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GEOTECHNICAL DESIGN REPORT
CORNSHOP BRIDGE NO. 0318
DEPOT STREET OVER STEVENS BROOK
MAINEDOT WIN 026236.00
BRIDGTON, MAINE

by
Haley & Aldrich, Inc.
Portland, Maine

for
Stantec Consulting Services Inc.
Portland, Maine

File No. 0205731-000
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Attention: Lauren Flanders, P.E.
Senior Structural Engineer

Subject: Geotechnical Design Report
Cornshop Bridge No. 0318
Depot Street over Stevens Brook
MaineDOT WIN 026236.00
Bridgton, Maine

Ladies and Gentlemen:

This geotechnical design report (GDR) presents the results of the geotechnical field investigation and laboratory testing program and provides geotechnical design recommendations intended for the design of the proposed bridge replacement of the existing Cornshop Bridge in Bridgton, Maine (see Figure 1, Project Locus). This GDR has been prepared to support Stantec Consulting Services Inc. (Stantec) preparation of the forthcoming Plans, Specifications, and Estimate (PS&E) submission to the Maine Department of Transportation (MaineDOT). This work has been completed in accordance with our proposal dated 5 November 2024, which was authorized by Stantec on 12 February 2025.

Project Background

EXISTING BRIDGE STRUCTURE

The existing 40-ft-long, single-span bridge carries Depot Street over Stevens Brook in Bridgton. We reviewed the existing bridge plan sheets (dated April 1946, included in Appendix C) and the Highway Bridge Inspection Report (dated 30 March 2021) provided by Stantec. Based on our review of the existing bridge drawings and inspection report, it is our understanding that the existing abutments and wingwalls consist of dry laid granite blocks. There appear to be voids between the granite blocks. The orientation of the void spaces between the granite blocks were observed to be irregular; there were voids between horizontal and vertical surfaces of the granite blocks. The void spaces were approximately 2 to 4 inches (in.) wide. Although not shown on the existing bridge drawings, the inspection report indicates that the abutments were constructed on top of “exposed timber grillage.” Based on a site visit conducted by Haley & Aldrich, Inc. (Haley & Aldrich) on 10 February 2023, the bottom of the existing bridge abutments is at approximately El. 393.5[1] and the lowest row of granite blocks for both existing abutments are bearing on timber grillage (refer to annotated existing bridge drawings and site visit measurements and photographs in Appendix C). Based on our field observations, we anticipate that the timber grillage bears on the alluvial deposits. It is not known how far behind the face of the existing abutments the timber grillage extends (though as noted below, the timber grillage was encountered during drilling test boring [boring] BB-BSB-102). Per discussions with Stantec, we understand some settlement of the existing abutments has occurred as well as scour beneath the existing abutments.

PROPOSED BRIDGE STRUCTURE

The new bridge will be an "on-alignment" replacement of the existing bridge. It is our understanding that there will be little to no change in the vertical profile of the bridge or approaches. We understand that the proposed bridge will be a single-span structure supported by integral abutments on driven steel H-piles.

Geologic Setting

According to Maine Geological Survey's Bridgton Quadrangle, Maine (2008), surficial geologic units mapped within the site vicinity consist of stream alluvium from Stevens Brook and glaciofluvial and glaciolacustrine deposits (which were encountered in the borings). According to the Geologic Map of Maine (1985), bedrock at the site is mapped as Carboniferous age muscovite GRANITE (which was encountered in the borings).

Geotechnical Field Investigation

Haley & Aldrich conducted a geotechnical field investigation at the site between 12 and 14 December 2022. The investigation consisted of drilling two borings (BB-BSB-101 and BB-BSB-102) to identify general subsurface conditions near the proposed bridge abutments. Two shallow, grab samples (GS-BSB-01 and GS-BSB-02) were also obtained upstream of the existing abutment to support Stantec's scour evaluations.

The boring locations were laid out in the field by Haley & Aldrich prior to the start of drilling. "As-drilled" boring locations and ground surface elevations were determined in the field by MaineDOT using global positioning system (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 on Figure 2, Site and Subsurface Exploration Location Plan.

