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GEOTECHNICAL DESIGN REPORT
CHASE MILLS BRIDGE NO. 5465 OVER GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Maine Department of Transportation
Augusta, Maine

File No. 0205517-001
November 2025

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

Subject: Geotechnical Design Report
Chase Mills Bridge No. 5465 over Gardner Lake Outlet
MaineDOT WIN 25529.00
Chases Mill Road, East Machias, Maine

Ladies and Gentlemen:

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit herewith our report entitled, "Geotechnical Design Report, Chase Mills Bridge No. 5465 over Gardner Lake Outlet, MaineDOT WIN 25529.00, Chases Mill Road, East Machias, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposal, dated 15 April 2025, which was executed by your Todd Pelletier, P.E. on 6 May 2025 and the provisions of our General Consultant Agreement (GCA) with the Maine Department of Transportation (MaineDOT), No. CT20200623000000000766.

Introduction

This Geotechnical Design Report (GDR) presents the results of preliminary (Phase I) and final (Phase II) geotechnical field investigation and laboratory testing programs, technical evaluations, and geotechnical design recommendations completed by Haley & Aldrich on behalf of MaineDOT for the proposed replacement bridge that will carry vehicular traffic on Chases Mill Road over Gardner Lake Outlet in East Machias, Maine (see Figure 1).

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

Plan locations of test borings (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 East Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

The proposed limit of work extends from approximately Sta. 13+00 (east) to Sta. 16+50 (west).

PROJECT LOCATION, EXISTING SITE CONDITIONS, AND EXISTING BRIDGE STRUCTURE

The existing bridge carries eastbound (EB) and westbound (WB) vehicular traffic on Chases Mill Road over Gardner Lake Outlet in East Machias, Maine, approximately 0.03 miles north of Lakeside Road (see Figure 1). Existing roadway grades within the limit of work are generally flat at El. 71 (approximate). The bottom of Gardner Lake Outlet, which flows from north (Gardner Lake) to south (Chase Mills Stream), within the limits of Chases Mills Road is approximately El. 58. A town-owned dam is located immediately upstream of the bridge and the bridge abutment wingwalls terminate at the dam structure. In addition, a fishway is located adjacent to the west abutment, between the bridge and the dam, which is oriented parallel with Chases Mills Road.

Based on our review of the historic, as-built bridge drawings and the Preliminary Design Report (PDR) that was prepared by Stantec Consulting Services, Inc. (Stantec; bridge and highway designer) on 6 September 2024, it is our understanding that the existing bridge was originally constructed in 1952 and consists of an approximate 25-ft long, 24-ft wide (curb-to-curb), single-span structure supported on two cast-in-place (CIP) concrete abutments that are presumably bearing on bedrock. Historic bridge drawings that were provided by MaineDOT are included in Appendix C.

Bridge inspection reports provided by MaineDOT and information included the PDR indicate that the existing superstructure has 50 percent cracking with active efflorescence staining and leakage and the fascia's have extensive cracking, efflorescence, and spalled concrete. The east abutment has deep scaling and cracking with complete section loss at the inlet wingwalls. The west abutment has moderate scaling and deep nested cracking, and the bituminous concrete wearing surface has isolated cracking. For these reasons, the existing bridge is considered structurally deficient and requires replacement.

PROPOSED BRIDGE STRUCTURE

Based on our review of the PDR, it is our understanding that Stantec has developed and evaluated multiple bridge replacement alternatives. Each alternative considered several factors including but not limited to overall project cost, maintenance of traffic, environmental impacts, and future bridge maintenance. The bridge replacement alternative recommended by Stantec in the PDR generally consists of the following:

- An approximate 46-ft long, 26-ft wide (curb-to-curb; 31-ft total width), single-span bridge superstructure that will be constructed using precast, prestressed concrete NEXT 28F beams.
- Substructures will consist of two full-height, cast-in-place (CIP) concrete abutments and footings bearing on seal concrete bearing on bedrock at the locations and elevations shown below.

Substructure	Station at Centerline of Alignment (ft)	Proposed Bottom of Abutment Elevation (ft, NAVD 88)
Abutment No. 1	Sta. 14+46	El. 60
Abutment No. 2	Sta. 14+92	El. 61

- A 24-in. thick roadway section (20 in. of aggregate subbase and 4 in. of bituminous concrete) that is 32-ft wide, which consists of two 11-ft wide travel lanes and two 3-ft wide shoulders.
- A horizontal roadway alignment that will roughly match the existing alignment. The proposed finished grade (vertical roadway profile) will be lowered by up to approximately 1 ft to meet minimum drainage requirements.
- Chases Mill Road will be closed during construction, which will require an approximate 5.5-mile-long detour around the site.

Existing and proposed site conditions are shown in Figure 2, Boring Location Plan.

Geologic Setting

Based on our review of the Maine Geological Survey's (MGS's) Surficial Geologic Map of the Gardiner Lake Quadrangle (see Reference 1), near-surface soils that are present in the vicinity of the site consist of glacial marine deposits (silt, clay, sand) and glacial till consisting of a mixture of sand, silt, clay, and stones. In addition, and according to the MGS's Bedrock Geologic Map of the Gardiner Lake Quadrangle (see Reference 2), bedrock at or in the immediate vicinity of the site is primarily mapped as shale, bedded lapilli, vitrophyre, and rhyolite of the Dennys Formation.

Geotechnical Field Investigations

Preliminary and final design phase geotechnical field investigations were completed at the site in support of the subject project.

The boring locations were laid out in the field by Haley & Aldrich by taping distances from existing site features prior to the start of drilling. "As-drilled" 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 "as-drilled" boring locations and ground surface elevations are summarized in Table I and are shown in Figure 2.

A Haley & Aldrich geologist or geotechnical engineer was present on-site full time to monitor the drilling and to document the soil, bedrock, and groundwater conditions encountered and to prepare boring logs. Soil and bedrock samples were collected and preserved in plastic containers and wooden boxes, respectively, and are currently being stored in our storage facility in Portland. The soil and bedrock samples that were not submitted for laboratory testing are available for review upon request.
All drilling and sampling activities were performed in accordance with MaineDOT requirements.

PRELIMINARY GEOTECHNICAL FIELD INVESTIGATION

Haley & Aldrich completed a preliminary (Phase I) geotechnical field investigation at the site in September 2022. A total of four test borings (borings), designated BB-EMGLO-101 through BB-EMGLO-104, were drilled adjacent to and on either side (east and west) of the existing bridge and in the WB travel lane.

The borings were drilled by New England Boring Contractors (NEBC) of Hermon, Maine using a Mobile Drill B-53 track-mounted drill rig. The borings were drilled to depths ranging from approximately 18 to 25 ft below existing 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 from the existing ground surface and then at standard 5-ft intervals by driving a 1-3/8-in. inside diameter (ID) split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the boring logs. The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler though the middle 12 in. of the 24-in. sampling interval. The drill rig was equipped with a calibrated automatic hammer (hammer) per MaineDOT requirements and Haley & Aldrich reviewed the hammer calibration report provided by NEBC, confirmed that the hammer was calibrated within 12 months of when drilling was completed, and confirmed the hammer efficiency factor. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.922; 92.2 percent theoretical hammer efficiency) divided by 0.6. The energy-corrected SPT N-values (N_{60}) are also shown on the boring logs.

Each boring was advanced approximately 10 to 11 ft into bedrock using a 2-in. (NQ-size) ID diamond-tipped core barrel.

FINAL GEOTECHNICAL FIELD INVESTIGATION

Haley & Aldrich completed a final (Phase II) geotechnical field investigation at the site in June 2025. Two borings, designated BB-EMGLO-201 and BB-EMGLO-202, were drilled adjacent to and on either side (east and west) of the existing bridge and in the EB travel lane.

The Phase II borings were drilled by NEBC to depths ranging from approximately 15 to 22 ft BGS using similar equipment and means and methods that were used to drill and sample the Phase I borings. Similar to the rig that was used to drill the Phase I borings, the drill rig used complete the Phase II borings was also equipped with a calibrated hammer per MaineDOT requirements and Haley & Aldrich reviewed the hammer calibration report provided by NEBC, confirmed that the hammer was calibrated within 12 months of when drilling was completed, and confirmed the hammer efficiency factor. The hammer efficiency factor is equal to (0.812; 81.2 percent theoretical hammer efficiency)

The borings were advanced approximately 5 to 8 ft into bedrock using a 2-in. (NQ-size) ID diamond-tipped core barrel.

Generalized Subsurface Conditions

The subsurface conditions encountered in the borings generally consist of fill soils overlying naturally-deposited marine soils, glacial till, and bedrock. Refer to Table II for a detailed summary of the soil units and thicknesses encountered in each boring. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the boring logs included

Appendix A. Refer to the interpretive subsurface profile in Figure 3 for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Bituminous Concrete	0.3 to 1	An approximate 3 to 12-in. thick layer of bituminous concrete was encountered at the ground surface in each boring.
Fill	5 to 9	Loose to very dense SAND with varying amounts of silt and gravel, dense Sandy GRAVEL with trace silt. Cobbles were encountered in the bottom 1 ft of fill in boring BB-EMGLO-102. *(encountered in each boring)*
Marine Deposit	3 to 9	Very loose to medium dense SAND with varying amounts of silt and gravel. *(encountered in each boring except BB-EMGLO-102 and BB-EMGLO-104)*
Glacial Till	0.4 to 3	Dense to very dense GRAVEL with varying amounts of silt and sand, loose SAND with little gravel and trace silt. (*encountered in each boring except BB-EMGLO-104 and BB-EMGLO-201)*
Bedrock	Bedrock was encountered in each boring at depths ranging from approximately 7 to 14 ft BGS (El. 57 to 64) and slopes up from west to east.	

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

BEDROCK CONDITIONS

As stated previously, approximately 5 to 11 ft of bedrock was sampled in the borings, the top of bedrock surface was encountered at depths ranging between approximately 7 and 14 ft BGS (El. 57 to 64), and the top of bedrock surface generally slopes up from west to east.

The sampled and recovered bedrock generally consisted of very hard, fresh to moderately weathered, grey and white, medium to coarse-grained GRANODIORITE. Primary joints were observed dipping at low to high angles and are very close to widely spaced, tight to open, and joint surfaces were rough.
Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the bedrock encountered in the borings drilled at the site ranged from 10 to 85 percent (average = 57 percent), indicating variable rock quality; from very poor to good in accordance with the MaineDOT Geotechnical Section “Key to Soil and Rock Descriptions and Terms Field Identification Information” document, dated May 2024.

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

GROUNDWATER CONDITIONS

Groundwater observation wells were not installed in any of the and therefore, long-term static water levels at the site could not be determined. Because of this, the following general observations were made relative to groundwater conditions during or immediately after drilling:

- recovered soil samples from each boring were visually observed to be “wet” at approximately 5 to 7 ft BGS (El. 64 to El. 66).
- water levels measured in each boring during or after drilling was approximately 2 to 10 ft BGS (El. 61 to El. 69).

In addition, the following water elevations in Gardner Lake Outlet were provided by Stantec:

Headwater Elevation	Water Elevation (ft, NAVD 88)
$Q_{1.1}$	El. 60.3
Q_{50}	El. 63.2
Q_{100}	El. 63.5

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

Geotechnical Laboratory Testing Program

A geotechnical laboratory testing program was undertaken on representative soil and bedrock samples recovered during the preliminary (Phase I) geotechnical investigation to aid in soil classification and determination of engineering properties. Laboratory testing was performed by GeoTesting Express of Acton, Massachusetts in accordance with applicable American Society for Testing Materials (ASTM) testing procedures. A summary of the laboratory testing and results is provided below.

Laboratory Test	ASTM Test Designation	Geologic Unit	No. of Tests	Range in Test Results[1]
Grain Size	ASTM D6913	Fill	8	AASHTO Classification: A-1-a, A-1-b, A-2-4 USCS Classification: SM, SP-SM, SW-SM, GP-GM
		Marine Deposit	2	AASHTO Classification: A-2-4 USCS Classification: SM
Organic Matter	ASTM D2974	Marine Deposit	1	Organic Matter = 6.7% Moisture Content = 40%
Compressive Strength and Elastic Moduli of Rock	ASTM D7012	Bedrock	7	Peak Compressive Stress: 7,000 to 34,926 psi Bulk Density: 161 to 166 pcf

Notes:
[1] psi = pounds per square in.; pcf = pounds per cubic foot

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

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 presented herein are provided for reference in Appendix D.

APPROACH EMBANKMENTS

As stated above, existing site grades along Chases Mill Road will be lowered by up to approximately 1 ft to meet proposed finish grades. Because of this and because soils subject to long term consolidation settlement were not encountered in the borings drilled at the site, we anticipate that post-construction settlement of the new approach roadways will be negligible.

Embankment stability evaluations were not completed because proposed raises in grade along Chases Mill Road are not anticipated and because the proposed slope inclinations will likely be less than the internal friction angle of soil. It is our opinion that the proposed embankment side slopes will be stable when constructed in accordance with the requirements of the MaineDOT Standard Specifications and Standard Details.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 using Method B. In instances where SPT N-values were equal to 0 (i.e., weight of rod or weight of hammer), were greater than 100 blows per foot (bpf) or where bedrock was present, default values of 1 and 100 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 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) =	0.151
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, S_1 (g) =	0.039
Peak seismic ground acceleration coeff. on rock, PGA (g) =	0.077
Horizontal response spectral acceleration coeff. at 0.2-s period modified by F_a, S_{DS} (g) =	0.181
Horizontal response spectral acceleration coeff. at 1.0-s period modified by F_v, S_{D1} (g) =	0.039
Peak seismic ground acceleration coefficient modified by F_{pga}, A_s (g) =	0.092

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 borings and the results of the laboratory testing, it is our opinion that the potential for saturated granular soils present at the site to liquefy during the design earthquake event is low.

ABUTMENT BREASTWALL AND WINGWALL FOUNDATION SUPPORT

As discussed above and as shown in the interpretive subsurface profile (Figure 3), the subsurface conditions along the proposed bridge alignment consist of fill overlying thin layers of marine and glacial till soils, and bedrock. Based on our discussions with Stantec, it is our understanding that mass concrete spread footings bearing on intact bedrock is the preferred alternative to support the proposed abutment breastwalls and wingwalls.

As stated above, the proposed bottom of Abutment 1 and Abutment 2 footing elevations are El. 60 and El. 61, respectively, which are slightly above the top of bedrock surface encountered in borings nearest the proposed abutment locations. Because of this and because all soil and loose, weathered and/or fractured rock will be removed, it is possible that a concrete seal or sub-footings will be needed to reach the proposed bottom of abutment footing level. Please note however, that variations in the top of bedrock surface can occur over relatively short distances, which could result in the need to excavate minor amounts of bedrock to reach the proposed bottom of abutment footing levels (i.e., if the top of rock surface is above the proposed bottom of abutment footing levels).

Additional foundation recommendations are summarized below. Please note that the design recommendations presented below assume that foundation subgrade preparation is completed in accordance with the recommendations provided in the Construction Considerations section of this GDR as well as the requirements of the Contract Documents (CDs; drawings, Standard Specifications, and Special Provisions).

- Bearing Resistance:
  - For the SER Limit State, mass concrete footings should be designed such that footing contact pressures do not exceed 16 kips per square foot (ksf). According to LRFD Section C10.6.2.5.1., settlement of footings is limited to 1.0 in. when designed for a maximum footing contact pressure of 16 ksf at the SER Limit State.
  - For the STR Limit State, footings should be designed for a factored bearing resistance of 18 ksf, using a resistance factor of 0.45.
  - For the EE Limit State, footings should be designed for a factored bearing resistance of 31 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 that 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 STR Limit State, based on factored loads, should not exceed 45 percent of the spread footing dimension in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the footing 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 breastwall 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, degrees)	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 Bridge Design Guide. If "anchorage" of footings to bedrock (e.g., rock dowels) is needed to provide additional sliding resistance between the footings and bedrock, supplemental geotechnical recommendations can be provided.

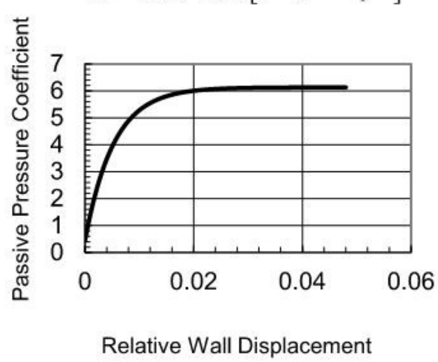
ABUTMENT BREASTWALL AND WINGWALL DESIGN

We recommend that the abutment breastwalls and wingwalls be designed for all relevant SER, STR, and EE Limit States and load combinations specified in AASHTO LRFD Sections 3.4.1 and 11.5.5. Additional recommendations are provided below.

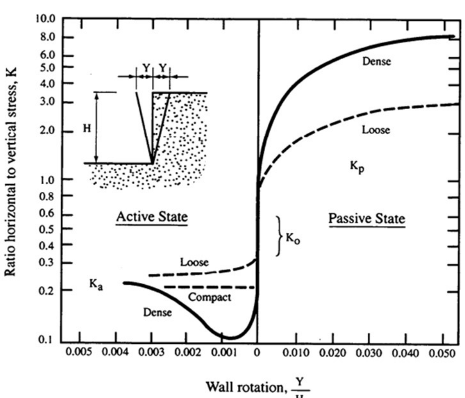
- Drainage:
  - The abutment breastwall and wingwall designs should include a drainage system to intercept any water and direct it to a suitable discharge point. We recommend that French drains be used in accordance with the requirements of Bridge Design Guide Section 5.4.1.9.
- Lateral Earth Pressures:
  - Recommendations presented below are based on the following:
    - Abutment breastwalls, backwalls, 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; interface friction angle = 24 degrees).
    - The abutment breastwall and wingwall backwalls are vertical.
    - Adequate drainage is provided, as recommended above, to eliminate the potential for unbalanced hydrostatic pressures to develop.
    - Backfill surfaces behind the abutment breastwalls are near-horizontal.
    - Backfill surfaces behind the abutment wingwalls vary from near-horizontal to 2H:1V.

- Semi-integral Abutments and Backwalls:
  - 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 uniform pressure distribution due to the height of the soil behind the superstructure. Therefore, considering the proposed roadway grade (+/- 0.5%), we recommend using a Rankine active earth pressure coefficient equal to 0.31.
  - Thermal expansion of the bridge superstructure will cause the backwalls to move towards the backfill, which will result in lateral earth pressures that vary between at-rest and full passive conditions. In accordance with BDG Section 5.4.3, the superstructure backwalls (below the end diaphragm) should be designed for full passive pressure. We recommend using the methodology developed by the Massachusetts Department of Transportation (MassDOT) or the Federal Highway Administration (FHWA), whichever is more stringent.

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$



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



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

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

  - We recommend a load factor for passive earth pressure equal to 1.5 be used to calculate factored passive lateral earth pressures consistent with MaineDOT standard practice.

- Wingwalls:
  - The Coulomb active earth pressure coefficients should be used for wall designs that are "gravity-shaped" or short-heeled, cantilever-types where the top of the stem wall interferes with the shear zone. In addition, Rankine active earth pressure coefficients should be used for long-heeled cantilever-type walls. Coulomb and Rankine active earth pressure coefficients are provided below for various backslope angles (e.g., horizontal, 2H:1V, 1.75H:1V).

Backslope Angle (deg.)	Rankine Active Earth Pressure Coefficient	Coulomb Active Earth Pressure Coefficient
0 (horizontal)	0.31	0.27
26.6 (2H:1V)	0.46	0.46
29.7 (1.75H:1V)	0.56	0.54

- Additional lateral earth pressures due to live load surcharge are required in accordance with BDG Section 3.6.8 for abutments (if approach slabs are not included) and wingwalls. When an abutment 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.

Frost Protection

As stated above, the proposed bridge abutment breastwalls and wingwalls will be supported by spread footings bearing on bedrock. Because of this, and in accordance with Bridge Design Guide Section 5.2.1, no minimum embedment depth for frost protection is required.

Construction Considerations

TEMPORARY EARTH SUPPORT

Based on the proposed elevation of the bottom of both abutments, a temporary support of excavation (SOE) system(s) may be needed to construct the substructures if conventional sloping/berms is not feasible, if the water level in the stream is higher than the top of bedrock surface, and/or use of the existing abutments is not sufficient in allowing excavation and subgrade preparation to occur in-the-dry.

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

BEDROCK SUBGRADE PREPARATION

The nature, slope, and degree of fracturing of the bedrock bearing surface will not be evident until foundation excavation for the abutment breastwalls and wingwalls is completed. We recommend that the bedrock surface be cleared of all loose, fractured and/or weathered/decomposed bedrock and soil (i.e., unsuitable material). If excavation of unsuitable material extends below the bearing level of the footings, we recommend that a tremie seal or sub-footing be placed from the bottom of excavation up to the proposed footing bearing level, after the surface has been inspected in the field by the Resident and/or project Geotechnical Engineer, as discussed below and in accordance with the requirements of the Standard Specifications.

- 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 resistance.
- Based on the proposed bearing level of the abutment breast and wingwall footings and the top of bedrock surface encountered in the borings we do not anticipate the need for bedrock removal using drilling and blasting techniques.
- It is possible that seepage of water from fractures and joints exposed in the bedrock surface could occur. We recommend that water inside of the temporary SOE system(s) 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.

IMPACT TO ADJACENT STRUCTURES

As stated above, town-owned dam and fishway structures are located immediately upstream of the existing bridge. Because of the proximity of these structures to the existing and proposed bridges, demolition and construction activities (e.g., excavation, dewatering, foundation subgrade preparation, placement and compaction of earthfill) have the potential to cause negative impacts. Because of this, we recommend that consideration be given to incorporating the following mitigation measures into the design and construction to reduce the potential for negative impacts to the dam and fishway structures.

- The proposed replacement bridge should be designed such that it is independent of the existing dam and fishway and does not rely on these structures for load resistance nor does the proposed bridge impart additional load(s) onto the dam or fishway.

- Excavation below the bottom and within the zone of influence (ZOI) of the dam and fishway should be prohibited. We recommend that during construction, the Contractor be made responsible for excavating down to and exposing the bottom of the structures and the information (i.e., structure bottom elevations) be used to adjust proposed bottom of footing levels to avoid excavating within the ZOI of the structures.
- Blasting is prohibited.
- Finished site grades in the immediate vicinity of the dam and fishway structures should match current site grades to avoid additional loading on the structures.
- Surface water runoff during and after construction should directed away from the dam and fishway to avoid surficial erosion (loss of material) or ponding of water that could impart additional load onto the structures.
- Construction equipment, material stockpiles, or other items capable of inducing surcharge loads onto the dam and fishway should not be located near the structures. Compaction of backfill materials should be completed using manually operated self-propelled vibratory plate compactors (i.e., not heavy, smooth-drum vibratory rollers).
- Installing instrumentation on the dam and fishway structures prior to the start of construction and monitoring the instrumentation periodically prior to, during, and after construction. We recommend the following instrumentation be installed and monitored by MaineDOT:
  - Seismographs – setup at multiple locations to measure vibrations and associated frequencies. Seismographs should be setup prior to the start of construction and read periodically to obtain baseline reading from which subsequent readings during construction will be compared. We recommend peak particle velocity (PPV) not exceed 0.2 in./sec. between 1 and 10 Hz and increasing linearly (on a logarithmic scale) from 0.2 in./sec. at 10 Hz to 0.9 in./sec. at 100 Hz.
  - Deformation Monitoring Points (DMPs) – setup at multiple locations to measure horizontal and vertical deformations. DMPs should be installed prior to the start of construction and read periodically to obtain baseline reading from which subsequent readings during construction will be compared. We recommend vertical and horizontal deformations not exceed the threshold and limiting values of ¼-in. and ½-in., respectively.
  - The Contractor should be contractually required to comply with the recommended threshold and limiting criteria at all time during construction.

SUBMITTAL REVIEWS

The Contract Documents (CDs; drawings, specifications, special provisions) should be written so that the requirements of the CDs are consistent with the design intent of the geotechnical recommendations outlined herein. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions, and specifications, and to facilitate design changes if subsurface conditions differ from those anticipated prior to the start of construction.

CONSTRUCTION MONITORING

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

- Review of instrumentation data collected prior to, during, and after construction.
- Excavation down to and exposure of the bottom of dam and fishway structures.
- Inspection of bedrock subgrade at Abutment Nos. 1 and 2 prior to reinforcing steel and concrete placement.

Limitations

This report is prepared for the exclusive use of MaineDOT relative to the Chase Mills Bridge replacement project in East Machias, Maine. 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 entities other than MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from MaineDOT and Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by such other person or entity without the written authorization of 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.

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

It is our understanding that this report may be included as a reference document in the documents that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the documents and that the information contained in the documents takes precedence over the information provided in this report.

Closure

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

Sincerely yours,
HALEY & ALDRICH, INC.

Eric Hunstein, E.I.
Geotechnical Engineer

Bryan C. Steinert, P.E.
Senior Associate

STATE OF MAINE BRYAN C. STEINERT No. 12008 LICENSED PROFESSIONAL ENGINEER

Enclosures:

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

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References

1. Map entitled, "Reconnaissance Surficial Geology of the Gardiner Lake Quadrangle, Maine, Open-File No. 82-4," prepared by the Maine Geological Survey, Department of Conservation, dated 1982.

2. Map entitled, "Reconnaissance Bedrock Geology of the Gardiner Lake Quadrangle, Maine, Open-File No. 78-3," prepared by the Maine Geological Survey, Department of Conservation, dated 1978.

