, open. Rock Quality=Fair Recovery=80% -BREWER FORMATION- R3 Core Times (min:sec): 27.4-28.4' (2:58); 28.4-29.4' (3:39); 29.4-29.7' (1:55)	
	R3	27.6/22	27.4 - 29.7	RQD = 54%							
30								107.4		29.7	
										Bottom of Exploration at 29.7 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector		**Boring No.:**	BB-BEA-203		
Location: Brewer and Eddington, Maine		**WIN:**	18915.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	135.7	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW-300#/16
Date Start/Finish:	11-9-2020/11-10-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+47.9, 57.4 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.6 ft
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.0 - 2.0	3/4/4/4	8	12	HSA			Brown, moist, stiff, SILT, little fine sand, trace coarse sand, roots, grass, fines slightly to mostly organic -TOPSOIL-(OL)	
								133.7		2.0 Note: Spun HSA to 5.0 ft. Occasional cobbles from 2.0 to 5.0 ft.	
5	2D	24/20	5.0 - 7.0	17/17/33/20	50	72				Light brown to grey-brown, moist, very dense, fine SAND, some silt, little medium sand, trace fine to coarse gravel -GLACIAL TILL-(SM) Note: Spun SSA to 10.0 ft. Frequent cobbles from 5.0 to 10.0 ft.	
10	3D	24/20	10.0 - 12.0	61/10/14/16	24	35	37	125.7		10.0 Olive-brown, moist, hard, SILT, trace fine to coarse sand, trace fine to coarse gravel, well bonded -GLACIAL TILL-(ML)	
							42			Note: Pushed NW casing to 10.0 ft. Washed ahead of casing from 10.0 to 15.0 ft. Drill action indicated mini boulders (12-in.) from 12.3 to 14.5 ft.	
							49				
							58				
							54				
15	4D	24/16	15.0 - 17.0	23/46/31/22	77	111	60			Olive-brown, wet, hard, SILT, some fine sand, trace medium to coarse sand, trace fine to coarse gravel, well bonded -GLACIAL TILL-(ML)	
							65				
							6 RC	118.2		Note: Drill action and cuttings indicated sound rock from 17.5 to 20.0 ft. Drove HW casing to 17.5 ft. 17.5 Top of Bedrock El. 118.2	
20	R1	60/60	20.0 - 25.0	RQD = 37%			NQ CORE			R1: Grey, aphanitic, METASILTSTONE, hard, fresh. Joints dipping steep to low angles, very close to moderately close, planar to undulating, rough, tight to open. Frequent calcite veins, occasional oxidation on joint surfaces. Rock Quality=Poor Recovery=100% -BREWER FORMATION- R1 Core Times (min:sec): 20.0-21.0' (4:10); 21.0-22.0' (5:25); 22.0-23.0' (5:51); 23.0-24.0' (4:19); 24.0-25.0' (3:12)	qp=8,733 psi (20.6'-21.7')
25											

Remarks:

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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-BEA-203 **WIN:** 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	135.7	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; HW-300#/
Date Start/Finish:	11-9-2020/11-10-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 107+47.9, 57.4 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.6 ft
Hammer Efficiency Factor: 0.867		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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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
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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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	60/50	25.0 - 30.0	RQD = 0%			NQ CORE			R2: Similar to R1, except common extremely fractured zones up to 10-in. thick with occasional silt infillings up to 2-in. thick between joints. Rock Quality=Very Poor Recovery=83% -BREWER FORMATION- R2 Core Times (min:sec): 25.0-26.0' (2:10); 26.0-27.0' (3:18); 27.0-28.0' (3:42); 28.0-29.0' (4:14); 29.0-30.0' (1:52)	
30								105.7		30.0 **Bottom of Exploration at 30.0 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 9/I-395 Connector			**Boring No.:**	BB-BEA-204
Location:	Brewer and Eddington, Maine			**WIN:**	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	138.7	**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:	11-5-2020/11-5-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 108+23.8, 63.5 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	13.6 ft
Hammer Efficiency Factor:	0.852	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.0 - 2.0	WOH/1/2/3	3	4	SSA	138.5		Brown, moist, soft, SILT, little fine sand, organics -TOPSOIL-(OL) 0.2	
							1			Brown, moist, soft, fine Sandy SILT, little medium to coarse sand, trace gravel, trace organics -GLACIAL TILL-(ML)	
							2				
							14	135.7		3.0	
							21				
5	2D	13.2/12	5.0 - 6.1	22/34/50(1")			43			Brown, moist, very dense, Silty fine SAND, trace coarse sand, trace gravel -GLACIAL TILL-(SM)	
							35				
							28				
							31				
							38				
10	3D	24/20	10.0 - 12.0	19/19/23/20	42	60	19	128.7		10.0 Brown, wet, hard, SILT, little coarse sand, little gravel -GLACIAL TILL-(ML)	
							23				
							23				
							18				
							10				
15	4D	24/13	15.0 - 17.0	14/15/23/23	38	54	HW			Brown, wet, hard, fine Sandy SILT, some coarse sand, little gravel, moderately bonded -GLACIAL TILL-(ML)	
20	5D	21.6/13	20.0 - 21.8	26/28/27/50(3")	55	78		118.7		20.0 Brown, wet, very dense, fine to medium SAND, some silt, some coarse sand, little gravel, moderately bonded -GLACIAL TILL-(SM)	
	R1	60/53	21.9 - 26.9	RQD = 87%			RC NQ	116.9		21.8 Top of Bedrock El. 116.9 R1: Grey, aphanitic, SILTSTONE, hard, fresh to slight weathering. Joints dipping at moderate to steep angle, close to moderate spacing, tight, rough, planar, few quartz veins 0.5 to 1 in. thickness. Rock Quality=Good	
							CORE				
25											

Remarks:

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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-BEA-204
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	138.7	**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#/
Date Start/Finish:	11-5-2020/11-5-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 108+23.8, 63.5 RT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	13.6 ft
Hammer Efficiency Factor:	0.852	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
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C = Consolidation Test

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.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ CORE			Recovery=88% -BREWER FORMATION- R1 Core Times (min:sec): 21.9-22.9' (3:06); 22.9-23.9' (3:12); 23.9-24.9' (2:39); 24.9-25.9' (3:32); 25.9-26.9' (4:46)	
	R2	50.4/53	26.9 - 31.1	RQD = 74%						R2: Grey, aphanitic, SILTSTONE, hard, fresh to slight weathering. Joints dipping at moderate to steep angles, close to moderate spacing, tight rough, planar. Frequent calcite/quartz veins (up to 0.5 in. thickness). Secondary low angle joints, very close to moderately close, planar rough, tight to open. Rock Quality=Fair Recovery=105% Note: R2 recovery and RQD includes portion of R1 that was not initially recovered. -BREWER FORMATION- R2 Core Times (min:sec): 26.9-27.9' (4:26); 27.9-28.9' (4:27); 28.9-29.9' (2:17); 29.9-31.1' (2:19)	qp=4,162 psi (26.9'-27.4')
30								107.6		31.1	
										Bottom of Exploration at 31.1 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 9/I-395 Connector
Location: Brewer and Eddington, Maine

Boring No.: HB-BE-217
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	135.1	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/16
Date Start/Finish:	11-9-2020/11-9-2020	**Drilling Method:**	HSA/NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 106+96.5, 2.9 LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor: 0.867		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	3/5/8/12	13	19	HSA			Brown, moist, very stiff, SILT, little fine sand, trace fine to coarse gravel, trace grass, roots -TOPSOIL-(OL)	
								133.1		2.0	
5	2D	22/10	5.0 - 6.8	13/19/20/50(4")	39	56				Brown, moist, very dense, fine to coarse SAND, some fine gravel, little silt, moderately bonded -GLACIAL TILL-(SM)	G#613870 A-1-b (0), SM
10	3D	24/24	10.0 - 12.0	20/20/28/31	48	69	68			Light brown, moist, very dense, Silty fine to coarse SAND, little fine gravel -GLACIAL TILL-(SM)	G#613871 A-4 (0), SM
							85				
							150			Note: Pushed NW casing to 10.0 ft. Used 140-lb automatic hammer to drive casing from 10.0 to 15.0 ft.	
							185				
							193				
15	4D	24/14	15.0 - 17.0	10/15/15/16	30	43	48			Olive-brown, moist, dense, medium SAND, some silt, little fine sand, little coarse gravel, trace coarse sand, trace fine gravel, well bonded -GLACIAL TILL-(SM)	G#613872 A-1-b (0), SM
							59				
							91				
							102			Note: Washed ahead of casing from 15.0 to 20.0 ft.	
							34				
20	5D	24/15	20.0 - 22.0	12/19/19/27	38	55	48			Olive-brown, wet, very dense, Silty fine to medium SAND, little fine gravel, trace coarse sand, well bonded -GLACIAL TILL-(SM)	G#613873 A-4 (0), SM
							63				
							69			Note: Washed ahead of casing from 20.0 to 25.0 ft.	
							68				
							70			Note: Drill action indicated cobbles from 22.8 to 24.2 ft.	
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.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Route 9/I-395 Connector Location: Brewer and Eddington, Maine	Boring No.: HB-BE-217 WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	135.1	**Auger ID/OD:**	HSA-3.25 in. ID
Operator:	J. Layfield	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	C. Toscano	**Rig Type:**	Mobile B-53 Truck	**Hammer Wt./Fall:**	SS-140#/30; NW-300#/
Date Start/Finish:	11-9-2020/11-9-2020	**Drilling Method:**	HSA/NW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 106+96.5, 2.9 LT	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	14.0 ft
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	20/17	25.0 - 26.7	26/32/46/50(2")	78	113	NW	110.1		25.0 Olive-brown, wet, hard, fine to medium Sandy SILT, little coarse sand, trace fine to coarse gravel, well bonded, cobbles likely present -GLACIAL TILL-(ML)	
							RC	108.6		26.5 Top of Bedrock El. 108.6	
	R1	60/58	28.5 - 33.5	RQD = 32%			NQ			R1: Grey, aphanitic, METASILTSTONE, slightly weathered. Joints dipping vertical to low angle, very close to close, open to tight. Frequent quartz veins up to 3-in. thick throughout core run. Occasional silt infillings on joint surfaces. Rock Quality=Poor Recovery=97% -BREWER FORMATION- R1 Core Times (min:sec): 28.5-29.5' (3:20); 29.5-30.5' (1:55); 30.5-31.5' (3:30); 31.5-32.5' (2:58); 32.5-33.5' (2:54)	
							CORE				
30											
								101.6		33.5 **Bottom of Exploration at 33.5 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Route 9/I-395 Connector	**Location:**	Brewer and Eddington, Maine	**Boring No.:**	HB-BE-218
				WIN:	18915.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	138.7	**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:	11-4-2020/11-4-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 109+01.6, 4.9 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	17.4 ft
Hammer Efficiency Factor:	0.852	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D/A	24/12	0.0 - 2.0	1/2/3/4	5	7	WOH	138.3		Brown, moist, soft, SILT, trace fine sand, organics, roots -TOPSOIL-(OL)	
										0.4	
							7			Brown, moist, loose, fine SAND, some silt, little gravel, trace medium sand, loosely bonded -GLACIAL TILL-(SM)	
							11				
							8				
							12				
5	2D	24/20	5.0 - 7.0	12/26/32/45	58	82	74	133.7		5.0 Brown, moist, hard, SILT, little fine gravel, trace fine to coarse sand, loosely bonded -GLACIAL TILL-(ML)	G#613874 A-4 (0), ML
							149				
							165				DS-1, DS-2, DS-3 Cohesion 803 psf Friction Angle 32.7
							143				
							152				
10	3D	24/16	10.0 - 12.0	21/21/22/19	43	61	HW			Brown, wet, hard, SILT, some fine sand, little medium sand, trace coarse sand, trace fine gravel, loosely bonded -GLACIAL TILL-(ML)	G#613875 A-4 (0), ML
15	4D	24/16	15.0 - 17.0	31/20/67/35	87	124				Brown, wet, hard, SILT, little fine sand, little fine gravel, trace medium to coarse sand, bonded -GLACIAL TILL-(ML)	G#613876 A-4 (0), ML
20	5D	24/15	20.0 - 22.0	8/65/50/53	115	163		118.7		20.0 Brown, wet, hard/very dense, SILT and fine to coarse GRAVEL, some fine to coarse sand, moderately bonded -GLACIAL TILL-(ML/GW)	G#613877 A-2-4 (0)
	R1	51.6/42	22.7 - 27.0	RQD = 64%			RC NQ CORE	116.6		22.1 Top of Bedrock El. 116.6 R1: Grey, aphanitic, SILTSTONE, hard, fresh to slightly weathered. Joints dipping at low to moderate angles, close to moderate spacing, tight to open. Secondary joints steep, moderate, tight to open. Quartz intrusion.	
25											

Remarks:

DS denotes direct shear test. DS-1, DS-2, DS-3 were run on a composite sample of 2D, 3D, and 4D.

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.: HB-BE-218
WIN: 18915.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	138.7	**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#/
Date Start/Finish:	11-4-2020/11-4-2020	**Drilling Method:**	SSA/HW Drive	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 109+01.6, 4.9 LT	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	17.4 ft
Hammer Efficiency Factor:	0.852	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ CORE			Rock Quality=Fair Recovery=81% -BREWER FORMATION- R1 Core Times (min:sec): 22.7-23.7' (4:33); 23.7-24.7' (3:51); 24.7-25.7' (3:17); 25.7-26.7' (4:03); 26.7-27.0' (2:01)	
	R2	38.4/21	27.0 - 30.2	RQD = 0%						R2: Grey, aphanitic, SILTSTONE, moderately hard, moderately weathered. Discernible joints dipping at steep to vertical angles, highly fractured throughout (gravel-sized pieces). Rock Quality=Very Poor Recovery=55% -BREWER FORMATION- R2 Core Times (min:sec): 27.0-28.0' (1:37); 28.0-29.0' (2:36); 29.0-30.2' (2:40)	
30								108.5		30.2	
										Bottom of Exploration at 30.2 feet below ground surface.	
35											
40											
45											
50											

Remarks:

DS denotes direct shear test. DS-1, DS-2, DS-3 were run on a composite sample of 2D, 3D, and 4D.

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.

ROCK CORE PHOTOGRAPHS
EASTERN AVENUE BRIDGE OVER INTERSTATE 395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER, MAINE

Top Row: BB-BEA-101: Run No. R1 25.0' (left) to 30.0' (middle); Run No. R2 30.0' (middle) to 30.5' (right)
Top Middle Row: BB-BEA-101: Run No. R2 continued 30.5' (left) to 32.2' (right); Run No. R3 32.2' (left) to 35.0' (right)
Bottom Middle Row: BB-BEA-102: Run No. R1 22.0' (left) to 27.0' (right)
Bottom Row: BB-BEA-102: Run No. R2 27.0' (left) to 32.0' (right)

ROCK CORE PHOTOGRAPHS
EASTERN AVENUE BRIDGE OVER INTERSTATE 395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER, MAINE

Top Row: BB-BEA-202: Run No. R1 20.2’ (left) to 25.2’ (right)
Top Middle Row: BB-BEA-202: Run No. R2 25.2’ (left) to 27.4’ (middle); Run No. R3 27.4’ (middle) to 29.7’ (right)

ROCK CORE PHOTOGRAPHS
EASTERN AVENUE BRIDGE OVER INTERSTATE 395/ROUTE 9 CONNECTOR
MAINEDOT WIN 018915.00
BREWER, MAINE

Top Row: HB-BE-217: Run No. R1 28.5’ (left) to 33.5’ (right)
Top Middle Row: BB-BEA-203: Run No. R1 20.0’ (left) to 25.0’ (right)
Bottom Middle Row: BB-BEA-203: Run No. R2 25.0’ (left) to 30.0’ (right)
Bottom Row: BB-BEA-201: Run No. R1 32.0’ (left) to 35.0’ (right)

Top Row: HB-BE-218: Run No. R1 22.7’ (left) to 27.0’ (middle-right); Run No. R2 27.0’ (middle-right) to 28.5’ (right)
Top Middle Row: HB-BE-218: Run No. R2 continued 28.5’ (left) to 30.2 (right)
Bottom Middle Row: BB-BEA-204: Run No. R1 21.9’ (left) to 26.9’ (right)
Bottom Row: BB-BEA-204: Run No. R2 26.9’ (left) to 31.1’ (right)

APPENDIX B

Observation Well Installation and Groundwater Monitoring Reports

HALEY ALDRICH

OBSERVATION WELL INSTALLATION REPORT

HALEY ALDRICH

Well No.	Observation Well
Boring No.	

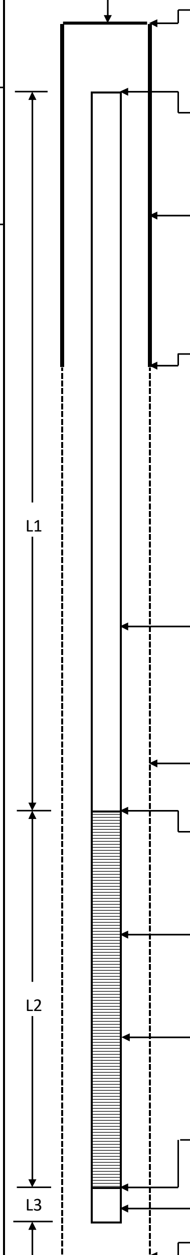
PROJECT	Eastern Avenue Bridge over Interstate 395/Rt. 9 Connector	**H&A FILE NO.**	132076-002
LOCATION	Brewer, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	N. Klausmeyer
CONTRACTOR	Northern Test Borings, Inc.	**DATE INSTALLED**	8/2/2018
DRILLER	M. Nadeau	**WATER LEVEL**	12.5 ft (depth below gs)

Ground El.	139.1 ft	**Location**	SEE PLAN	☑ **Guard Pipe**
El. Datum	NAVD 88			☐ **Roadway Box**

SOIL/ROCK CONDITIONS	**BOREHOLE BACKFILL**
Granular -FILL-	-FILTER SAND-
	2.0 ft
	-BENTONITE-
	4.0 ft
6.5 ft	
-GLACIAL TILL-	-FILTER SAND-
15.0	**15.0**

(Bottom of Exploration)

(Numbers refer to depth from ground surface in feet)



Type of protective cover	Locking Cap	
Height of top of guard pipe above ground surface	2.0	ft
Height of top of riser pipe above ground surface	1.9	ft
Type of protective casing:	Steel Guard Pipe	
Length	7.0	ft
Inside Diameter	4.0	in
Depth of bottom of guard pipe	3.0	ft

Type of Seals	**Top of Seal (ft)**	**Thickness (ft)**
Concrete	--	--
Bentonite Seal	3.0	1.0

Type of riser pipe:	Sch. 40 PVC	
Inside diameter of riser pipe	2.0	in
Type of backfill around riser	Bentonite/Sand	
Diameter of borehole	4.0	in
Depth to top of well screen	5.0	ft
Type of screen	Sch. 40 PVC	
Screen gauge or size of openings	0.010	in
Diameter of screen	2.0	in
Type of backfill around screen	No. 2 Filter Sand	
Depth of bottom of well screen	15.0	ft
Bottom of Silt trap	15.2	ft
Depth of bottom of borehole	15.2	ft

(Not to Scale)

6.9 ft + 10.0 ft + 0.2 ft = 17.1 ft

Riser Pay Length (L1) + Length of screen (L2) + Length of silt trap (L3) = Pay length

COMMENTS:

GROUNDWATER MONITORING REPORT

OW/PZ NUMBER
Observation Well

PROJECT	Eastern Avenue Bridge over Interstate 395/Route 9 Connector	**H&A FILE NO.**	132076-002
LOCATION	Brewer, Maine	**PROJECT MGR.**	B. Steinert
CLIENT	Maine Department of Transportation	**FIELD REP.**	N. Klausmeyer
CONTRACTOR	Northern Test Borings, Inc.	**DATE**	4/20/2020

ELEVATION OF REFERENCE POINT (ft) 139.1 **REFERENCE POINT:** Ground Surface ☑ PVC ☐ Other ☐

Date	Time	Elapsed Time (days)	Depth of Water from Reference Point (ft)	Elevation of Water (ft)	Remarks	Read By
8/6/2018	10:15 AM	0	10.9	128.2	Intial Reading - 80s & Sunny	NLK
8/10/2018	11:10 AM	4	10.2	128.9	80s & Sunny	NLK
8/13/2018	12:30 PM	7	Dry	Dry	70s & Overcast	NLK
9/4/2018	2:13 PM	29	12.6	126.5	80s & Sunny	NLK
9/10/2018	12:49 PM	35	13.2	125.9	60s & Cloudy	NLK
10/5/2018	11:18 AM	60	15.0	124.1	50s & Sunny	MLS
10/16/2018	8:00 AM	71	Dry	Dry	40s & Sunny	HH
11/8/2018	8:00 AM	94	5.1	134.0	50's & Sunny; after several days of rain	HH
12/19/2018	1:00 PM	135	5.3	133.8	20s & Sunny w/ 5" of snow	NLK
10/30/2019	12:34 PM	450	5.5	133.7	50's and cloudy	HH
12/12/2019	12:14 PM	493	3.7	135.4	20's and sunny	JKF
4/4/2020	10:41 AM	607	1.8	137.3	50's and sunny	HH
6/2/2020	10:55 AM	666	6.6	132.5	50's and sunny	JF
2/1/2021	6:30 AM	910	4.1	135.0	0's and cloudy	JF
3/24/2021	10:15 AM	961	3.6	135.5	40's and sunny	JF
4/19/2021	2:44 PM	987	3.2	135.9	50's and sunny	JF
5/12/2021	10:20 AM	1010	3.0	136.1	50's and cloudy	JF

G:\PROJECTS\132076 - brewer eddington\002 - Exploration + Laboratory Testing Programs\Field\Ground Water Monitoring Reports