The borings were drilled by New England Boring Contractors of Hermon, Maine using a Mobile B53 track-mounted drill rig. The borings were drilled to depths ranging from 51.9 ft to 57 ft below ground surface (BGS). The borings were advanced using cased-washed drilling methods and a combination of

[1] Elevations referenced herein are in feet (ft), estimated based on provided existing survey information received from Stantec, and reference the North American Vertical Datum of 1988 (NAVD 88).

solid-stem augers and 4-in. (HW-size) outside diameter (OD) steel casing. Soil samples were generally collected continuously through the in-situ fill soils and then typically at 5-ft intervals thereafter. Soils samples were collected by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the boring logs. Drilling and sampling were performed in accordance with MaineDOT specifications. The drill rig was equipped with an automatic hammer calibrated annually per MaineDOT requirements (Appendix A of MaineDOT Geotechnical Drilling Contract Specifications, revised June 2007). A calculated hammer efficiency of 0.93 was calculated for the calibrated automatic hammer system.

The number of hammer blows required to advance the sampler through each 6-in. interval was recorded. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor (0.93) divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected SPT N-values are shown on the boring logs (see Appendix A).

Fill soil samples obtained during drilling were screened with a photoionization detector (PID) to determine the presence of volatile organic compounds (VOCs). No elevated readings of VOCs were measured with the PID during drilling (i.e., PID values were zero parts per million).

The borings were advanced approximately 10 ft to 16 ft into bedrock using a 2-in. (NX-size) ID diamond-tipped core barrel.

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

Generalized Subsurface Conditions

The subsurface conditions encountered consisted of the following geologic units presented in order of increasing depth below ground surface: fill, alluvial deposits, organic deposits, glaciofluvial deposits, glaciolacustrine deposits, glacial till, and bedrock. Refer to Tables II and III for summaries of encountered soil and bedrock conditions. A general description of each soil/bedrock unit is provided separately below. Detailed soil and bedrock descriptions are provided on the boring logs included in Appendix A. Refer to Figure 3, Interpretative Subsurface Profile for a graphical representation of the subsurface conditions present along the proposed bridge alignment. Please note that the soil descriptions provided on the boring logs and summarized below do not represent actual field conditions other than at the specific boring locations. The actual conditions will likely vary from those described and shown herein.

SOIL UNIT DESCRIPTIONS

Soil Unit	Approximate Range in Encountered Thickness (ft)	Generalized Visual Description
Fill	7.5 to 11.5	Loose to very dense, fine to medium SAND, little silt, trace to little coarse sand, trace to little fine gravel, trace coarse gravel, well- to poorly-graded. An approximately 1.7-ft-thick layer of wood was drilled through at boring BB-BSB-102 only (likely existing abutment timber grillage). An approximately 0.5-ft-thick layer of bituminous concrete was encountered at the ground surface in all borings. *(Encountered in all borings.)*
Alluvial Deposit	1.5 to 2.0	Loose, fine SAND, some silt, trace medium to coarse sand, trace organics/fibers. Medium stiff, Sandy SILT. (*Encountered in all borings.*)
Organic Deposit	0.5 to 1.5	Medium stiff, Organic SILT, some fine to medium sand, with root fibers. (*Encountered in all borings.*)
Glaciofluvial Deposit	6.8 to 15.0	Loose, fine to medium SAND to dense, Gravelly fine to medium SAND, with varying amounts of gravel, sand, and silt, well- to poorly-graded. (*Encountered in all borings. An upper and lower glaciofluvial deposit was encountered in boring BB-BSB-101.)*
Glaciolacustrine Deposit	5.0 to 9.0	Very soft, SILT, little fine sand, trace clay to very stiff to hard, Silty CLAY. *(Encountered in all borings.)*
Glacial Till	2.7 to 5.5	Hard, Clayey SILT, little fine to coarse sand. Very dense, fine to Silty fine SAND, trace medium to coarse sand, little silt. *(Encountered in all borings.)*

BEDROCK CONDITIONS

Bedrock was cored in both borings. The top of the bedrock surface ranged from approximately 40.7 ft to 41.3 ft BGS (El. 363.4 to El. 362.1). Bedrock encountered at the site consisted of moderately hard to hard GRANITE and hard BASALT. Highly fractured zones were observed in the BASALT core samples from approximately El. 363.4 to El. 356.6 in boring BB-BSB-102.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the core run. RQD values for bedrock encountered at the site ranged between 0 and 80 percent, which correlates to a Rock Quality rating of very poor to good.