TABLE I
Subsurface Exploration Location Data
Chase Mills Bridge No. 5465 over Gardner Lake Outlet
MaineDOT WIN 25529.00
Chases Mill Road, East Machias, Maine

Haley & Aldrich, Inc. File No.: 0205517-001

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Station[4]	Offset Distance and Direction[4,5] (ft)	Horizontal Coordinates (ft)[3]	
				Northing (Y)	Easting (X)
BB-EMGLO-101	71.1	15+03	8.2 RT	336,856	2,430,216
BB-EMGLO-102	71.2	14+88	8.0 RT	336,844	2,430,227
BB-EMGLO-103	71.2	14+44	8.3 RT	336,805	2,430,251
BB-EMGLO-104	71.4	14+27	8.5 RT	336,790	2,430,258
BB-EMGLO-201	70.9	14+97	5.6 LT	336,843	2,430,210
BB-EMGLO-202	71.0	14+37	6.0 LT	336,793	2,430,241

Notes:
1 Boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
2 Ground surface elevations at 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 Horizontal coordinates of borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference the NAD83, Maine 2000 East Zone coordinate system.
4 Station and offset information shown is considered approximate, is relative to the project baseline, and was determined by Haley & Aldrich, Inc.
5 LT = offset distance left of baseline (looking upstation); RT = offset distance right of baseline (looking upstation)

	Individual	Date
Prepared By:	EMH	8/4/2025
Checked By:	DMH	8/15/2025
Reviewed By:	BCS	10/13/2025

TABLE II
Subsurface Exploration Subsurface Data
Chase Mills Bridge No. 5465 over Gardner Lake Outlet
MaineDOT WIN 25529.00
Chases Mill Road, East Machias, Maine

Haley & Aldrich, Inc. File No.: 0205517-001

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Stratigraphic Data[3] (ft)											Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration[2] (ft)
		Bituminous Concrete/Fill[4]			Marine Deposit			Glacial Till			Bedrock			
		Depth to Top	Elev. of Top	Thick.	Depth to Top	Elev. of Top	Thick.	Depth to Top	Elev. of Top	Thick.	Depth to Top	Elev. of Top		
BB-EMGLO-101	71.1	0.0	71.1	6.5	7.0	64.1	3.0	10.0	61.1	1.5	11.5	59.6	21.4	49.7
BB-EMGLO-102	71.2	0.0	71.2	9.6	NE	NE	NE	9.0	62.2	3.2	12.2	59.0	22.5	48.7
BB-EMGLO-103	71.2	0.0	71.2	6.5	7.0	64.2	6.0	13.0	58.2	1.1	14.1	57.1	24.5	46.7
BB-EMGLO-104	71.4	0.0	71.4	6.7	NE	NE	NE	NE	NE	NE	7.0	64.4	17.6	53.8
BB-EMGLO-201	70.9	0.0	70.9	5.0	5.0	65.9	8.6	NE	NE	NE	13.6	57.3	22.0	48.9
BB-EMGLO-202	71.0	0.0	71.0	5.5	5.5	65.5	3.5	9.0	62.0	0.4	9.4	61.6	14.6	56.4

Notes:
1 Boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
2 Ground surface elevations at 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 boring.
4 A 0.3- to 1.0-ft thick layer of bituminous concrete was encountered at the ground surface in each boring.

	Individual	Date
Prepared By:	EMH	8/4/2025
Checked By:	DMH	8/15/2025
Reviewed By:	BCS	10/13/2025

TABLE III
Subsurface Exploration Bedrock Core Data
Chase Mills Bridge No. 5465 over Gardner Lake Outlet
MaineDOT WIN 25529.00
Chases Mill Road, East Machias, Maine

Haley & Aldrich, Inc. File No.: 0205517-001

Test Boring No.[1]	Ground Surface Elevation[2] (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery[3]		Rock Quality Designation[4,5]			Physical Rock Parameters		Peak Comp. Strength[6] (psi)	Lithologic, Rock Mass and Discontinuity Description[7]
			No.	Depth Below Ground Surface (ft) Top	Bottom	Midpoint	Total Length (ft)	Recovered Length (in.)	%	Length (in.)	%	Rock Quality	Weathering	Estimated Field Strength		
BB-EMGLO-101	71.1	NQ (2")	R1	11.5	16.5	14.0	5.0	60	100%	33	55%	Fair	Slight	Very hard	--	Grey and white, medium to coarse-grained Gneissic GRANODIORITE. Primary joints dipping at moderate to high angles, are closely spaced, rough, and tight to open, weakly foliated
			R2	16.5	21.4	19.0	4.9	56	95%	32	54%	Fair	Slight	Very hard	--	
BB-EMGLO-102	71.2	NQ (2")	R1	12.5	17.5	15.0	5.0	60	100%	37	62%	Fair	Slight	Very hard	7,000 (14.9'-15.3')	Grey and white, medium to coarse-grained Gneissic GRANODIORITE. Primary joints dipping at high to low angles. Secondary joints dipping horizontal to high angle, are very close to close, rough, and tight to open, weak to moderately foliated, highly fractured from 15.9 to 16.3 ft
			R2	17.5	22.5	20.0	5.0	58	97%	32	53%	Fair	Slight to Moderate	Very hard	14,117 (18.5'-18.7'); 16,562 (20.5'-20.8')	
BB-EMGLO-103	71.2	NQ (2")	R1	14.5	19.5	17.0	5.0	58	97%	51	85%	Good	Fresh to Slight	Very hard	29,799 (18.8'-19.2')	Grey and white, medium to coarse-grained GRANODIORITE. Primary joints dipping at high and low angles, are very close to wide, rough, and tight to open, weakly foliated
			R2	19.5	24.5	22.0	5.0	55	92%	29	48%	Poor	Fresh to Slight	Very hard	32,933 (21.2'-21.5')	
BB-EMGLO-104	71.4	NQ (2")	R1	7.6	12.6	10.1	5.0	58	97%	33	55%	Fair	Slight to Moderate	Very hard	34,926 (9.7'-10.0')	Grey, medium to coarse-grained Gneissic GRANODIORITE. Primary joints dipping at low and high angles. Secondary joints dipping vertically, are very close to close, rough, and tight to open
			R2	12.6	17.6	15.1	5.0	60	100%	42	70%	Fair	Slight	Very hard	30,814 (16.8'-17.2')	
BB-EMGLO-201	70.9	NQ (2")	R1	14.0	18.0	16.0	4.0	48	100%	5	10%	Very Poor	Fresh to Slight	Very hard	--	Grey and white, fine to coarse-grained GRANODIORITE. Primary joints dipping horizontal to low angle, are close, mostly rough, and are mostly open.
			R2	18.0	22.0	20.0	4.0	48	100%	31	65%	Poor	Fresh to Slight	Very hard	--	
BB-EMGLO-202	71.0	NQ (2")	R1	10.0	14.6	12.3	4.6	54	98%	38	69%	Fair	Fresh to Slight	Very hard	--	Grey and white, fine to coarse-grained GRANODIORITE. Primary joints dipping at low to moderate angles, are close to moderately closed, are rough, and are mostly open.

Notes:
1 Boring locations are shown on Figure 2, Boring Location Plan and Interpretive Subsurface Profile.
2 Ground surface elevations at 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 Total core recovery (TCR) is the length of core recovered divided by the length of the run.
4 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, dated May 2024.
6 Peak compressive strength was determined in accordance with ASTM D7012 - Method D. Refer to Appendix B for the laboratory test results.
7 Refer to the boring logs and rock core photographs in Appendix A for additional information.

	Individual	Date
Prepared By:	EMH	8/4/2025
Checked By:	DMH	8/15/2025
Reviewed By:	BCS	10/13/2025

SITE COORDINATES: 44°45'21"N, 67°21'37"W

MAP SOURCE: USGS

HALEY ALDRICH

CHASE MILLS BRIDGE NO. 5465 OVER GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 3,000 FEET
AUGUST 2025

FIGURE 1

0205517_000_LOCUS HALEYALDRICH\EHUNSTEIN

HALEY ALDRICH

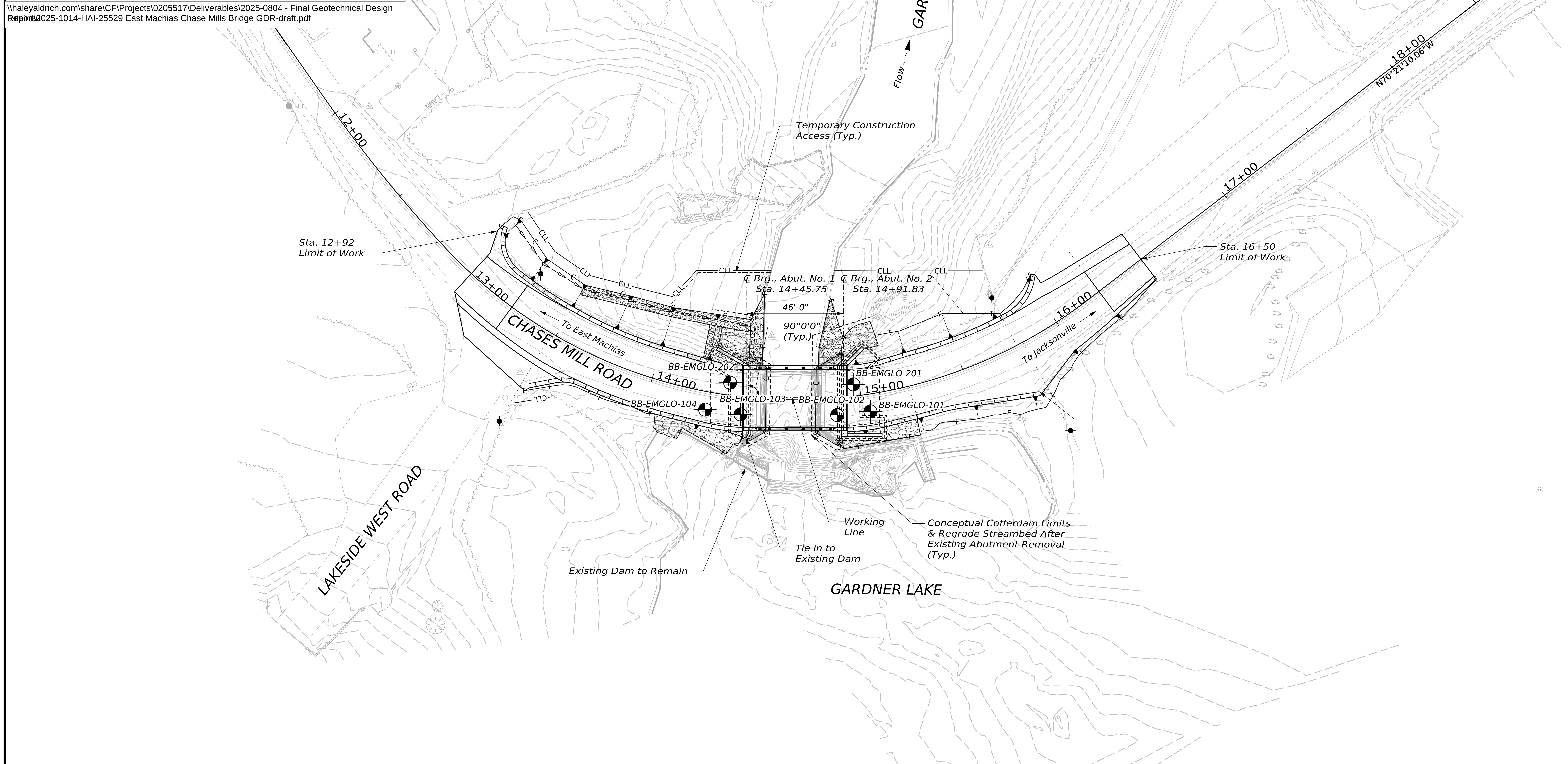
CHASE MILLS BRIDGE NO. 5465 OVER
GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

BORING LOCATION PLAN

SCALE: AS SHOWN
OCTOBER 2025

FIGURE 2

\\haleyaldrich.com\share\CF\Projects\0205517\Deliverables\2025-0804 - Final Geotechnical Design Report\2025-1014-HAI-25529 East Machias Chase Mills Bridge GDR-draft.pdf



LEGEND:

BB-EMGLO-101 — Designation and as-drilled location of preliminary phase test boring drilled by New England Boring Contractors and monitored by Haley & Aldrich, Inc. in August 2023.

BB-EMGLO-201 — Designation and as-drilled location of preliminary phase test boring drilled by New England Boring Contractors and monitored by Haley & Aldrich, Inc. in June 2025.

Location and orientation of interpretive subsurface profile

NOTES:

1. Existing site and topographic information and project stationing were taken from electronic files provided by Stantec on June 19, 2025.
2. As-drilled locations of test borings were determined in the field by Maine DOT using GPS survey equipment.
3. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

HALEY ALDRICH

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

02552900

BRIDGE PLAN
WIN 025529.00
BRIDGE NO. 5465

		BY	DATE
PROJ. MANAGER	M. WIGHT		
DESIGN-DETAILED	KFP/EMH	KFP/EMH	JUN 2025
CHECKED-REVIEWED	EMH/BCS	EMH/BCS	JUN 2025
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

EAST MACHIAS
CHASE MILL BRIDGE

BORING LOCATION PLAN

SHEET NUMBER
5
OF 34

USERNAME:KPOSTOLOWSKI DATE:6/26/2025

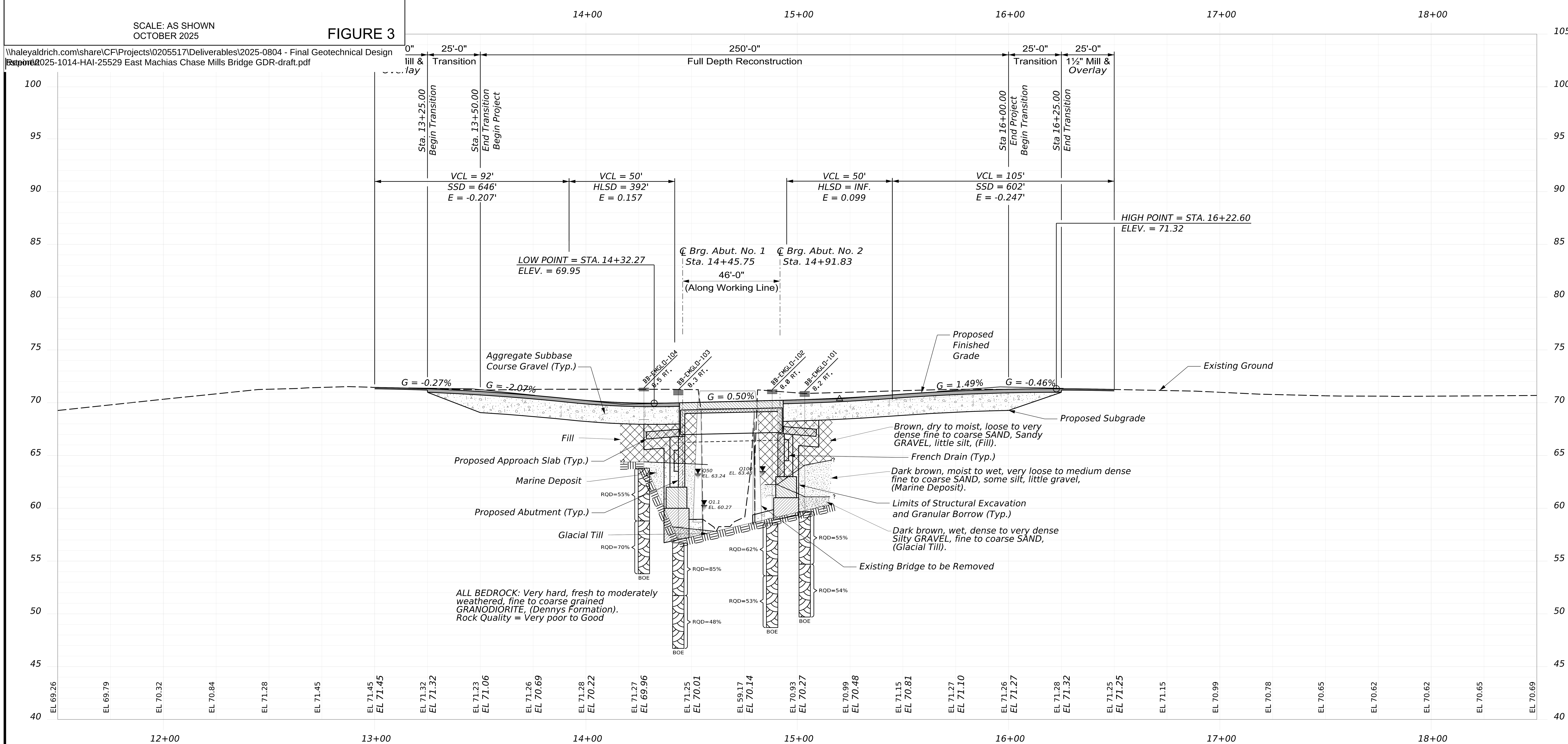
FILENAME:\\HALEYALDRICH.COM\SHARE\CF\PROJECTS\0205517\CAD\CLIENT FILES\2025-0619 - ORD FILES\0205517_PLAN_CHASESMILLRD.DGN

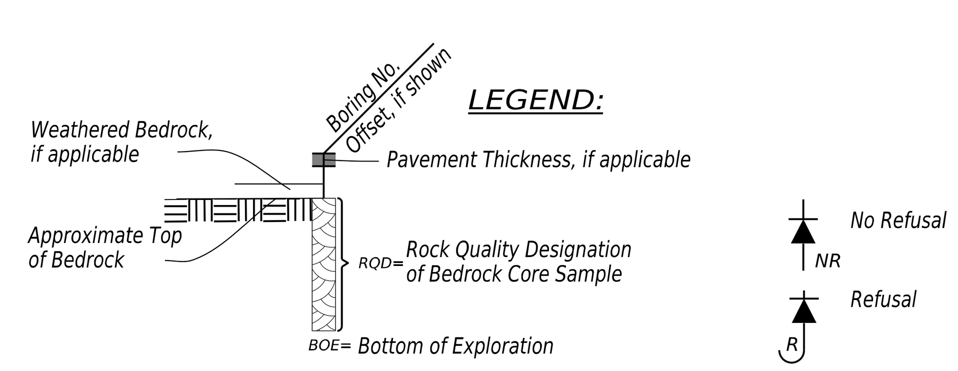
CHASE MILLS BRIDGE NO. 5465 OVER
GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

INTERPRETIVE SUBSURFACE PROFILE

SCALE: AS SHOWN
OCTOBER 2025

FIGURE 3





NOTE:

1. 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 information refer to the exploration logs.
2. This generalized subsurface profile was developed using the borings drilled in the westbound lane of Chases Mill Road (ie., -100 series borings).
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 Haley & Aldrich, Inc.

PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

Scale in Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
2552900
WIN 025529.00
BRIDGE PLANS
BRIDGE NO. 5465

EAST MACHIAS
CHASES MILL ROAD
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
6
OF 34

Username: kpostolowski
Date: 6/26/2025

Filename: \\haleyaldrich.com\share\CF\Projects\0205517\CAD\Client Files\2025-0619 - ORD Files\0205517_Profile_ChasesMillRd.dgn

APPENDIX A
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

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

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_{60}-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_{60}-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))

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: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-101
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	71.1	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#30"
Date Start/Finish:	09-06-2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 15+3.4, 8.2 ft Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	10.0 ft
Hammer Efficiency Factor:	0.922	**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							HW	70.6		-BITUMINOUS CONCRETE- 0.5	
	1D	24/14	1.0 - 3.0	7-17-15-14	32	49				Brown-grey, dry, dense, fine to coarse SAND, some fine gravel, little silt, trace coarse gravel, well-graded, (Fill).	G#690514 A-1-b, SM
							30				
	2D	24/10	3.0 - 5.0	10-15-14-5	29	45	13			Brown, dry, dense, fine to coarse SAND, some fine gravel, little silt, trace coarse gravel, well-graded, (Fill).	G#690514 A-1-b, SM
							12				
5	3D	24/17	5.0 - 7.0	3-4-3-3	7	11	13			Brown to dark brown, moist, medium dense, fine to coarse SAND, some silt, little fine gravel, trace coarse gravel, moderately bonded, contains decomposed wood fragments, (Fill). 7.0	G#690515 A-2-4, SM
							-	64.1			
	4D	24/20	7.0 - 9.0	3-3-2-2	5	8	-			Dark brown, moist, loose, fine to medium SAND, some silt, trace coarse sand, poorly-graded, decomposed wood fragments, trace shells, organic odor, (Marine Deposit).	G#690516 A-2-4, SM OM#690524 OM=6.7% WC=40%
							43				
							40			10.0	
10	5D	15/8	10.0 - 11.3	11-23-37(3")	60	92	-	61.1		Grey-brown, moist, very dense, Silty GRAVEL, some medium to coarse sand, trace fine sand, moderately bonded, (Glacial Till). 11.5	
	R1	60/60	11.5 - 16.5	RQD = 55%			- NQ CORE	59.6		Top of Bedrock El. at 59.6 R1: Very hard, slightly weathered, grey and white, medium to coarse-grained Gneissic GRANODIORITE. Joints are moderately dipping to high angle, close, rough, planar, tight to open. DENNYS FORMATION Rock Mass Quality = Fair Recovery = 100% R1 Core Times (min:sec) 11.5-12.5 ft (5:34) 12.5-13.5 ft (5:15) 13.5-13.8 ft (7:15) 13.8-14.6 ft (5:45) 14.6-15.6 ft (6:15) 15.6-16.5 ft (6:12)	
15											
	R2	59/56	16.5 - 21.4	RQD = 54%						R2: Very hard, slightly weathered, grey and white, medium to coarse-grained GRANODIORITE. Joints dipping at moderate to high angles, close, rough, planar, tight to open, weakly foliated. DENNYS FORMATION Rock Mass Quality = Fair Recovery = 95% R2 Core Times (min:sec) 16.5-17.5 ft (8:30) 17.5-18.1 ft (8:50) 18.1-18.5 ft (3:20) 18.5-19.5 ft (8:15) 19.5-20.5 ft (4:30) 20.5-21.4 ft (9:05)	
20								49.7		21.4	
25										**Bottom of Exploration at 21.4 feet below ground surface.**	
30											

Remarks:

1. OM = Organic Matter

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.