APPENDIX C

Laboratory Test Results

Client:	Haley & Aldrich, Inc.				
Project:	Rt 9/I-395 Eastern Ave Bridge				
Location:	Brewer and Eddington, ME			Project No:	GTX-308855
Boring ID:	---	Sample Type:	---	Tested By:	GA
Sample ID:	---	Test Date:	10/03/18	Checked By:	emm
Depth :	---	Test Id:	474421		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BEA-101	3D	5-7 ft	Moist, grayish brown silty sand with gravel	7.6
BB-BEA-101	4D	10-11.4 ft	Moist, grayish brown silty sand with gravel	8.5
BB-BEA-101	5D	15-17 ft	Moist, grayish brown silty sand with gravel	11.3
BB-BEA-102	4D	10-12 ft	Moist, dark grayish brown sandy silt	10.9
BB-BEA-102	5D	15-17 ft	Moist, dark grayish brown silty gravel with sand	9.7

Notes: Temperature of Drying : 110º Celsius

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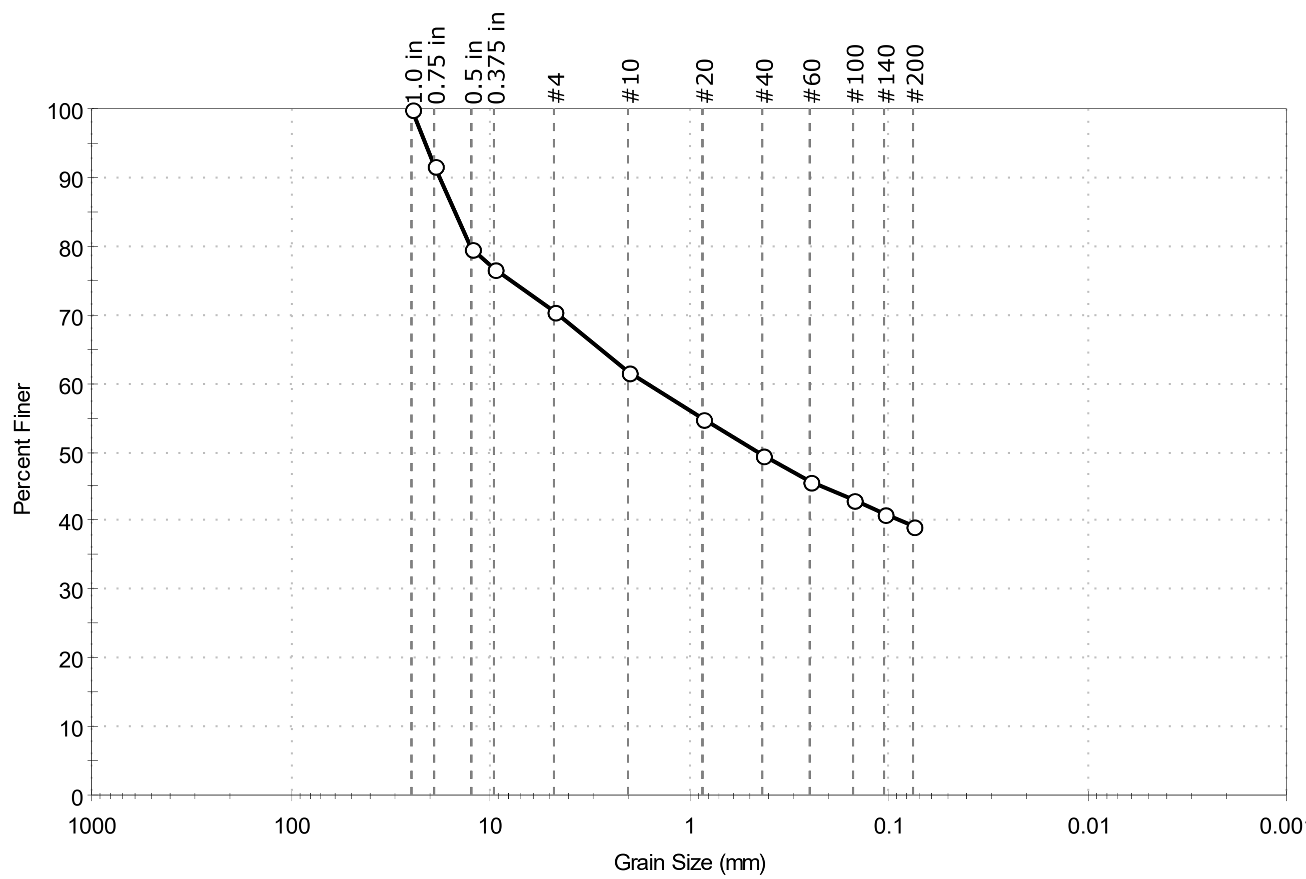
Client:	Haley & Aldrich, Inc.		
Project:	Rt 9/I-395 Eastern Ave Bridge		
Location:	Brewer and Eddington, ME	Project No:	GTX-308855
Boring ID: BB-BEA-101	Sample Type: jar	Tested By:	GA
Sample ID: 3D	Test Date: 10/03/18	Checked By:	emm
Depth : 5-7 ft	Test Id: 474412		
Test Comment:	---		
Visual Description:	Moist, grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	29.5	31.3	39.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	92		
0.5 in	12.50	80		
0.375 in	9.50	77		
#4	4.75	70		
#10	2.00	62		
#20	0.85	55		
#40	0.42	50		
#60	0.25	46		
#100	0.15	43		
#140	0.11	41		
#200	0.075	39		

Coefficients

D_{85} =15.0398 mm D_{30} =N/A

D_{60} =1.6089 mm D_{15} =N/A

D_{50} =0.4451 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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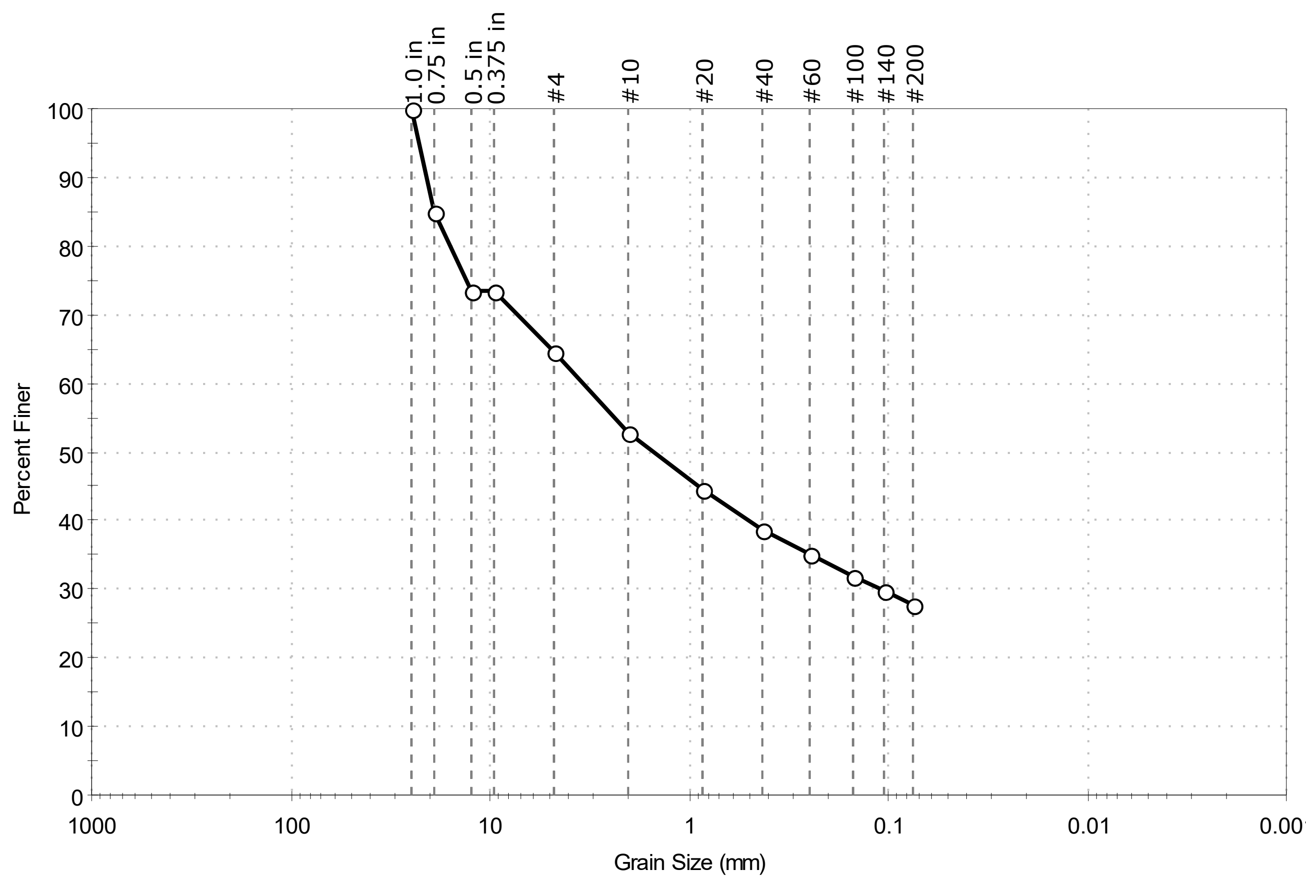
Client:	Haley & Aldrich, Inc.				
Project:	Rt 9/I-395 Eastern Ave Bridge				
Location:	Brewer and Eddington, ME			Project No:	GTX-308855
Boring ID:	BB-BEA-101	Sample Type:	jar	Tested By:	GA
Sample ID:	4D	Test Date:	10/03/18	Checked By:	emm
Depth :	10-11.4 ft	Test Id:	474413		

Test Comment: ---
Visual Description: Moist, grayish brown silty sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	35.4	36.8	27.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	74		
0.375 in	9.50	74		
#4	4.75	65		
#10	2.00	53		
#20	0.85	44		
#40	0.42	39		
#60	0.25	35		
#100	0.15	32		
#140	0.11	30		
#200	0.075	28		

Coefficients

D_{85} =19.0103 mm
D_{30} =0.1113 mm
D_{60} =3.3917 mm
D_{15} =N/A
D_{50} =1.4948 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

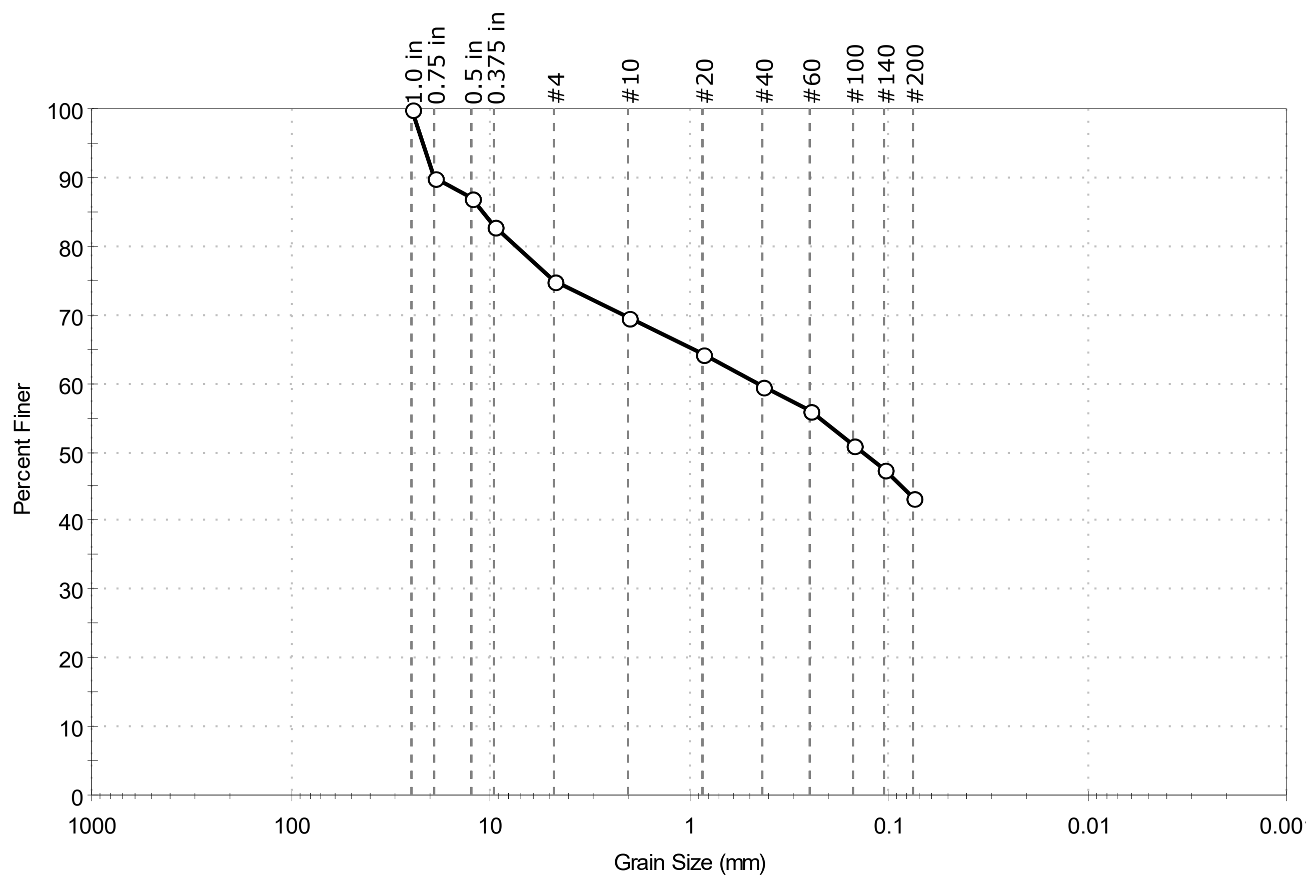
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/4/2018 11:22:42 AM

Client:	Haley & Aldrich, Inc.		
Project:	Rt 9/I-395 Eastern Ave Bridge		
Location:	Brewer and Eddington, ME	Project No:	GTX-308855
Boring ID:	BB-BEA-101	Sample Type: jar	Tested By: GA
Sample ID:	5D	Test Date: 10/03/18	Checked By: emm
Depth :	15-17 ft	Test Id: 474414	
Test Comment:	---		
Visual Description:	Moist, grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.0	31.6	43.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	90		
0.5 in	12.50	87		
0.375 in	9.50	83		
#4	4.75	75		
#10	2.00	70		
#20	0.85	64		
#40	0.42	60		
#60	0.25	56		
#100	0.15	51		
#140	0.11	47		
#200	0.075	43		

Coefficients

D_{85} =10.9699 mm D_{30} =N/A

D_{60} =0.4428 mm D_{15} =N/A

D_{50} =0.1358 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

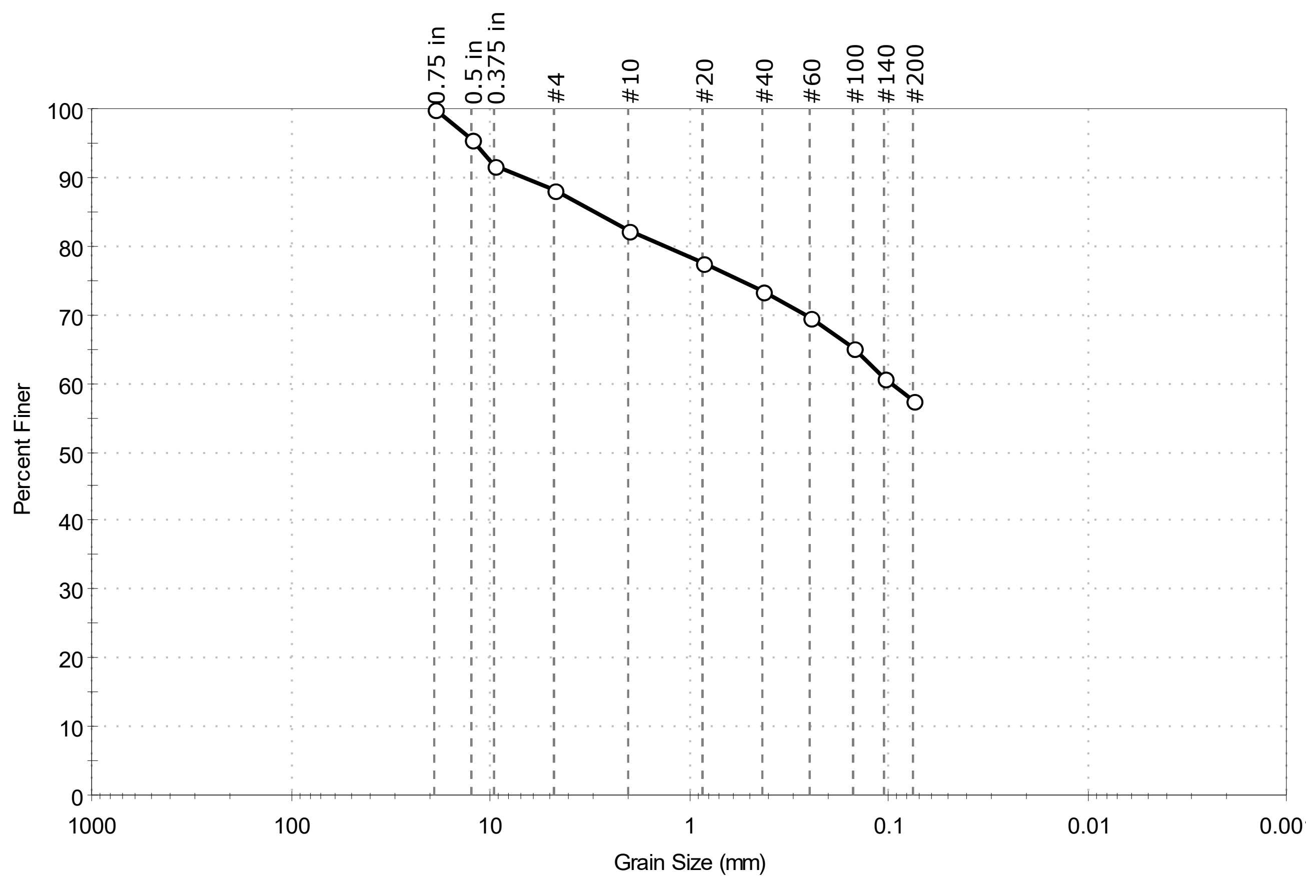
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/4/2018 11:22:50 AM

Client:	Haley & Aldrich, Inc.		
Project:	Rt 9/I-395 Eastern Ave Bridge		
Location:	Brewer and Eddington, ME	Project No:	GTX-308855
Boring ID: BB-BEA-102	Sample Type: jar	Tested By:	GA
Sample ID: 4D	Test Date: 10/03/18	Checked By:	emm
Depth : 10-12 ft	Test Id: 474415		
Test Comment:	---		
Visual Description:	Moist, dark grayish brown sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.8	30.6	57.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	92		
#4	4.75	88		
#10	2.00	82		
#20	0.85	78		
#40	0.42	73		
#60	0.25	70		
#100	0.15	65		
#140	0.11	61		
#200	0.075	58		

Coefficients

D_{85} =2.9612 mm D_{30} =N/A

D_{60} =0.0973 mm D_{15} =N/A

D_{50} =N/A D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

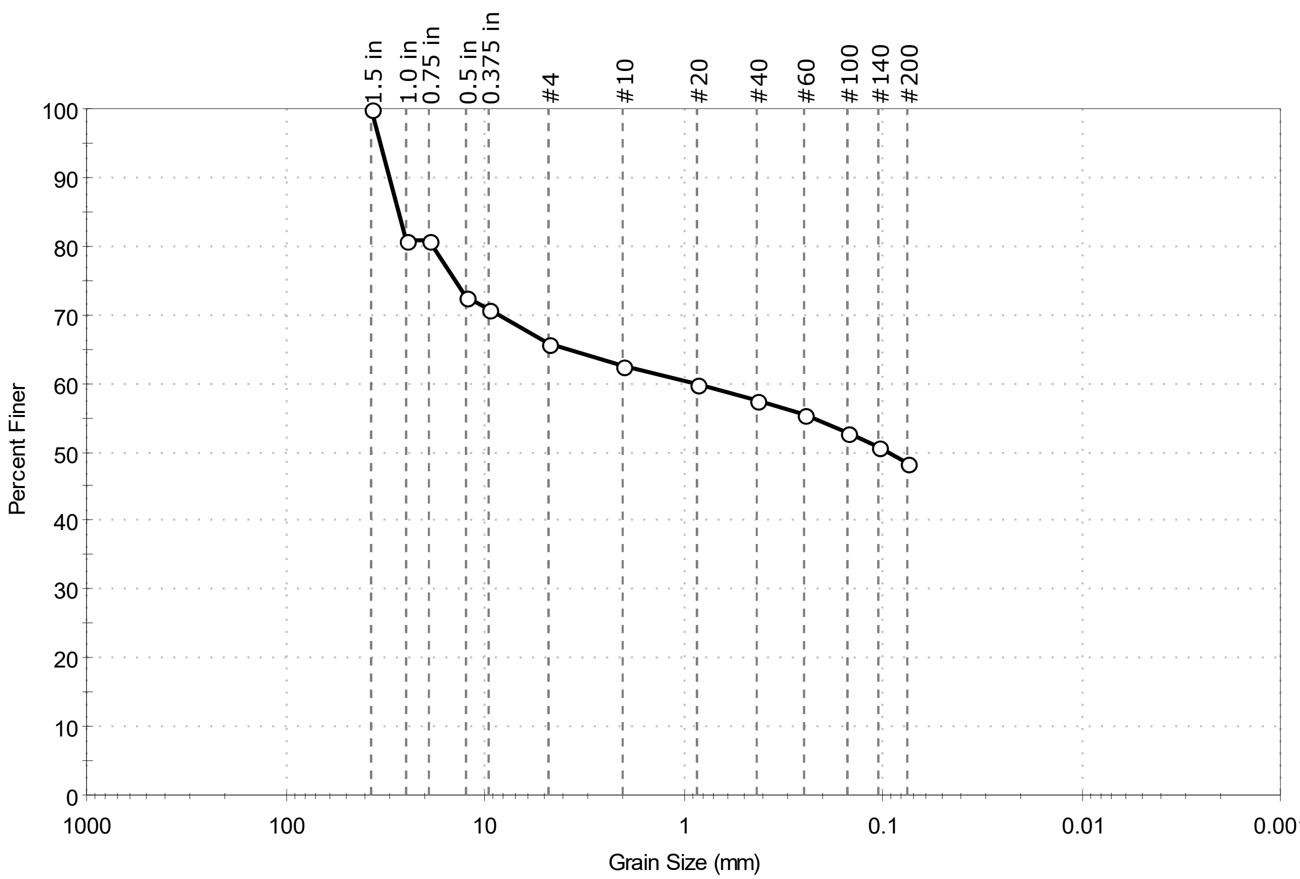
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/4/2018 11:22:51 AM