Photographs of the bedrock cores are included in Appendix A.

WATER ELEVATIONS

Groundwater levels were recorded at approximately El. 395 and El. 392 based on measurements made during drilling, as shown on the boring logs in Appendix A. Please note that these readings were taken at the times and conditions immediately after drilling and may fluctuate. Please also note that observation wells were not installed in either of the borings so variation in static water levels at the site have not been determined.

The following existing conditions stream data was provided by Stantec on the draft PS&E plans:

- $Q_{1.1}$: El. 395.2
- Q_{50}: El. 400.7
- Q_{100}: El. 401.5

Laboratory Testing Program

A laboratory testing program was undertaken on representative soil and bedrock samples collected during the field investigation. The program was designed to assist in soil classification/identification and the completion of geotechnical engineering and scour evaluations (to be performed by Stantec). The bedrock testing was performed to understand the compressive strength of the bedrock which is needed to perform micropile rock socket length evaluations (if needed). In general, laboratory testing was performed on disturbed soil samples collected during SPT sampling and on cored bedrock samples. Laboratory testing was performed by GeoTesting Express of Acton, Massachusetts and R.W. Gillespie & Associates, Inc. of Biddeford, Maine. Geotechnical laboratory testing was performed in accordance with applicable ASTM International testing procedures.

The laboratory testing program included performing ten grain size analyses, two Atterberg limits, and three compressive strength and elastic moduli of bedrock tests. The laboratory test results are provided in Appendix B. A summary of laboratory test results is provided below.

Grain Size Analyses (ASTM D6913)

Boring/ Exploration No.	Soil Sample No.	Sample Depth (ft)	Percent Finer than No. 200 Sieve (%)	USCS Classification	Strata
BB-BSB-101	4D	6.0 to 8.0	14.7	SM	Fill
BB-BSB-101	5D	8.0 to 10.0	25.6	SM	Alluvial Deposit
BB-BSB-101	7D	12.0 to 14.0	2.3	SP	Glaciofluvial Deposit (Upper)
BB-BSB-101	9D	20.0 to 22.0	1.2	SP	Glaciofluvial Deposit (Upper)
BB-BSB-101	11D	30.0 to 32.0	9.9	SP-SM	Glaciofluvial Deposit (Lower)
BB-BSB-102	3D	4.0 to 6.0	19.5	SM	Fill
BB-BSB-102	8D	14.0 to 16.0	8.6	SP-SM	Glaciofluvial Deposit
BB-BSB-102	9D	20.0 to 22.0	3.2	SP	Glaciofluvial Deposit
GS[2]-BSB-01	GS1	0.0 to 1.0	6.6	SP-SM	Glaciofluvial Deposit
GS-BSB-02	GS2	0.0 to 1.0	13.7	SM	Glaciofluvial Deposit

Atterberg Limits (ASTM D4318)

Boring No.	Soil Sample No.	Sample Depth (ft)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification	Strata
BB-BSB-101	10D	25.0 to 27.0	27	Non-plastic	Non-plastic	Non-plastic	ML	Glaciolacustrine Deposit
BB-BSB-102	11D	30.0 to 32.0	30	49	22	27	CL	Glaciolacustrine Deposit

Bedrock Core Testing (ASTM D7012 – Method D)

Boring No.	Bedrock Core No.	Sample Depth (ft)	Bedrock Type	Peak Compressive Stress (ksf)	Young's Modulus (psi)
BB-BSB-101	R1	46.0 to 46.4	Granite	4,051	4,850,000 to 5,160,000
BB-BSB-102	R3	45.3 to 45.6		3,956	3,790,000 to 5,210,000
BB-BSB-102	R4	50.2 to 50.6		3,664	3,790,000 to 4,810,000

[2] GS = Grab Sample (obtained for scour evaluations).