Boring No.: BB-EMGLO-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-102
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	71.2	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	09-07-2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 14+88.1, 8.0 ft Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	10.3 ft
Hammer Efficiency Factor:	0.922	**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							HW	70.8		-BITUMINOUS CONCRETE- 0.4	
	1D	24/11	1.0 - 3.0	16-6-6-10	12	18				Brown, dry, medium dense, medium SAND, some fine gravel, little fine and coarse sand, trace coarse gravel, trace silt, poorly-graded, (Fill).	G#690517 A-1-a, SP-SM
	2D	24/7	3.0 - 5.0	2-7-6-7	13	20				Brown, dry, medium dense, medium SAND, some fine gravel, little fine and coarse sand, trace coarse gravel, trace silt, poorly-graded, (Fill).	G#690517 A-1-a, SP-SM
5	3D	24/9	5.0 - 7.0	9-6-8-8	14	22				Brown, wet, medium dense, medium SAND, some fine gravel, little fine and coarse sand, trace coarse gravel, trace silt, slightly bonded, poorly-graded, (Fill).	G#690518 A-1-b, SP-SM
	4D	24/7	7.0 - 9.0	7-8-11-13	19	29				Brown, wet, medium dense, medium SAND, some fine gravel, little fine and coarse sand, trace coarse gravel, trace silt, moderately bonded, poorly-graded, (Fill).	G#690518 A-1-b, SP-SM
								62.2		9.0 Cobbles	
10	5D	24/2	10.0 - 12.0	7-4-2-1	6	9	27	61.2		10.0 Note: 2-in. gravel fragment.	
							17			Grey-brown, wet, loose, fine to coarse SAND, little fine to coarse gravel, trace silt, well graded, (Glacial Till). 12.2	
	6D	2/2	12.0 - 12.2	16(2")			14	59.0		Top of Bedrock El. at 59.0	
	R1	60/60	12.5 - 17.5	RQD = 62%			NQ CORE			R1: Very hard, slightly weathered, grey and white, medium to coarse-grained Gneissic GRANODIORITE. Joints dipping at high angles. Secondary horizontal joint, very close to close, rough, planar, undulating,tight to open, weakly foliated. Highly fractured zones from 13.0 to 13.5 ft, and approximately 15.9 to 16.3 ft. DENNYS FORMATION Rock Mass Quality = Fair Recovery = 100% R1 Core Times (min:sec) 12.5-13.5 ft (7:05) 13.5-14.5 ft (14:15) 14.5-15.5 ft (7:00) 15.5-16.5 ft (9:20) 16.5-17.5 ft (13:15)	qp=7,000 psi (14.9'-15.3')
15											
	R2	60/58	17.5 - 22.5	RQD = 53%						R2: Very hard, slightly to moderately weathered, grey and white, medium to coarse-grained Gneissic GRANODORITE. Joints horizontal to low angle. Secondary high angle joints, very close to close, rough, planar, tight to open, moderately foliated. DENNYS FORMATION Rock Mass Quality = Fair Recovery = 97% R2 Core Times (min:sec) 17.5-18.5 ft (9:15) 18.5-19.5 ft (7:10) 19.5-20.5 ft (1:30) 20.5-21.5 ft (3:45) 21.5-22.5 ft (2:45) 22.5	qp=14,117 psi (18.5'-18.7') qp=16,562 psi (20.5'-20.8')
20											
								48.7		**Bottom of Exploration at 22.5 feet below ground surface.**	
25											
30											

Remarks:

1. psi = pounds per square inch

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: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-103
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	71.2	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	09-08-2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta, 14+43.7, 8.3 ft Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	8.6 ft
Hammer Efficiency Factor:	0.922	**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							HW	70.7		-BITUMINOUS CONCRETE- 0.5	
	1D	24/9	1.0 - 3.0	13-13-11-14	24	37				Brown, dry, dense, Sandy fine GRAVEL, trace silt, poorly-graded, (Fill).	G#690519 A-1-a, GP-GM
5								66.2		5.0	
	2D	24/4	5.0 - 7.0	2-2-2-3	4	6	6			Dark brown, dry, loose, fine to medium SAND, some fine gravel, little coarse sand, little silt, poorly-graded, (Fill).	G#690520 A-1-b, SM
							13	64.2		7.0	
	3D	24/5	7.0 - 9.0	2-1-1-1	2	3	17			Dark brown, wet, very loose, fine to medium SAND, some silt, trace coarse sand and fine gravel, poorly-graded, slight organic odor, (Marine Deposit).	G#690521 A-2-4, SM
							9				
	MD	24/0	9.0 - 11.0	WOH/24"			-			No recovery	
10							2				
							2				
							1	58.2		13.0	
	5D	13/7	13.0 - 14.1	25-17-9(1")	26	40	21	57.1		Dark brown-grey, wet, dense, Silty fine to coarse GRAVEL, trace fine to coarse sand, trace wood, moderately bonded, (Glacial Till). 14.1	
	R1	60/58	14.5 - 19.5	RQD = 85%			NQ			Top of Bedrock El. at 57.1 R1: Very hard, fresh to slightly weathered, grey and white, medium to coarse-grained GRANODIORITE. Joints dipping at high and low angles, wide spacing, rough, planar, tight to open, weakly foliated. DENNYS FORMATION Rock Mass Quality = Good Recovery = 97% R1 Core Times (min:sec) 14.5-15.5 ft (3:15) 15.5-16.5 ft (2:30) 16.5-17.5 ft (3:20) 17.5-18.5 ft (2:15) 18.5-19.5 ft (2:00)	qp=29,799 psi (18.8'-19.2')
15							CORE				
	R2	60/55	19.5 - 24.5	RQD = 48%						R2: Similar to R1, except joint spacing very close to close. Note: Quartz vein (2-in. thick) at approximately 22.6 ft, and quartz vein from approximately 23.3 to 23.6 ft. DENNYS FORMATION Rock Mass Quality = Poor Recovery = 92% R2 Core Times (min:sec) 19.5-20.5 ft (2:30) 20.5-21.5 ft (3:00) 21.5-22.5 ft (4:00) 22.5-23.5 ft (4:20)	qp=32,933 psi (21.2'-21.5')
20											
								46.7		24.5	
25											
30											

Remarks:

1. psi = pounds per square inch

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-103
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	71.2	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	09-08-2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta, 14+43.7, 8.3 ft Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	8.6 ft
Hammer Efficiency Factor: 0.922		**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.
30										**Bottom of Exploration at 24.5 feet below ground surface.**	
35											
40											
45											
50											
55											
60											

Remarks:

1. psi = pounds per square inch

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Chase Mills Bridge #5465 over Gardner Lake Outlet			**Boring No.:**	BB-EMGLO-104
Location:	Chases Mill Road, East Machias, Maine			**WIN:**	025529.00
Driller:	New England Boring Contractors	**Elevation (ft.)**	71.4	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	J. Ilunga	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	09-12-2022	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 14+27.4, 8.5 ft Rt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	2.3 ft
Hammer Efficiency Factor:	0.922	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HW	71.1	-BITUMINOUS CONCRETE- (0.3)	
	1D	24/17	1.0 - 3.0	13-13-10-5	23	35			Brown, dry, dense, Gravelly fine to coarse SAND, trace silt, well graded, (Fill). (3.0)	G#690522 A-1-a, SW-SM
	2D	24/8	3.0 - 5.0	3-2-3-3	5	8		68.4	Brown, moist, loose, fine to medium SAND, some silt, little coarse sand, trace fine gravel, moderately bonded, poorly-graded, (Fill).	G#690523 A-2-4, SM
5	3D	18/9	5.0 - 6.5	1-1-14(6")	15	23			Dark brown, moist, medium dense, fine to medium SAND, some silt, little coarse sand, trace fine gravel, moderately bonded, poorly-graded, organic silt pocket, (Fill). (7.0)	G#690523 A-2-4, SM
							48	64.4		
	R1	60/58	7.6 - 12.6	RQD = 55%			NQ CORE		Top of Bedrock El. at 64.4 R1: Very hard, slightly to moderately weathered, grey, medium to coarse-grained Gneissic GRANODIORITE. Joints dipping at low and high angles. Secondary vertical angle, very close to close, rough, planar, tight to open. DENNYS FORMATION Rock Quality = Fair Recovery = 97% R1 Core Times (min:sec) 7.6-8.6 ft (4:10) 8.6-9.6 ft (3:15) 9.6-10.6 ft (1:30) 10.6-11.6 ft (3:00) 11.6-12.6 ft (2:30)	qp=34,926 psi (9.7'-10.0')
10										
	R2	60/60	12.6 - 17.6	RQD = 70%					R2: Similar to R1, except slightly weathered. DENNYS FORMATION Rock Quality = Fair Recovery = 100% R2 Core Times (min:sec) 12.6-13.6 ft (6:00) 13.6-14.6 ft (2:30) 14.6-15.6 ft (5:15) 15.6-16.6 ft (1:30) 16.6-17.6 ft (3:00) (17.6)	qp=30,814 psi (16.8'-17.2')
15										
								53.8		
									Bottom of Exploration at 17.6 feet below ground surface.	
20										
25										
30										

Remarks:

1. psi = pounds per square inch

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: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-201
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	70.9	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	E. Hunstein	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	6/2/2025-6/2/2025	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 14+97.0, 5.6 ft Lt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	9.3 ft
Hammer Efficiency Factor:	0.812	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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MU = Unsuccessful Thin Wall Tube Sample Attempt
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MV = Unsuccessful Field Vane Shear Test Attempt
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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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							PUSH		BITUMINOUS CONCRETE	
								70.0	0.9	
	1D	24/5	1.0 - 3.0	18/23/19/20	42	57			Grey-brown, dry, very dense, fine to coarse SAND, some fine to coarse gravel, little silt, (Fill).	
	2D	24/12	3.0 - 5.0	11/6/5/7	11	15			Brown, dry to moist, medium dense, fine SAND, little silt, little fine gravel, little medium to coarse sand, (Fill).	
5								65.9	5.0	
	3D	24/6	5.0 - 7.0	7/10/6/4	16	22			Grey-brown and black, wet, medium dense, fine to coarse SAND, trace silt, trace fine gravel, bottom 1 in. contains more silt, gravel in shoe, (Marine Deposit).	
	4D	24/8	7.0 - 9.0	6/7/7/8	14	19			Similar to 3D, (Marine Deposit).	
	5D	24/2	9.0 - 11.0	8/3/2/2	5	7			Grey-brown and black, wet, loose, fine to medium SAND, (Marine Deposit).	
10										
	6D	24/6	11.0 - 13.0	1/3/2/2	5	7			Grey-brown and black, wet, loose, fine SAND, trace medium sand, trace silt, wood in shoe of spoon, (Marine Deposit).	
	7D	2/2	13.0 - 13.2	50(2")				57.3	Grey-brown and black, wet, very dense, fine SAND, trace silt, wood in shoe of spoon, (Marine Deposit).	
									13.6	
	R1	48/48	14.0 - 18.0	RQD = 10%			NQ CORE		Top of Bedrock at Elev 57.3 R1: Grey and white, fine to coarse grained GRANODIORITE, very hard, fresh to slightly weathered. Joints dipping horizontal to low angle, close, mostly rough, fresh to discolored, mostly open, minor soil infilling, infrequent quartz intrusions. DENNYS FORMATION Rock Quality = Very Poor Recovery = 100% R1 Core Times (min:sec) 14.0-15.0 ft (3:08) 15.0-16.0 ft (3:08) 16.0-17.0 ft (2:56) 17.0-18.0 ft (3:41)	
15										
	R2	48/48	18.0 - 22.0	RQD = 65%					R2: Grey, white, purple, fine to coarse grained GRANODIORITE, very hard, fresh to slightly weathered. Joints dipping horizontal to low angle, close, mostly rough, mostly fresh, mostly open, no infilling, frequent quartz intrusions. DENNYS FORMATION Rock Quality = Poor Recovery = 100% R2 Core Times (min:sec) 18.0-19.0 ft (2:59) 19.0-20.0 ft (2:52) 20.0-21.0 ft (3:16) 21.0-22.0 ft (3:24)	
20										
								48.9		
									22.0	
25									**Bottom of Exploration at 22.0 feet below ground surface.**	
30										

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.

Boring No.: BB-EMGLO-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Chase Mills Bridge #5465 over Gardner Lake Outlet
Location: Chases Mill Road, East Machias, Maine

Boring No.: BB-EMGLO-202
WIN: 025529.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	71.0	**Auger ID/OD:**	--
Operator:	G. McDougal	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	E. Hunstein	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	SS-140#/30";HW-140#/30"
Date Start/Finish:	6/2/2025-6/2/2025	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2.0 in. ID
Boring Location:	Sta. 14+37.1, 6.0 ft Lt.	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	
Hammer Efficiency Factor:	0.812	**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							PUSH			BITUMINOUS CONCRETE	
	1D	24/8	1.0 - 3.0	29/16/9/8	25	34		70.0		1.0 — Brown-grey, dry, dense, fine to coarse SAND, little fine to coarse gravel, trace silt, asphalt pieces throughout sample, well graded, (Fill).	
	2D	24/6	3.0 - 5.0	6/8/7/7	15	20				Brown, dry, medium dense, fine to medium SAND, some fine to coarse gravel, little silt, (Fill).	
5	3D	24/1	5.5 - 7.5	2/1/WOH/WOH				65.5		5.5 — Brown, wet, very loose, fine to coarse SAND, little fine gravel, little silt, well graded, (Marine Deposit).	
	4D	18/6	7.5 - 9.0	1/WOH/WOH						Grey-brown, wet, very loose, fine SAND, some silt, little fine to coarse gravel, trace medium to coarse sand, trace wood, (Marine Deposit).	
	5D	4/4	9.0 - 9.3	50(4")			RC	62.0		9.0 — Brown, wet, very dense, coarse GRAVEL, some fine to coarse sand, trace fine gravel, trace silt, (Glacial Till).	
10	R1	55/54	10.0 - 14.6	RQD = 69%			NQ CORE	61.6		9.4 — Top of Bedrock at Elev 61.6 Note: Advanced roller cone to 10.0 ft R1: Grey and white, fine to coarse grained GRANODIORITE, very hard, fresh to slightly weathered. Joints dipping at low to moderate angles, close to moderate, rough, fresh to discolored, mostly open, no infilling. DENNYS FORMATION Rock Quality = Fair Recovery = 98% R1 Core Times (min:sec) 10.0-11.0 ft (2:33) 11.0-12.0 ft (2:25) 12.0-13.0 ft (2:07) 13.0-14.0 ft (2:15) 14.0-14.6 ft (2:37)	
15								56.4		14.6 — **Bottom of Exploration at 14.6 feet below ground surface.**	
20											
25											
30											

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.

ROCK CORE PHOTOGRAPHS
CHASE MILLS BRIDGE NO. 5465 OVER GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

Top Row: BB-EMGLO-101: Run No. R1 11.5’ (left) to 16.5’ (right)
Top Middle Row: BB-EMGLO-101: Run No. R2 16.5’ (left) to 21.4’ (right)
Bottom Middle Row: BB-EMGLO-102: Run No. R1 12.5’ (left) to 17.5’ (right)
Bottom Row: BB-EMGLO-102: Run No. R2 17.5’ (left) to 22.5’ (right)

ROCK CORE PHOTOGRAPHS
CHASE MILLS BRIDGE NO. 5465 OVER GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

Top Row: BB-EMGLO-103: Run No. R1 14.5’ (left) to 19.5’ (right)
Top Middle Row: BB-EMGLO-103: Run No. R2 19.5’ (left) to 24.5’ (right)
Bottom Middle Row: BB-EMGLO-104: Run No. R1 7.6’ (left) to 12.6’ (right)
Bottom Row: BB-EMGLO-104: Run No. R2 12.6’ (left) to 17.6’ (right)

ROCK CORE PHOTOGRAPHS
CHASE MILLS BRIDGE NO. 5465 OVER GARDNER LAKE OUTLET
MAINEDOT WIN 25529.00
CHASES MILL ROAD, EAST MACHIAS, MAINE

Top Row: BB-EMGLO-202: Run No. R1 10.0’ (left) to 14.6’ (right)
Top Middle Row: BB-EMGLO-201: Run No. R1 14.0’ (left) to 18.0’ (right)
Bottom Middle Row: BB-EMGLO-201: Run No. R1 18.0’ (left) to 22.0’ (right)
Bottom Row: Empty

APPENDIX B
Laboratory Test Results

Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246

Boring ID:	BB-EMGLO-101	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	10/24/22	Checked By:	bfs
Depth :	7.0'-9.0'	Test Id:	690524		

Test Comment: ---
Visual Description: Moist, very dark brown silty sand with gravel and organics
Sample Comment: ---

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content,%	Ash Content,%	Organic Matter,%
BB-EMGLO-101	4D	7.0'-9.0'	Moist, very dark brown silty sand with gravel and organics	40	93.3	6.7

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

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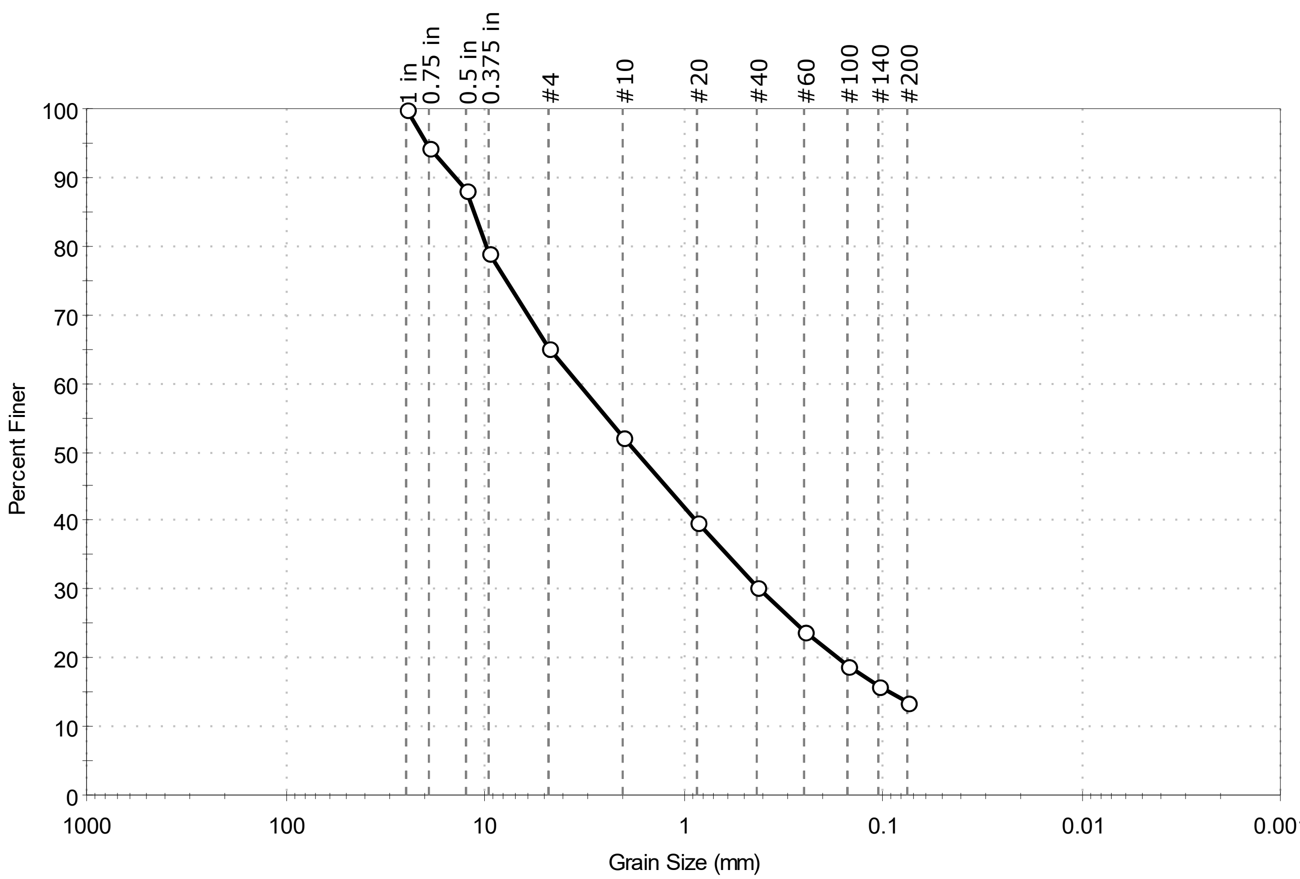
Client:	Haley & Aldrich, Inc.
Project:	Chase Mills Bridge 5465
Location:	East Machias, ME
Project No:	GTX-316246
Boring ID:	BB-EMGLO-101
Sample ID:	1D+2D
Depth :	1.0-5.0'
Sample Type:	jar
Test Date:	10/24/22
Test Id:	690514
Tested By:	ckg
Checked By:	bfs
Test Comment:	---
Visual Description:	Moist, brown silty sand with gravel
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.8	51.8	13.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	88		
0.375 in	9.50	79		
#4	4.75	65		
#10	2.00	52		
#20	0.85	40		
#40	0.42	30		
#60	0.25	24		
#100	0.15	19		
#140	0.11	16		
#200	0.075	13		

Coefficients

D_{85} =11.3884 mm | D_{30} =0.4148 mm

D_{60} =3.3428 mm | D_{15} =0.0930 mm

D_{50} =1.7035 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

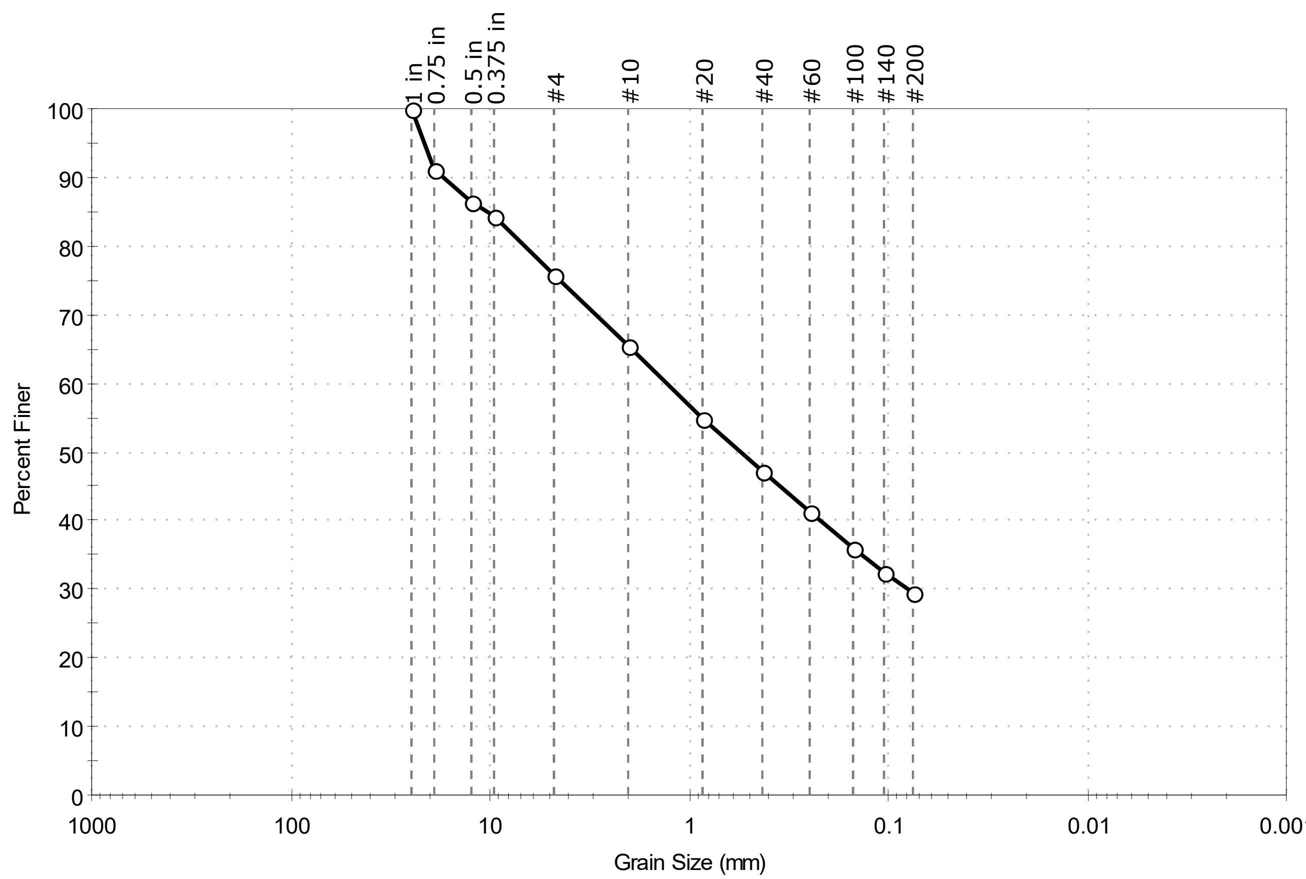
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:31 AM

Client:	Haley & Aldrich, Inc.
Project:	Chase Mills Bridge 5465
Location:	East Machias, ME
Project No:	GTX-316246
Boring ID:	BB-EMGLO-101
Sample Type:	jar
Tested By:	ckg
Sample ID:	3D
Test Date:	10/24/22
Checked By:	bfs
Depth :	5.0'-7.0'
Test Id:	690515
Test Comment:	---
Visual Description:	Moist, reddish brown silty sand with gravel
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	24.3	46.1	29.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	87		
0.375 in	9.50	84		
#4	4.75	76		
#10	2.00	65		
#20	0.85	55		
#40	0.42	47		
#60	0.25	41		
#100	0.15	36		
#140	0.11	32		
#200	0.075	30		

Coefficients

D_{85} =10.3614 mm

D_{30} =0.0785 mm

D_{60} =1.2886 mm

D_{15} =N/A

D_{50} =0.5510 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

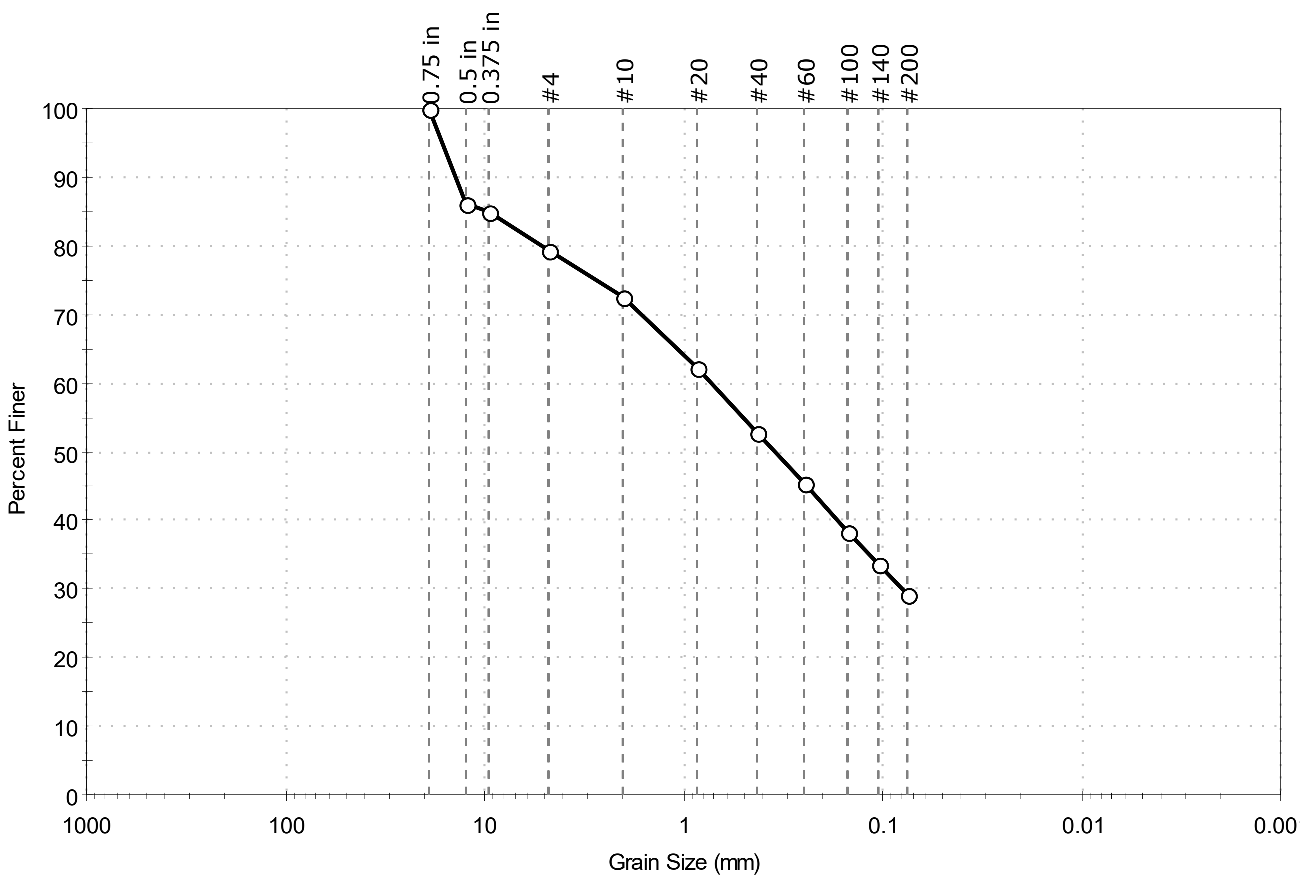
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:34 AM

Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID:	BB-EMGLO-101	Sample Type: jar	Tested By: ckg
Sample ID:	4D	Test Date: 10/24/22	Checked By: bfs
Depth :	7.0'-9.0'	Test Id: 690516	
Test Comment:	---		
Visual Description:	Moist, very dark brown silty sand with gravel and organics		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.6	50.2	29.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	86		
0.375 in	9.50	85		
#4	4.75	79		
#10	2.00	72		
#20	0.85	62		
#40	0.42	53		
#60	0.25	45		
#100	0.15	38		
#140	0.11	34		
#200	0.075	29		

Coefficients

D_{85} =9.7267 mm
D_{30} =0.0798 mm
D_{60} =0.7205 mm
D_{15} =N/A
D_{50} =0.3475 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

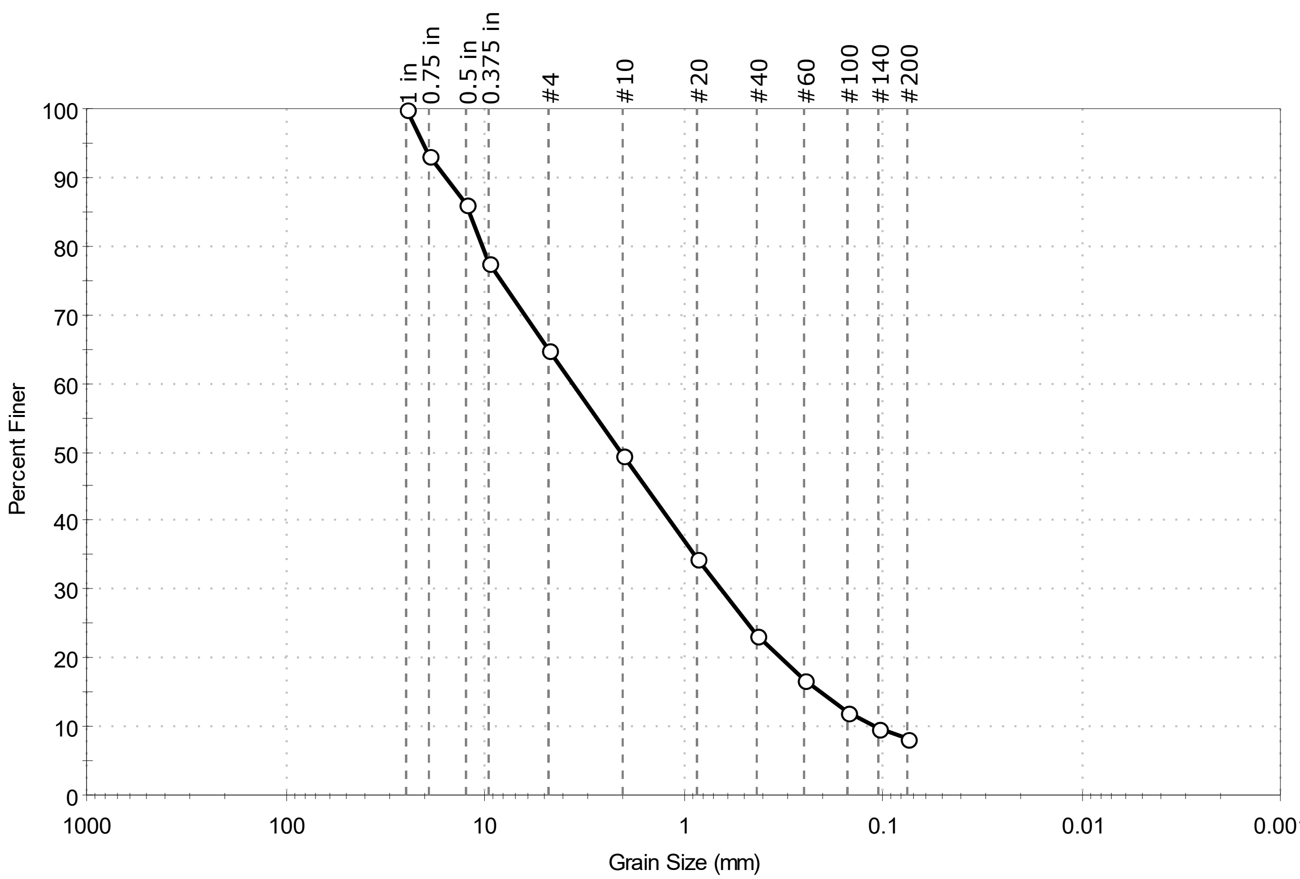
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:37 AM

Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID: BB-EMGLO-102	Sample Type: jar	Tested By:	ckg
Sample ID: 1D+2D	Test Date: 10/24/22	Checked By:	bfs
Depth : 1.0'-5.0'	Test Id: 690517		
Test Comment:	---		
Visual Description:	Moist, brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	35.1	56.6	8.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	93		
0.5 in	12.50	86		
0.375 in	9.50	78		
#4	4.75	65		
#10	2.00	50		
#20	0.85	34		
#40	0.42	23		
#60	0.25	17		
#100	0.15	12		
#140	0.11	10		
#200	0.075	8.3		

Coefficients

D_{85} =12.0480 mm D_{30} =0.6426 mm

D_{60} =3.6006 mm D_{15} =0.2066 mm

D_{50} =2.0341 mm D_{10} =0.1081 mm

C_u =33.308 C_c =1.061

Classification

ASTM N/A

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

Sample/Test Description

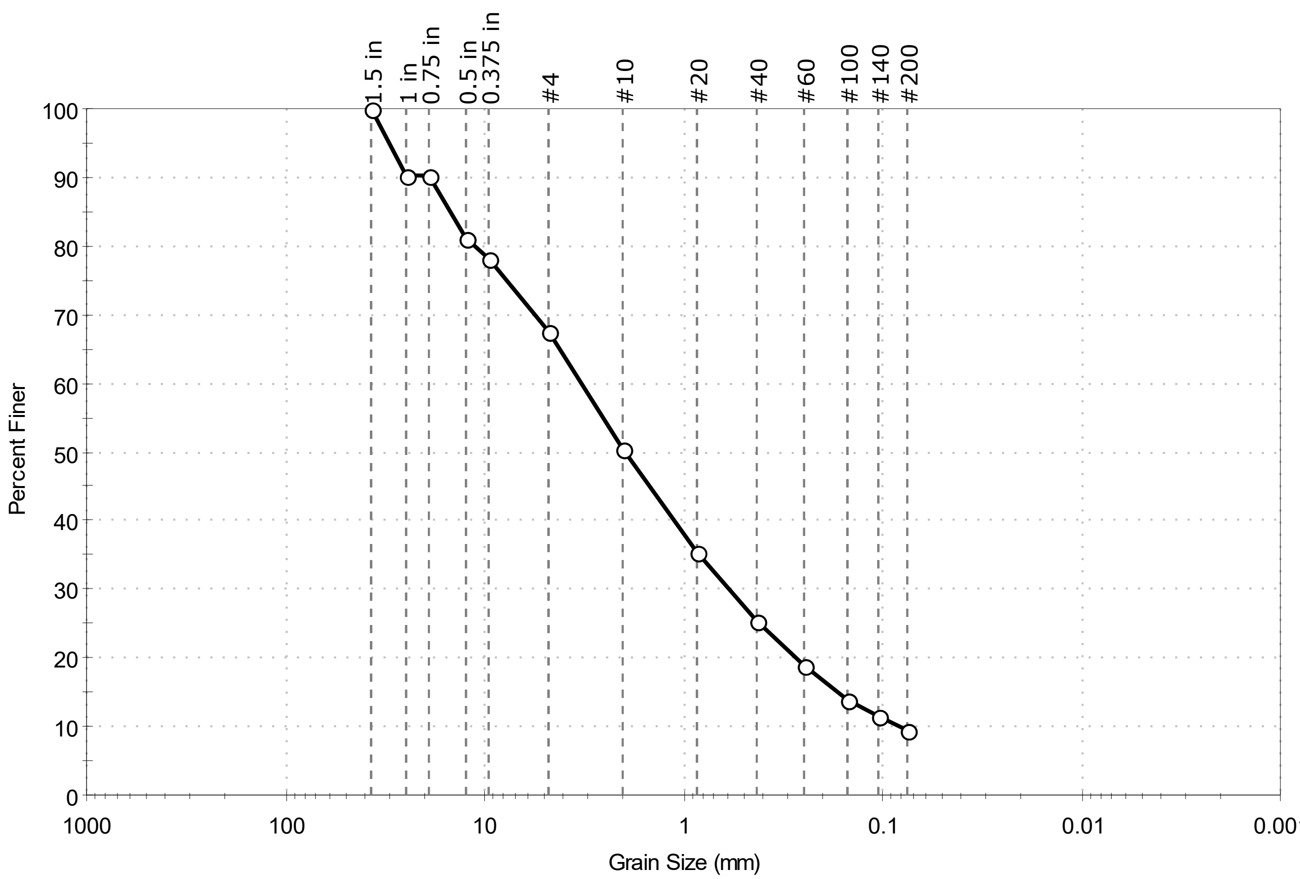
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:38 AM

Client:	Haley & Aldrich, Inc.
Project:	Chase Mills Bridge 5465
Location:	East Machias, ME
Project No:	GTX-316246
Boring ID:	BB-EMGLO-102
Sample Type:	jar
Tested By:	ckg
Sample ID:	3D+4D
Test Date:	10/24/22
Checked By:	bfs
Depth :	5.0'-9.0'
Test Id:	690518
Test Comment:	---
Visual Description:	Moist, dark brown sand with silt and gravel
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	32.5	58.0	9.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	90		
0.75 in	19.00	90		
0.5 in	12.50	81		
0.375 in	9.50	78		
#4	4.75	67		
#10	2.00	51		
#20	0.85	35		
#40	0.42	25		
#60	0.25	19		
#100	0.15	14		
#140	0.11	11		
#200	0.075	9.5		

Coefficients

D_{85} =14.9443 mm
D_{30} =0.5816 mm
D_{60} =3.2480 mm
D_{15} =0.1666 mm
D_{50} =1.9402 mm
D_{10} =0.0823 mm
C_u =39.465
C_c =1.265

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:39 AM

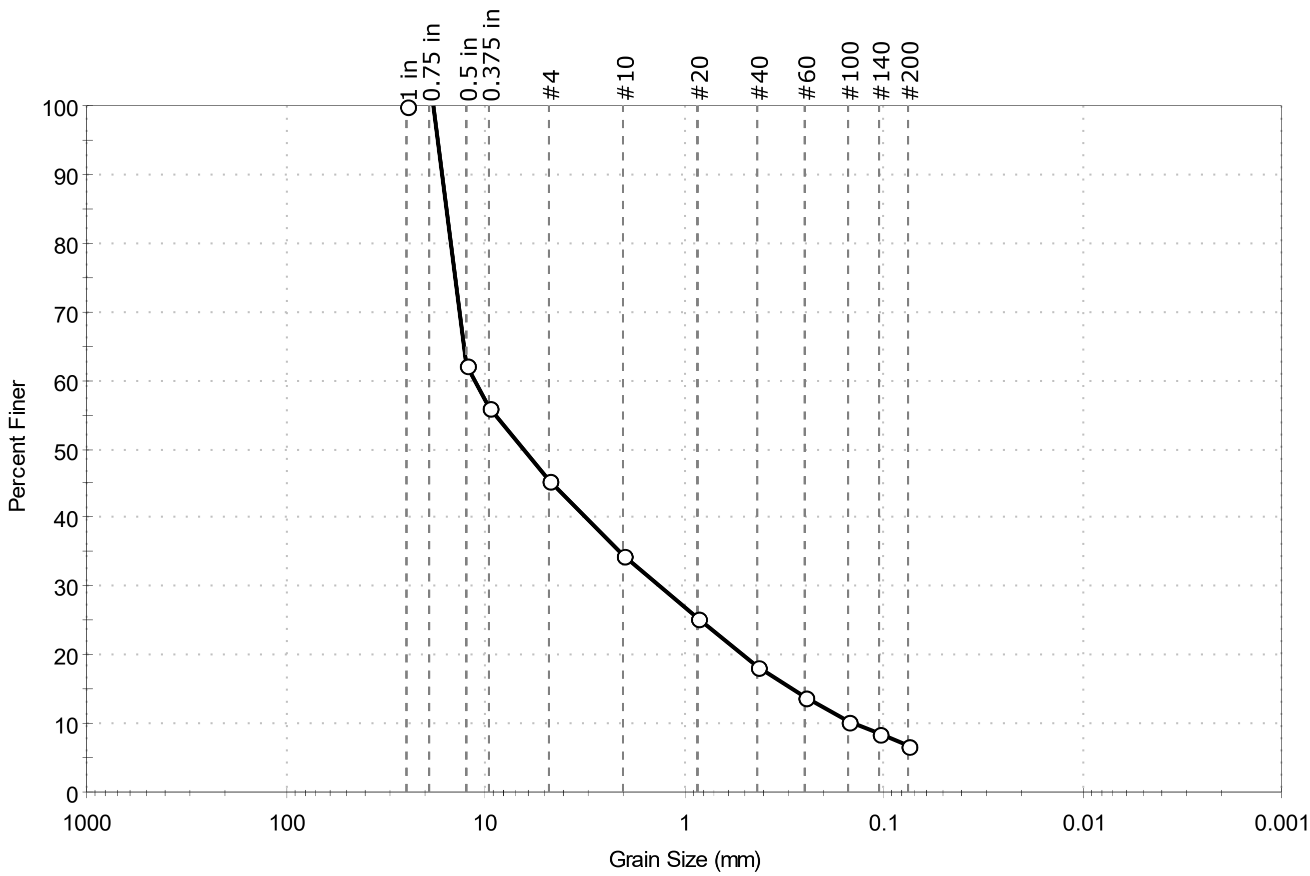
Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID: BB-EMGLO-103	Sample Type: jar	Tested By:	ckg
Sample ID: 1D	Test Date: 10/24/22	Checked By:	bfs
Depth : 1.0'-3.0'	Test Id: 690519		

Test Comment: ---

Visual Description: Moist, brown gravel with silt and sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	54.7	38.4	6.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.5 in	12.50	62		
0.375 in	9.50	56		
#4	4.75	45		
#10	2.00	35		
#20	0.85	25		
#40	0.42	18		
#60	0.25	14		
#100	0.15	10		
#140	0.11	8		
#200	0.075	6.9		

Coefficients

D_{85} =15.7545 mm	D_{30} =1.3105 mm
D_{60} =11.3529 mm	D_{15} =0.2868 mm
D_{50} =6.4146 mm	D_{10} =0.1397 mm
C_u =81.266	C_c =1.083

Classification

ASTM N/A

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

Sample/Test Description

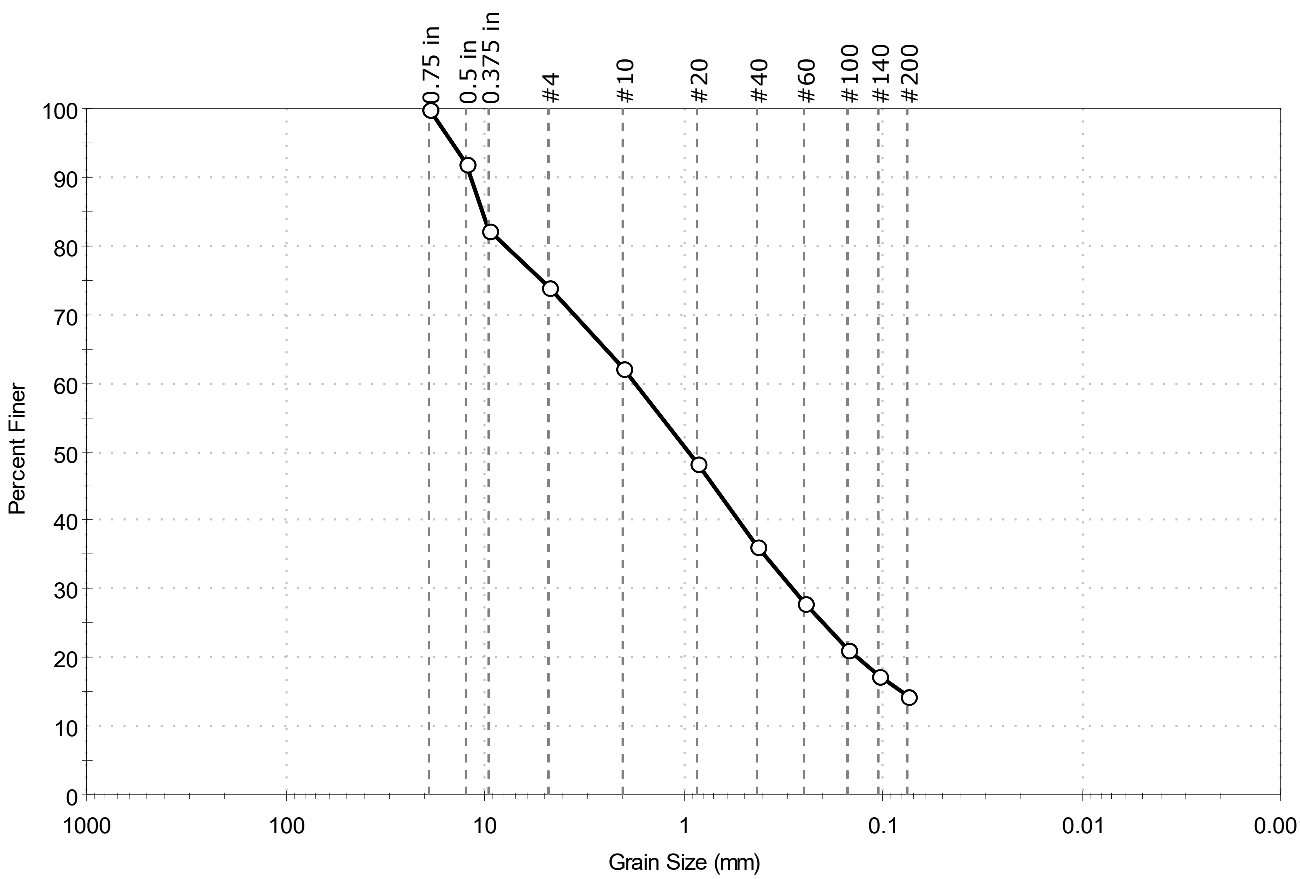
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:40 AM

Client:	Haley & Aldrich, Inc.				
Project:	Chase Mills Bridge 5465				
Location:	East Machias, ME			Project No:	GTX-316246
Boring ID:	BB-EMGLO-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	10/24/22	Checked By:	bfs
Depth :	5.0'-7.0'	Test Id:	690520		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	26.1	59.5	14.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	92		
0.375 in	9.50	82		
#4	4.75	74		
#10	2.00	62		
#20	0.85	49		
#40	0.42	36		
#60	0.25	28		
#100	0.15	21		
#140	0.11	17		
#200	0.075	14		

Coefficients

D_{85} =10.2501 mm D_{30} =0.2850 mm

D_{60} =1.7366 mm D_{15} =0.0800 mm

D_{50} =0.9319 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

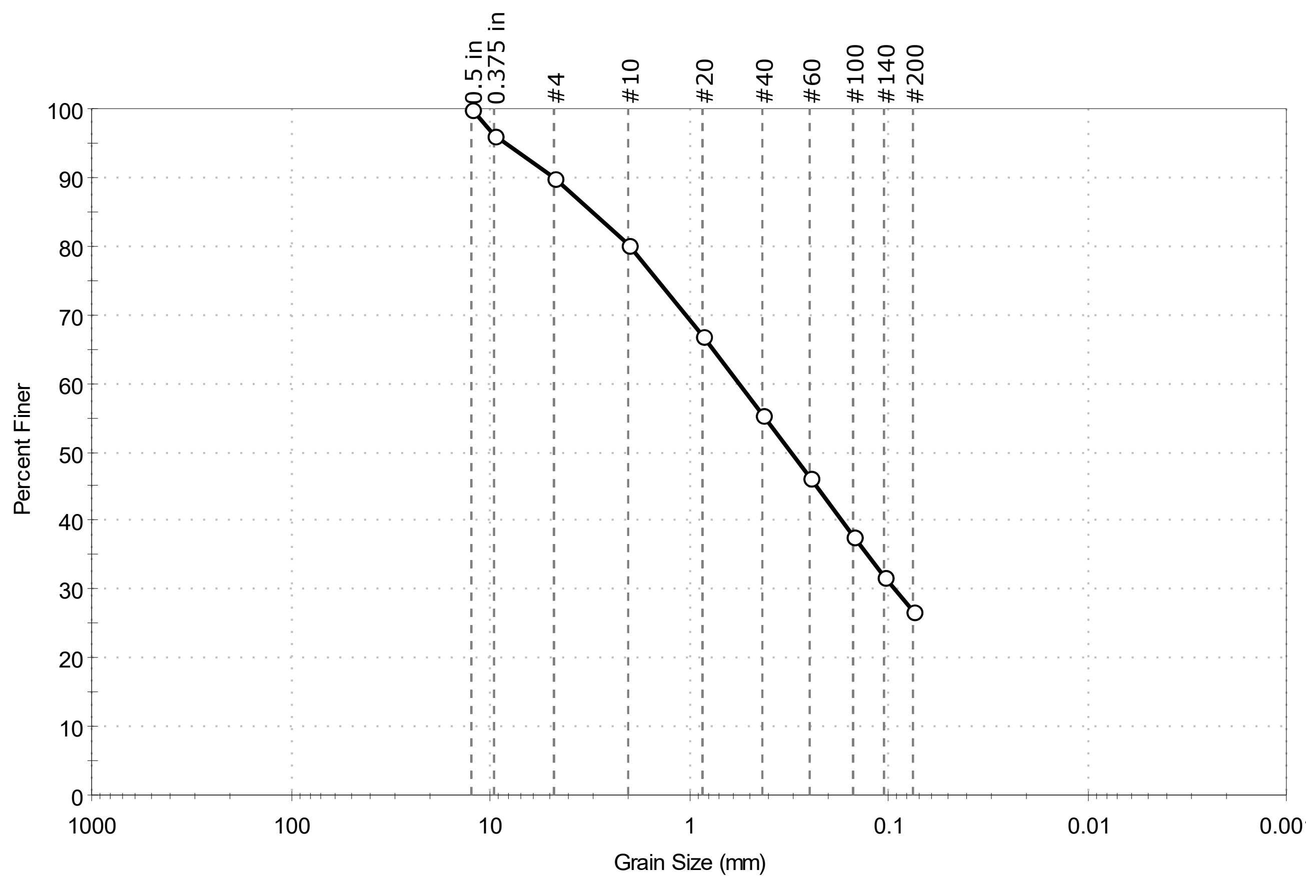
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:41 AM

Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID: BB-EMGLO-103	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 10/24/22	Checked By:	bfs
Depth : 7.0'-9.0'	Test Id: 690521		
Test Comment:	---		
Visual Description:	Moist, dark brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.0	63.2	26.8

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	90		
#10	2.00	80		
#20	0.85	67		
#40	0.42	55		
#60	0.25	46		
#100	0.15	38		
#140	0.11	32		
#200	0.075	27		

Coefficients

D_{85} =3.0408 mm | D_{30} =0.0929 mm
D_{60} =0.5621 mm | D_{15} =N/A
D_{50} =0.3097 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