Client:	Haley & Aldrich, Inc.
Project:	Rt 9/I-395 Eastern Ave Bridge
Location:	Brewer and Eddington, ME
Project No:	GTX-308855
Boring ID:	BB-BEA-102
Sample Type:	jar
Tested By:	GA
Sample ID:	5D
Test Date:	10/03/18
Checked By:	emm
Depth :	15-17 ft
Test Id:	474416
Test Comment:	---
Visual Description:	Moist, dark grayish brown silty gravel with sand
Sample Comment:	---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.2	17.5	48.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1.0 in	25.00	81		
0.75 in	19.00	81		
0.5 in	12.50	72		
0.375 in	9.50	71		
#4	4.75	66		
#10	2.00	62		
#20	0.85	60		
#40	0.42	57		
#60	0.25	55		
#100	0.15	53		
#140	0.11	51		
#200	0.075	48		

Coefficients

D_{85} =27.3552 mm　D_{30} =N/A

D_{60} =0.8581 mm　D_{15} =N/A

D_{50} =0.0939 mm　D_{10} =N/A

C_u =N/A　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

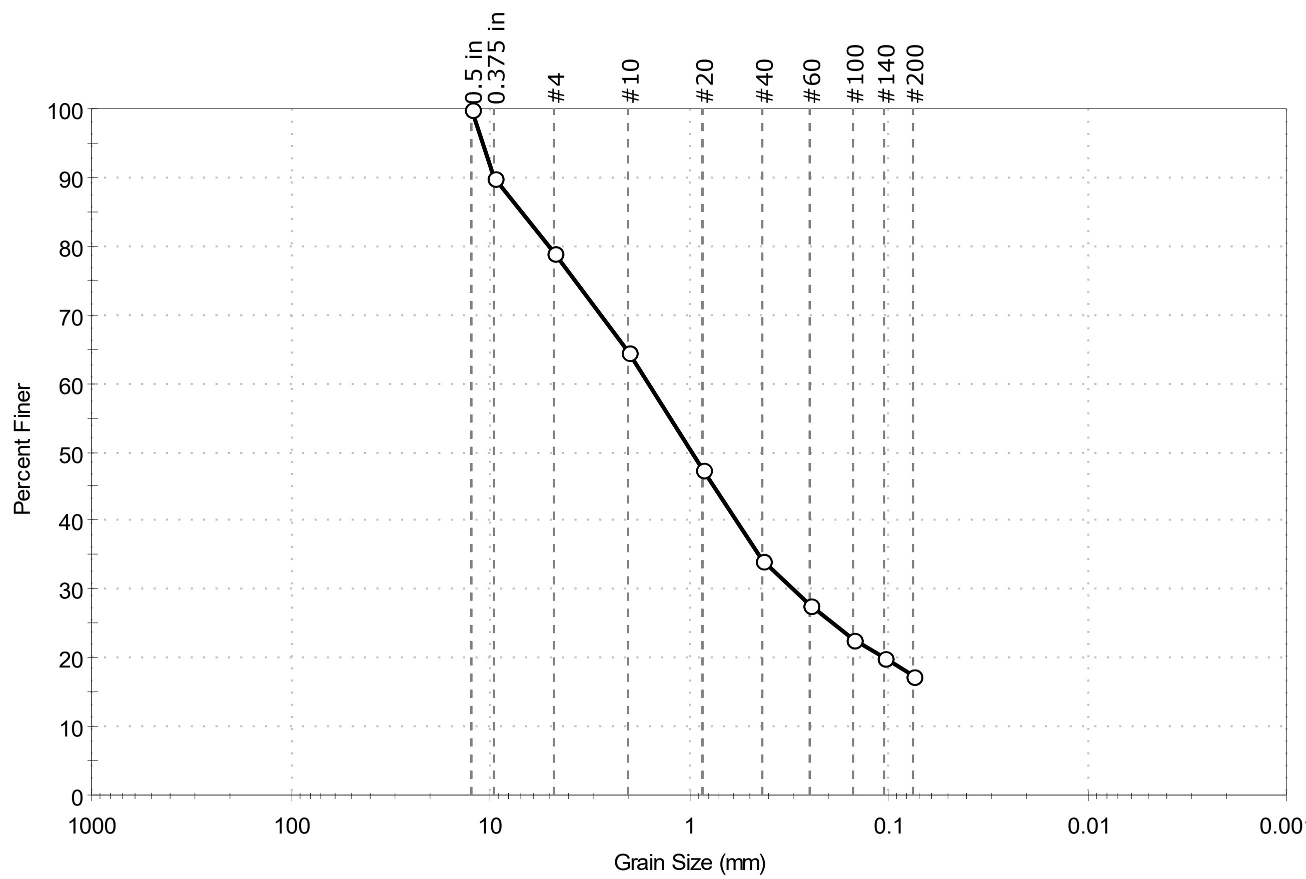
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Hwy, Brewer-Eddington		
Location:	Brewer, ME	Project No:	GTX-313370
Boring ID:	HB-BE-217	Sample Type: jar	Tested By: GA
Sample ID:	2D	Test Date: 03/29/21	Checked By: emm
Depth :	5-6.8	Test Id: 613870	
Test Comment:	---		
Visual Description:	Moist, yellowish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.9	61.8	17.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	90		
#4	4.75	79		
#10	2.00	65		
#20	0.85	47		
#40	0.42	34		
#60	0.25	28		
#100	0.15	23		
#140	0.11	20		
#200	0.075	17		

Coefficients

D_{85} =6.8992 mm
D_{30} =0.3021 mm
D_{60} =1.5938 mm
D_{15} =N/A
D_{50} =0.9676 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

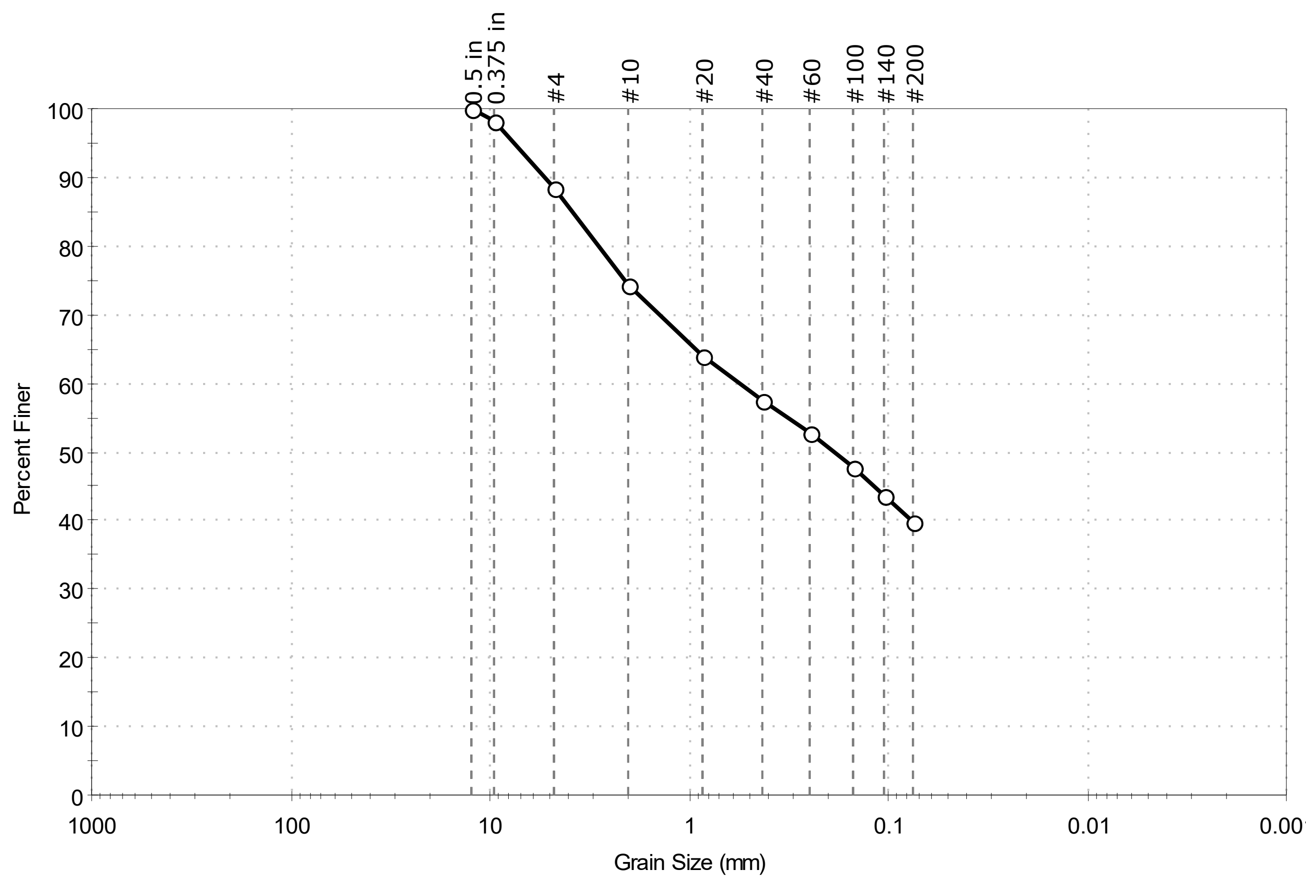
Sand/Gravel Hardness : HARD

Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Hwy, Brewer-Eddington		
Location:	Brewer, ME	Project No:	GTX-313370

Boring ID:	HB-BE-217	Sample Type:	jar	Tested By:	GA
Sample ID:	3D	Test Date:	03/29/21	Checked By:	emm
Depth :	10-12	Test Id:	613871		

Test Comment: ---
Visual Description: Moist, brown silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.5	48.8	39.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	89		
#10	2.00	74		
#20	0.85	64		
#40	0.42	57		
#60	0.25	53		
#100	0.15	48		
#140	0.11	44		
#200	0.075	40		

Coefficients

D_{85} =3.8378 mm D_{30} =N/A
D_{60} =0.5524 mm D_{15} =N/A
D_{50} =0.1894 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM   N/A

AASHTO   Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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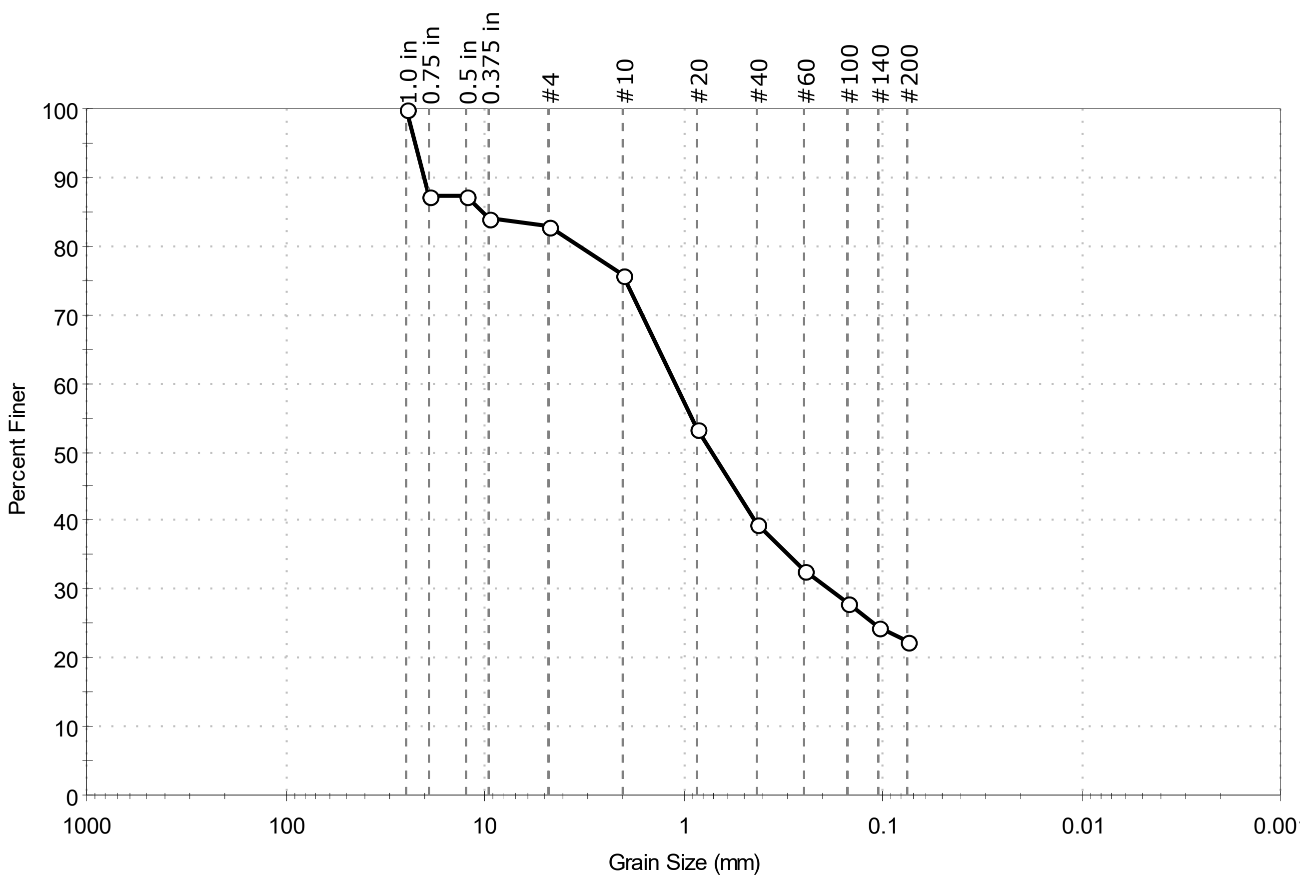
Client: Haley & Aldrich, Inc.			
Project: I-395/Rte 9 Connector Hwy, Brewer-Eddington			
Location: Brewer, ME		Project No: GTX-313370	
Boring ID: HB-BE-217	Sample Type: jar	Tested By: GA	
Sample ID: 4D	Test Date: 03/29/21	Checked By: emm	
Depth : 15-17	Test Id: 613872		

Test Comment: ---
Visual Description: Moist, olive brown clayey sand with gravel
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.1	60.6	22.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	87		
0.5 in	12.50	87		
0.375 in	9.50	84		
#4	4.75	83		
#10	2.00	76		
#20	0.85	53		
#40	0.42	40		
#60	0.25	33		
#100	0.15	28		
#140	0.11	25		
#200	0.075	22		

Coefficients

D_{85} =10.2490 mm | D_{30} =0.1860 mm
D_{60} =1.0932 mm | D_{15} =N/A
D_{50} =0.7175 mm | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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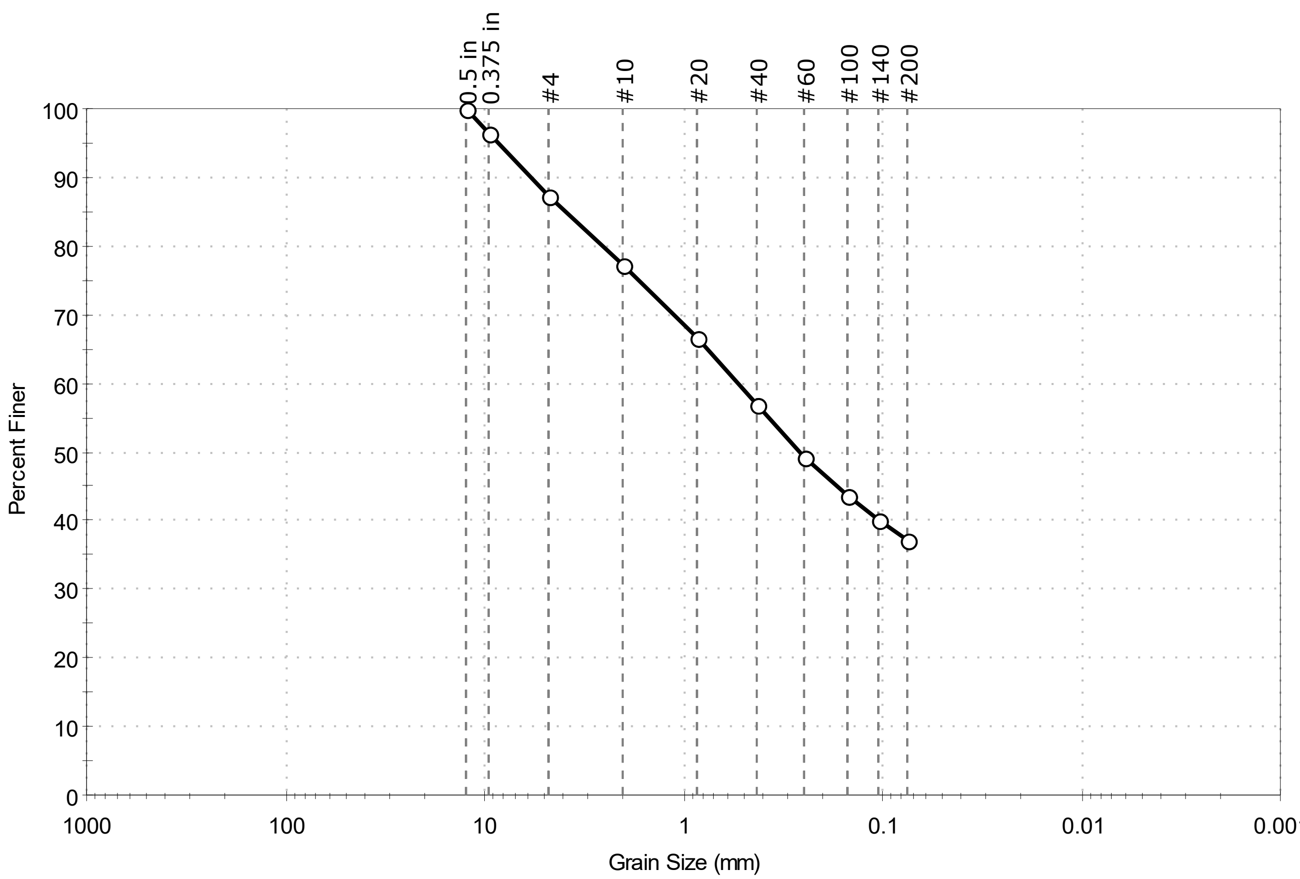
Client: Haley & Aldrich, Inc.			
Project: I-395/Rte 9 Connector Hwy, Brewer-Eddington			
Location: Brewer, ME		Project No:	GTX-313370
Boring ID: HB-BE-217	Sample Type: jar	Tested By:	GA
Sample ID: 5D	Test Date: 03/29/21	Checked By:	emm
Depth : 20-22	Test Id: 613873		
Test Comment: ---			
Visual Description: Moist, grayish brown clayey sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.7	50.1	37.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	96		
#4	4.75	87		
#10	2.00	77		
#20	0.85	67		
#40	0.42	57		
#60	0.25	49		
#100	0.15	44		
#140	0.11	40		
#200	0.075	37		