Geotechnical Design Recommendations

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

- AASHTO Load and Resistance Factor Design Bridge Design Specifications, Tenth Edition, 2024, referred to herein as AASHTO LRFD.
- MaineDOT Bridge Design Guide, August 2003, with Interim Revisions through June 2018, referred to herein as the BDG.

APPROACH EMBANKMENT DESIGN CONSIDERATIONS

Based on recent coordination with Stantec, it is our understanding that the proposed bridge approach embankments will not involve raises-in-grade or roadway widening. Given these conditions, we do not anticipate that immediate settlement, long-term consolidation settlement, or global instability will impact the performance of the bridge approach embankments.

SEISMIC DESIGN

Seismic Site Class and Design Parameters

The seismic site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 by correlating SPT N-values and undrained shear strengths to shear wave velocity (V_s).

Per AASHTO LRFD Section 3.10.3.1, site class was determined by the average shear wave velocity in the upper 100 ft of soil and/or bedrock. Because shear wave velocity was not measured directly, AASHTO LRFD allows V_s to be estimated using correlations as stated above. Site class was derived using each of the average shear wave velocity ($\bar{v}_s$), $\bar{v}_s/1.33$, and $1.3\bar{v}_s$.

Based on the thickness, descriptions, and estimated shear wave velocities of the overburden soils, as well as the depth to bedrock at the site, Seismic Site Class C, C/D, and D were calculated using the different mean shear wave velocities listed above. Risk-targeted spectral accelerations were determined based on the geographic site location and the different seismic site classifications using the United States Geological Survey Seismic Design Web Service and are included in Appendix D. The AASHTO LRFD design response spectra are 5% damped, risk-targeted design response spectra.

Liquefaction Evaluation

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

The liquefaction evaluations were completed using a design acceleration coefficient (A_s) at structural period equal to zero, 0.132g, based on Seismic Site Class C/D, which was determined to provide the highest A_s value, compared to Seismic Site Classes C and D.

Based on the simplified liquefaction evaluation using the data from the borings, it is our opinion that the site soils are not susceptible to liquefaction. The factor of safety in the samples from all the borings was greater than 1.0.

BRIDGE ABUTMENT FOUNDATION SUPPORT

Based on the subsurface conditions encountered in the borings, it is our opinion that glacial till and bedrock are the shallowest materials suitable to support the bridge abutments. Based on scour susceptibility of near surface soils, discussions with Stantec, and observations made during our site visit, we do not consider supporting the bridge on shallow foundations bearing in the glaciofluvial deposit to be feasible, even though the existing bridge abutments are founded in this stratum. We recommend that the bridge abutments be supported on steel H-piles driven to end bearing in/on bedrock.

Driven Steel H-piles

Based on discussions with Stantec and the draft plan set that we received from Stantec on 1 October 2025, we understand the bridge abutments will be supported on driven steel HP14x89 H-piles advanced through overburden soils to end bearing in/on bedrock.

AXIAL COMPRESSION PILE RESISTANCE

Structural Resistance

The structural axial compression resistance of HP14x89 piles was calculated in accordance with AASHTO LRFD Sections 6.9.2 and 6.9.4. The structural resistance factor (Section 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving where use of a pile tip is necessary is 0.50. In addition, resistance factors for Service and Extreme Limit State loading are 1.0. The nominal and factored structural resistances of the steel HP14x89 H-pile section (with F_y = 50 ksi) at the Service, Strength, and Extreme Event Limit States are summarized below.

Steel H-pile Section	**Nominal Structural Resistance (kips)**	**Factored Structural Resistance (kips)**		
		Service Limit State (ϕ=1.0)	**Strength Limit State (ϕ=0.50)**	**Extreme Limit State (ϕ=1.0)**
HP14x89	1,026	1,026	513	1,026

We note that the structural resistance calculated only addresses axial resistance. If the piles are subjected to lateral loads, then the structural resistance under combined axial load and flexure should be evaluated. The unbraced length used in the evaluation above from Sections 6.9.2 and 6.9.4 was assumed to be zero.