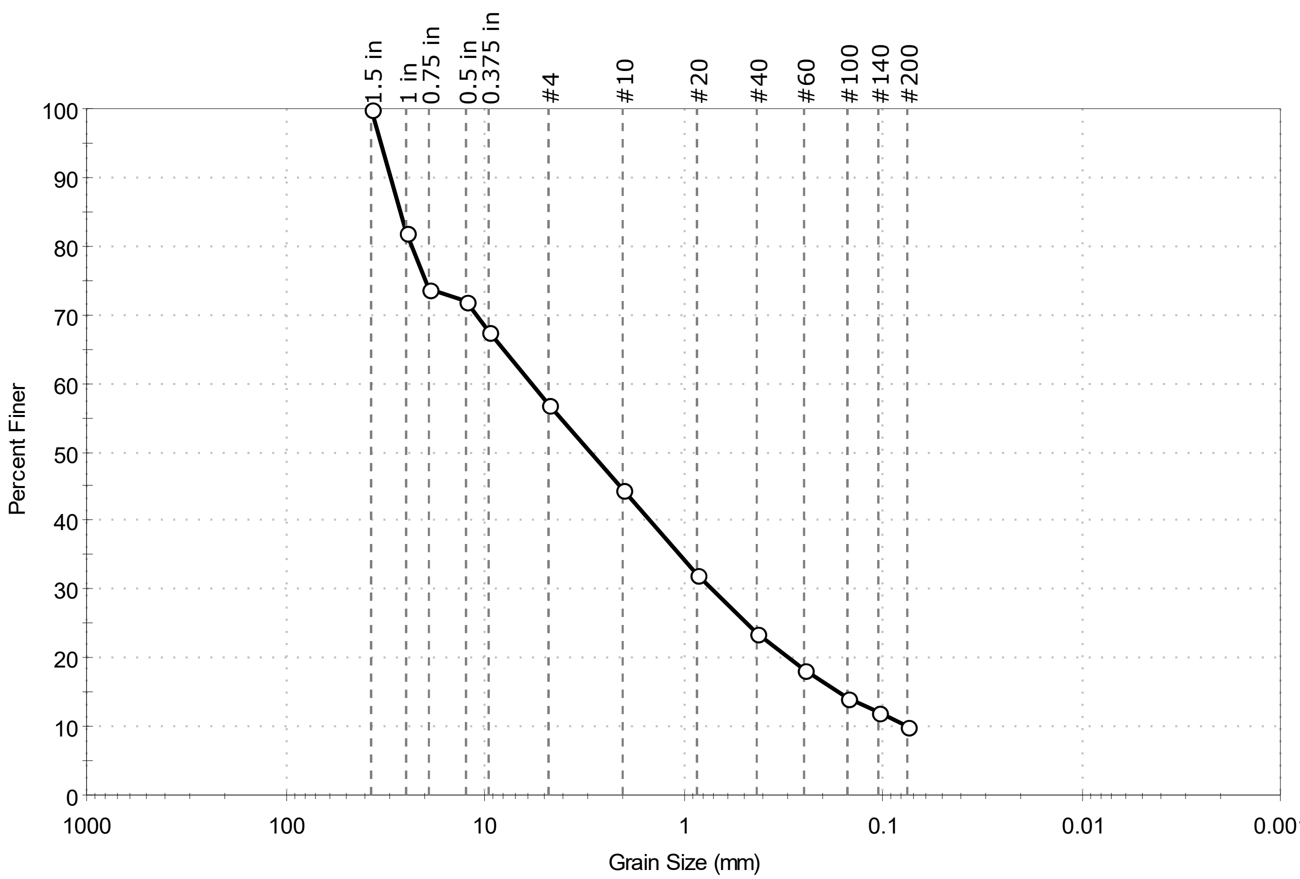
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 10/25/2022 10:51:42 AM

Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID: BB-EMGLO-104	Sample Type: jar	Tested By:	ckg
Sample ID: 1D	Test Date: 10/24/22	Checked By:	bfs
Depth : 1.0'-3.0'	Test Id: 690522		
Test Comment:	---		
Visual Description:	Moist, brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	43.2	46.8	10.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	82		
0.75 in	19.00	74		
0.5 in	12.50	72		
0.375 in	9.50	67		
#4	4.75	57		
#10	2.00	45		
#20	0.85	32		
#40	0.42	24		
#60	0.25	18		
#100	0.15	14		
#140	0.11	12		
#200	0.075	10		

Coefficients

D_{85} =26.7392 mm	D_{30} =0.7108 mm
D_{60} =5.8338 mm	D_{15} =0.1649 mm
D_{50} =2.9287 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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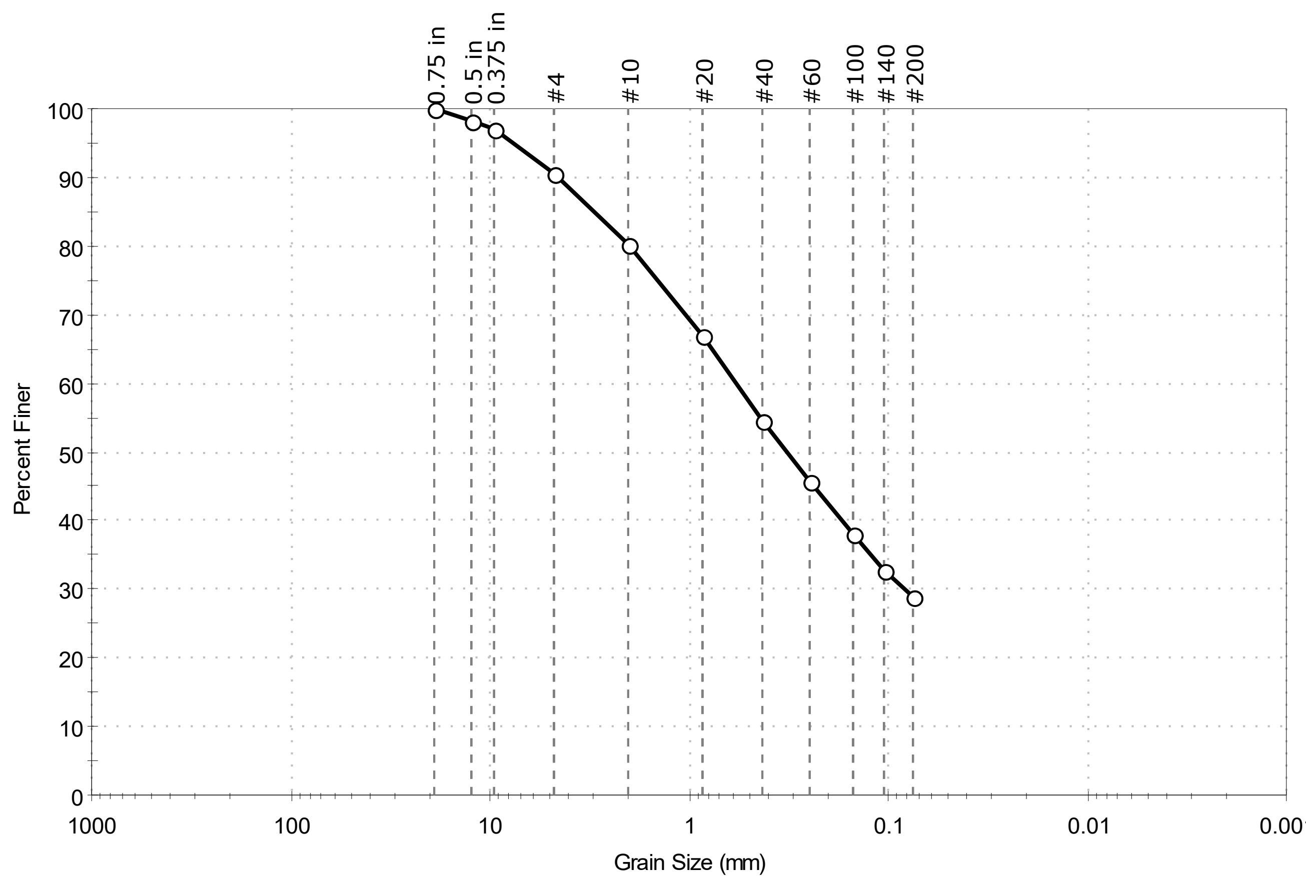
Client:	Haley & Aldrich, Inc.		
Project:	Chase Mills Bridge 5465		
Location:	East Machias, ME	Project No:	GTX-316246
Boring ID:	BB-EMGLO-104	Sample Type: jar	Tested By: ckg
Sample ID:	2D+3D	Test Date: 10/24/22	Checked By: bfs
Depth :	3.0'-6.5'	Test Id: 690523	
Test Comment:	---		
Visual Description:	Moist, dark brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.3	61.9	28.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	98		
0.375 in	9.50	97		
#4	4.75	91		
#10	2.00	80		
#20	0.85	67		
#40	0.42	55		
#60	0.25	46		
#100	0.15	38		
#140	0.11	33		
#200	0.075	29		

Coefficients

D_{85} =2.9606 mm
D_{30} =0.0832 mm
D_{60} =0.5777 mm
D_{15} =N/A
D_{50} =0.3227 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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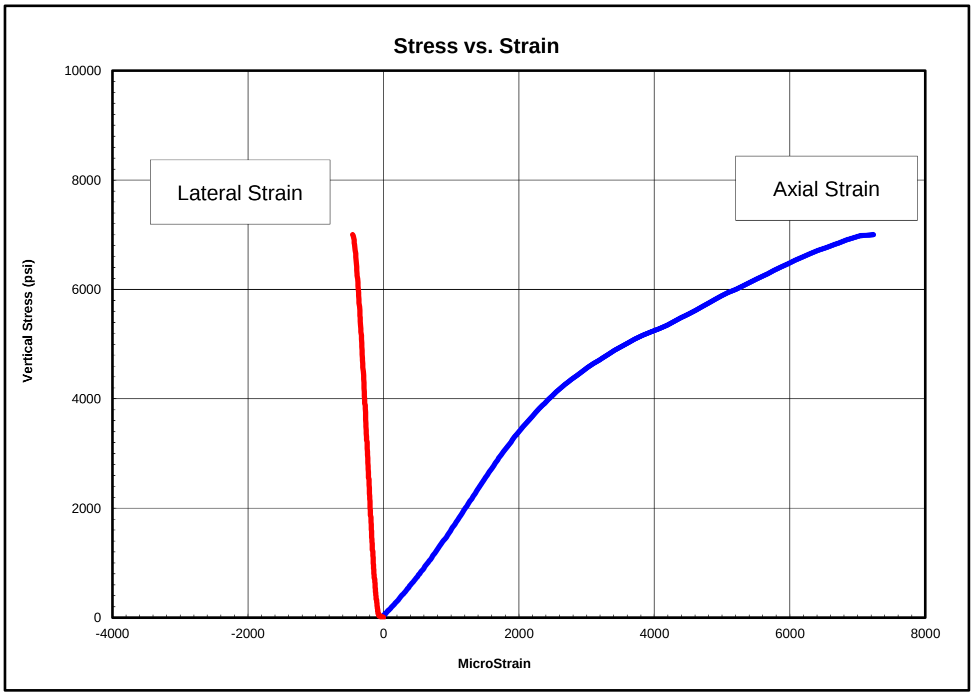
Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-102
Sample ID:	R1
Depth, ft:	14.9-15.28
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 7,000 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
700-2600	1,800,000	0.08
2600-4400	1,420,000	0.06
4400-6300	642,000	0.03

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:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-102		
Sample ID:	R1		
Depth:	14.9-15.28 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:	4.35	4.35	4.35		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	591.45				
Bulk Density, lb/ft^3	165			**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	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00040	-0.00050
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	-0.00010	-0.00020	-0.00030

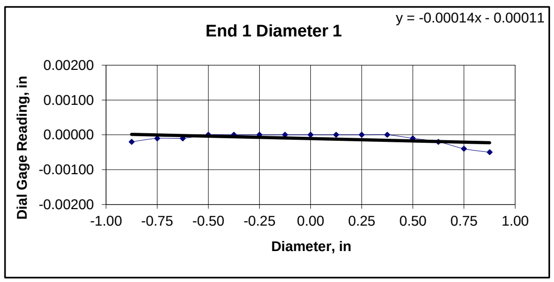
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00030

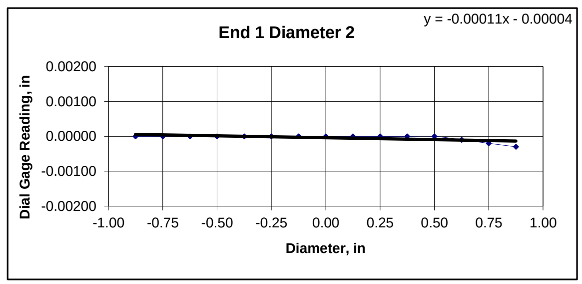
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

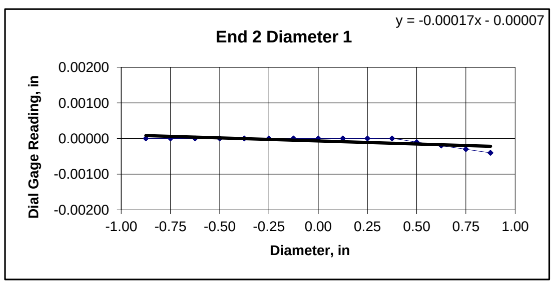
Difference between max and min readings, in:
0° = 0.0004 90° = 0.0002

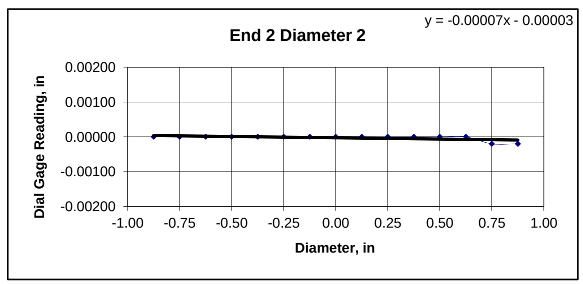
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00014
Angle of Best Fit Line:	0.00786
End 2:	
Slope of Best Fit Line	0.00017
Angle of Best Fit Line:	0.00982
Maximum Angular Difference:	0.00196

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00622
End 2:	
Slope of Best Fit Line	0.00007
Angle of Best Fit Line:	0.00426
Maximum Angular Difference:	0.00196

Parallelism Tolerance Met? **YES**
Spherically Seated

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

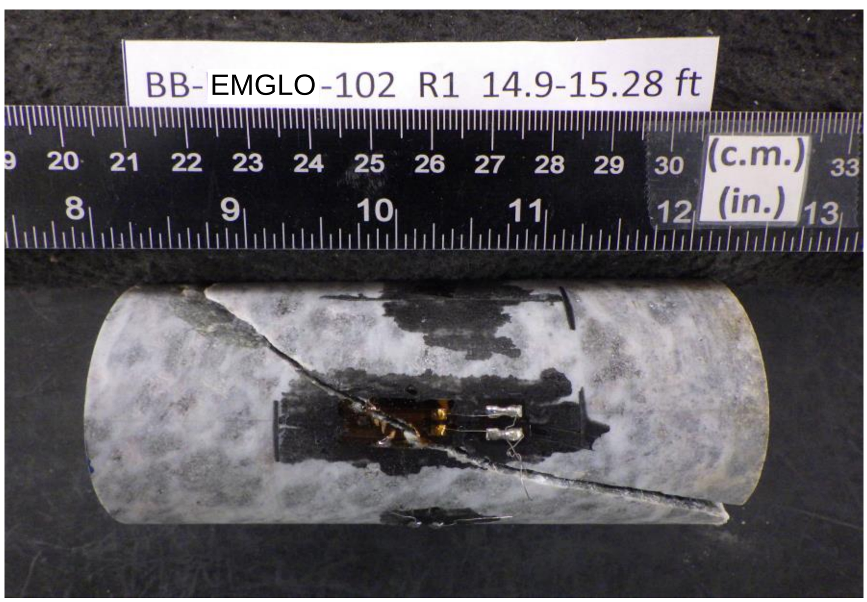
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	2.000	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00030	2.000	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00040	2.000	0.00020	0.011	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-102
Sample ID:	R1
Depth, ft:	14.9-15.28

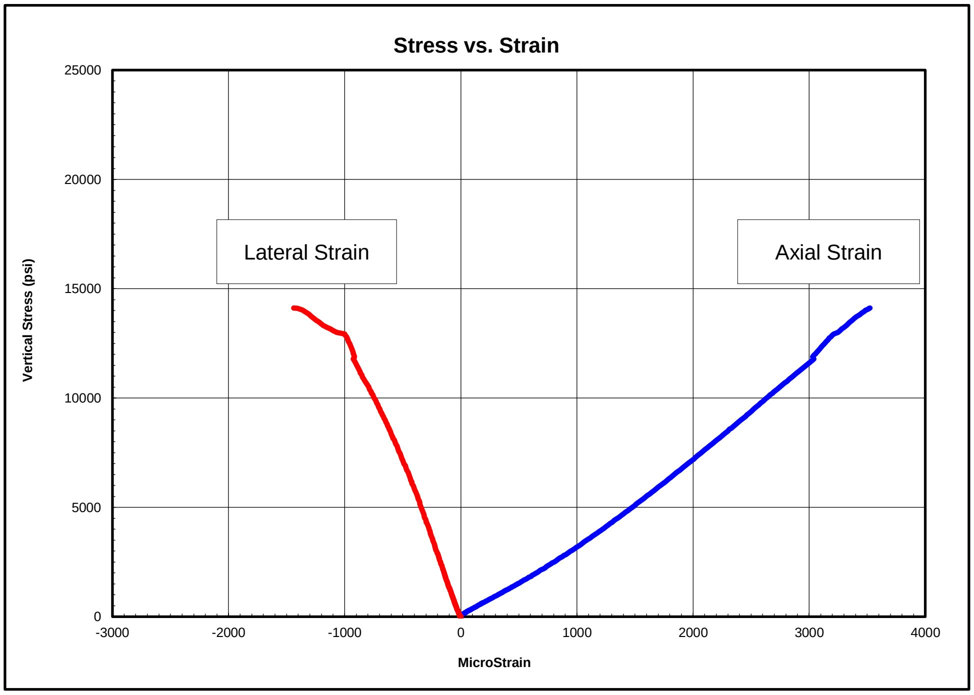
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-102
Sample ID:	R2
Depth, ft:	18.51-18.7
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: 14,117 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1400-5200	3,530,000	0.23
5200-8900	4,270,000	0.34
8900-12700	4,750,000	0.43

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:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-102		
Sample ID:	R2		
Depth:	18.51-18.7 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:	4.46	4.47	4.47		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	595.24				
Bulk Density, lb/ft^3	161	**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	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	-0.00010	-0.00010	-0.00020	-0.00030

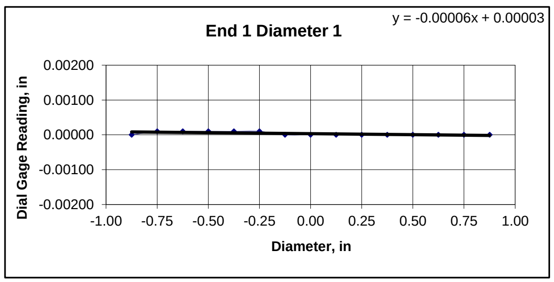
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00040

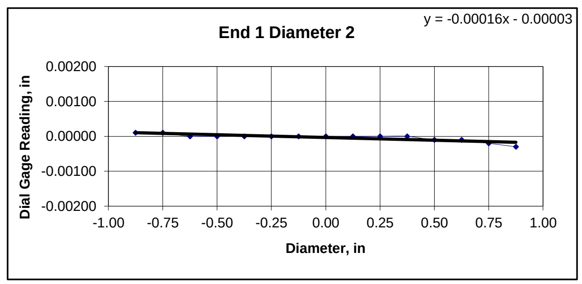
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000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

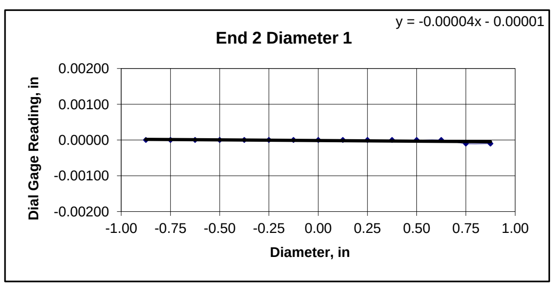
Difference between max and min readings, in:
0° = 0.0001 90° = 0.0002

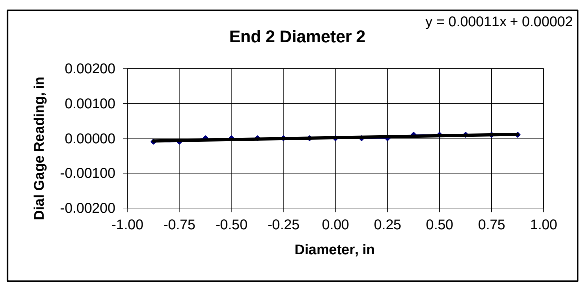
Maximum difference must be < 0.0020 in. Difference = ± 0.00020

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00327
End 2:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00213
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00016
Angle of Best Fit Line:	0.00900
End 2:	
Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00622
Maximum Angular Difference:	0.00278

Parallelism Tolerance Met? **YES**
Spherically Seated

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

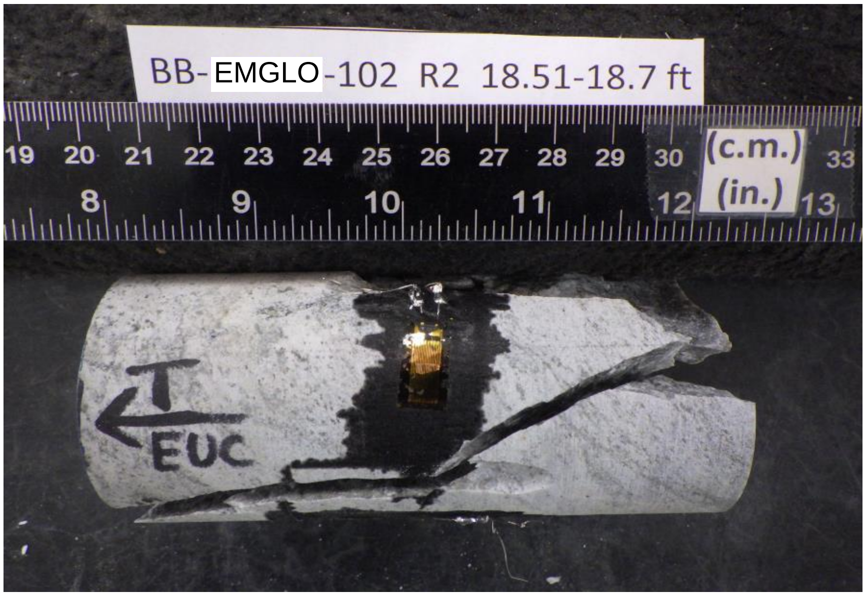
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	2.000	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	2.000	0.00020	0.011	YES
END 2					
Diameter 1, in	0.00010	2.000	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES

Maximum angle of departure must be < 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-102
Sample ID:	R2
Depth, ft:	18.51-18.7

After cutting and grinding

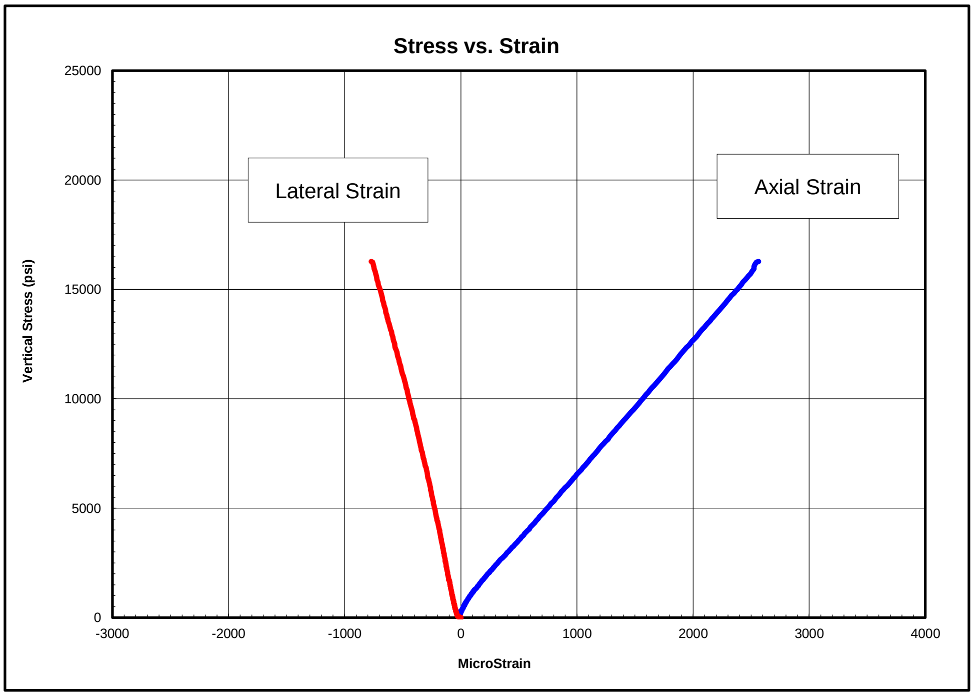


After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-102
Sample ID:	R2
Depth, ft:	20.45-20.83
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: 16,562 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1700-6100	5,890,000	0.22
6100-10500	6,110,000	0.28
10500-14900	6,130,000	0.31

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:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-102		
Sample ID:	R2		
Depth:	20.45-20.83 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:	4.50	4.50	4.50		
Specimen Diameter, in:	2.00	2.00	2.00		
Specimen Mass, g:	609.39				
Bulk Density, lb/ft^3	164			**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 ≤ 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	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	0.00000	-0.00010

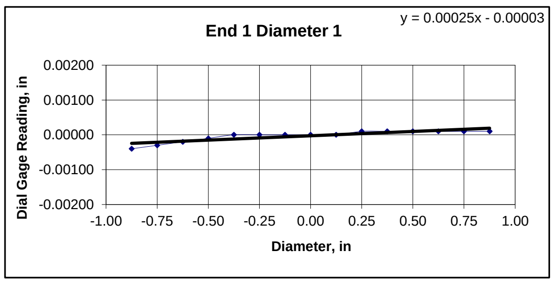
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00030

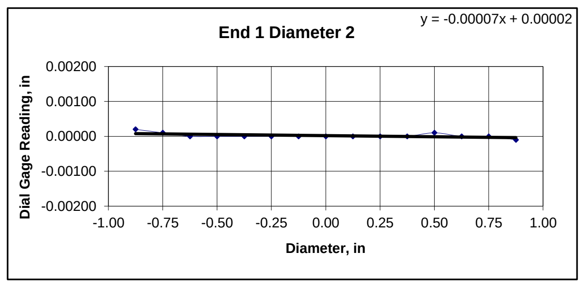
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0.0005 90° = 0.0002