Coefficients

D_{85} =3.9135 mm D_{30} =N/A

D_{60} =0.5275 mm D_{15} =N/A

D_{50} =0.2613 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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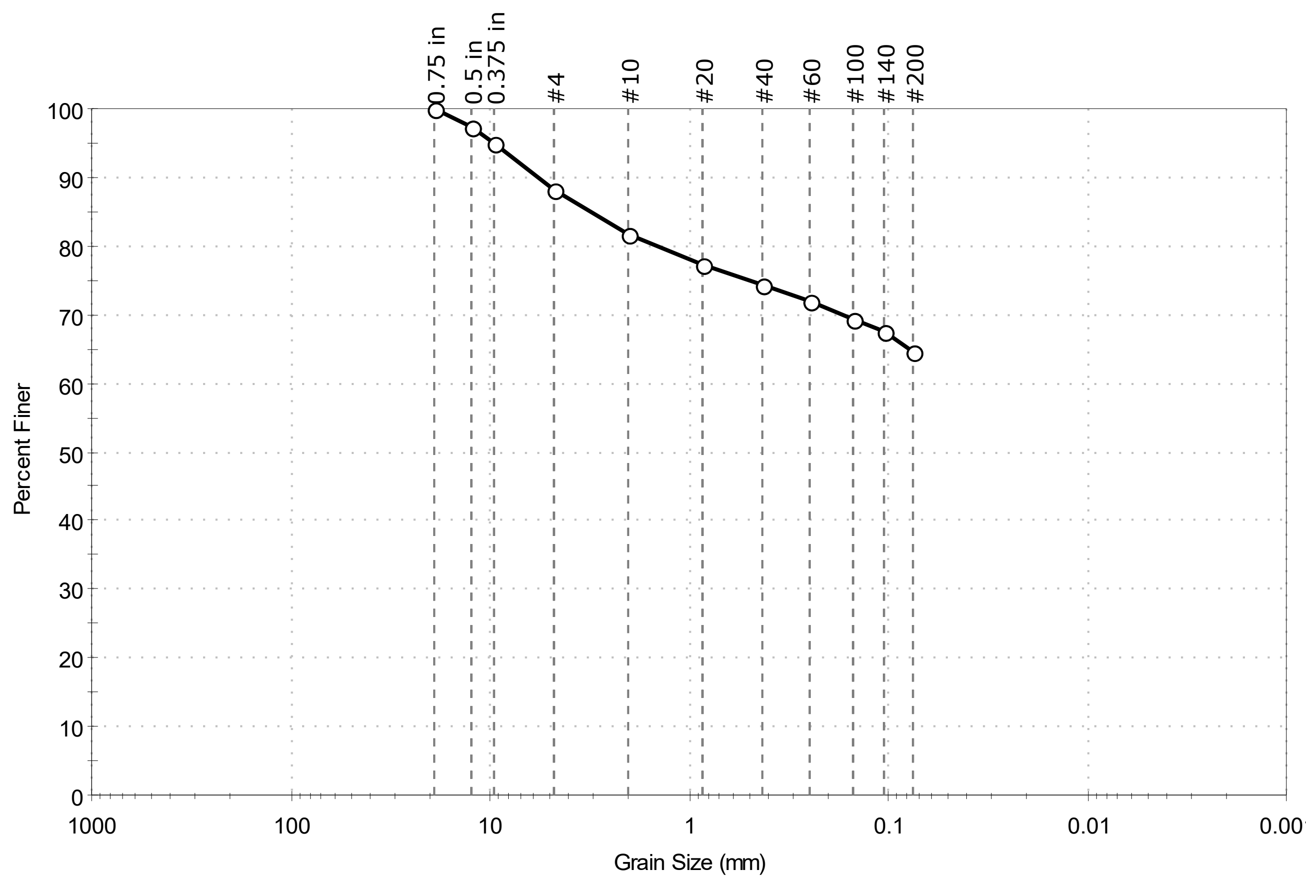
Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Hwy, Brewer-Eddington		
Location:	Brewer, ME	Project No:	GTX-313370
Boring ID: HB-BE-218	Sample Type: jar	Tested By:	ckg
Sample ID: 2D	Test Date: 03/25/21	Checked By:	emm
Depth : 5-7	Test Id: 613874		
Test Comment:	---		
Visual Description:	Moist, light olive brown sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	11.7	23.6	64.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	95		
#4	4.75	88		
#10	2.00	82		
#20	0.85	77		
#40	0.42	74		
#60	0.25	72		
#100	0.15	69		
#140	0.11	67		
#200	0.075	65		

Coefficients

D_{85} =3.0778 mm D_{30} =N/A
D_{60} =N/A D_{15} =N/A
D_{50} =N/A D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

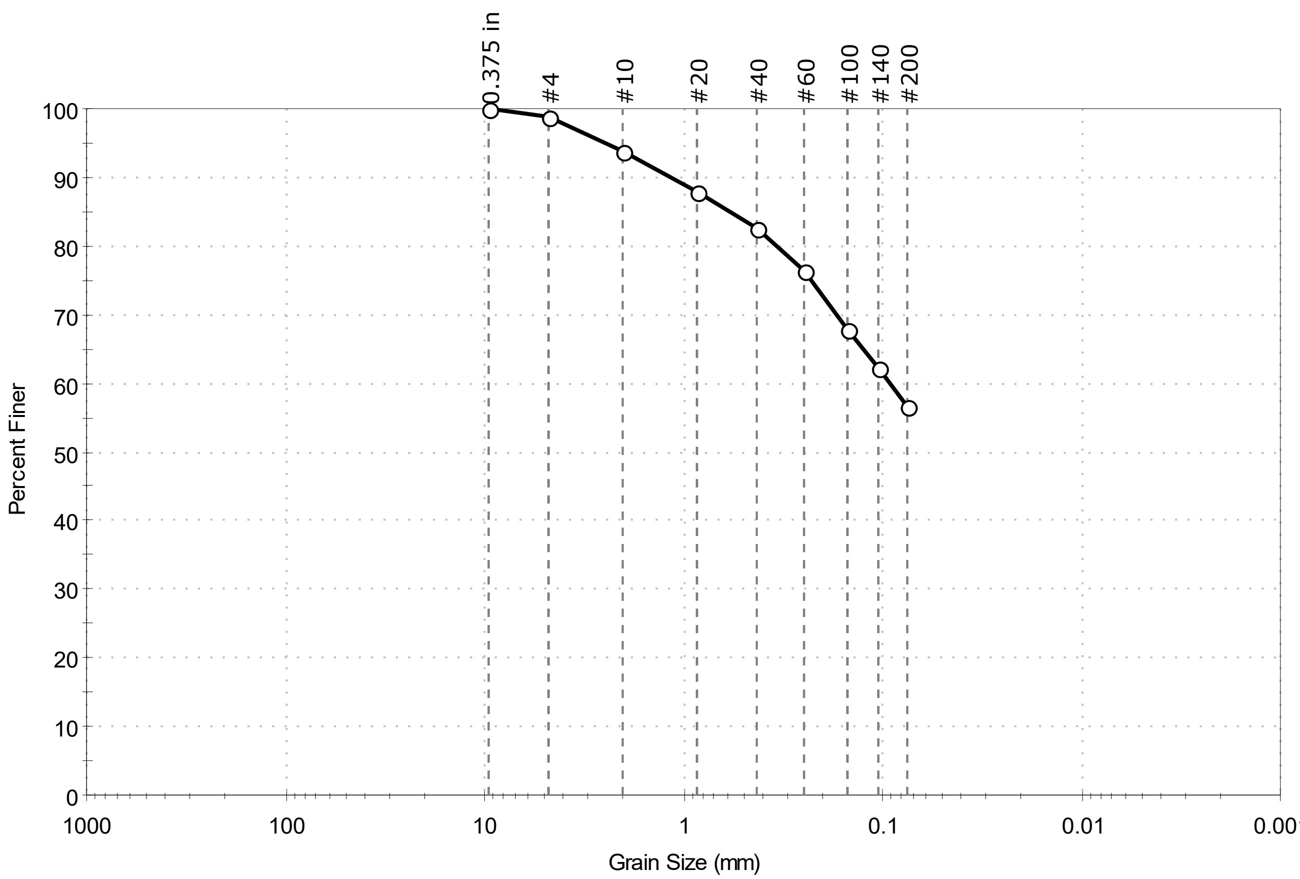
printed 4/9/2021 5:12:27 PM

GeoTesting EXPRESS

Client: Haley & Aldrich, Inc.			
Project: I-395/Rte 9 Connector Hwy, Brewer-Eddington			
Location: Brewer, ME		Project No:	GTX-313370
Boring ID: HB-BE-218	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 03/24/21	Checked By:	emm
Depth : 10-12	Test Id: 613875		

Test Comment: ---
Visual Description: Moist, brown sandy silt
Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.1	42.1	56.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	94		
#20	0.85	88		
#40	0.42	83		
#60	0.25	76		
#100	0.15	68		
#140	0.11	62		
#200	0.075	57		

Coefficients

D_{85} =0.5901 mm
D_{30} =N/A
D_{60} =0.0921 mm
D_{15} =N/A
D_{50} =N/A
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

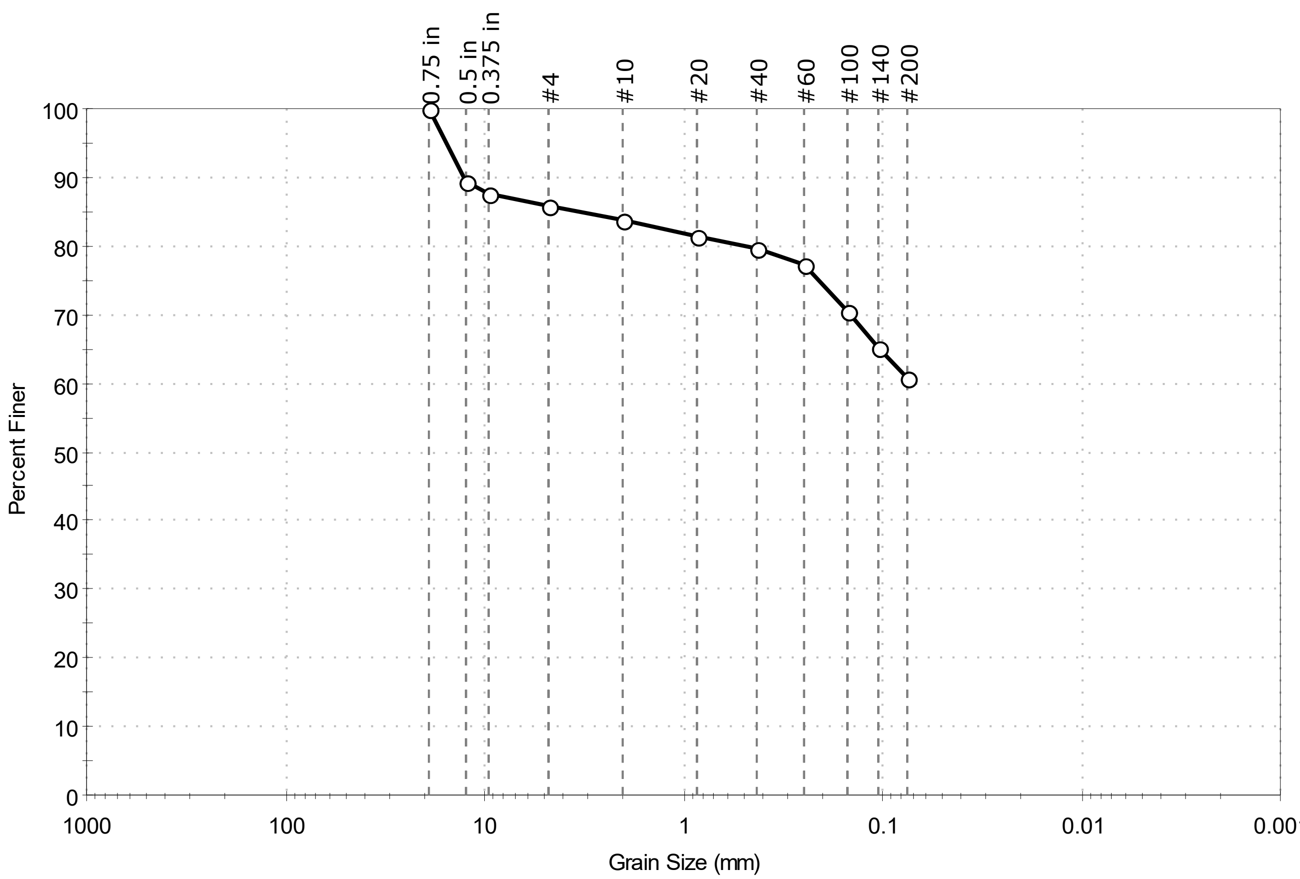
Sand/Gravel Hardness : HARD

printed 4/9/2021 5:12:28 PM

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.		
Project:	I-395/Rte 9 Connector Hwy, Brewer-Eddington		
Location:	Brewer, ME	Project No:	GTX-313370
Boring ID:	HB-BE-218	Sample Type: jar	Tested By: ckg
Sample ID:	4D	Test Date: 03/24/21	Checked By: emm
Depth :	15-17	Test Id: 613876	
Test Comment:	---		
Visual Description:	Moist, grayish brown sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.1	25.2	60.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	90		
0.375 in	9.50	88		
#4	4.75	86		
#10	2.00	84		
#20	0.85	81		
#40	0.42	80		
#60	0.25	77		
#100	0.15	70		
#140	0.11	65		
#200	0.075	61		

Coefficients

D_{85} = 3.2272 mm D_{30} = N/A

D_{60} = N/A D_{15} = N/A

D_{50} = N/A D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

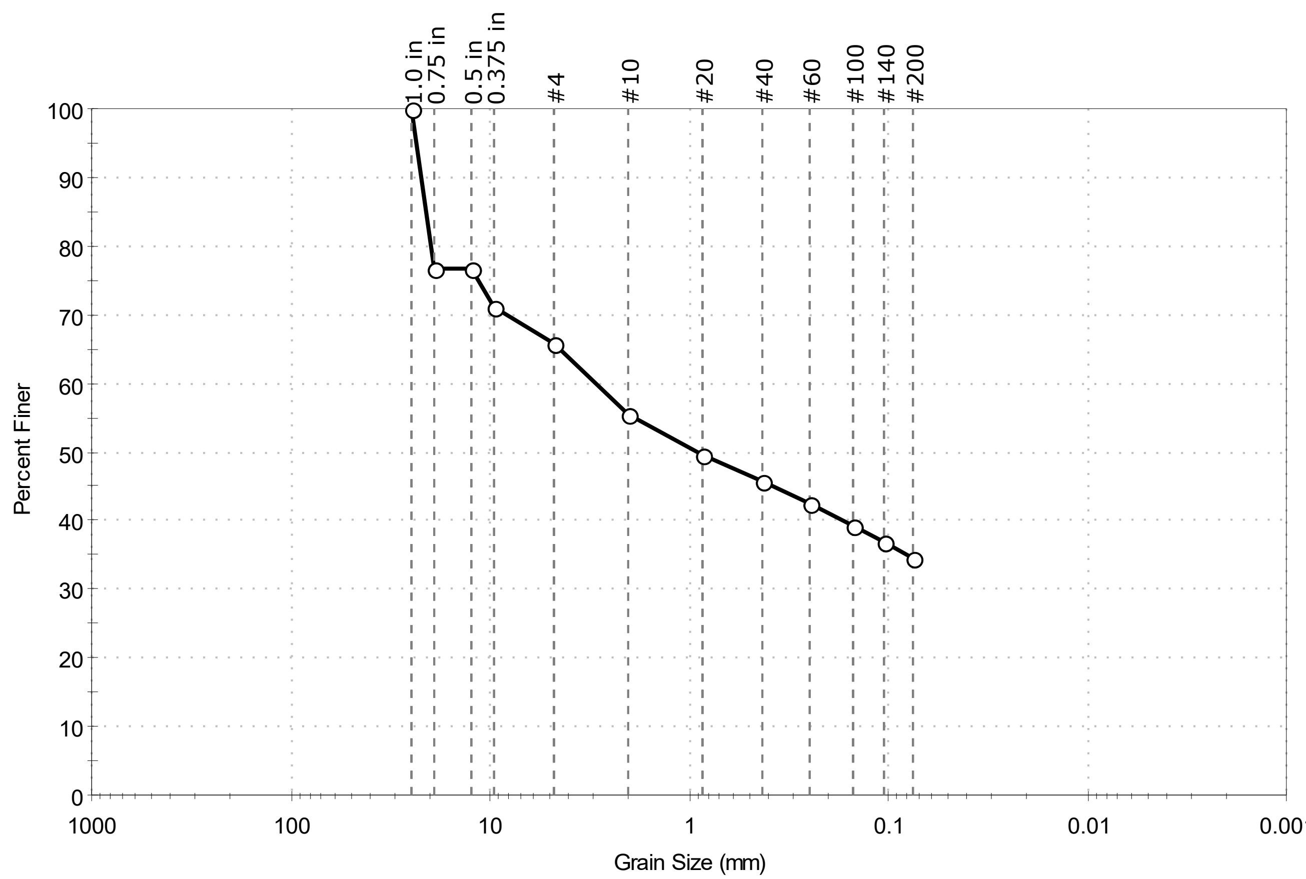
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 4/9/2021 5:12:29 PM

Client:	Haley & Aldrich, Inc.			
Project:	I-395/Rte 9 Connector Hwy, Brewer-Eddington			
Location:	Brewer, ME		Project No:	GTX-313370
Boring ID:	HB-BE-218	Sample Type: jar	Tested By:	GA
Sample ID:	5D	Test Date: 03/29/21	Checked By:	emm
Depth :	20-22	Test Id: 613877		
Test Comment:	---			
Visual Description:	Moist, grayish brown clayey gravel with sand			
Sample Comment:	---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.3	31.3	34.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	77		
0.5 in	12.50	77		
0.375 in	9.50	71		
#4	4.75	66		
#10	2.00	55		
#20	0.85	50		
#40	0.42	46		
#60	0.25	42		
#100	0.15	39		
#140	0.11	37		
#200	0.075	34		

Coefficients

D_{85} =20.9485 mm D_{30} =N/A
D_{60} =2.9426 mm D_{15} =N/A
D_{50} =0.8887 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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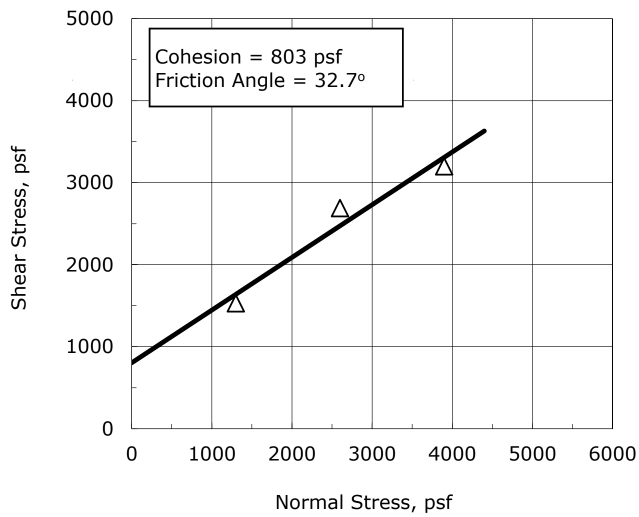
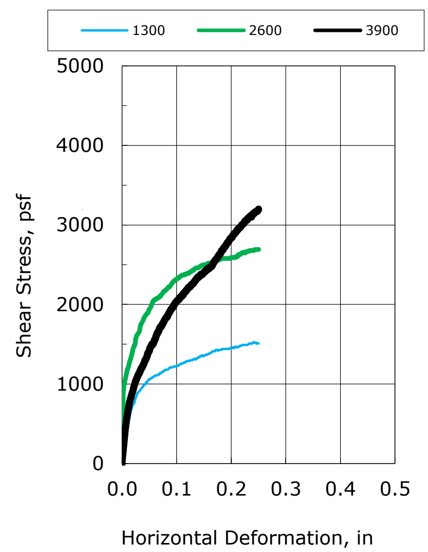
Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Hwy, Brewer-Eddington
Project Location:	Brewer, ME
GTX #:	313370
Test Date:	04/05/21
Tested By:	mp
Checked By:	njh
Boring ID:	HB-BE-218
Sample ID:	2D, 3D, 4D
Depth, ft:	5-7
Visual Description:	Moist, grayish brown sandy silt

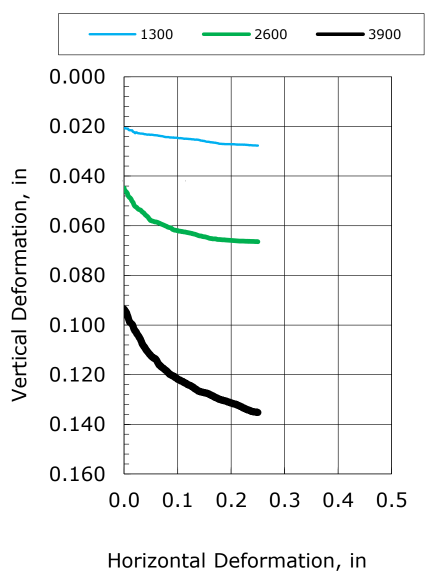
Direct Shear Test of Soils Under Consolidated Drained Conditions by ASTM D3080





Test No.:	DS-1	DS-2	DS-3
Initial Diameter, in:	2.5	2.5	2.5
Initial Height, in:	1.0	1.0	1.0
Initial Mass, grams:	168	168	168
Initial Dry Density, pcf:	115.4	115.4	115.4
Initial Moisture Content, %:	12.7	12.7	12.7
Initial Bulk Density, pcf:	130.0	130.0	130.0
Initial Degree of Saturation:	74.2	74.2	74.2
Initial Void Ratio:	0.46	0.46	0.46
Final Dry Density, pcf:	118.7	123.6	133.4
Final Moisture Content, %:	16.5	17.8	17.6
Final Bulk Density, pcf:	138.2	145.6	156.8
Normal Stress, psf:	1300	2600	3900
Maximum Shear Stress, psf:	1530	2690	3200
Shear Rate, in/min:	0.0004	0.0004	0.0004

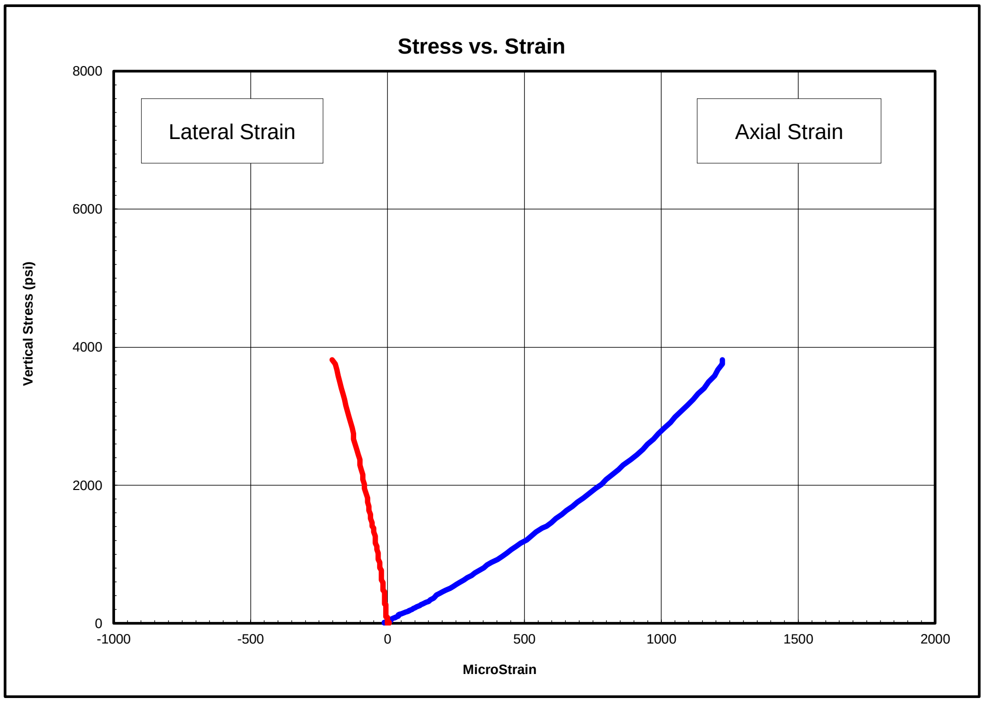
Sample Type:	reconstituted
Estimated Specific Gravity:	2.70
Liquid Limit:	---
Plastic Limit:	---
Plasticity Index:	---
% Passing #200 sieve:	---
Soil Classification:	---
Group Symbol:	---



Notes:
Material greater than #5 sieve screened out of sample prior to testing
Moisture content obtained before shear from sample trimmings
Moisture Content determined by ASTM D2216
Target Compaction: 130 pcf at the as-recieved moisture content. Values specified by client.
Values for cohesion and friction angle determined from best-fit straight line to the data for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site-specific conditions.
"---" indicates testing required to determine these values was not requested.