Drivability Resistance

The engineering design of the H-piles also includes consideration of drivability resistance in accordance with AASHTO LRFD Section 10.7.8. Drivability evaluations were conducted using the computer program GRL WEAP 14 developed by GRL Engineers, Inc. to determine the following: 1) if the piles could be impact driven through the hard/very dense glacial till to bedrock; and 2) what nominal resistance could be achieved using hammer sizes typically used by local pile driving contractors without damaging or overstressing the piles while keeping the penetration resistance below 15 blows per inch (bpi), which is the upper limit of penetration resistance allowed by MaineDOT.

Drivability evaluations were conducted assuming that a Delmag D36-32 single acting diesel hammer with a maximum rated energy equal to 91 kip-ft will be used to install the piles. Evaluations were completed at both abutment locations based on the subsurface conditions present at the site. The controlling drivability resistances for the HP14x89 steel H-piles, applicable to both abutments, are summarized below.

Steel H-pile Section	Drivability Resistance (kips)	
	Nominal	Factored (ϕ_{dyn}=0.65)
HP14x89	750	488

Based on the results of the drivability evaluations, we anticipate that the piles will be able to be advanced through the overburden soils without exceeding both the AASHTO LRFD limit of 45 kips per square inch (ksi; 90 percent of F_y for 50 ksi steel piles) and the MaineDOT penetration resistance limit of 15 bpi. The piles will encounter refusal on bedrock and will achieve the nominal drivability resistance noted above.

Please keep in mind that the drivability resistances summarized above are based on an assumed pile hammer size, assumed hammer efficiency, hammer assembly, and the assumption that the piles penetrate through the entire soil overburden and end bear in/on bedrock. If the actual, factored axial compressive pile loads vary or if the actual pile-hammer system used to install the piles is different than the assumed system, additional evaluations will be required to determine the nominal and factored drivability resistances that can be achieved at a reasonable penetration resistance without overstressing the piles.

Summary of Axial Compressive Pile Resistances

The factored axial compressive H-pile resistance is controlled by the lesser of the Strength Limit State drivability and structural resistances since the H-piles will be installed using impact hammers and the minimum nominal resistance will be confirmed using dynamic testing. As shown below, drivability resistance will control pile design. The abutment factored resistances (structural and drivability) are summarized below.

Steel H-pile Section	Factored Structural Resistance (kips)	Factored Drivability Resistance (kips)	Governing Factored Resistance (kips)
HP14x89	513	488	488

LATERAL PILE RESISTANCE

At the request of Stantec, Haley & Aldrich developed soil profiles and soil properties at the abutments for Stantec's use in conducting soil structure interaction/lateral pile evaluations. Recommended strength and lateral soil properties and profiles are summarized below.

Abutment No. 1 (north)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pounds per cubic ft, pcf)	Undrained Shear Strength, S_u (pounds per square-ft, psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pounds per cubic in., pci)	Strain Factor, ε_{50}
Fill	Sand (Reese)	402.9	52.6	N/A	30	60	N/A
Alluvial Deposit	Sand (Reese)	395.4	57.6	N/A	28	20	N/A
Organic Deposit	Soft Clay (Matlock)	393.4	37.6	525	N/A	N/A	0.02
Glaciofluvial Deposit (Upper)	Sand (Reese)	391.9	57.6	N/A	28	20	N/A
Glaciolacustrine Deposit	Soft Clay (Matlock)	379.4	57.6	250	N/A	N/A	0.02
Glaciofluvial Deposit (Lower)	Sand (Reese)	374.4	57.6	N/A	35	125	N/A
Glacial Till	Sand (Reese)	367.6	67.6	N/A	40	125	N/A
Bedrock	N/A	362.1	N/A	N/A	N/A	N/A	N/A