Maximum difference must be < 0.0020 in. Difference = ± 0.00025

Flatness Tolerance Met? **YES**





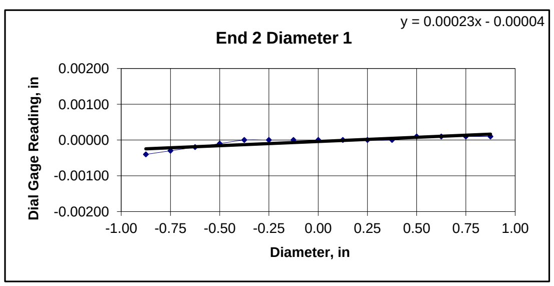
DIAMETER 1

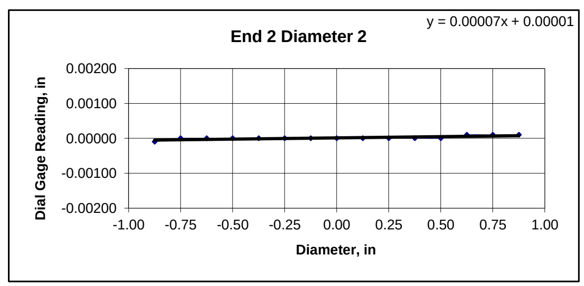
End 1:
Slope of Best Fit Line 0.00025
Angle of Best Fit Line: 0.01424

End 2:
Slope of Best Fit Line 0.00023
Angle of Best Fit Line: 0.01342

Maximum Angular Difference: 0.00082

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00007
Angle of Best Fit Line: 0.00377

End 2:
Slope of Best Fit Line 0.00007
Angle of Best Fit Line: 0.00409

Maximum Angular Difference: 0.00033

Parallelism Tolerance Met? **YES**
Spherically Seated

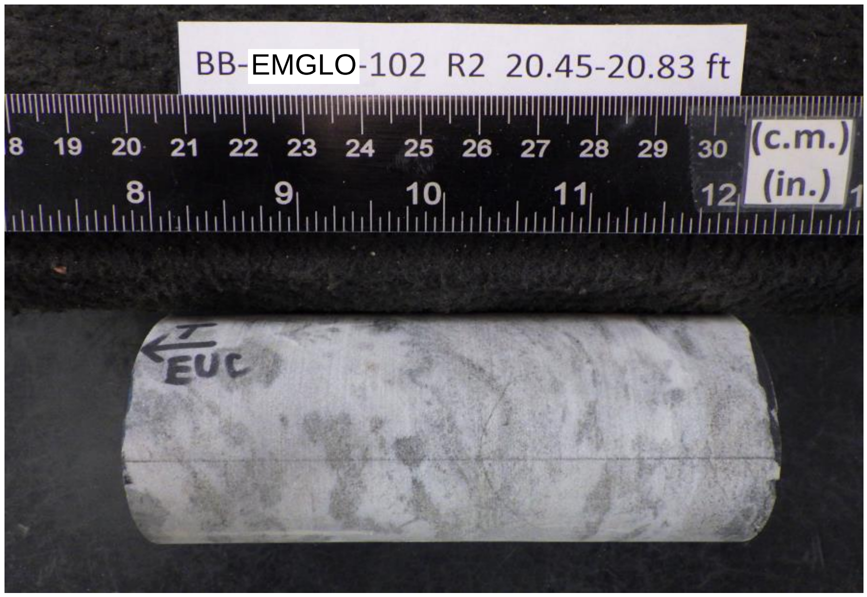
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	2.000	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00030	2.000	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00050	2.000	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES

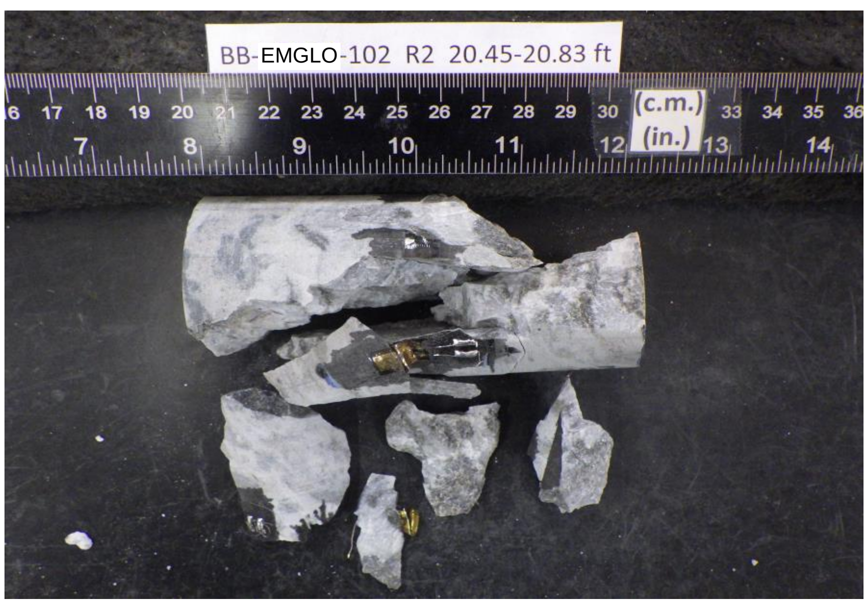
Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-102
Sample ID:	R2
Depth, ft:	30.45-20.83



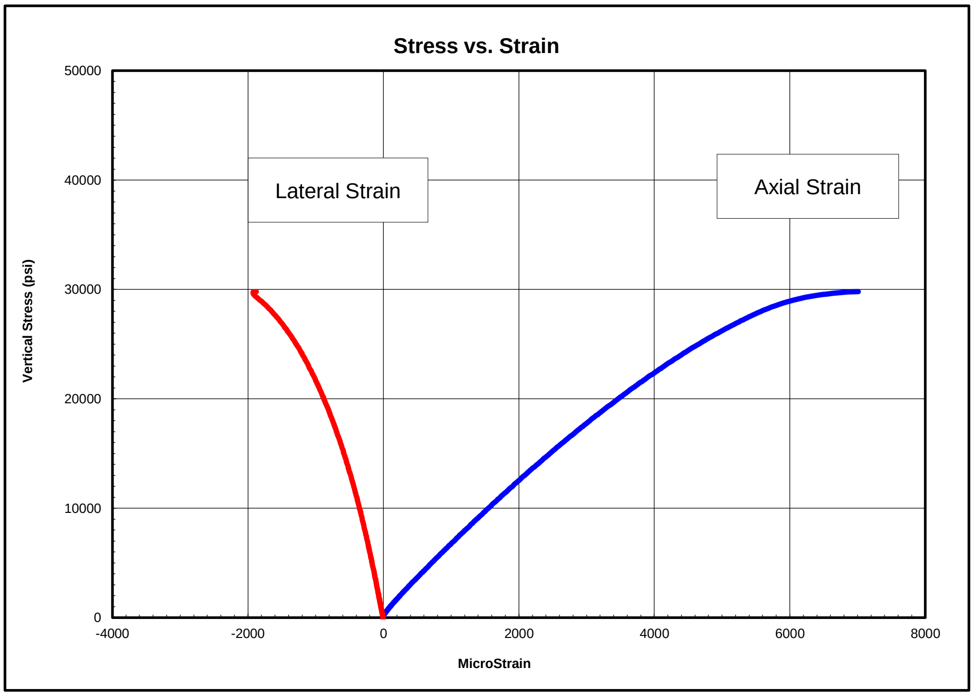
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-103
Sample ID:	R1
Depth, ft:	18.84-19.22
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: 29,799 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
3000-10900	6,040,000	0.22
10900-18900	5,260,000	0.27
18900-26800	4,110,000	0.35

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

Client:	Haley & Aldrich, Inc.	Test Date:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-103		
Sample ID:	R1		
Depth:	18.84-19.22 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	4.44	4.44	
Specimen Diameter, in:	2.00	2.00	2.00	
Specimen Mass, g:	609.97			
Bulk Density, lb/ft^3	166	**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	0.00080	0.00070	0.00060	0.00050	0.00040	0.00030	0.00010	0.00000	-0.00020	-0.00040	-0.00050	-0.00070	-0.00080	-0.00090	-0.00100
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010

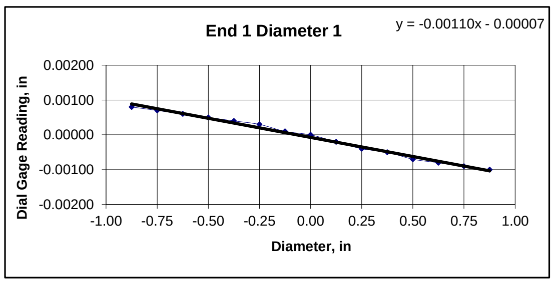
Difference between max and min readings, in:
0° = 0.00180 90° = 0.00020

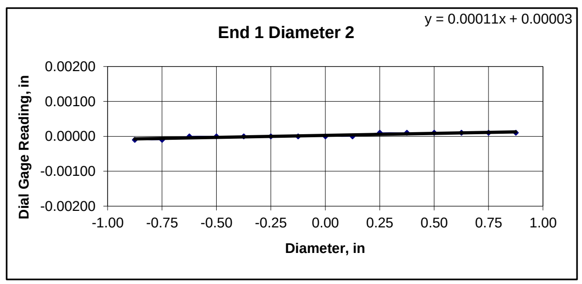
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00090	0.00080	0.00070	0.00060	0.00040	0.00030	0.00020	0.00000	-0.00020	-0.00040	-0.00060	-0.00070	-0.00080	-0.00090	-0.00100
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020

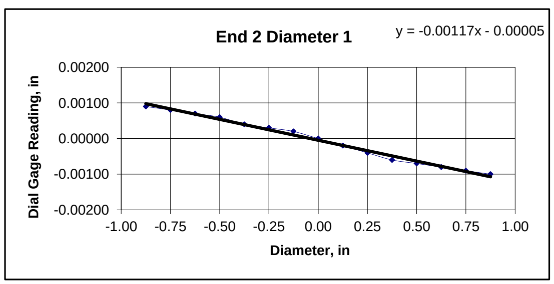
Difference between max and min readings, in:
0° = 0.0019 90° = 0.0002

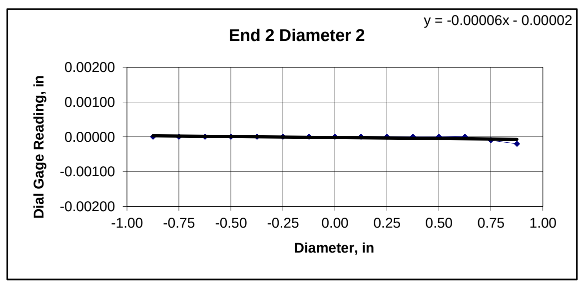
Maximum difference must be < 0.0020 in. Difference = ± 0.00095

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00110
Angle of Best Fit Line: 0.06286

End 2:
Slope of Best Fit Line 0.00117
Angle of Best Fit Line: 0.06712

Maximum Angular Difference: 0.00426

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00011
Angle of Best Fit Line: 0.00655

End 2:
Slope of Best Fit Line 0.00006
Angle of Best Fit Line: 0.00327

Maximum Angular Difference: 0.00327

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	0.00180	2.000	0.00090	0.052	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00190	2.000	0.00095	0.054	YES
Diameter 2, in (rotated 90°)	0.00020	2.000	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-103
Sample ID:	R1
Depth, ft:	18.84-19.22

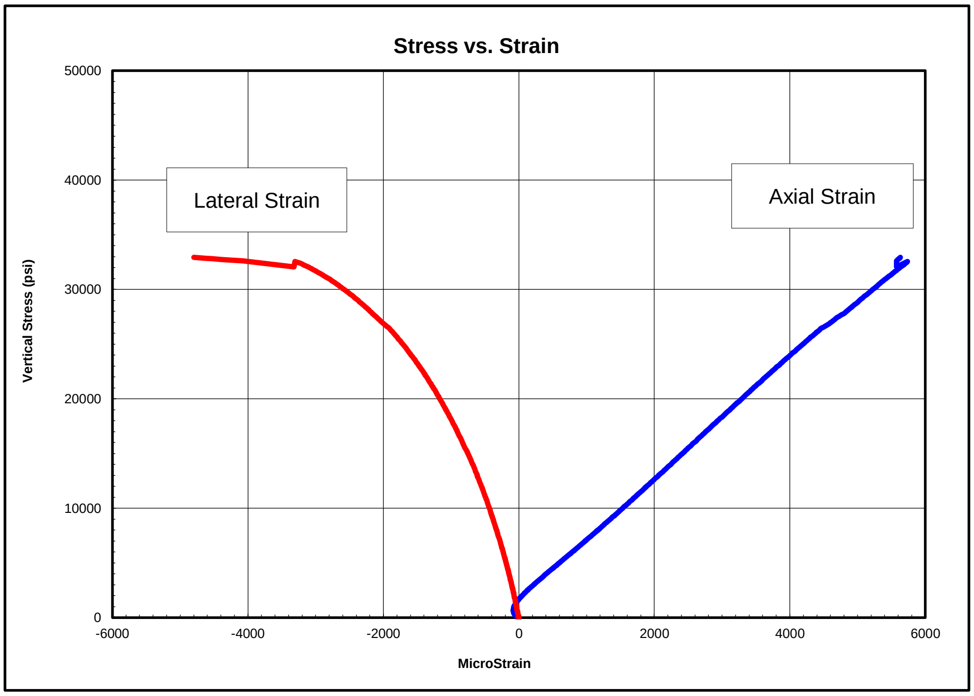
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-103
Sample ID:	R2
Depth, ft:	21.16-21.54
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: 32,933 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
3300-12100	5,360,000	0.27
12100-20900	5,710,000	0.45
20900-29600	5,060,000	---

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:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-103		
Sample ID:	R2		
Depth:	21.16-21.54 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:	4.47	4.48	4.48	
Specimen Diameter, in:	2.00	2.00	2.00	
Specimen Mass, g:	608.28			
Bulk Density, lb/ft^3	164	**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**

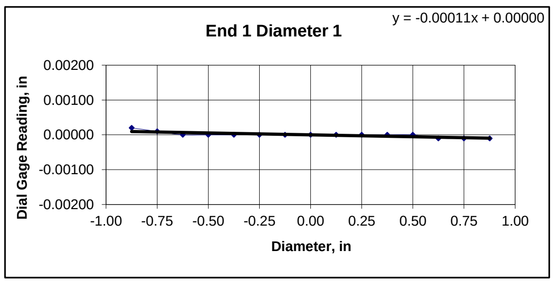
END FLATNESS AND PARALLELISM (Procedure FP1)

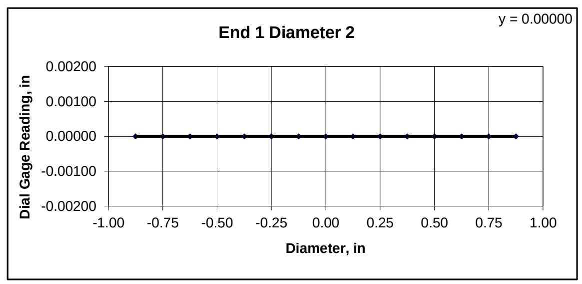
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

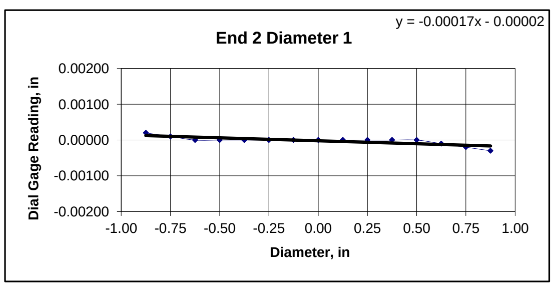
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00000

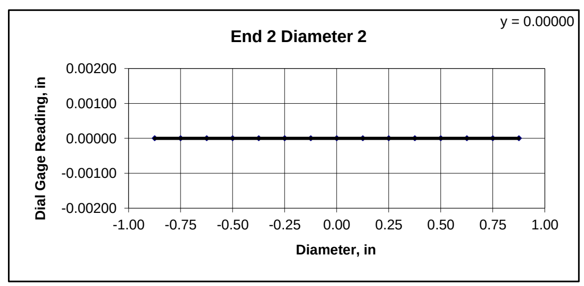
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0.0005 90° = 0
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00011
Angle of Best Fit Line:	0.00622
End 2:	
Slope of Best Fit Line	0.00017
Angle of Best Fit Line:	0.00949
Maximum Angular Difference:	0.00327

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00000
End 2:	
Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00000
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	2.000	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00000	2.000	0.00000	0.000	YES
END 2					
Diameter 1, in	0.00050	2.000	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00000	2.000	0.00000	0.000	YES

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

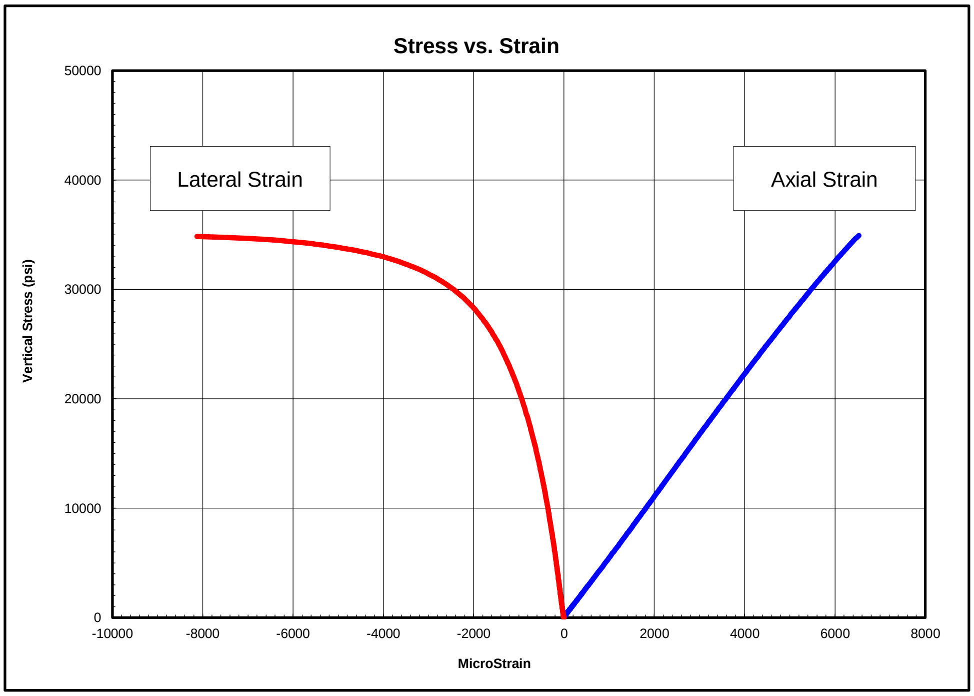
Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-103
Sample ID:	R2
Depth, ft:	21.16-21.54

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-104
Sample ID:	R1
Depth, ft:	9.66-10.04 ft
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: 34,926 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
3500-12800	5,580,000	0.21
12800-22100	5,600,000	0.38
22100-31400	5,210,000	---

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:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-104		
Sample ID:	R1		
Depth:	9.66-10.04 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:	607.95				
Bulk Density, lb/ft^3	166			**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	-0.00010	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010

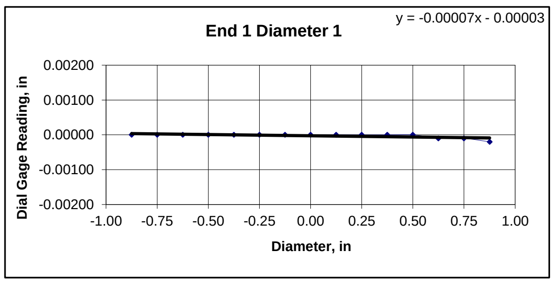
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00010

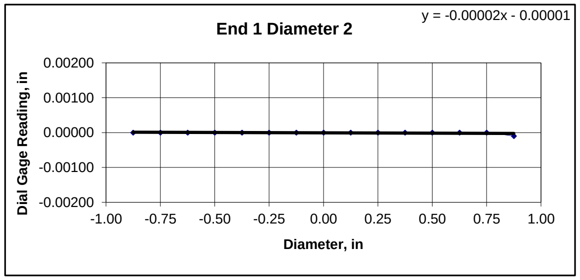
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000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

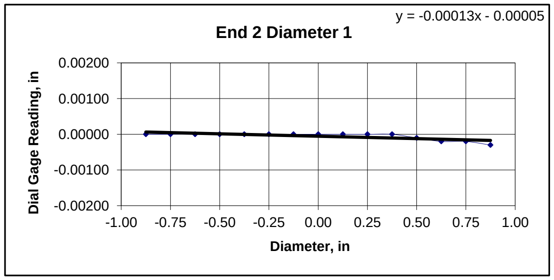
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0001

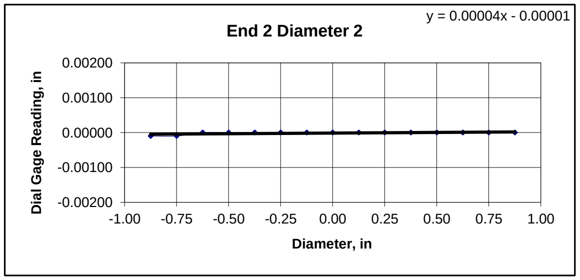
Maximum difference must be < 0.0020 in. Difference = ± 0.00015

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:		
	Slope of Best Fit Line	0.00007
	Angle of Best Fit Line:	0.00409
End 2:		
	Slope of Best Fit Line	0.00013
	Angle of Best Fit Line:	0.00769
Maximum Angular Difference:		0.00360

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	0.00004
	Angle of Best Fit Line:	0.00213
Maximum Angular Difference:		0.00098

Parallelism Tolerance Met? **YES**
Spherically Seated

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

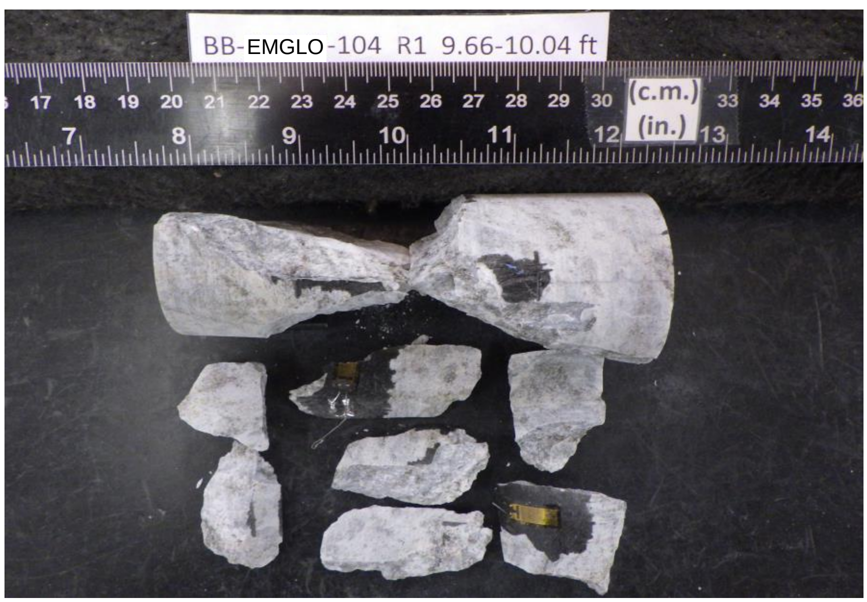
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	2.000	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00010	2.000	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00030	2.000	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00010	2.000	0.00005	0.003	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-104
Sample ID:	R1
Depth, ft:	9.66-10.04

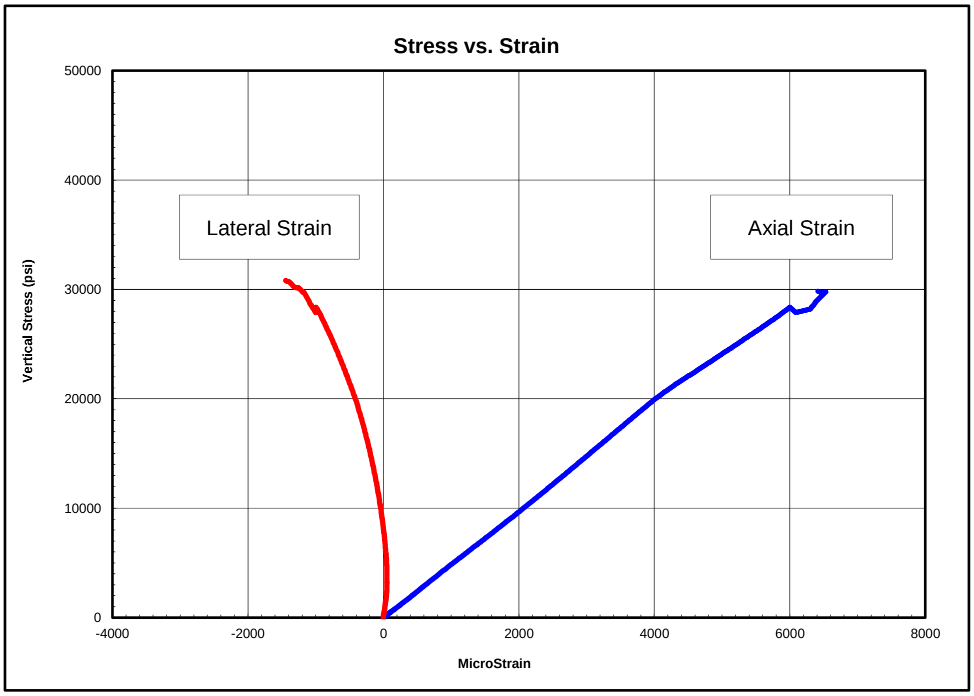
After cutting and grinding



After break

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-EMGLO-104
Sample ID:	R2
Depth, ft:	16.8-17.17
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: 30,814 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
3100-11300	4,830,000	0.08
11300-19500	5,180,000	0.20
19500-27700	4,140,000	0.27