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	10/1/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BEA-101
Sample ID:	R1
Depth, ft:	28.7-29.6
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 3,818 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
400-1400	2,510,000	0.11
1400-2400	3,130,000	0.16
2400-3400	3,940,000	0.25

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.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/I-395 Eastern Ave Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308855		
Boring ID:	BB-BEA-101		
Sample ID:	R1		
Depth:	28.7-29.6 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.09	4.09	4.09
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	557.54		
Bulk Density, lb/ft^3	168	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **NO**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	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	0.00020	0.00030	0.00030	0.00030	0.00030

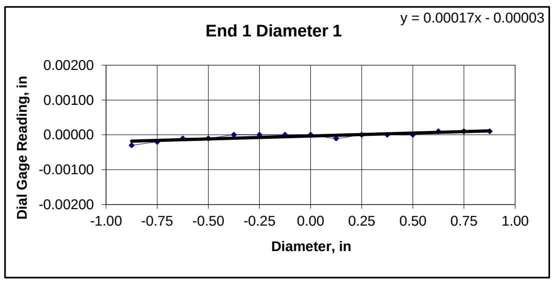
Difference between max and min readings, in:
0° = 0.00040 90° = 0.00090

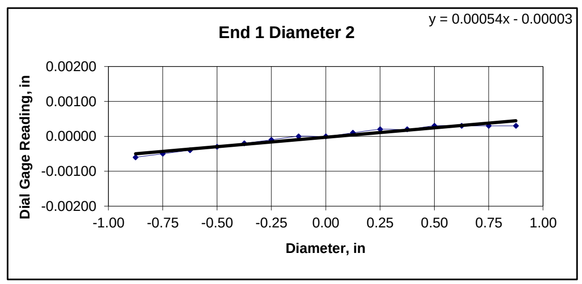
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00030	-0.00030	-0.00030

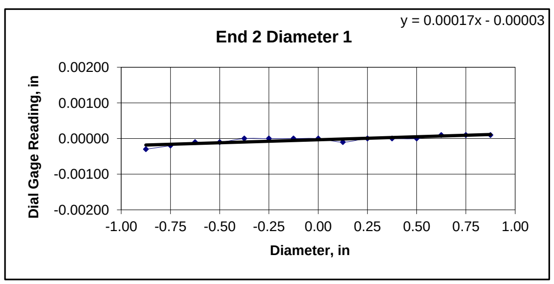
Difference between max and min readings, in:
0° = 0.0004 90° = 0.0009

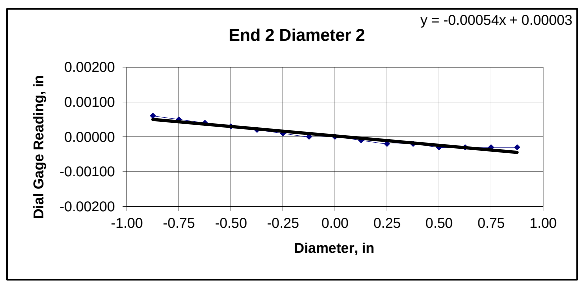
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00045

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00017
	Angle of Best Fit Line:	0.00966
End 2:	Slope of Best Fit Line	0.00017
	Angle of Best Fit Line:	0.00966
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00054
	Angle of Best Fit Line:	0.03094
End 2:	Slope of Best Fit Line	0.00054
	Angle of Best Fit Line:	0.03094
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

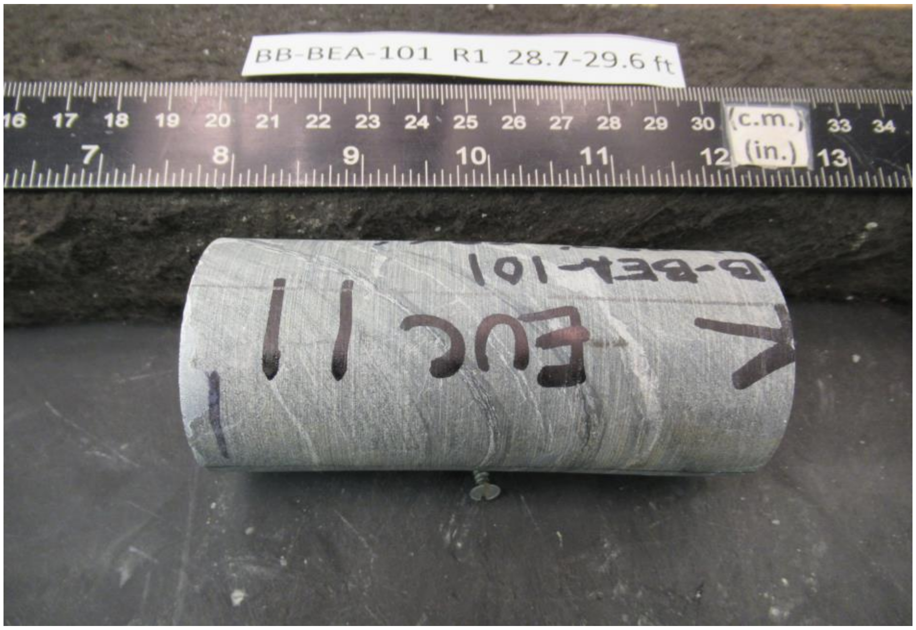
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00040	1.980	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00090	1.980	0.00045	0.026	YES
END 2					
Diameter 1, in	0.00040	1.980	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00090	1.980	0.00045	0.026	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-101
Sample ID:	R1
Depth, ft:	28.7-29.6

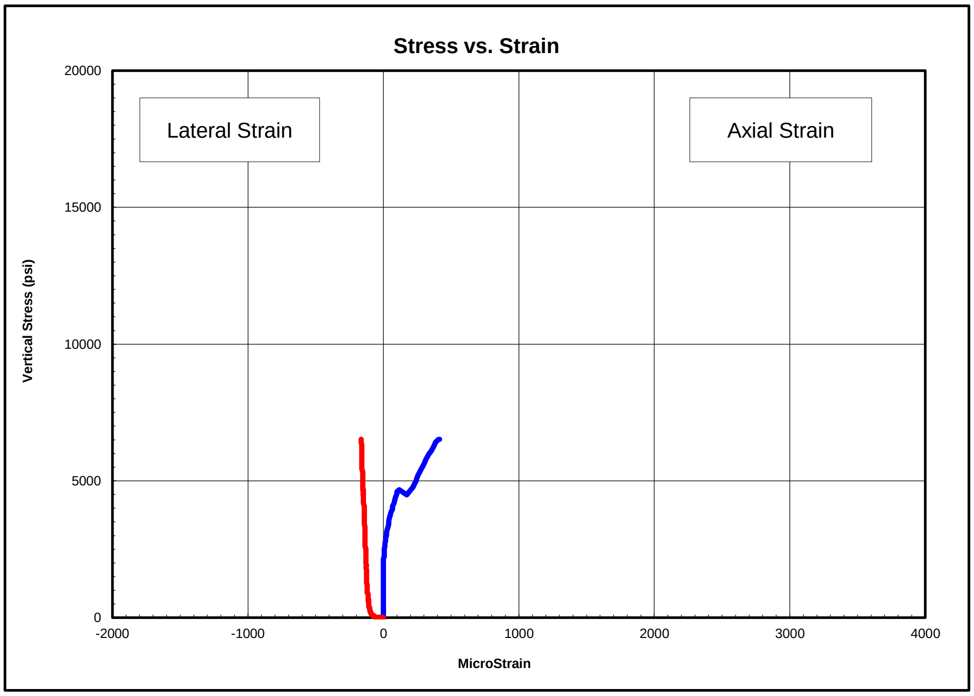


After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	10/1/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BEA-101
Sample ID:	R2
Depth, ft:	31.6-32.2
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 6,520 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2400	---	---
2400-4100	25,400,000	0.21
4100-5900	5,680,000	0.05

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/I-395 Eastern Ave Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308855		
Boring ID:	BB-BEA-101		
Sample ID:	R2		
Depth:	31.6-32.2 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.33	4.34	4.34
Specimen Diameter, in:	1.98	1.99	1.99
Specimen Mass, g:	593.79		
Bulk Density, lb/ft^3	168		
Length to Diameter Ratio:	2.2		

Minimum Diameter Tolerence Met? YES

Length to Diameter Ratio Tolerance Met? YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010

Difference between max and min readings, in:

0° = 0.00040 90° = 0.00030

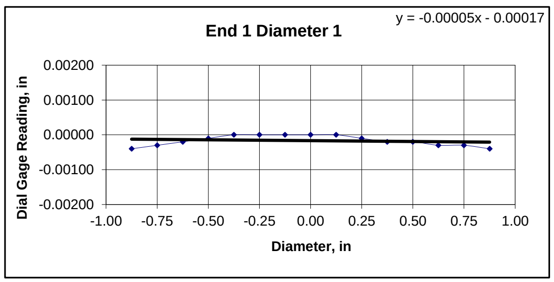
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010

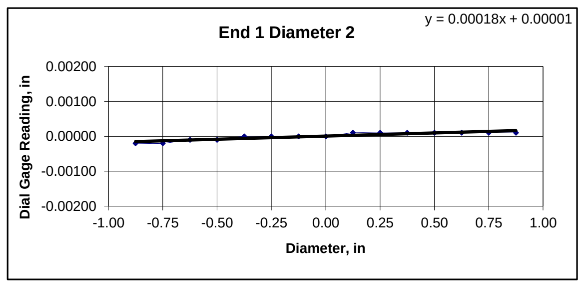
Difference between max and min readings, in:

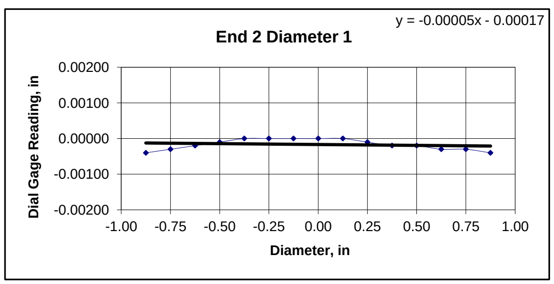
0° = 0.0004 90° = 0.0003

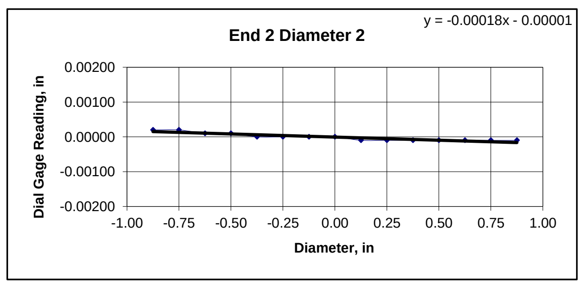
Maximum difference must be < 0.0020 in. Difference = ± 0.00020

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00005
	Angle of Best Fit Line:	0.00278
End 2:	Slope of Best Fit Line	0.00005
	Angle of Best Fit Line:	0.00278
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? YES

Spherically Seated

DIAMETER 2

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? 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?
Diameter 1, in	0.00040	1.985	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.985	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00040	1.985	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.985	0.00015	0.009	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

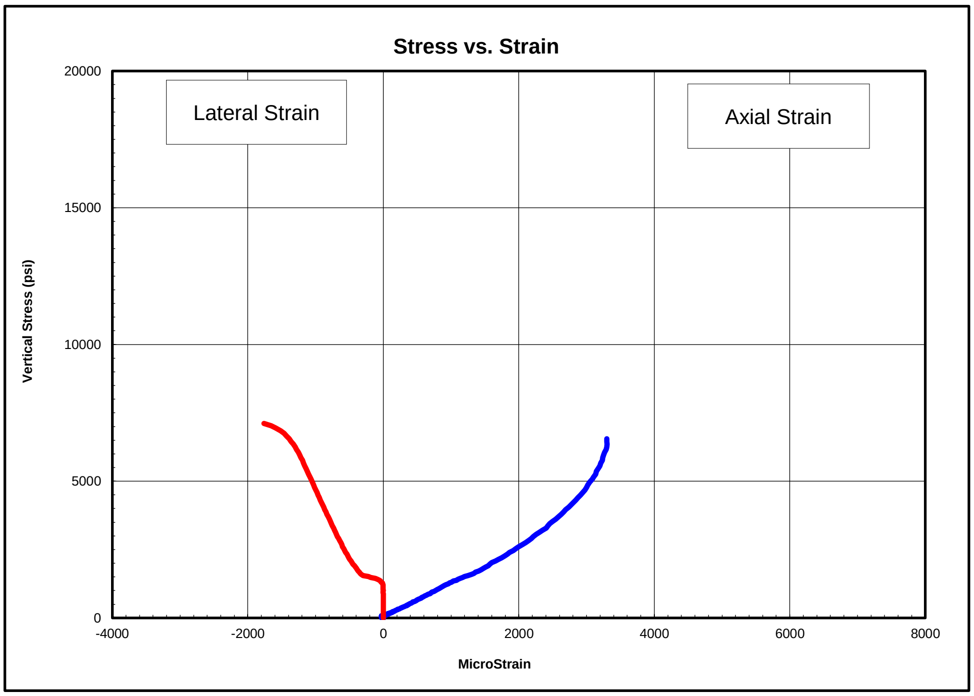
Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-101
Sample ID:	R2
Depth, ft:	31.6-32.2

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Connector
Project Location:	Brewer and Eddington, ME
GTX #:	308853
Test Date:	10/12/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BEA-102
Sample ID:	R1
Depth, ft:	22.84-23.20
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 7,261 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2700	1,250,000	---
2700-4600	2,160,000	0.41
4600-6500	5,070,000	---

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:	10/5/2018
Project Name:	Rt 9/I-395 Connector	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308853		
Boring ID:	BB-BEA-102		
Sample ID:	R1		
Depth:	22.84-23.20 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.17	4.18	4.18		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	573.64				
Bulk Density, lb/ft^3	170			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? NO

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? NO

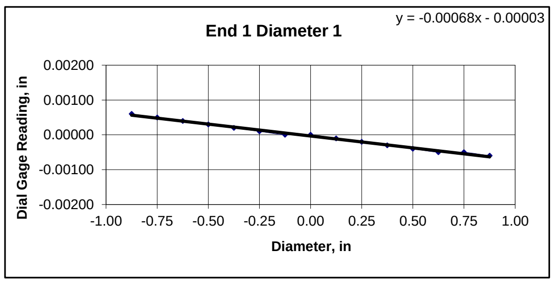
END FLATNESS AND PARALLELISM (Procedure FP1)

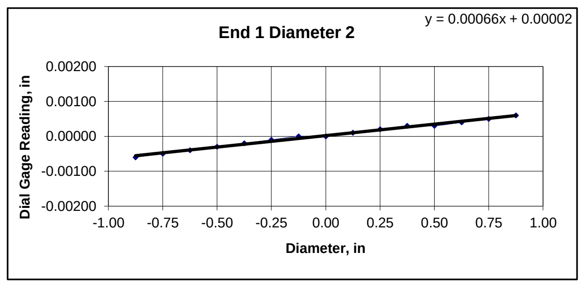
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050	-0.00060
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00040	0.00050	0.00060

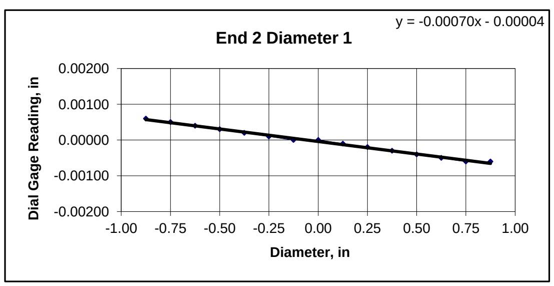
Difference between max and min readings, in:
0° = 0.00120 90° = 0.00120

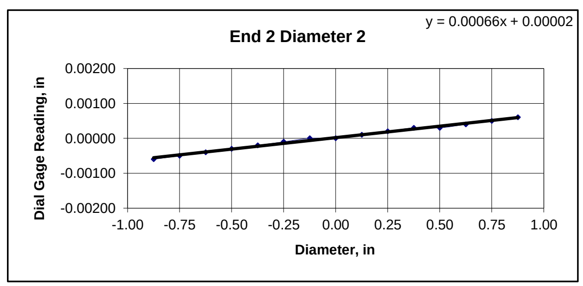
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00060	0.00050	0.00040	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00060
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00040	0.00050	0.00060

Difference between max and min readings, in:
0° = 0.0012 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 0.00068
Angle of Best Fit Line: 0.03912

End 2:
Slope of Best Fit Line 0.00070
Angle of Best Fit Line: 0.04011

Maximum Angular Difference: 0.00098

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00066
Angle of Best Fit Line: 0.03765

End 2:
Slope of Best Fit Line 0.00066
Angle of Best Fit Line: 0.03765

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

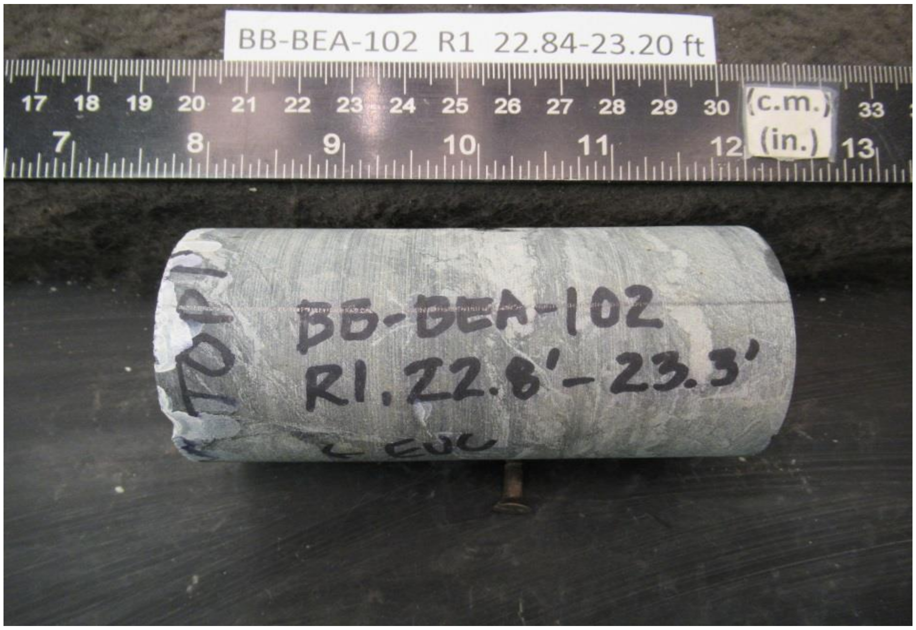
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

Maximum angle of departure must be ≤ 0.25°

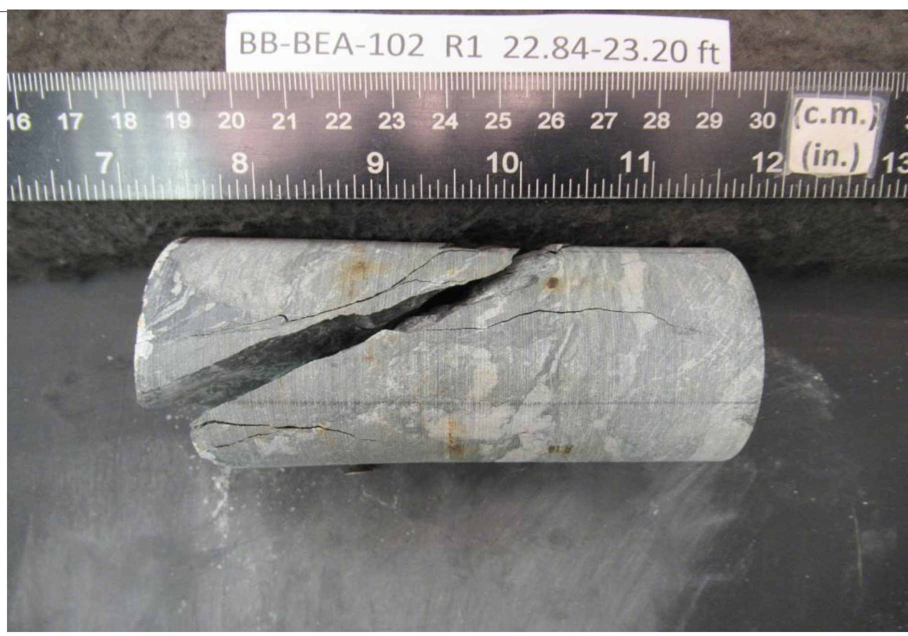
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00120	1.980	0.00061	0.035	YES
Diameter 2, in (rotated 90°)	0.00120	1.980	0.00061	0.035	YES
END 2					
Diameter 1, in	0.00120	1.980	0.00061	0.035	YES
Diameter 2, in (rotated 90°)	0.00120	1.980	0.00061	0.035	YES