Abutment No. 2 (south)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ε_{50}
Fill	Sand (Reese)	403.6	52.6	N/A	30	60	N/A
Alluvial/Organic Deposit	Soft Clay (Matlock)	392.1	57.6	580	N/A	N/A	0.02
Glaciofluvial Deposit	Sand (Reese)	390.1	57.6	N/A	30	60	N/A
Glaciolacustrine Deposit	Stiff Clay with Free Water	375.1	57.6	1,580	N/A	N/A	0.005
Glacial Till	Sand (Reese)	366.1	67.6	N/A	40	125	N/A
Bedrock	N/A	363.4	N/A	N/A	N/A	N/A	N/A

ABUTMENT DESIGN

For structural design of integral abutments, we recommend use of passive earth pressures, using a Coulomb passive earth pressure coefficient, k_p, of 6.6.

Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005 (where H is measured from bottom of pile cap to top of abutment). If the calculated displacements are significantly less than that required to develop full passive pressure (i.e., y/H less than 0.005), the Rankine passive earth pressure coefficient, 3.25, should be used.

Soil properties of Soil Type 4, granular borrow, should be used for abutment backfill material. The backfill properties per BDG Section 3.6.1 are as follows:

- Friction Angle, ϕ = 32 degrees
- Total Unit Weight, γ = 125 pcf
- Soil-Concrete Interface Friction Angle, δ = 24 degrees

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

We recommend that the abutment design include a drainage system in accordance with the requirements of BDG Sections 5.4.1.9 and 5.4.2.13 to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutments. Using a freedraining backfill material and providing adequate drainage will substantially reduce the potential for unbalanced hydrostatic pressures from developing.

ESTIMATED PILE LENGTHS

Based on the encountered depth to bedrock, we estimate driven piles will be approximately 42- to 43-ft-long, incorporating an additional 5 ft for the purposes of developing pile quantities for bid. This estimated pile length is based on the following assumptions:

- The tops of piles at El. 397.
- The bottom of the proposed abutments at El. 395.
- The top of bedrock is at approximately El. 362 to El. 363.
- Considering the poor quality of the upper 5 ft of the bedrock at both abutments, we anticipate that driven piles could gain capacity 2 ft to 3 ft below the top of bedrock (approximately El. 359 to El. 360).

The estimated pile lengths do not account for locations where bedrock may be deeper or shallower than was encountered in the preliminary borings, damaged piles, additional pile length required for dynamic testing instrumentation (per ASTM D4945; for driven piles), or additional pile length needed to accommodate the Contractor's leads and driving equipment (for driven piles).

PILE EMBEDMENT, SPACING, CLEARANCE, AND PILE MATERIAL

We recommend that minimum pile spacing, clearance, and embedment (into the abutment pile caps) meet the requirements of AASHTO LRFD Section 10.7.1.2. We also recommend that the piles be equipped with a pile tip in accordance with MaineDOT Standard Specification 501.048. Based on the potential for hard driving and sloping bedrock, we recommend the pile tip consist of a HP-77750-B manufactured by Associated Pipe & Fitting, or equivalent, fabricated from cast steel conforming to ASTM A148 Grade 90/60.

DOWNDRAG FORCES

As discussed above, since there are minimal proposed embankment grading changes from the existing conditions, we do not anticipate that downdrag forces will act on the foundation piles.

CORROSION AND DETERIORATION

The geotechnical engineering design of the proposed piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. Based on our visual review of the soil samples and our experience on similar projects with similar soil conditions, it is our opinion that the in-situ soils have low corrosion potential. However, laboratory testing to investigate corrosion potential has not been performed. Therefore, the net factored pile resistance recommended above includes a reduction in pile cross-sectional area for steel degradation of 1/16 in.

FROST PROTECTION

The minimum depth of embedment/cover for footings or other below-grade structures was evaluated in accordance with BDG Section 5.2.1. Based on a design freezing index equal to 1,800 freezing degree days, we recommend that walls or other foundations bear a minimum of 6 ft below the lowest adjacent ground surface exposed to freezing. However, in accordance with BDG Section 5.4.2.8 standard details and discussions with MaineDOT, the bottom of the integral abutment pile cap should be a minimum of 4 ft below the adjacent ground surface.