Notes:

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

Client:	Haley & Aldrich, Inc.	Test Date:	11/10/2022
Project Name:	Chase Mills Bridge 5465	Tested By:	jab
Project Location:	East Machias, ME	Checked By:	smd
GTX #:	316246		
Boring ID:	BB-EMGLO-104		
Sample ID:	R2		
Depth:	16.8-17.17 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:	4.57	4.57	4.57
Specimen Diameter, in:	2.00	2.00	2.00
Specimen Mass, g:	617.77		
Bulk Density, lb/ft^3	164	**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 ≤ 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	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00020
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020

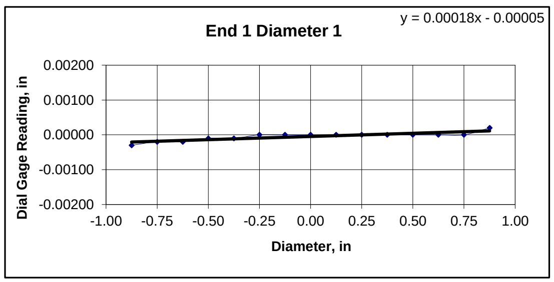
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00040

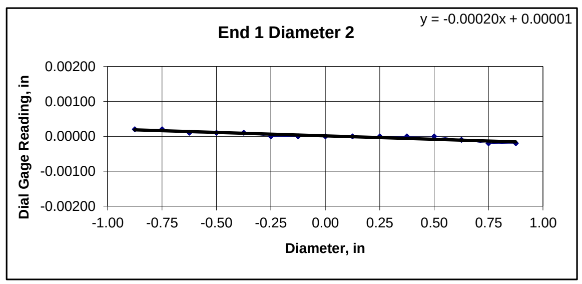
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00020	-0.00010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

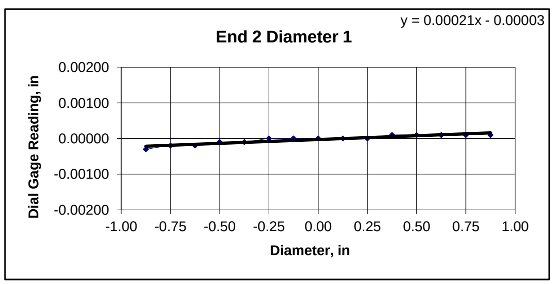
Difference between max and min readings, in:
0° = 0.0004 90° = 0.0005

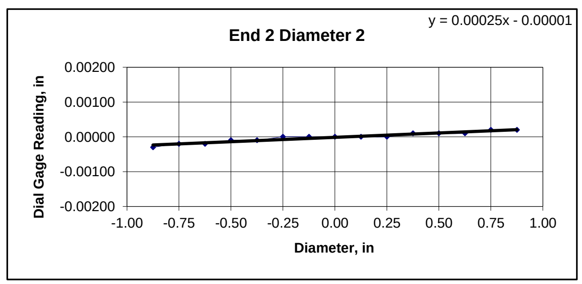
Maximum difference must be < 0.0020 in. Difference = ± 0.00025

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00018
	Angle of Best Fit Line:	0.01048
End 2:	Slope of Best Fit Line	0.00021
	Angle of Best Fit Line:	0.01228
Maximum Angular Difference:		0.00180

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00020
	Angle of Best Fit Line:	0.01130
End 2:	Slope of Best Fit Line	0.00025
	Angle of Best Fit Line:	0.01441
Maximum Angular Difference:		0.00311

Parallelism Tolerance Met? YES
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	2.000	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00040	2.000	0.00020	0.011	YES
END 2					
Diameter 1, in	0.00040	2.000	0.00020	0.011	YES
Diameter 2, in (rotated 90°)	0.00050	2.000	0.00025	0.014	YES

Maximum angle of departure must be ≤ 0.25°

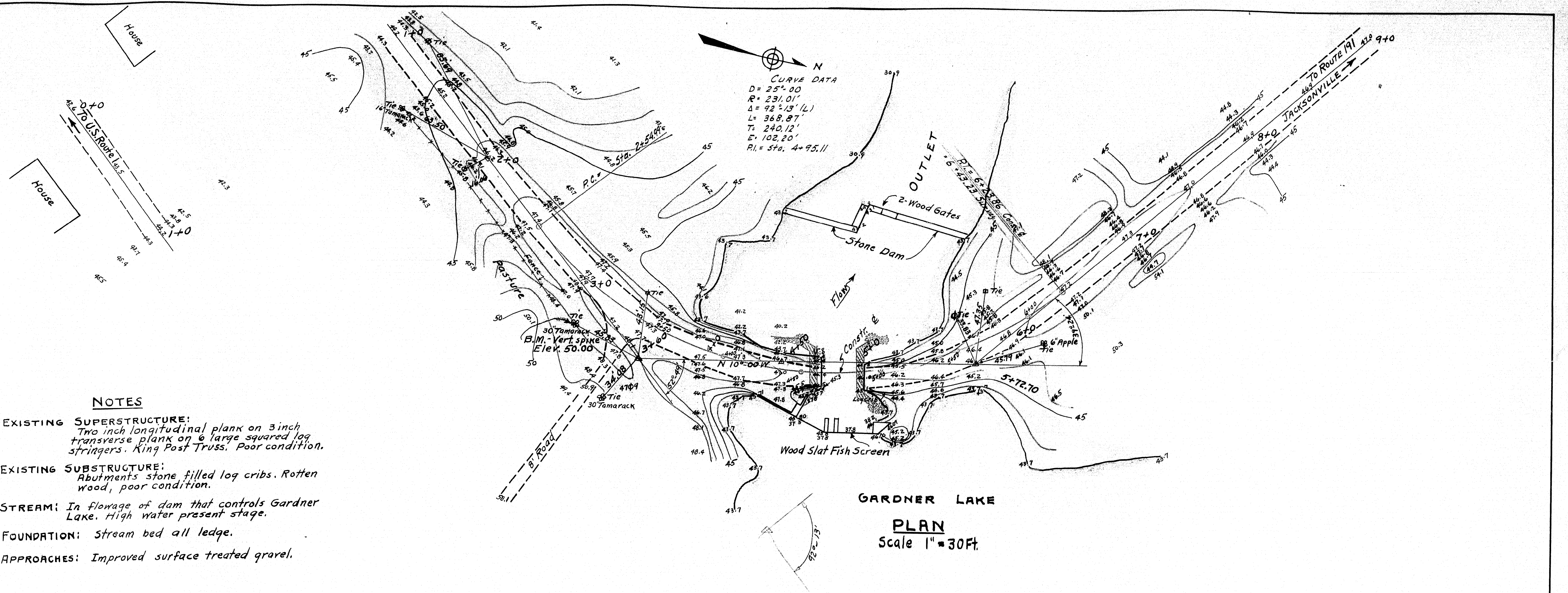
Perpendicularity Tolerance Met? YES

Client:	Haley & Aldrich, Inc.
Project Name:	Chase Mills Bridge 5465
Project Location:	East Machias, ME
GTX #:	316246
Test Date:	11/11/2022
Tested By:	bp
Checked By:	smd
Boring ID:	BB-EMGLO-104
Sample ID:	R2
Depth, ft:	16.8-17.17

After cutting and grinding

After break

APPENDIX C
Historic Bridge Drawings



PLAN

Scale 1" = 30 Ft.

NOTES

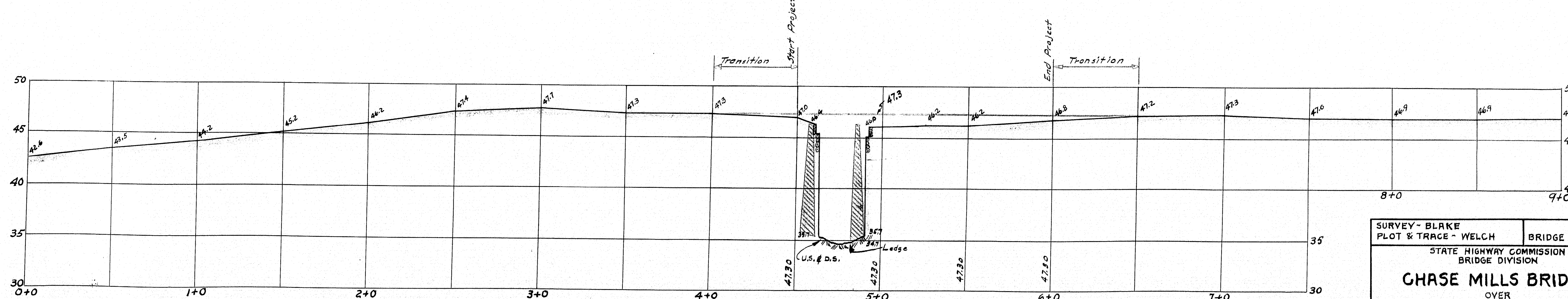
EXISTING SUPERSTRUCTURE: Two inch longitudinal plank on 3 inch transverse plank on 6 large squared log stringers. King Post Truss. Poor condition.

EXISTING SUBSTRUCTURE: Abutments stone filled log cribs. Rotten wood, poor condition.

STREAM: In flowage of dam that controls Gardner Lake. High water present stage.

FOUNDATION: Stream bed all ledge.

APPROACHES: Improved surface treated gravel.



PROFILE

SCALE - HOR. 1" = 30' ~ VERT. 1" = 5'

SURVEY - BLAKE PLOT & TRACE - WELCH	BRIDGE - 5465

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CHASE MILLS BRIDGE

OVER

GARDNER LAKE OUTLET

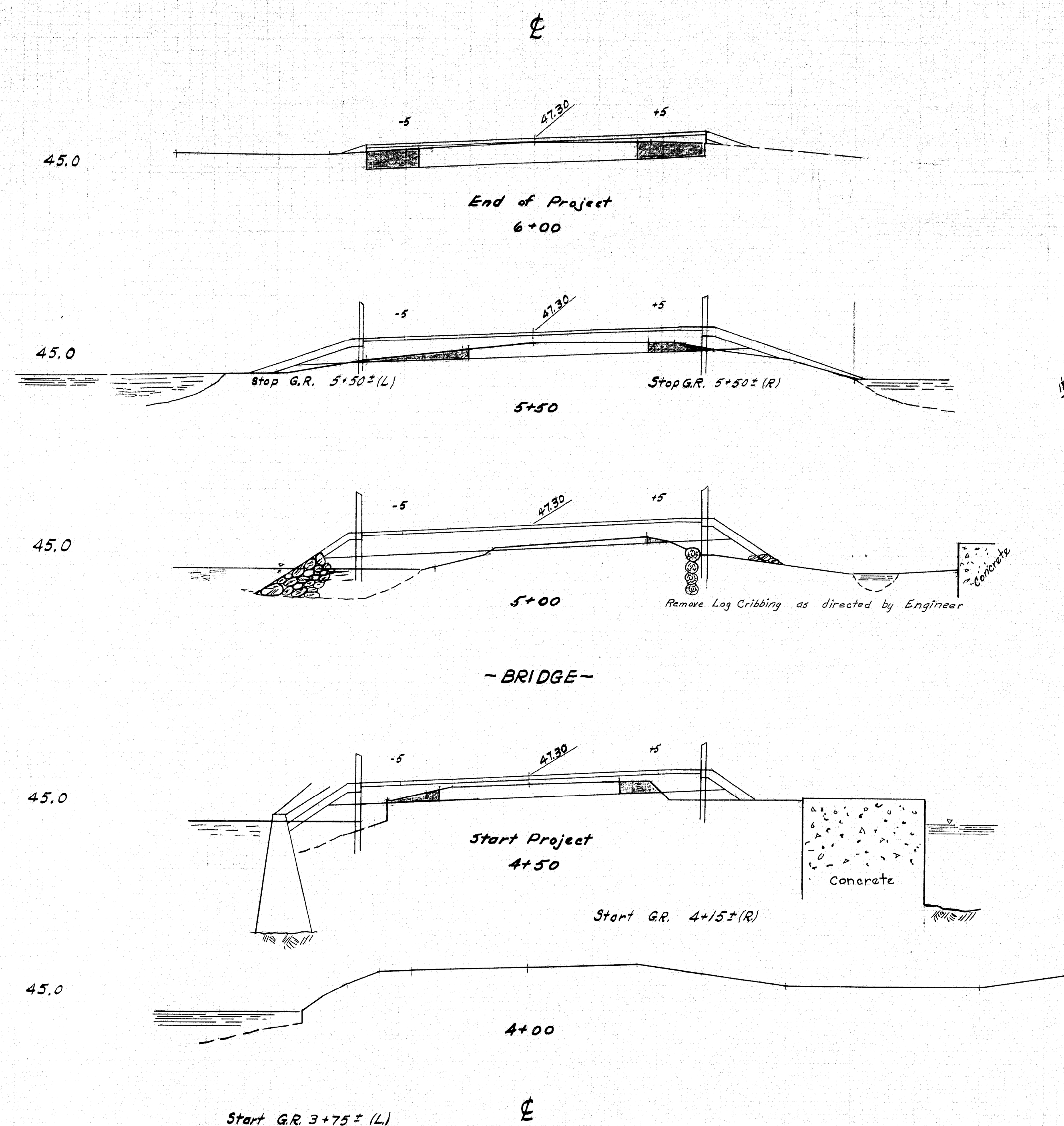
IN THE TOWN OF

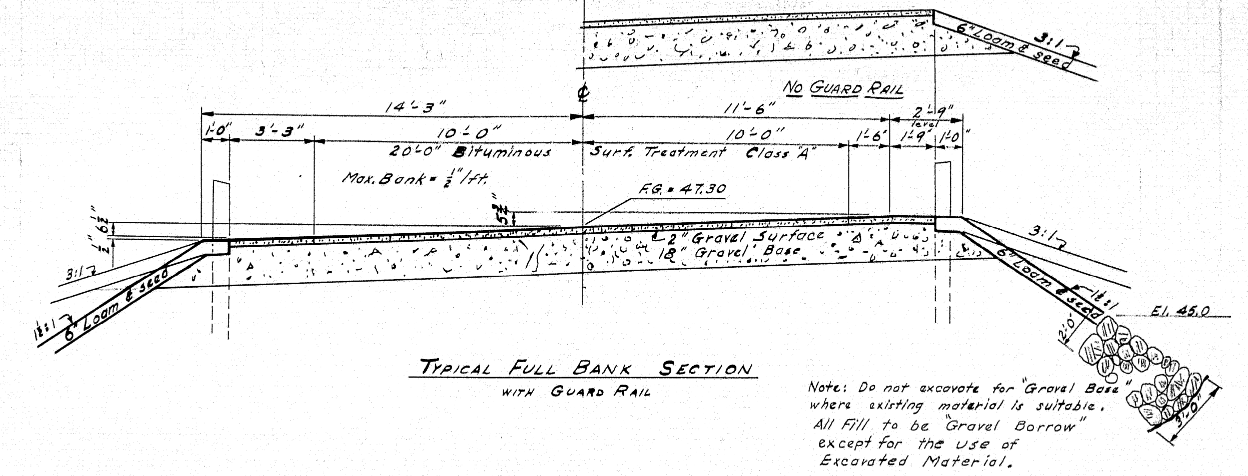
EAST MACHIAS
WASHINGTON COUNTY

SURVEY

SHEET 1 OF 4 AUGUSTA, MAINE JAN. 1951

54-10





Note: Do not excavate for "Gravel Base" where existing material is suitable. All Fill to be "Gravel Borrow" except for the use of Excavated Material.

PLOT— MAZEIKO CHECK— ABR	BRIDGE 5465

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

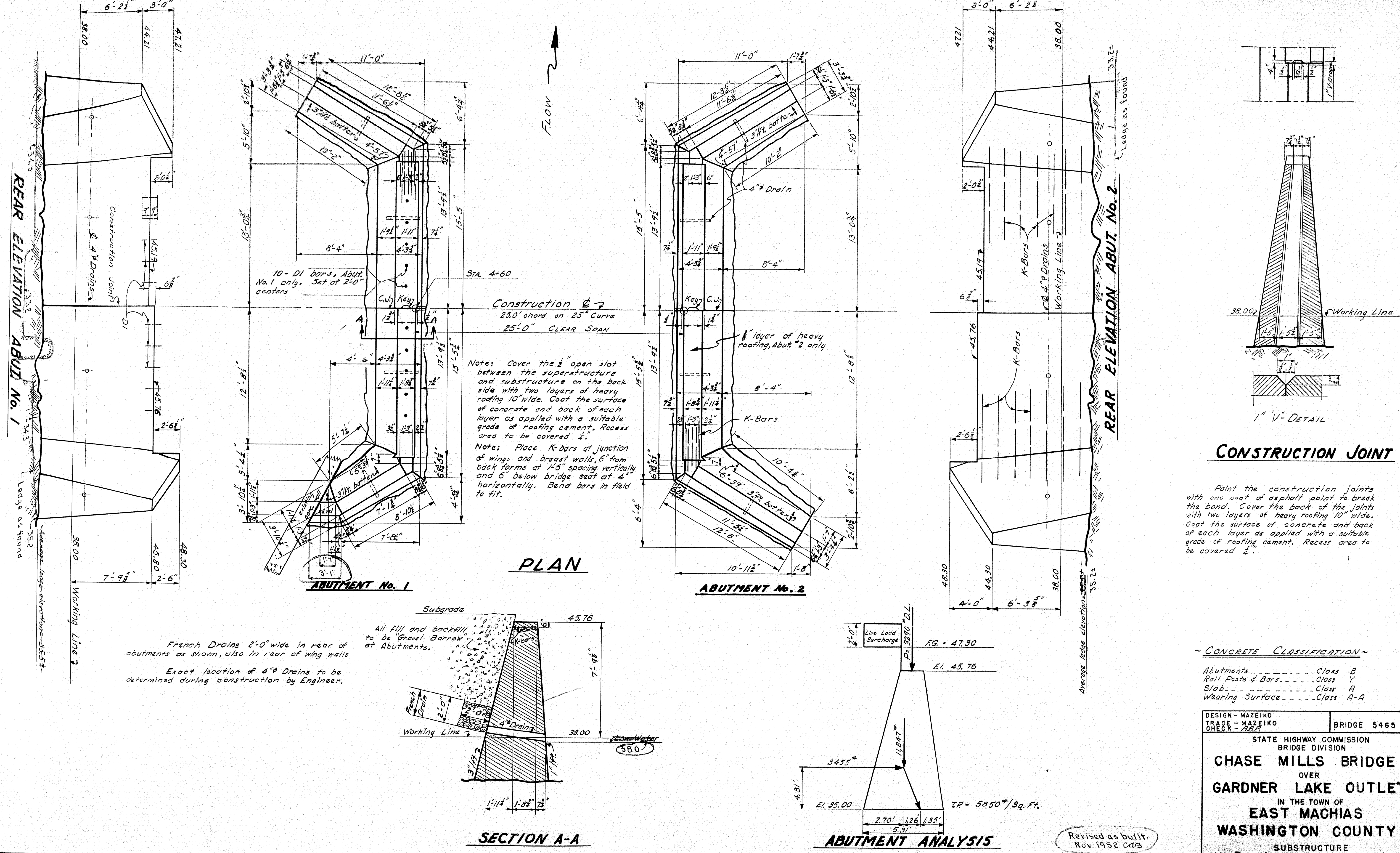
CHASE MILLS BRIDGE
OVER
GARDNER LAKE OUTLET
IN THE TOWN OF
EAST MACHIAS
WASHINGTON COUNTY

CROSS SECTIONS

SHEET 2 OF 4 AUGUSTA, MAINE FEB. 1952

54-11

0 1 2 3 4 5 INCHES



Note: Cover the 1/2" open slot between the superstructure and substructure on the back side with two layers of heavy roofing 10" wide. Coat the surface of concrete and back of each layer as applied with a suitable grade of roofing cement. Recess area to be covered 1/4".

Note: Place K-bars at junction of wings and breast walls, 6" from back forms at 1'-6" spacing vertically and 6" below bridge seat at 4" horizontally. Bend bars in field to fit.

French Drains 2'-0" wide in rear of abutments as shown, also in rear of wing walls

Exact location of 4"φ Drains to be determined during construction by Engineer.

CONSTRUCTION JOINT

Paint the construction joints with one coat of asphalt paint to break the bond. Cover the back of the joints with two layers of heavy roofing 10" wide. Coat the surface of concrete and back of each layer as applied with a suitable grade of roofing cement. Recess area to be covered 1/4".

~ CONCRETE CLASSIFICATION ~

Abutments	Class B
Rail Posts & Bars	Class Y
Slab	Class A
Wearing Surface	Class A-A

Revised as built. Nov. 1952 CAB

DESIGN - MAZEIKO
TRACE - MAZEIKO
CHECK - ABP.

BRIDGE 5465

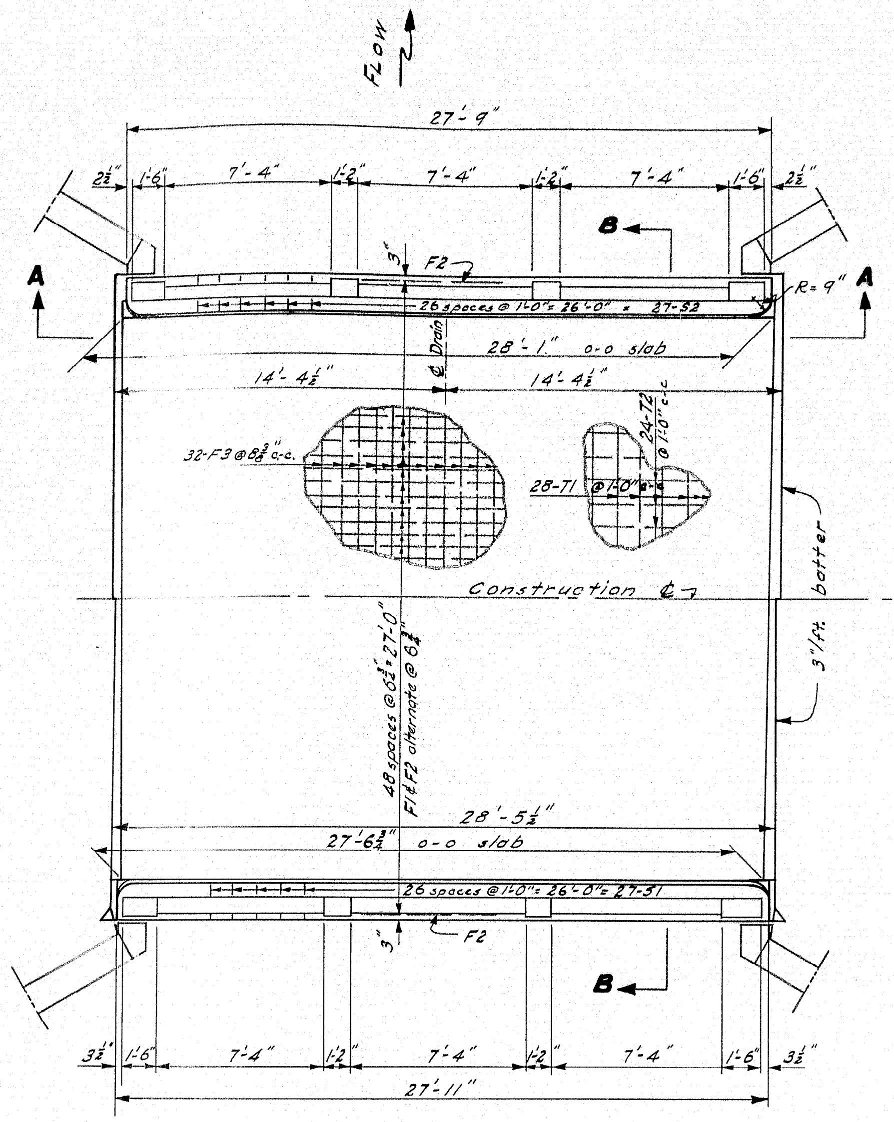
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CHASE MILLS BRIDGE
OVER
GARDNER LAKE OUTLET
IN THE TOWN OF
EAST MACHIAS
WASHINGTON COUNTY