Perpendicularity Tolerance Met? YES

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Connector
Project Location:	Brewer and Eddington, ME
GTX #:	308853
Test Date:	10/12/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-102
Sample ID:	R1
Depth, ft:	22.84-23.20



After cutting and grinding

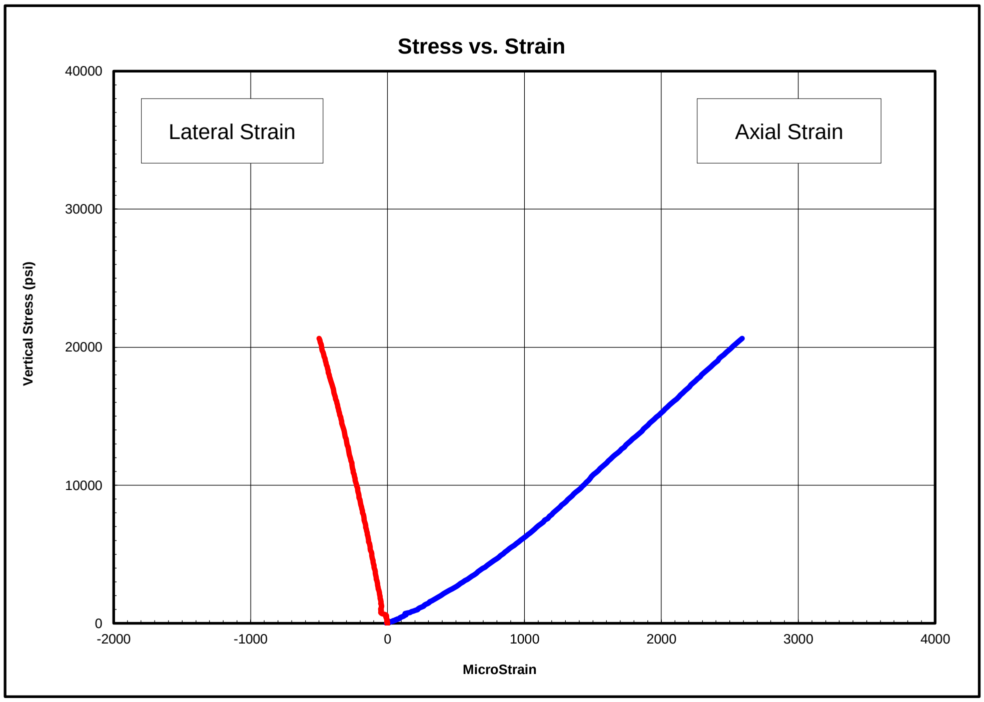


After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	10/1/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	BB-BEA-102
Sample ID:	R2
Depth, ft:	29.3-30
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 20,635 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2100-7600	7,230,000	0.15
7600-13100	9,330,000	0.21
13100-18600	9,230,000	0.24

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.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	9/27/2018
Project Name:	Rt 9/I-395 Eastern Ave Bridge	Tested By:	tlm
Project Location:	Brewer and Eddington, ME	Checked By:	jsc
GTX #:	308855		
Boring ID:	BB-BEA-102		
Sample ID:	R2		
Depth:	29.3-30 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.18	4.18	4.18
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	566.49		
Bulk Density, lb/ft^3	167	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **NO**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040

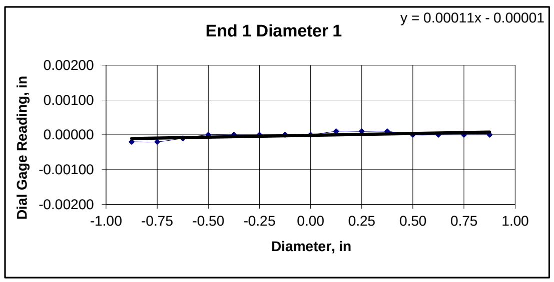
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00040

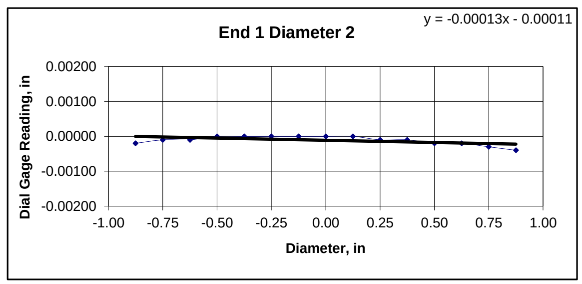
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040

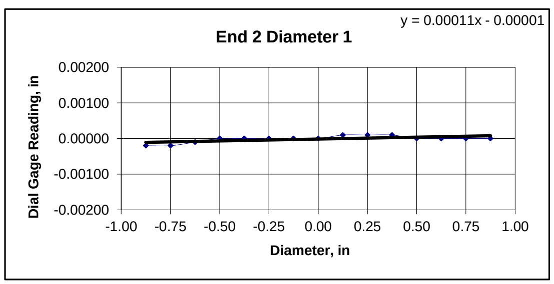
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0004

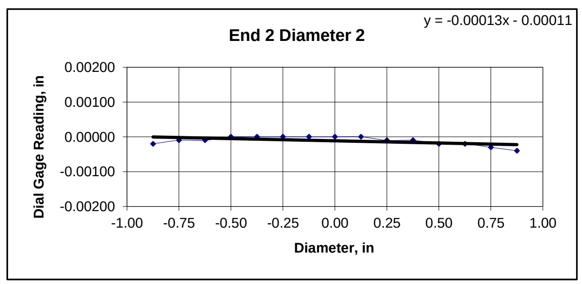
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00606
End 2:		
	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00606
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:		
	Slope of Best Fit Line	0.00013
	Angle of Best Fit Line:	0.00720
End 2:		
	Slope of Best Fit Line	0.00013
	Angle of Best Fit Line:	0.00720
Maximum Angular Difference:		0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

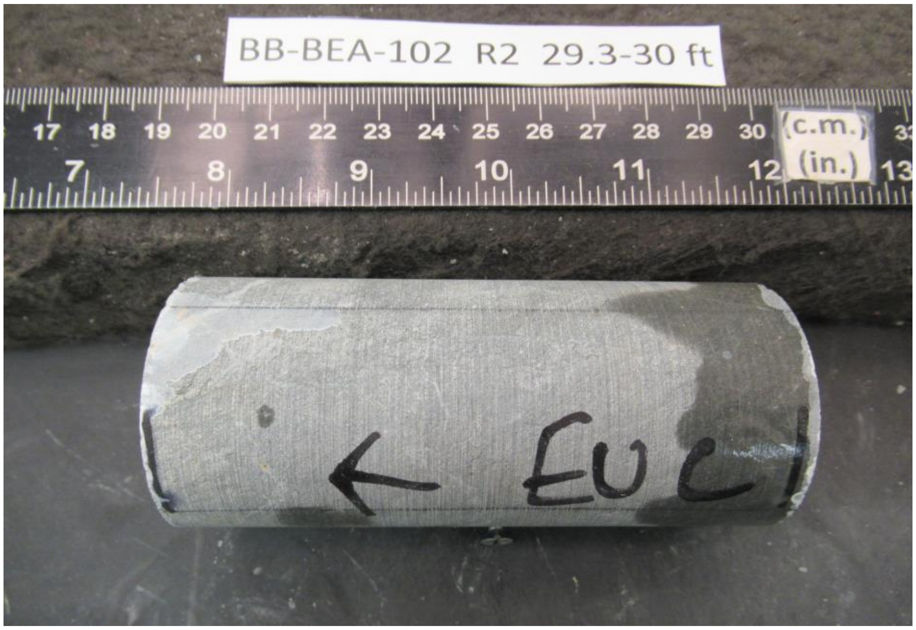
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Rt 9/I-395 Eastern Ave Bridge
Project Location:	Brewer and Eddington, ME
GTX #:	308855
Test Date:	9/28/2018
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-102
Sample ID:	R2
Depth, ft:	29.3-30

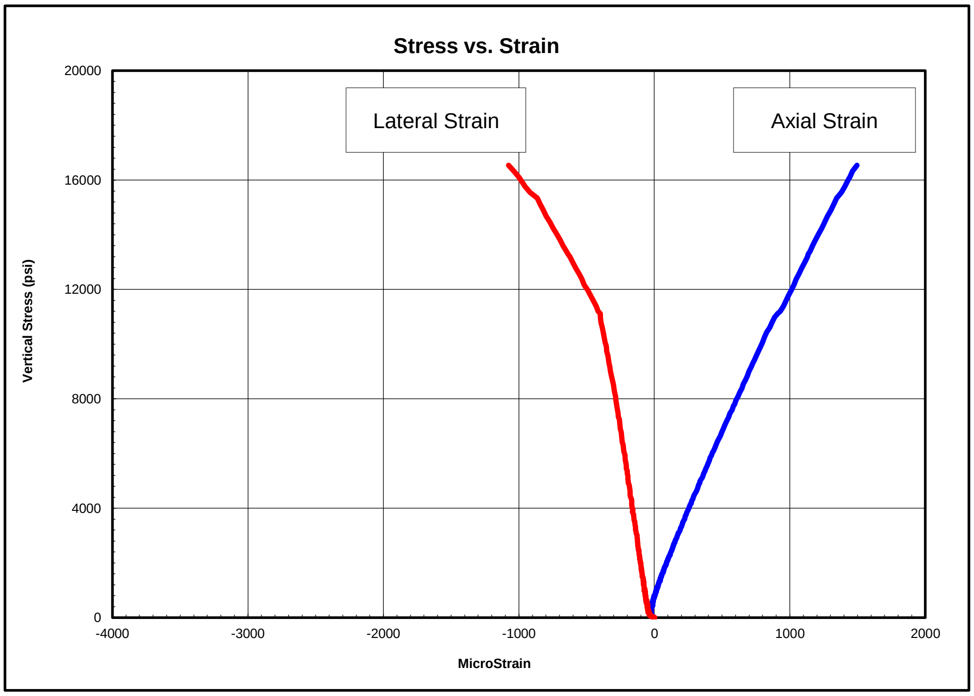


After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/25/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-201
Sample ID:	R1
Depth, ft:	33-33.7
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: 16,538 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
1700-6100	11,900,000	0.36
6100-10500	11,000,000	0.38
10500-14900	9,580,000	---

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.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)	Tested By:	cmh
Project Location:	Brewer, ME	Checked By:	smd
GTX #:	313321		
Boring ID:	BB-BEA-201		
Sample ID:	R1		
Depth:	33-33.7 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.43	4.43	4.43		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	610.32				
Bulk Density, lb/ft^3	170			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness 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	-0.00040	-0.00040	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00030	0.00030
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	0.00000	0.00000	0.00000	0.00000	-0.00010

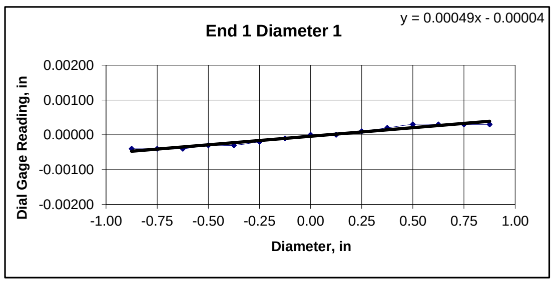
Difference between max and min readings, in:
0° = 0.00070 90° = 0.00010

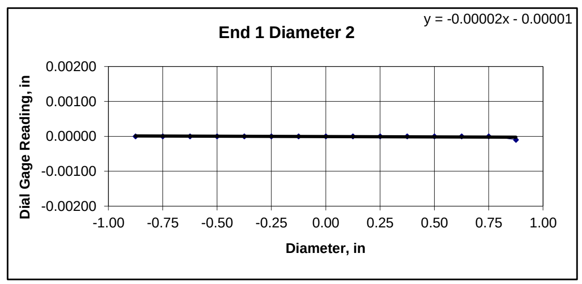
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00050	-0.00040	-0.00040	-0.00030	-0.00020	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00020	0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010

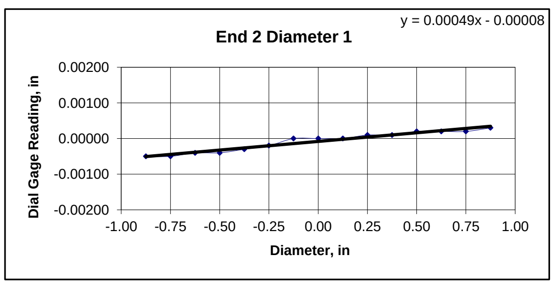
Difference between max and min readings, in:
0° = 0.0008 90° = 0.0001

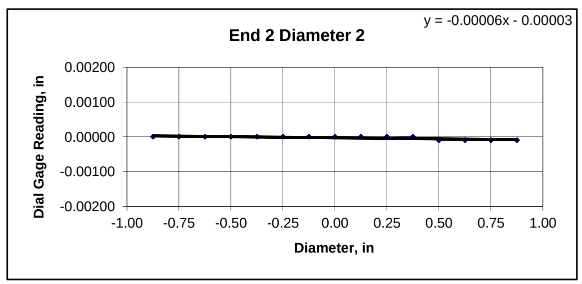
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00040

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00049
Angle of Best Fit Line:	0.02816
End 2:	
Slope of Best Fit Line	0.00049
Angle of Best Fit Line:	0.02783
Maximum Angular Difference:	0.00033

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115
End 2:	
Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00360
Maximum Angular Difference:	0.00246

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?
Diameter 1, in	0.00070	1.980	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00080	1.980	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-BEA-201
Sample ID:	R1
Depth, ft:	33-33.7

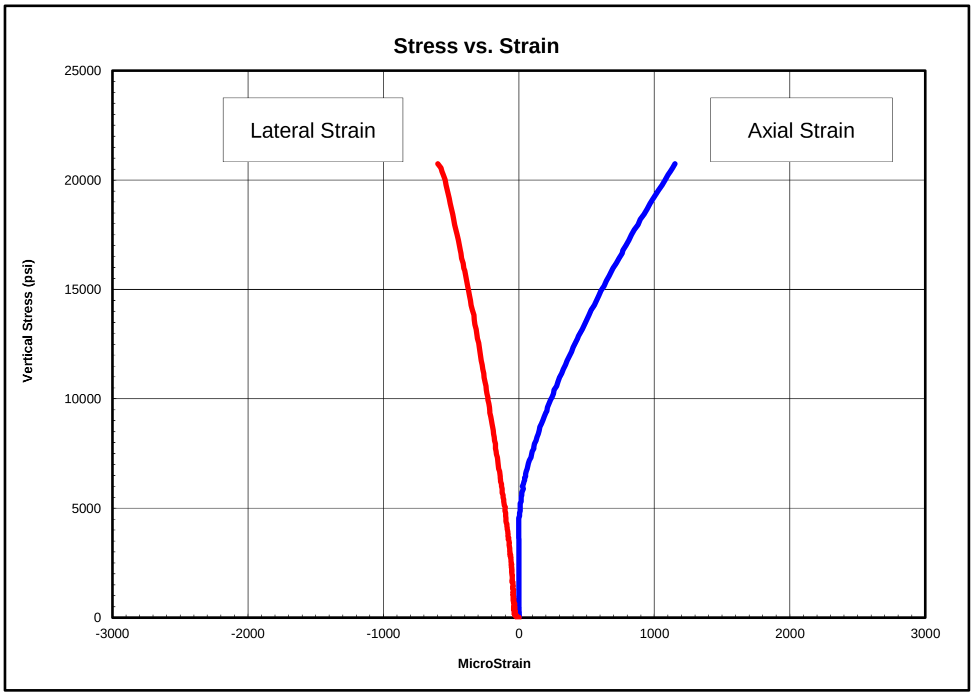
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/25/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-202
Sample ID:	R1
Depth, ft:	22.2-22.9
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 20,742 psi

The strain values recorded within the first 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
2100-7600	58,200,000	---
7600-13100	15,300,000	0.41
13100-18700	11,600,000	0.38

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 (Eastern Ave)	Tested By:	cmh
Project Location:	Brewer, ME	Checked By:	smd
GTX #:	313321		
Boring ID:	BB-BEA-202		
Sample ID:	R1		
Depth:	22.2-22.9 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.44	4.45	4.45		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	614.21				
Bulk Density, lb/ft^3	167			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **YES**

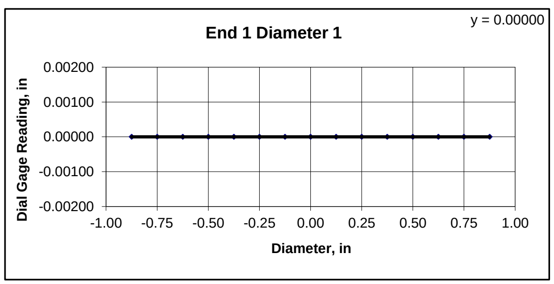
END FLATNESS AND PARALLELISM (Procedure FP1)

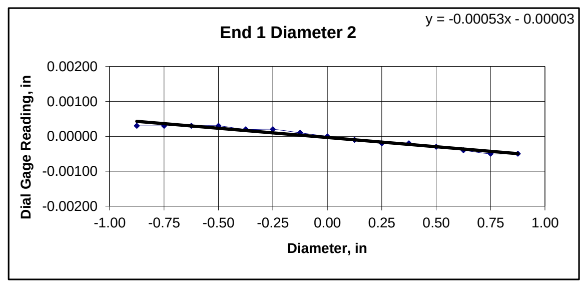
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050

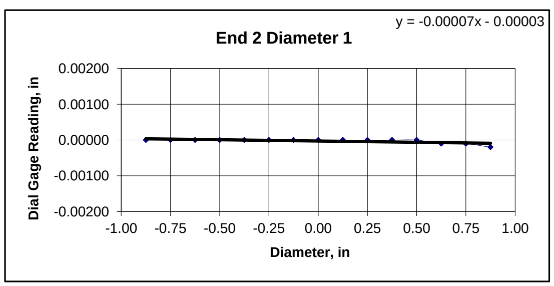
Difference between max and min readings, in:
0° = 0.00000 90° = 0.00080

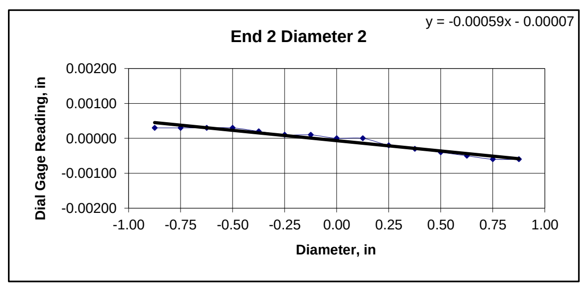
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00060

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	0.00000
Angle of Best Fit Line:	0.00000
End 2:	
Slope of Best Fit Line	0.00007
Angle of Best Fit Line:	0.00409
Maximum Angular Difference:	0.00409

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00053
Angle of Best Fit Line:	0.03028
End 2:	
Slope of Best Fit Line	0.00059
Angle of Best Fit Line:	0.03389
Maximum Angular Difference:	0.00360

Parallelism Tolerance Met? **YES**
Spherically Seated

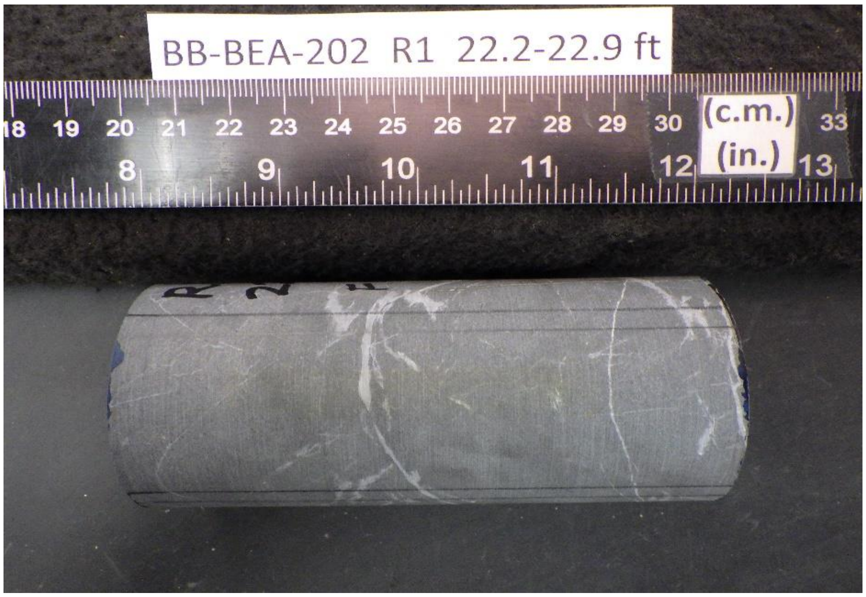
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00000	2.000	0.00000	0.000	YES
Diameter 2, in (rotated 90°)	0.00080	2.000	0.00040	0.023	YES
END 2					
Diameter 1, in	0.00020	2.000	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00090	2.000	0.00045	0.026	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/25/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-BEA-202
Sample ID:	R1
Depth, ft:	22.2-22.9