Construction Considerations

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

PRE-EXCAVATION PRIOR TO PILE DRIVING

Pre-excavation through the fill and removal of granite blocks and timber grillage (located beneath the granite blocks) at the proposed driven pile locations will be required in some locations prior to the start of pile driving.

DYNAMIC PILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the minimum required nominal pile resistances in the field using dynamic testing methods. The piles should be driven to a nominal resistance equal to the maximum factored axial compressive pile load divided by a resistance factor equal to 0.65 in accordance with AASHTO LRFD Table 10.5.5.2.3-1 (i.e., 750 kips = 488 kips/0.65). We recommend that the Contractor perform two dynamic pile load tests with 24-hour (minimum) restrike tests at each abutment (four total) to evaluate hammer system efficiencies, driving stresses in the pile, and the nominal resistance of the piles. We recommend that the dynamic testing be completed prior to production pile driving and that CAPWAP analysis be performed on each pile installed during the dynamic test program (four total). The CAPWAP results will be used to finalize driving criteria for the production piles to assure that the piles achieve the necessary resistance without being overstressed.

SUBMITTAL REVIEWS

The Contract Drawings and Special Provisions (if needed) should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. Standard Specifications require that the Contractor and the Contractor’s engineer perform analyses and submit results to Stantec and MaineDOT for review. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor’s analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions, and specifications, and to facilitate design changes 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 MaineDOT, qualified by geotechnical training and experience be present at the site to provide monitoring during the foundation construction activities listed below.

- Dynamic testing of driven test piles.
- Installation of production driven piles.
- Placement and compaction of compacted fills.

Limitations and Closure

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

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

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

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

Sincerely yours,
HALEY & ALDRICH, INC.

Nathan A. Sherwood, P.E.
Senior Project Manager

Erin A. Force, P.E.
Senior Associate

STATE OF MAINE
ERIN A. FORCE
No. 12207
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 – Site and Subsurface Exploration Location Plan
- Figure 3 – Interpretive Subsurface Profile
- Appendix A – Boring Logs and Bedrock Core Photographs
- Appendix B – Laboratory Test Results
- Appendix C – Annotated Existing Bridge Drawings and Site Visit Measurements and Photographs
- Appendix D – Geotechnical Calculations

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References

Maine Geological Survey's Bridgton Quadrangle, Maine (2008).

Geologic Map of Maine (1985).

TABLE I
Subsurface Exploration Location Data
Cornshop Bridge over Stevens Brook, Bridge No. 0318
MaineDOT WIN No. 26236.00
Bridgton, Maine

Haley & Aldrich, Inc. File No.: 0205731-000

Test Boring No.[1]	Ground Surface Elevation (ft)[3]	Station[4]	Offset Distance (ft) & Direction[4,5]	Horizontal Coordinates[2]	
				Northing (Y)	Easting (X)
BB-BSB-101	403.4	11+63.7	6.4 LT	444,872	897,054
BB-BSB-102	404.1	12+13.9	5.0 RT	444,828	897,080

Notes:

1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.

2 As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference NAD83, Maine 2000 coordinate system.

3 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet, and reference the North American Vertical Datum of 1988 (NAVD 88).

4 Station and offset information shown are approximate and are relative project baseline and were determined by Haley & Aldrich based on information provided by MaineDOT.