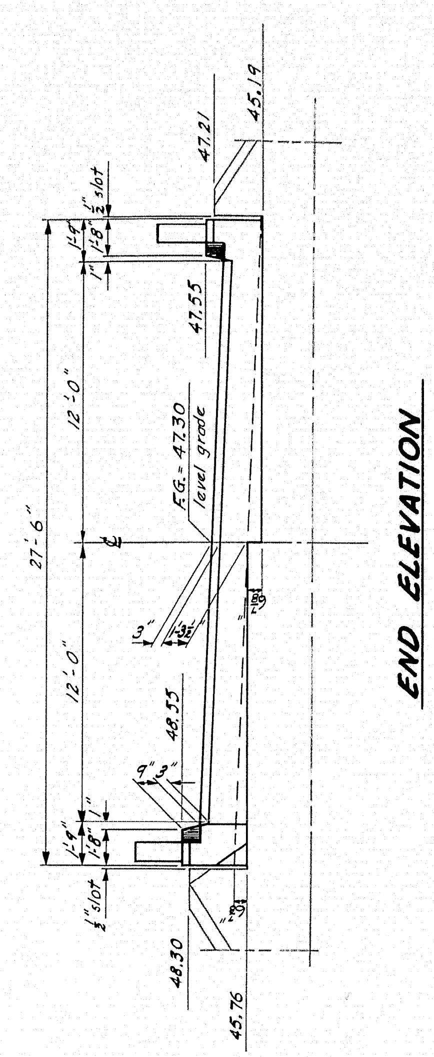
SUBSTRUCTURE

SHEET 3 OF 4 AUGUSTA, MAINE FEB. 1952

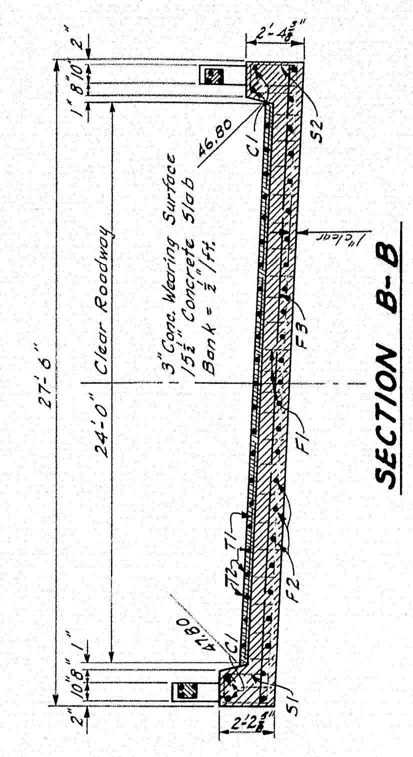
51-12



PLAN

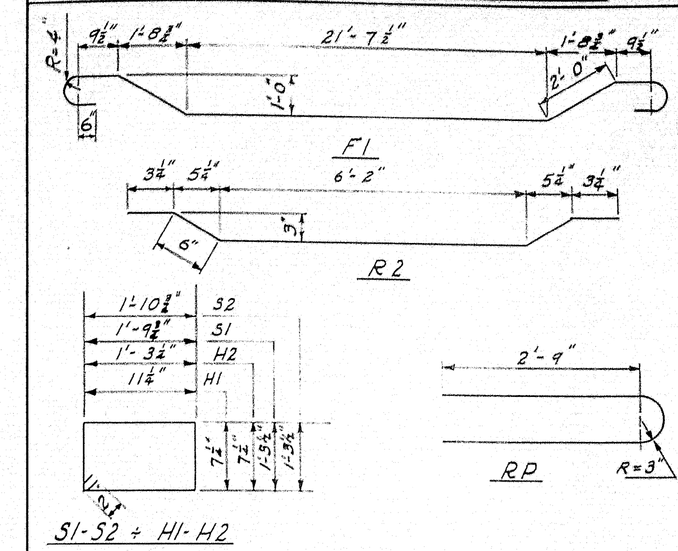


END ELEVATION



SECTION B-B

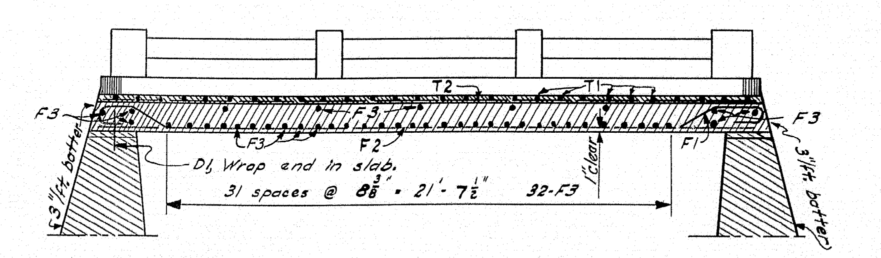
REINFORCING STEEL



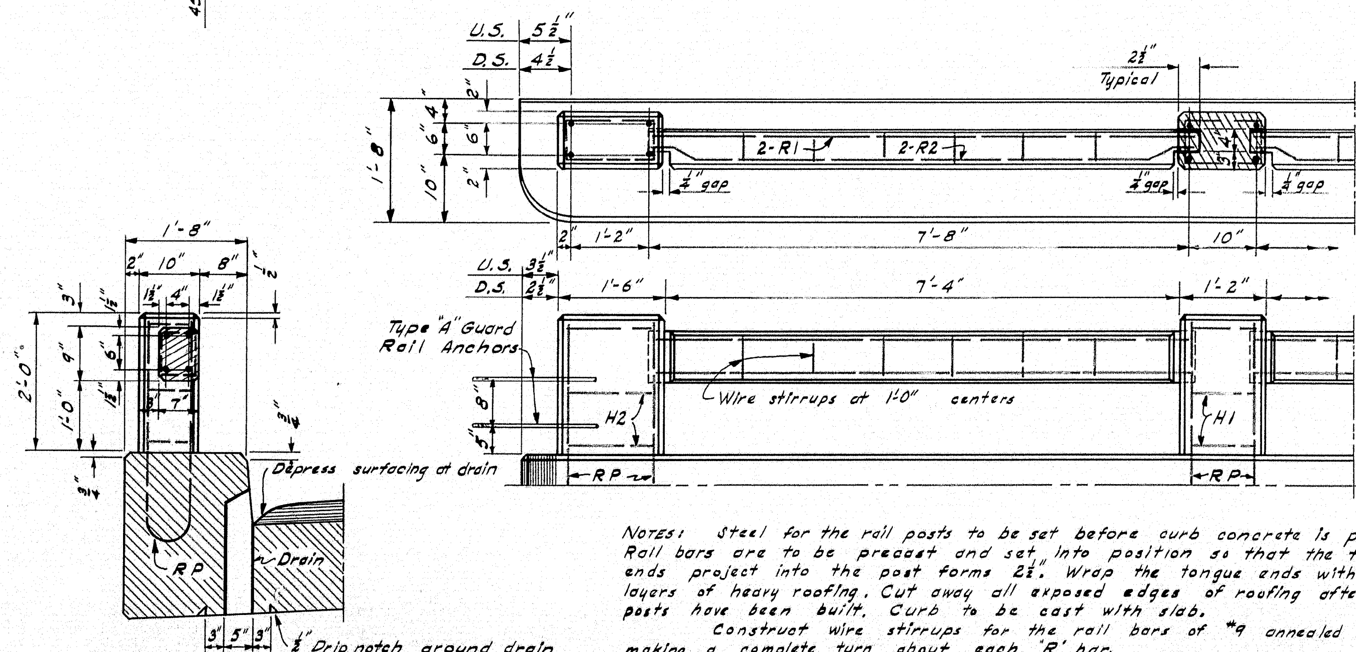
NOTE: All dimensions to ℄ of bars.

BENT BARS					STRAIGHT BARS				
Mark	Size	No.	Length	Location	Mark	Size	No.	Length	Location
F1	#10	24	30'-4"	Slab	F2	#10	25	27'-10"	Slab
S1	#3	27	6'-6½"	Curb U.S.	F3	#5	43	27'-2"	Slab
S2	#3	27	6'-8½"	Curb D.S.	T1	#3	28	23'-8"	Wearing Surf.
H1	#3	12	3'-5"	Int. Rail Posts	T2	#3	24	27'-4"	Wearing Surf.
H2	#3	12	4'-1"	End Rail Posts	R1	#4	12	7'-7"	Rail Bar
R2	#4	12	7'-8½"	Rail Bar	C1	#4	4	27'-0"	Curb
RP	#6	16	6'-3½"	All Rail Posts	D1	#6	10	1'-6"	Dowels
					K	#6	48	8'-0"	Abutments

All reinforcing steel shall conform to ASTM Specifications A305-49



SECTION A-A

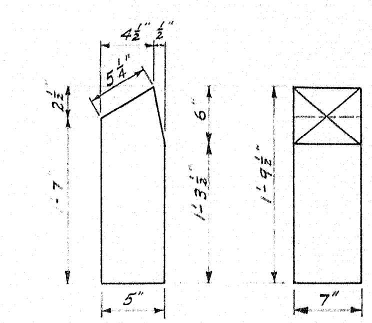


NOTES: Steel for the rail posts to be set before curb concrete is placed. Rail bars are to be precast and set into position so that the tongue ends project into the post forms 2½". Wrap the tongue ends with two layers of heavy roofing. Cut away all exposed edges of roofing after the posts have been built. Curb to be cast with slab.

Construct wire stirrups for the rail bars of #9 annealed wire making a complete turn about each "R" bar.

Chamfer all exposed edges of concrete ½" unless otherwise noted.

RAIL DETAIL



DRAIN DETAIL

24 gage Galvanized Iron
1 Required

-DESIGN-

Loading H-15-44
f_s = 18,000
f_c = 1,200
n = 10

DESIGN – MAZEIKO
TRACE – MAZEIKO
CHECK – A.B.P.

BRIDGE 5465

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CHASE MILLS BRIDGE
OVER
GARDNER LAKE OUTLET
IN THE TOWN OF
EAST MACHIAS
WASHINGTON COUNTY

SUPERSTRUCTURE

SHEET 4 OF 4 AUGUSTA, MAINE FEB 1952

APPENDIX D
Geotechnical Calculations

Seismic Site Class and Design Parameters

PROBLEM STATEMENT & OBJECTIVE

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

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the six borings near the proposed substructures (BB-EMGLO-101 through BB-EMGLO-104 and BB-EMGO-201 through BB-EMGLO-202), recommend a **Seismic Site Class C.**

REFERENCES

1. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018
2. AASHTO LRFD Bridge Design Specifications, 2020.

AVAILABLE INFORMATION

1. Boring logs dated September 2022 and June 2025 by New England Boring Contractors.
2. Elevations reference NAVD 88.

ASSUMPTIONS

1. Where SPT N data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N for the extended profile was then assumed based on the available information.
2. SPT N value for bedrock = 100 blows/ft; SPT N value for WOH = 1 blow/ft

PROCEDURE

1. Check the site against the three categories of Site Class F, 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 B will be used for the seismic Site Class evaluation.

SITE CLASS DEFINITIONS

(Table from AASHTO LRFD Bridge Design Specifications, Ninth Edition, 2020)

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10.0 ft of soft clay defined as soil with PI > 20, w > 40 percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays (H > 10.0 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25.0 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

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

where:

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

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

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile

PI = plasticity index (ASTM D4318)

w = moisture content (ASTM D2216)

HALEY ALDRICH	CALCULATIONS	File No.:	205517-000
		Sheet:	4 of 6
Client:	Maine Department of Transportation	Date:	17-May-23
Project:	Chase Mills Bridge #5465 over Gardner Lake Outlet	Computed by:	EMH
Subject:	Seismic Site Class Calculation	Checked by:	BCS

CALCULATIONS - METHOD B

Boring ID: BB-EMGLO-101
Ground Surface El.: 71.1

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1	70.1	SAND (Fill)	3.0	32	0.094
2D	3	68.1	SAND (Fill)	2.0	29	0.069
3D	5	66.1	SAND (Fill)	2.0	7	0.286
4D	7	64.1	SAND (Marine Deposit)	3.0	5	0.600
5D	10	61.1	Silty GRAVEL (Glacial Till)	1.5	60	0.025
	11.5	59.6	Bedrock	88.5	100	0.885
			Totals =	100.0		1.958

N-bar (blows/ft) = 51.1
Site Class = C

Boring ID: BB-EMGLO-102
Ground Surface El.: 71.2

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1	70.2	SAND (Fill)	3.0	12	0.250
2D	3	68.2	SAND (Fill)	2.0	13	0.154
3D	5	66.2	SAND (Fill)	2.0	14	0.143
4D	7	64.2	SAND (Fill)	3.0	19	0.158
5D	10	61.2	SAND (Glacial Till)	2.0	6	0.333
6D	12	59.2	SAND (Glacial Till)	0.2	100	0.002
	12.2	59	Bedrock	87.8	100	0.878
			Totals =	100.0		1.918

N-bar (blows/ft) = 52.1
Site Class = C

CALCULATIONS - METHOD B

Boring ID: BB-EMGLO-103
Ground Surface El.: 71.2

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1	70.2	Sandy GRAVEL (Fill)	3.0	24	0.125
2D	5	66.2	SAND (Fill)	4.0	4	1.000
3D	7	64.2	SAND (Marine Deposit)	2.0	2	1.000
4D	9	62.2	No Recovery	4.0	1	4.000
5D	13	58.2	Silty GRAVEL (Glacial Till)	1.1	26	0.042
	14.1	57.1	Bedrock	85.9	100	0.859
			Totals =	100.0		7.026

N-bar (blows/ft) = 14.2
Site Class = E

Boring ID: BB-EMGLO-104
Ground Surface El.: 71.4

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	1	70.4	Gravelly SAND (Fill)	3.0	23	0.130
2D	2	69.4	SAND (Fill)	2.0	5	0.400
3D	5	66.4	SAND (Fill)	2.0	15	0.133
	7	64.4	Bedrock	93.0	100	0.930
			Totals =	100.0		1.594

N-bar (blows/ft) = 62.7
Site Class = C

CALCULATIONS - METHOD B

Boring ID: BB-EMGLO-201
Ground Surface El.: 70.9

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2	68.9	SAND (Fill)	3.0	42	0.071
2D	4	66.9	SAND (Fill)	2.0	11	0.182
3D	6	64.9	SAND (Marine Deposit)	2.0	16	0.125
4D	8	62.9	SAND (Marine Deposit)	2.0	14	0.143
5D	10	60.9	SAND (Marine Deposit)	2.0	5	0.400
6D	12	58.9	SAND (Marine Deposit)	2.0	5	0.400
7D	13.1	57.8	SAND (Marine Deposit)	0.6	50	0.012
	13.6	57.3	Bedrock	86.4	100	0.864
			Totals =	100.0		1.321

N-bar (blows/ft) = 75.7
Site Class = C

Boring ID: BB-EMGLO-202
Ground Surface El.: 71.4

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	2	69.4	SAND (Fill)	3.0	25	0.120
2D	4	67.4	SAND (Fill)	2.5	15	0.167
3D	6.5	64.9	SAND (Marine Deposit)	2.0	1	2.000
4D	8.3	63.15	SAND (Marine Deposit)	1.5	1	1.500
5D	9.2	62.25	GRAVEL (Glacial Till)	0.4	50	0.008
	9.4	62	Bedrock	90.6	100	0.906
		Totals =		100.0		4.701

N-bar (blows/ft) = 21.3
Site Class = D

SUMMARY

Boring	N-bar	Site Class
BB-EMGLO-101	51.1	C
BB-EMGLO-102	52.1	C
BB-EMGLO-103	14.2	E
BB-EMGLO-104	62.7	C
BB-EMGLO-201	75.7	C
BB-EMGLO-202	21.3	D

CONCLUSION

Recommend **Seismic Site Class C**.

Bearing Resistance

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 18 ksf for the strength limit state is recommended.
A factored bearing resistance of 16 ksf for the service limit state for 1.0 in. settlement is recommended.
A factored bearing resistance of 31 ksf for the extreme event limit state is recommended.

AVAILABLE INFORMATION

1. Borings BB-EMGLO-101 through BB-EMGLO-104 and BB-EMGO-201 through BB-EMGLO-202 drilled by New England Boring Contractors in September 2022.
2. Rock compressive strength laboratory test results.

REFERENCES

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

ELEVATION DATUM

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

ASSUMPTIONS

1. The proposed abutments will be immediately behind the existing abutments. The bottom of footing elevations will be approximately at the top of bedrock.

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 2020:

10.6.3.2 - Bearing Resistance of Rock

10.6.3.2.1 - General

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

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

10.6.3.2.2 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). <u>This procedure is based on the unconfined compressive strength of the intact rock core sample</u>. 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 C10.6.2.5.1-1 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 2020 presumptive bearing resistance for service limit state for settlement stated.

PROCEDURE FOR EXTREME EVENT LIMIT STATE

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

11.5.8 - Resistance Factors for Extreme Event Limit state

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 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 2020 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 Top of Rock (ft)	Peak Compressive Strength (psi)	Failure Type
2	BB-EMGLO-102	71.2	R1	15.1	56.1	2.9	7,000	Intact
			R2	18.6	52.6	6.4	14,117	Intact
			R2	20.7	50.6	8.5	16,562	Intact
1	BB-EMGLO-103	71.2	R1	19.0	52.2	4.9	29,799	Intact
			R2	21.4	49.9	7.3	32,933	Intact
	BB-EMGLO-104	71.4	R1	9.9	61.6	2.9	34,926	Intact
			R2	17.0	54.4	10.0	30,814	Intact

SUMMARY OF BEDROCK DATA AT SITE

Abutment No.	Test Boring No.	Ground Surface Elevation	Rock Core No.	Avg. Rock Core Depth (BGS)	Avg. Rock Core Elevation	Depth of Rock Core Below Top of Rock (ft)	Rock Core Run Recovery (%)	Rock Quality Designation (RQD, %)
2	BB-EMGLO-101	71.1	R1	14.0	57.1	2.5	100	55
			R2	19.0	52.1	7.5	95	54
	BB-EMGLO-102	71.2	R1	15.0	56.2	2.8	100	62
			R2	20.0	51.2	7.8	97	53
	BB-EMGLO-201	70.9	R1	16	54.9	2.4	100	10
			R2	20	50.9	6.4	100	65
1	BB-EMGLO-103	71.2	R1	17.0	54.2	2.9	97	85
			R2	22.0	49.2	7.9	92	48
	BB-EMGLO-104	71.4	R1	10.1	61.3	3.1	97	55
			R2	15.1	56.3	8.1	100	70
	BB-EMGLO-202	71.0	R1	12.3	58.7	5.3	98	69

PARAMETERS FOR CALCULATIONS

1. Unconfined compressive strength information below is based on the average of the test results shown, which represents intact material failures. Recall 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:	57	%
Average RQD from Abutment 1 test borings:	65	%
Average RQD from Abutment 2 test borings:	50	%

Use average RQD at each proposed bridge abutment for final design.

Average peak compressive strength at proposed bridge abutments:	23,736	psi
	3,418	ksf
Average peak compressive strength at Abutment 1:	32,118	psi
	4,625	ksf
Average peak compressive strength at Abutment 2:	12,560	psi
	1,809	ksf

Use average peak compressive strength at each proposed bridge abutment for final design.

Determine bearing resistances at each abutment. Recommend the controlling resistances to account for bedrock variability.

Abutment 1 - 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	4625 ksf	> 4,320 ksf	15
RQD	65%	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 0.05-0.2 in (observed in photos)	Slicken-sided surfaces Joints open 0.05-0.2 in	6
Groundwater Condition	Moist only (interstitial water)	Moist only (interstitial water)	7
Joint Strike and Dip	Fair	Fair	-7
		Total Rating =	44

Abutment 1 - Strength Limit State Continued

Determine s and m
Assume the rock type E

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 polymineralic 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:

$$\frac{m}{m_1} = e^{\left(\frac{RMR-1}{14}\right)} \qquad \text{Equation 18}$$

m1 is the value of m for *intact* rock

$$s = e^{\left(\frac{RMR-1}{6}\right)} \qquad \text{Equation 19}$$

Rock Quality	Rock Type	RMR	m_1	m	s
Fair	E	44	25.00	4.58E-01	8.84E-05

Abutment 1 - Strength Limit State Continued

Semi-empirical method by Carter and Kulhawy 1988:

q_u =	32,118	psi	average of laboratory test results below the footing bearing levels
m =	0.458		
s =	8.84E-05		
q_{ult} =	350.1	ksf	Equation 82b
ϕ =	0.45		from Table 10.5.5.2.2-1
q_R =	157.5	ksf	Equation 82b

Abutment 1 - Service Limit State

Based on Table C10.6.2.5.1-1 the service limit state for bearing resistance on massive crystalline igneous and metamorphic rock is recommended at 120 ksf for settlements of 1 in.

Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense 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

Abutment 1 - Extreme Event Limit State

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

q_{ult} = 350.1 ksf

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

q_R = 280.1 ksf

Abutment 2 - 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	1809 ksf	1,080 - 2,160 ksf	7
RQD	50%	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 0.05-0.2 in (observed in photos)	Slicken-sided surfaces Joints open 0.05-0.2 in	6
Groundwater Condition	Moist only (interstitial water)	Moist only (interstitial water)	7
Joint Strike and Dip	Unfavorable	Unfavorable	-15
		Total Rating =	28

Abutment 2 - Strength Limit State Continued

Determine s and m
Assume the rock type E

Table 17 from NCHRP Report 651:

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

Table 19 from NCHRP Report 651:

Rock quality	Constants	Rock type A = Carbonate rocks with well developed crystal cleavage— *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 polymineralic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite* E = Coarse-grained polymineralic igneous and metamorphic crystalline rocks—*amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
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:

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

m1 is the value of m for *intact* rock

$$s = e^{\left(\frac{RMR-1}{6}\right)} \quad \text{Equation 19}$$

Rock Quality	Rock Type	RMR	m_1	m	s
Poor	E	28	25.00	0.146	0.0000061

Abutment 2 - Strength Limit State Continued

Semi-empirical method by Carter and Kulhawy 1988:

q_u =	12,560	psi	average of laboratory test results below the footing bearing levels
m =	0.146		
s =	6.14E-06		
q_{ult} =	39.2	ksf	Equation 82b
ϕ =	0.45		from Table 10.5.5.2.2-1
q_R =	17.6	ksf	Equation 82b

Abutment 2 - Service Limit State

Based on Table C10.6.2.5.1-1 the service limit state for bearing resistance on weathered or broken rock is recommended at 16 ksf for settlements of 1 in.

Table C10.6.2.5.1-1—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

Abutment 2 - Extreme Event Limit State

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

q_{ult} = 39.2 ksf

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

q_R = 31.3 ksf

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The recommended factored bearing resistance for the strength limit state is 18 ksf, based on the computed bearing resistance for Abutment 2.

Service Limit State

The recommended presumptive value for broken bedrock is 16 ksf for the service limit state for a settlement up to 1.0 in. Broken bedrock was selected due to the presence of weathering and highly fractured zones encountered in some of the cored rock.

Extreme Event Limit State

The recommended factored bearing resistance for the extreme event limit state is 31 ksf, based on the computed bearing resistance for Abutment 2.

Use bearing resistance calculated at Abutment 2 to design both abutment footings to account for potential variability in bedrock.

Sliding Resistance

PROBLEM STATEMENT AND OBJECTIVE

Determine the coefficient of friction between the abutment breastwall and wingwall footings and bedrock, and resistance factors for sliding for the Service, Strength, and Extreme Event Limit States.

EXECUTIVE SUMMARY

The coefficient of friction between the footing and bedrock is =	**0.70**
The resistance factor for sliding at the Strength Limit State is =	**0.8**
The resistance factor for sliding at the Service and Extreme Event Limit States is =	**1.0**

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018.

AVAILABLE INFORMATION

1. Borings BB-EMGLO-101 through BB-EMGLO-104 drilled by New England Boring Contractors in September 2022.

ASSUMPTIONS

1. The abutment footings will bear directly on bedrock.
2. The rock surface will be prepared in-the-dry and in accordance with Section 5.3.4.2 of the BDG.

CALCULATIONS

Coefficient of Friction Between Concrete and Bedrock

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

Estimated footing-rock interface friction angle (δ):

35 deg., friction angle for mass concrete on clean sound rock (AASHTO LRFD Table C3.11.5.3-1)

Recommended δ =	**35**	deg., friction angle between footing/seal and bedrock
Recommended tanδ =	**0.70**	coefficient of friction

Resistance Factors

Service Limit State

AASHTO LRFD Section 10.5.5.1 prescribes a Service Limit State resistance factor equal to = **1.0**

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** (AASHTO LRFD Table 10.5.5.2.2-1)

Extreme Event Limit State

Section 10.5.5.3.3 of AASHTO LRFD prescribes a resistance factor of of foundations in the Extreme Event Limit State. **1.0**

Lateral Earth Pressures

CALCULATIONS		
	File No.	205517-001
	Sheet	1 of 1
Client: Maine Department of Transportation	Date	8-Aug-25
Project: Chase Mills Bridge No. 5465 over Gardner Lake Outlet	Computed by	EMH
Subject: Active Lateral Earth Pressure Coefficients for Semi-Integral Abutments and Wingwalls	Checked by	BCS

PROBLEM STATEMENT & OBJECTIVE

Calculate static active earth pressure coefficients for abutment breastwalls and wingwalls.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020.
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through June 2018.
3. Ddesign drawings prepared by Stantec.

ASSUMPTIONS

1. Abutment breastwalls and wingwalls are backfilled with Granular Borrow.
2. Free draining material used to backfill walls and drains are provided (i.e., no hydrostatic pressures develop).

SOIL PROPERTIES AND WALL 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	26.6	29.7		degrees (horiz., 2H:1V, 1.75H:1V)
Backface of Wall Angle to Horizontal, α =	90	degrees			per Stantec dwg's
Soil and Wall Friction Angle, δ =	24	degrees			Soil Type 4, BDG Table 3-3

RANKINE STATIC ACTIVE LATERAL EARTH PRESSURE COEFFICIENT (K_a)

$$K_a = \cos\beta \cdot \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

MaineDOT BDG Section 3.6.5.2

$K_{a,Rankine}$ =	**0.31**	**0.46**	**0.56**

COULOMB STATIC ACTIVE LATERAL EARTH PRESSURE COEFFICIENT (K_a)

$$K_a = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha \cdot \sin(\alpha-\delta)\cdot\left(1+\sqrt{\frac{\sin(\phi+\delta)\cdot\sin(\phi-\beta)}{\sin(\alpha-\delta)\cdot\sin(\beta+\alpha)}}\right)^2}$$

MaineDOT BDG Section 3.6.6

$K_{a,Coulomb}$ =	**0.27**	**0.46**	**0.54**