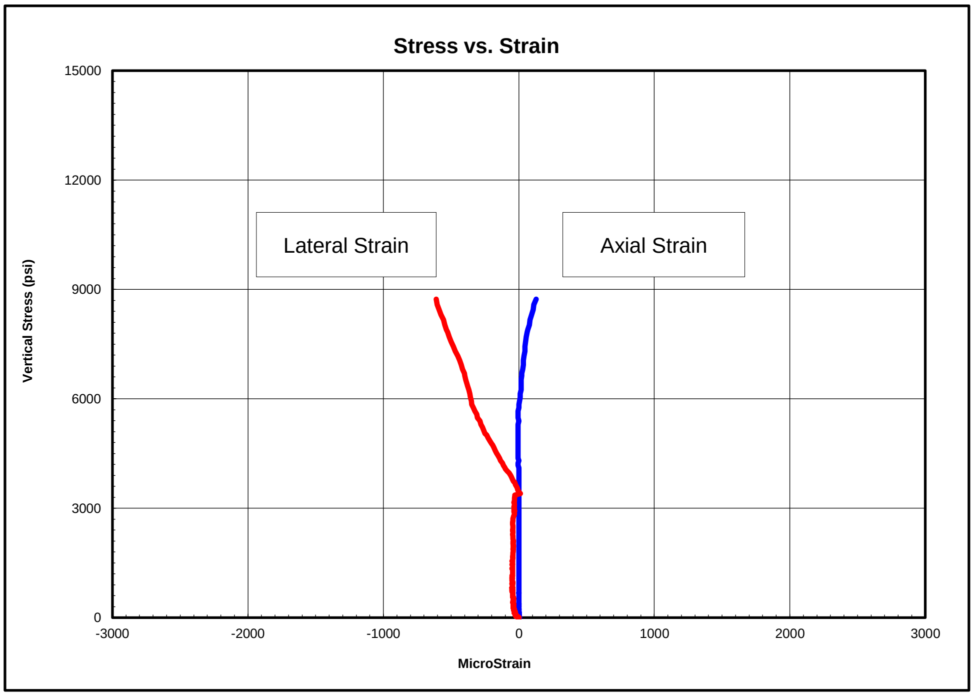
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/25/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-203
Sample ID:	R1
Depth, ft:	20.6-21.7
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 8,733 psi

The strain gauges failed to record meaningful data in order to produce Young's Modulus and Poisson's Ratio for the first and second stress ranges.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
900-3200	---	---
3200-5500	---	---
5500-7900	35,100,000	---

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 (Eastern Ave)	Tested By:	cmh
Project Location:	Brewer, ME	Checked By:	smd
GTX #:	313321		
Boring ID:	BB-BEA-203		
Sample ID:	R1		
Depth:	20.6-21.7 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.48	4.48	4.48		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	617.94				
Bulk Density, lb/ft^3	169			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **YES**

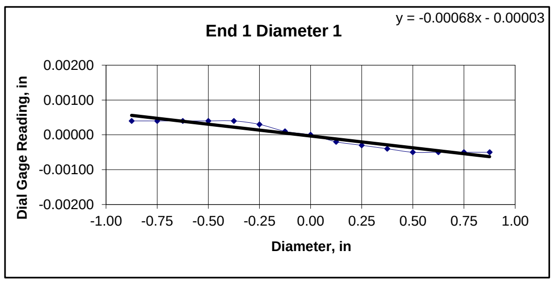
END FLATNESS AND PARALLELISM (Procedure FP1)

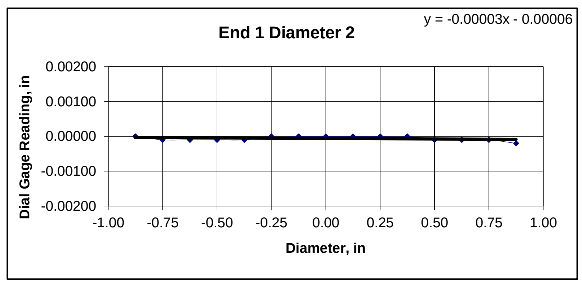
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00040	0.00040	0.00040	0.00040	0.00030	0.00010	0.00000	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050	-0.00050	-0.00050
Diameter 2, in (rotated 90°)	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020

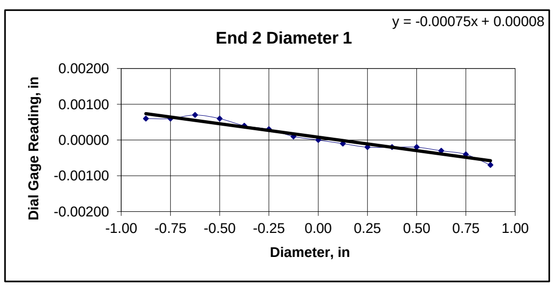
Difference between max and min readings, in:
0° = 0.00090 90° = 0.00020

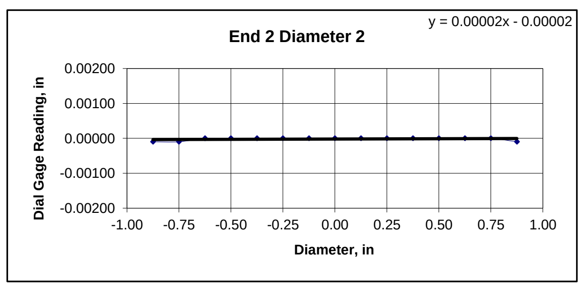
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00060	0.00060	0.00070	0.00060	0.00040	0.00030	0.00010	0.00000	-0.00010	-0.00020	-0.00020	-0.00020	-0.00030	-0.00040	-0.00070
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

Difference between max and min readings, in:
0° = 0.0014 90° = 0.0001
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00070
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00068
	Angle of Best Fit Line:	0.03880
End 2:	Slope of Best Fit Line	0.00075
	Angle of Best Fit Line:	0.04305
Maximum Angular Difference:		0.00426

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00003
	Angle of Best Fit Line:	0.00180
End 2:	Slope of Best Fit Line	0.00002
	Angle of Best Fit Line:	0.00098
Maximum Angular Difference:		0.00082

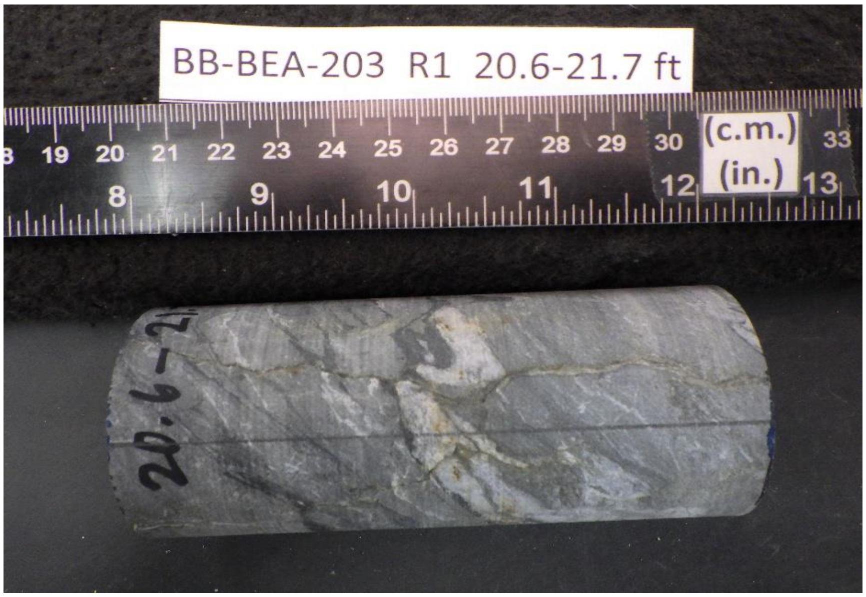
Parallelism Tolerance Met? **YES**
Spherically Seated

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?
Diameter 1, in	0.00090	1.990	0.00045	0.026	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00140	1.990	0.00070	0.040	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES

Maximum angle of departure must be $\leq$ 0.25°
Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-BEA-203
Sample ID:	R1
Depth, ft:	20.6-21.7

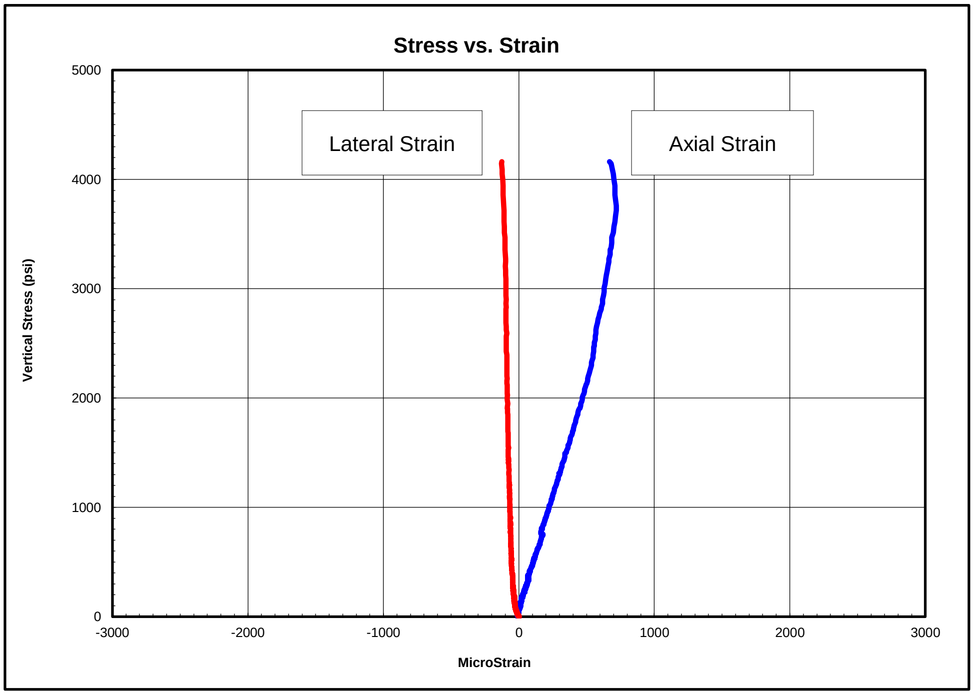


After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/25/2021
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-BEA-204
Sample ID:	R2
Depth, ft:	26.9-27.4
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 4,162 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
400-1500	4,160,000	0.10
1500-2600	4,660,000	0.08
2600-3700	7,310,000	0.13

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.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	3/22/2021
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)	Tested By:	cmh
Project Location:	Brewer, ME	Checked By:	smd
GTX #:	313321		
Boring ID:	BB-BEA-204		
Sample ID:	R2		
Depth:	26.9-27.4 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.49	4.49	4.49		
Specimen Diameter, in:	2.00	1.99	2.00		
Specimen Mass, g:	622.38				
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**		YES	
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**		YES	

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

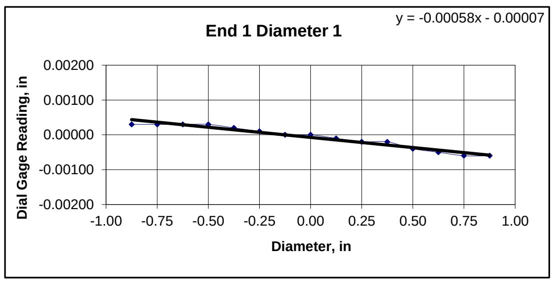
END FLATNESS AND PARALLELISM (Procedure FP1)

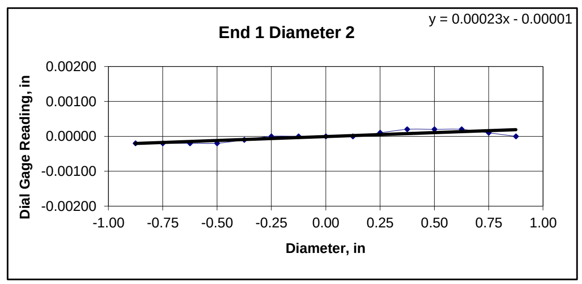
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00040	-0.00050	-0.00060	-0.00060
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	0.00020	0.00020	0.00010	0.00000

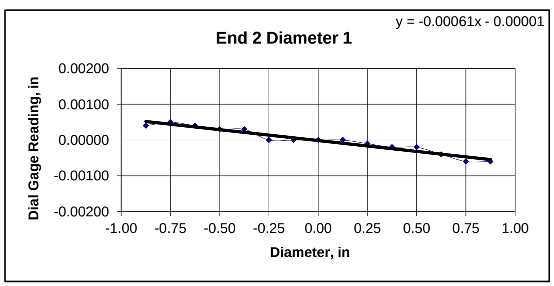
Difference between max and min readings, in:
0° = 0.00090 90° = 0.00040

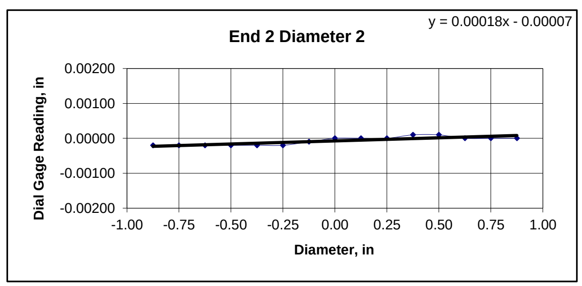
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00040	0.00050	0.00040	0.00030	0.00030	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00040	-0.00060	-0.00060
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0011 90° = 0.0003
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00055
Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00058
Angle of Best Fit Line:	0.03340
End 2:	
Slope of Best Fit Line	0.00061
Angle of Best Fit Line:	0.03487
Maximum Angular Difference:	0.00147

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00023
Angle of Best Fit Line:	0.01293
End 2:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01015
Maximum Angular Difference:	0.00278

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?
Diameter 1, in	0.00090	1.995	0.00045	0.026	YES
Diameter 2, in (rotated 90°)	0.00040	1.995	0.00020	0.011	YES
END 2					
Diameter 1, in	0.00110	1.995	0.00055	0.032	YES
Diameter 2, in (rotated 90°)	0.00030	1.995	0.00015	0.009	YES

Maximum angle of departure must be $\leq$ 0.25°
Perpendicularity Tolerance Met? YES

Client:	Haley & Aldrich, Inc.
Project Name:	I-395/Rte 9 Connector Bridge (Eastern Ave)
Project Location:	Brewer, ME
GTX #:	313321
Test Date:	3/26/2021
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-BEA-204
Sample ID:	R2
Depth, ft:	26.9-27.4

After cutting and grinding

After break

APPENDIX D

Geotechnical Design Calculations

HALEY ALDRICH

Seismic Site Class

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-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-BEA-101, BB-BEA-102, BB-BEA-201 through BB-BEA-204 and HB-BE-217 & HB-BE-218), recommend a **Seismic Site Class C**. Borings BB-BEA-202, BB-BEA-204, and HB-BE-218 classified as Site Class D however, based on a sensitivity study the Site Class D designation is a function of low SPT N-values in the upper 5 ft of the soil profile.

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 and 2 August 2018 drilled by Northern Test Borings, Inc. (monitored by Haley & Aldrich, Inc.).
2. Boring logs dated 4, 5, 9 and 10 November 2020 drilled by New England Boring Contractors (monitored by Haley & Aldrich, Inc.).
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.
3. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

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:

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

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.

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.

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, $\overline{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\overline{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\overline{v}_s$ < 2,500 ft/s, or with either $\overline{N}$ > 50 blows/ft, or $\overline{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\overline{v}_s$ < 1,200 ft/s, or with either 15 < $\overline{N}$ < 50 blows/ft, or 1.0 < $\overline{s}_u$ < 2.0 ksf
E	Soil profile with $\overline{v}_s$ < 600 ft/s or with either $\overline{N}$ < 15 blows/ft or $\overline{s}_u$ < 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 $\overline{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • 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:

$\overline{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile

$\overline{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile

$\overline{s}_u$ = 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)

CALCULATIONS - METHOD B

Exploration ID: BB-BEA-101
Ground Surface El.: 139.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1.5	138	SAND (Fill)	2.5	43	0.058
2D	3.5	136	SILT (Fill)	2.5	13	0.192
3D	6	133.5	silty SAND (Glacial Till)	5.0	58	0.086
4D	10.7	128.8	SAND (Glacial Till)	5.0	50	0.100
5D	16	123.5	silty SAND (Glacial Till)	5.0	37	0.135
6D	21	118.5	SAND (Glacial Till)	3.0	31	0.097
7D	23.6	115.9	SAND (Glacial Till)	1.5	50	0.030
R1-R3	24.5	115	BEDROCK	75.5	100	0.755
			Totals =	100.0		1.454

N-bar (blows/ft) = 69
Site Class = C

Exploration ID: BB-BEA-102
Ground Surface El.: 139.3

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1.4	137.9	SAND (Fill)	2.5	30	0.083
2D	3.5	135.8	SILT (Fill)	2.5	12	0.208
3D	5.9	133.4	SAND (Fill)	1.5	39	0.038
4D	11	128.3	SILT (Glacial Till)	7.0	37	0.189
5D	16	123.3	silty SAND (Glacial Till)	6.5	38	0.171
6D	20.5	118.8	SAND (Glacial Till)	1.1	50	0.022
R1-R3	21.1	118.2	BEDROCK	78.9	100	0.789
			Totals =	100.0		1.501

N-bar (blows/ft) = 67
Site Class = C

HALEY ALDRICH	CALCULATIONS	File No.	132076-007
		Sheet	5 of 7
Client	Maine Department of Transportation	Date	9-Feb-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Seismic Site Class Evaluation	Checked by	BWC

CALCULATIONS - METHOD B

Exploration ID: BB-BEA-201
Ground Surface El.: 136.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	136.5	SAND (Fill)	2.0	19	0.105
2D	5	131.5	SAND (Glacial Till)	8.0	66	0.121
3D	10	126.5	SAND (Glacial Till)	5.0	67	0.075
4D	15	121.5	SILT (Glacial Till)	5.0	32	0.156
MD	20	116.5	SILT (Glacial Till)	5.0	57	0.088
5D	25	111.5	WEATHERED BEDROCK	5.0	100	0.050
6D	30	106.5	WEATHERED BEDROCK	1.0	100	0.010
-	31	105.5	BEDROCK	69.0	100	0.690
			Totals =	100.0		1.295

N-bar (blows/ft) = 77
Site Class = C

Exploration ID: BB-BEA-202
Ground Surface El.: 137.1

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	137.1	SILT	6.0	2	3.000
2D	6	131.1	SILT (Glacial Till	4.0	91	0.044
3D	10	127.1	AND (Glacial Ti	5.0	94	0.053
4D	15	122.1	SILT (Glacial Till)	5.2	33	0.158
-	20.2	116.9	BEDROCK	79.8	100	0.798
			Totals =	100.0		4.053

N-bar (blows/ft) = 25
Site Class = D

CALCULATIONS - METHOD B

Exploration ID: BB-BEA-203
Ground Surface El.: 135.7

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	135.7	SILT	2.0	8	0.250
2D	5	130.7	SAND (Glacial Till)	8.0	50	0.160
3D	10	125.7	SILT (Glacial Till)	5.0	24	0.208
4D	15	120.7	SILT (Glacial Till)	2.5	77	0.032
-	17.5	118.2	BEDROCK	82.5	100	0.825
			Totals =	100.0		1.476

N-bar (blows/ft) = 68
Site Class = C

Exploration ID: BB-BEA-204
Ground Surface El.: 138.7

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	138.7	SILT (Glacial Till)	3.0	3	1.000
2D	5	133.7	SAND (Glacial Till)	7.0	100	0.070
3D	10	128.7	SILT (Glacial Till)	5.0	42	0.119
4D	15	123.7	SILT (Glacial Till)	5.0	38	0.132
5D	20	118.7	SAND (Glacial Till)	1.8	55	0.033
-	21.8	116.9	BEDROCK	78.2	100	0.782
			Totals =	100.0		2.135

N-bar (blows/ft) = 47
Site Class = D

CALCULATIONS - METHOD B

Exploration ID: HB-BE-217
Ground Surface El.: 135.8

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	135.8	SILT	2.0	13	0.154
2D	5	130.8	SILT (Glacial Till)	8.0	39	0.205
3D	10	125.8	SAND (Glacial Till)	5.0	48	0.104
4D	15	120.8	SILT (Glacial Till)	5.0	30	0.167
5D	20	115.8	SILT (Glacial Till)	5.0	38	0.132
6D	15	120.8	SILT (Glacial Till)	1.5	78	0.019
-	26.5	109.3	BEDROCK	73.5	100	0.735
			Totals =	100.0		1.516

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

Exploration ID: HB-BE-218
Ground Surface El.: 138.7

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	138.7	SAND (Glacial Till)	5.0	5	1.000
2D	5	133.7	SAND (Glacial Till)	5.0	58	0.086
3D	10	128.7	SAND (Glacial Till)	5.0	43	0.116
4D	15	123.7	SAND (Glacial Till)	5.0	87	0.057
5D	20	118.7	GRAVEL (Glacial Till)	2.1	115	0.018
-	22.1	116.6	BEDROCK	77.9	100	0.779
			Totals =	100.0		2.057

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

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Bearing Resistance

HALEY ALDRICH	CALCULATIONS	File No.	132076-007
		Sheet	1 of 8
Client	Maine Department of Transportation	Date	29-Mar-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Bearing Resistance of Bedrock for Abutment Footings	Updated by	SSM
		Checked by	BCS

PROBLEM STATEMENT & OBJECTIVE

Calculate the factored bearing resistance at the service, strength and extreme limit states for the proposed abutment footings bearing on bedrock.

EXECUTIVE SUMMARY

A factored bearing resistance of	52	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	93	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, 8th Edition, 2017.
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:

Abutment 1 =	111.0	ft (LW)	113.0	ft (BW, RW)
Abutment 2 =	116.9	ft (LW, BW, RW)		

2. The peak compressive strength of bedrock is based on laboratory test data (see page 4 for a summary of laboratory test results).

PROCEDURE FOR STRENGTH LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2017:

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 Semiemprical 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:

$q_{ult} = q_u(\sqrt{s}+(m\sqrt{s} + s)^{0.5})$ Equation 82b 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 ϕ from Table 10.5.5.2.2-1 in AASHTO LRFD 2017 for bearing resistance of footings on rock

PROCEDURE FOR SERVICE LIMIT STATE

1. See bearing resistance for footing on rock guidance from AASHTO LRFD 2017:

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 2017:

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 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE Walls.

2. Use nominal resistance calculated for the Strength Limit State and apply a resistance factor of 0.8 from AASHTO LRFD 2017 Section 11.5.8 to obtain the factored resistance.

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	BB-BEA-101	139.5	R1	29.2	110.3	0.7	3,818	discontinuity
			R2	31.9	107.6	3.4	6,520	discontinuity
	BB-BEA-201	136.5	R1	33.4	103.1	7.9	16,538	intact
	BB-BEA-202	137.1	R1	22.6	114.5	-3.5	20,742	intact
2	BB-BEA-102	139.3	R1	23.0	116.3	0.6	7,261	discontinuity
			R2	29.7	109.6	7.3	20,635	intact
	BB-BEA-203	135.7	R1	21.2	114.5	2.4	8,733	discontinuity
	BB-BEA-204	138.7	R2	27.2	111.5	5.4	4,162	discontinuity

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	BB-BEA-101	139.5	R1	27.5	112	-1.0	83	53
			R2	31.1	108.4	2.6	100	62
			R3	33.6	105.9	5.1	94	71
	BB-BEA-201	136.5	R1	33.5	103.0	8.0	89	25
	BB-BEA-202	137.1	R1	22.7	114.4	-3.4	77	48
			R2	26.3	110.8	0.2	91	46
			R3	28.6	108.5	2.5	80	54
2	BB-BEA-102	139.3	R1	24.5	114.8	2.1	97	92
			R2	29.5	109.8	7.1	87	55
	BB-BEA-203	135.7	R1	22.5	113.2	3.7	100	37
			R2	27.5	108.2	8.7	83	0
	BB-BEA-204	138.7	R1	24.4	114.3	2.6	88	87
			R2	29.0	109.7	7.2	105	74

PARAMETERS FOR CALCULATIONS

1. Exclude yellow-highlighted cells, which represent data above the bearing level of the abutment footings.
2. Unconfined compressive strength information below is based on the average of the test results shown, which represents both intact material failures as well as failures along discontinuities. In our opinion, this is conservative considering that the methodology presented by Carter and Kulhawy (1988) is based on the unconfined compressive strength of the intact rock core samples.

Average RQD from borings at proposed bridge abutments:	55	%
Average RQD from Abutment 1 test borings:	52	%
Average RQD from Abutment 2 test borings:	58	%
Average peak compressive strength at proposed bridge abutments:	9,667	psi
Average peak compressive strength at Abutment 1:	8,959	psi
Average peak compressive strength at Abutment 2:	10,198	psi
Use average peak compressive strength at proposed bridge abutments:	9,667	psi
	1,392	ksf

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		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in thick or • Joints open 0.05–0.2 in • Continuous joints	• Soft gouge >0.2 in thick or • Joints open >0.2 in • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal/hr	400–2,000 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	1,392 ksf	1080 - 2160 ksf	7
RQD	55%	50% to 75%	13
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 Hard joint wall rock	20
Groundwater Condition	Moist only (interstitial water)	Moist only (interstitial water)	7
Joint Strike and Dip	Fair	Fair	-7
		Total Rating =	50

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	B	C	D	E
		A = Carbonate rocks with well developed crystal cleavage— *dolomite, limestone, and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale, and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polyminerallic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite* E = Coarse-grained polymineralic igneous and metamorphic crystalline rocks—*amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*				
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 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: *RMR* = 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: *RMR* = 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: *RMR* = 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: *RMR* = 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: *RMR* = 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:

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

m1 is the value of m for *intact* rock

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

Rock Quality	Rock Type	RMR	m_1	m	s
Fair	B	50	10.00	2.81E-01	2.40E-04

Strength Limit State Continued

Semi-empirical method by Carter and Kulhawy 1988:

q_u =	9,667	psi	average of laboratory test results below the footing bearing levels
m =	0.281		
s =	2.40E-04		
q_{ult} =	116.0	ksf	Equation 82b
ϕ =	0.45		from Table 10.5.5.2.2-1
q_R =	52.2	ksf	Equation 82b

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	Bearing Resistance (ksf) 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 Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

Extreme Event Limit State

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

q_{ult} = 116.0 ksf

Using a resistance factor of 0.8 from Section 11.5.8, the factored bearing resistance is the following:

q_R = 92.8 ksf

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The recommended factored bearing resistance for the strength limit state is 52 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 93 ksf

Sliding Resistance

HALEY ALDRICH

HALEY ALDRICH	CALCULATIONS	File No.	132076-007
		Sheet	1 of 2
Client	Maine Department of Transportation	Date	2-Apr-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Sliding Resistance for Footing on Bedrock	Updated by	SSM
		Checked by	BCS

PROBLEM STATEMENT AND OBJECTIVE

Determine the coefficient of friction between the footing and bedrock, resistance factor for sliding for the Strength Limit State, and resistance factor for sliding for the Extreme Event Limit State for the footing on bedrock, assuming the bedrock surface is prepared in-the-dry

EXECUTIVE SUMMARY

The coefficient of friction between the footing and bedrock is = **0.7**
The resistance factor for sliding at the Strength Limit State is = **0.8**
The resistance factor for sliding at the Extreme Event Limit State is = **1.0**

REFERENCES

1. AASHTO LRFD Bride Design Specifications, 8th Edition, 2017.
2. MaineDOT BDG 2003 with 2014 interims

AVAILABLE INFORMATION

1. Haley & Aldrich test borings BB-BEA-101, BB-BEA-102, BB-BEA-201, BB-BEA-202, BB-BEA-203 and BB-BEA-204.

ASSUMPTIONS

1. Abutment footing will bear on intact SILTSTONE bedrock.

CALCULATIONS

Coefficient of Friction Between Concrete and Bedrock

Nominal sliding resistance between the cast-in-place concrete footing and bedrock is dependent on the coefficient of friction (tanδ) at the interface between the footing and bedrock.

Estimated footing-rock interface friction angle (δ):

35 deg., friction angle for mass concrete on clean sound rock (AASHTO LRFD Table 3.11.5.3-1)

Recommended δ =	35	deg., friction angle between footing/seal and bedrock
Recommended tanδ =	0.7	coefficient of friction

Resistance Factors

Service Limit State

AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on bedrock.
For retaining walls, AASHTO LRFD prescribes a sliding resistance factor of = **1.0** (Section 11.5.7).

Strength Limit State

AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on bedrock.
For cast-in-place concrete on sand, the sliding resistance factor is = **0.8** (Table 10.5.5.2.2-1)

Extreme Event Limit State

Section 10.5.5.3.2 of AASHTO LRFD prescribes a resistance factor of **1.0** for the design of foundations in the Extreme Event Limit State.

Table C3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

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

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

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

Lateral Earth Pressures

HALEY ALDRICH

HALEY ALDRICH	CALCULATIONS	File No.	130732-007
		Sheet	1 of 5
Client	Maine Department of Transportation	Date	29-Mar-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Lateral Earth Pressure Coefficients for Abutment No. 1	Checked by	BWC

Objective

-Calculate the active, at-rest, and passive lateral earth pressure coefficients to design the proposed Abutment No. 1 and wingwalls

Assumptions

-Due to sloping backfill conditions at each abutment, calculate different lateral earth pressure coefficients for each proposed abutment and wingwalls

-Abutments and wingwalls and their footings are backfilled with Granular Borrow based on H&A recomendations.

-Free draining retaining wall, no hydrostatic pressure.

References

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Maine DOT Bridge Design Guide, August 2003, with interim revisions through March 2014

EARTH PRESSURE COEFFICIENTS FOR PROPOSED ABUTMENT NO. 1

Soil Properties and Geometry

designates input cell

Total Unit Weight, ϒ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	0.6	degrees	Preliminary bridge profile indicates 1% bridge grade
Backface of Wall Angle to Horizontal, ϴ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

Static Active Lateral Earth Pressure Coefficient, K_a

$K_a = \sin^2(\Theta+\phi')/r(\sin^2\Theta\sin(\Theta-\delta))$ AASHTO LRFD Eq. 3.11.5.3-1

where $r = [1 + \sqrt{(\sin(\phi+\delta)\sin(\phi-\beta)/(\sin(\Theta-\delta)\sin(\Theta+\beta))}]^2$ AASHTO LRFD Eq. 3.11.5.3-2

K_a = 0.28

At-Rest Lateral Earth Pressure Coefficient, K_o

$K_0 = 1-\sin\phi$ AASHTO LRFD Eq. 3.11.5.2-1

K_o = 0.47

Passive Lateral Earth Pressure Coefficient, K_p

Rankine Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is less than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_{p,Rankine}$ = tan (45 + ϕ'/2)^2

$K_{p,Rankine}$ = 3.25 Das, Principles of Geotechnical Engineering, 7th Ed., Eq. 13.22

Coulomb Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is greater than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_p = \sin^2(\Theta-\phi')/r(\sin^2\Theta\sin(\Theta+\delta))$ BDG Section 3.6.6

where $r = [1 - \sqrt{(\sin(\phi+\delta)\sin(\phi+\beta)/(\sin(\Theta+\delta)\sin(\Theta+\beta))}]^2$ BDG Section 3.6.6

$K_{p,Coulomb}$ = 8.70

HALEY ALDRICH	CALCULATIONS	File No.	130732-007
		Sheet	2 of 5
Client	Maine Department of Transportation	Date	29-Mar-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Lateral Earth Pressure Coefficients for Abutment No. 1 Wingwalls	Checked by	BWC

Objective

-Calculate the active, at-rest, and passive lateral earth pressure coefficients to design the proposed Abutment No. 1 and wingwalls

Assumptions

-Due to sloping backfill conditions at each abutment, calculate different lateral earth pressure coefficients for each proposed abutment and wingwalls

-Abutments and wingwalls and their footings are backfilled with Granular Borrow based on H&A recomendations.

-Free draining retaining wall, no hydrostatic pressure.

References

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Maine DOT Bridge Design Guide, August 2003, with interim revisions through March 2014

EARTH PRESSURE COEFFICIENTS FOR PROPOSED ABUTMENT NO. 1 WINGWALLS

Soil Properties and Geometry

designates input cell

Total Unit Weight, ϒ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	19	degrees	Maximum Slope Angle Indicated by MDOT
Backface of Wall Angle to Horizontal, Θ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

Static Active Lateral Earth Pressure Coefficient, K_a

$K_a = \sin^2(\Theta+\phi')/r(\sin^2\Theta\sin(\Theta-\delta))$ AASHTO LRFD Eq. 3.11.5.3-1

where $r = [1 + \sqrt{\sin(\phi+\delta)\sin(\phi-\beta)/(\sin(\Theta-\delta)\sin(\Theta+\beta))}]^2$ AASHTO LRFD Eq. 3.11.5.3-2

K_a = 0.37

At-Rest Lateral Earth Pressure Coefficient, K_o

$K_0 = 1-\sin\phi$ AASHTO LRFD Eq. 3.11.5.2-1

K_o = 0.47

Passive Lateral Earth Pressure Coefficient, K_p

Rankine Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is less than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_{p,Rankine} = \tan(45 + \phi'/2)^2$

$K_{p,Rankine}$ = 3.25 Das, Principles of Geotechnical Engineering, 7th Ed., Eq. 13.22

Coulomb Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is greater than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_p = \sin^2(\Theta-\phi')/r(\sin^2\Theta\sin(\Theta+\delta))$ BDG Section 3.6.6

where $r = [1 - \sqrt{\sin(\phi+\delta)\sin(\phi+\beta)/(\sin(\Theta+\delta)\sin(\Theta+\beta))}]^2$ BDG Section 3.6.6

$K_{p,Coulomb}$ = 42.35

HALEY ALDRICH	CALCULATIONS	File No.	130732-003
		Sheet	3 of 5
Client	Maine Department of Transportation	Date	29-Mar-21
Project	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by	JAD
Subject	Lateral Earth Pressure Coefficients for Abutment No. 2	Checked by	BWC

Objective

-Calculate the active, at-rest, and passive lateral earth pressure coefficients to design the proposed Abutment No. 2 and wingwalls

Assumptions

-Due to sloping backfill conditions at each abutment, calculate different lateral earth pressure coefficients for each proposed abutment and wingwalls

-Abutments and wingwalls and their footings are backfilled with Granular Borrow based on H&A recomendations.

-Free draining retaining wall, no hydrostatic pressure.

References

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Maine DOT Bridge Design Guide, August 2003, with interim revisions through March 2014

EARTH PRESSURE COEFFICIENTS FOR PROPOSED ABUTMENT NO. 2

Soil Properties and Geometry

designates input cell

Total Unit Weight, ϒ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	-0.6	degrees	Preliminary bridge profile indicates 1% bridge grade
Backface of Wall Angle to Horizontal, Θ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

Static Active Lateral Earth Pressure Coefficient, K_a

$K_a = \sin^2(\Theta+\phi')/r(\sin^2\Theta\sin(\Theta-\delta))$ — AASHTO LRFD Eq. 3.11.5.3-1

where $r = [1 + \sqrt{\sin(\phi+\delta)\sin(\phi-\beta)/(\sin(\Theta-\delta)\sin(\Theta+\beta))}]^2$ — AASHTO LRFD Eq. 3.11.5.3-2

K_a = 0.27

At-Rest Lateral Earth Pressure Coefficient, K_o

$K_0 = 1-\sin\phi$ — AASHTO LRFD Eq. 3.11.5.2-1

K_o = 0.47

Passive Lateral Earth Pressure Coefficient, K_p

Rankine Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is less than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_{p,Rankine}$ = tan (45 + ϕ'/2)^2

$K_{p,Rankine}$ = 3.25 — Das, Principles of Geotechnical Engineering, 7th Ed., Eq. 13.22

Coulomb Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is greater than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_p = \sin^2(\Theta-\phi')/r(\sin^2\Theta\sin(\Theta+\delta))$ — BDG Section 3.6.6

where $r = [1 - \sqrt{\sin(\phi+\delta)\sin(\phi+\beta)/(\sin(\Theta+\delta)\sin(\Theta+\beta))}]^2$ — BDG Section 3.6.6

$K_{p,Coulomb}$ = 8.07

Objective

-Calculate the active, at-rest, and passive lateral earth pressure coefficients to design the proposed Abutment No. 2 and wingwalls

Assumptions

-Due to sloping backfill conditions at each abutment, calculate different lateral earth pressure coefficients for each proposed abutment and wingwalls

-Abutments and wingwalls and their footings are backfilled with Granular Borrow based on H&A recomendations.

-Free draining retaining wall, no hydrostatic pressure.

References

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020
2. Maine DOT Bridge Design Guide, August 2003, with interim revisions through March 2014

EARTH PRESSURE COEFFICIENTS FOR PROPOSED ABUTMENT NO. 2 WINGWALLS

Soil Properties and Geometry

designates input cell

Total Unit Weight, ϒ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	19	degrees	Maximum Slope Angle Indicated by MDOT
Backface of Wall Angle to Horizontal, ϴ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

Static Active Lateral Earth Pressure Coefficient, K_a

$K_a = \sin^2(\Theta+\phi')/r(\sin^2\Theta\sin(\Theta-\delta))$ AASHTO LRFD Eq. 3.11.5.3-1

where $r = [1 + \sqrt{\sin(\phi+\delta)\sin(\phi-\beta)/(\sin(\Theta-\delta)\sin(\Theta+\beta))}]^2$ AASHTO LRFD Eq. 3.11.5.3-2

K_a = 0.37

At-Rest Lateral Earth Pressure Coefficient, K_o

$K_0 = 1-\sin\phi$ AASHTO LRFD Eq. 3.11.5.2-1

K_o = 0.47

Passive Lateral Earth Pressure Coefficient, K_p

Rankine Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is less than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_{p,Rankine} = \tan(45 + \phi'/2)^2$

$K_{p,Rankine}$ = 3.25 Das, Principles of Geotechnical Engineering, 7th Ed., Eq. 13.22

Coulomb Theory

If the ratio of lateral abutment movement to **abutment height (y/H) is greater than 0.005**, we recommend using Rankine theory to calculate the passive lateral earth pressure coefficient

$K_p = \sin^2(\Theta-\phi')/r(\sin^2\Theta\sin(\Theta+\delta))$ BDG Section 3.6.6

where $r = [1 - \sqrt{\sin(\phi+\delta)\sin(\phi+\beta)/(\sin(\Theta+\delta)\sin(\Theta+\beta))}]^2$ BDG Section 3.6.6

$K_{p,Coulomb}$ = 42.35

Friction Angle= 32

Slope Behind Wall, β (deg)	K_a
0	0.307
1	0.307
2	0.308
3	0.308
4	0.309
5	0.311
6	0.312
7	0.314
8	0.316
9	0.318
10	0.321
11	0.324
12	0.328
13	0.331
14	0.336
15	0.341
16	0.346
17	0.352
18	0.358
19	0.366
20	0.374
21	0.383
22	0.393
23	0.405
24	0.418
25	0.434
26	0.451
27	0.473
28	0.498
29	0.531
30	0.574
31	0.639
32	0.848

TABLE 11-3
Rankine active earth pressure coefficients K_a using Eq. (11-7*a*)

β	ϕ = 26	28	30	32	34	36	38	40	42
0	0.3905	0.3610	0.3333	0.3073	0.2827	0.2596	0.2379	0.2174	0.1982
5	0.3959	0.3656	0.3372	0.3105	0.2855	0.2620	0.2399	0.2192	0.1997
10	0.4134	0.3802	0.3495	0.3210	0.2944	0.2696	0.2464	0.2247	0.2044
15	0.4480	0.4086	0.3729	0.3405	0.3108	0.2834	0.2581	0.2346	0.2129
20	0.5152	0.4605	0.4142	0.3739	0.3381	0.3060	0.2769	0.2504	0.2262
25	0.6999	0.5727	0.4936	0.4336	0.3847	0.3431	0.3070	0.2750	0.2465
30	—	—	0.8660	0.5741	0.4776	0.4105	0.3582	0.3151	0.2784
35	—	—	—	—	—	0.5971	0.4677	0.3906	0.3340
40	—	—	—	—	—	—	—	0.7660	0.4668

Since $\cos\beta$ is a permanent entry it is convenient to include it with K_a' of Eq. (11-7) or K_p' of Eq. (11-8), giving, e.g.,

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad (11\text{-}7a)$$

	CALCULATIONS	File No.:	132076-003
		Sheet:	5 of 5
Client:	Maine Department of Transportation	Date:	29-Mar-21
Project:	Eastern Avenue Bridge over I-395/Route 9 Connector - WIN 18915.00	Computed by:	JAD
Subject:	Lateral Earth Pressure Coefficients for Abutment No. 2 and Wingwalls	Checked by:	BWC

SEISMIC LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020

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

where:

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

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

Note: Abutment 1 has a backfill surface of -.6 degrees and abutment 2 has a backfill angle of 0.6 degrees.

Note: Abutment 1 and 2 Wingwalls have a maximum backfill angle of 19 degrees.

CALCULATIONS

ϕ'_f (deg)	PGA (g)	F_{PGA}	k_h (g)	k_v (g)	θ_{MO} (deg)	δ (deg)	β (deg)	i (deg)	K_{AE}
32	0.066	1.20	0.08	0.00	4.5	24	0.0	0.60	0.33
32	0.066	1.20	0.08	0.00	4.5	24	0.0	-0.60	0.32
32	0.066	1.20	0.08	0.00	4.5	24	0.0	19.00	0.47
32	0.066	1.20	0.08	0.00	4.5	24	0.0	19.00	0.47

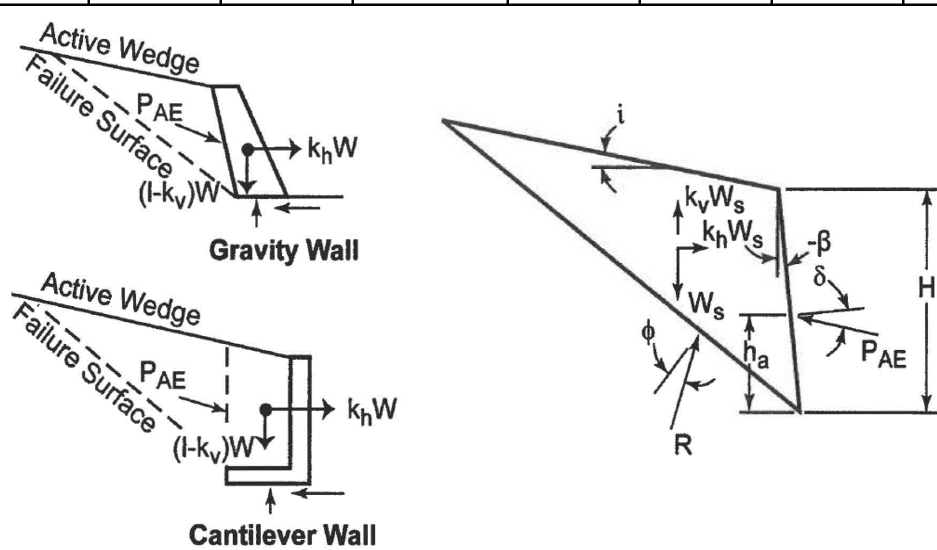

Figure A11.3.1-1—Mononobe-Okabe Method Force Diagrams