5 LT = offset distance toward left direction; RT = offset distance toward right direction; ft = feet.

	Individual	Date
Prepared By:	NAS	10/16/2025
Reviewed By:	EAF	10/17/2025

TABLE II
Subsurface Exploration Subsurface Data
Cornshop Bridge over Stevens Brook, Bridge No. 0318
MaineDOT WIN No. 26236.00
Bridgton, Maine

Haley & Aldrich, Inc. File No.: 0205731-000

Boring No.[1]	Estimated Ground Surface Elevation[2] (ft)	Stratigraphy Data																				Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration[2] (ft)
		Fill[3,4]			Alluvial Deposit			Organic Deposit			Glaciofluvial Deposit[5]			Glaciolacustrine Deposit			Glacial Till			Bedrock			
		Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)		
BB-BSB-101	403.4	0.5	402.9	7.5	8.0	395.4	2.0	10.0	393.4	1.5	11.5 (upper)/ 29.0 (lower)	391.9 (upper)/ 374.4 (lower)	12.5 (upper)/ 6.8 (lower)	24.0	379.4	5.0	35.8	367.6	5.5	41.3	362.1	51.9	351.5
BB-BSB-102	404.1	0.5	403.6	11.5	12.0	392.1	1.5	13.5	390.6	0.5	14.0	390.1	15.0	29.0	375.1	9.0	38.0	366.1	2.7	40.7	363.4	57.0	347.1

Notes:
1 Boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
2 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet (ft), and reference the North American Vertical Datum of 1988 (NAVD 88).
3 Bituminous concrete was encountered in the upper 0.5 ft of each boring.
4 An approximately 1.7-ft-thick layer of wood (likely existing timber grillage) was encountered at boring BB-BSB-102 only.
5 Upper and lower layers were encountered at boring BB-BSB-101 only.

	Individual	Date
Prepared By:	EMH	2/1/2023
Checked By:	NAS	10/16/2025
Reviewed By:	EAF	10/16/2025

TABLE III
Subsurface Exploration Bedrock Core Data
Cornshop Bridge over Stevens Brook, Bridge No. 0318
MaineDOT WIN No. 26236.00
Bridgton, Maine

Haley & Aldrich, Inc. File No.: 0205731-000

Boring No.[1]	Estimated Ground Surface Elevation[2] (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery[3]		Rock Quality Designation[4,5]		Physical Rock Parameters		Peak Compressive Strength[6] (ksf)	Lithologic, Rock Mass, and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	%	Designation	Weathering	Estimated Field Strength		
				Top	Bottom	Midpoint									
BB-BSB-101	403.4	NX (2")	R1	42.0	46.5	44.3	4.5	3.3	74%	22%	Very Poor	Slightly	Moderately Hard	4,051	Orange-brown with black and tan, fine to coarse-grained, muscovite GRANITE. Primary joints horizontal to low angles, spacing close to moderate, discontinuity aperatures open, rough. Secondary joints dipping at steep to vertical angles, spacing moderately close, discontinuity aperatures open. Orange-brown color due to weathered feldspar through sample.
			R2	46.5	51.9	49.2	5.4	5.4	100%	77%	Good	Fresh to Slightly	Moderately Hard	-	
BB-BSB-102	404.1	NX (2")	R1	42.0	45.0	43.5	3.0	1.5	50%	0%	Very Poor	Slightly	Hard	-	Grey, aphanitic to fine-grained, BASALT. Primary joints dipping at vertical angles, spacing very close, discontinuity angles open. Calcite coatings on some joint surfaces. Highly fractured throughout.
			R2	45.0	47.0	46.0	2.0	1.3	67%	0%	Very Poor	Slightly	Hard	-	
			R3	47.0	52.0	49.5	5.0	5.0	100%	62%	Fair	Fresh to Slightly	Hard	3,956	Black and white, fine to coarse-grained, muscovite GRANITE, primary joints horizontal, spacing close to moderately close, aperatures open, rough. Slight iron oxide discoloration on joint surfaces
			R4	52.0	57.0	54.5	5.0	4.4	88%	80%	Good	Very Slightly	Hard	3,664	

Notes:
[1] Boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet (ft), and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
[4] RQD = rock quality designation. RQD is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
[5] Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information.
[6] Peak compressive strength was determined in accordance with ASTM D7012 - Method D.

	Individual	Date
Prepared By:	EMH	2/2/2023
Checked By:	NAS	10/16/2025
Reviewed By:	EAF	10/16/2025

SITE COORDINATES: 44°03'10"N, 70°42'21"W

MAP SOURCE: USGS

HALEY ALDRICH

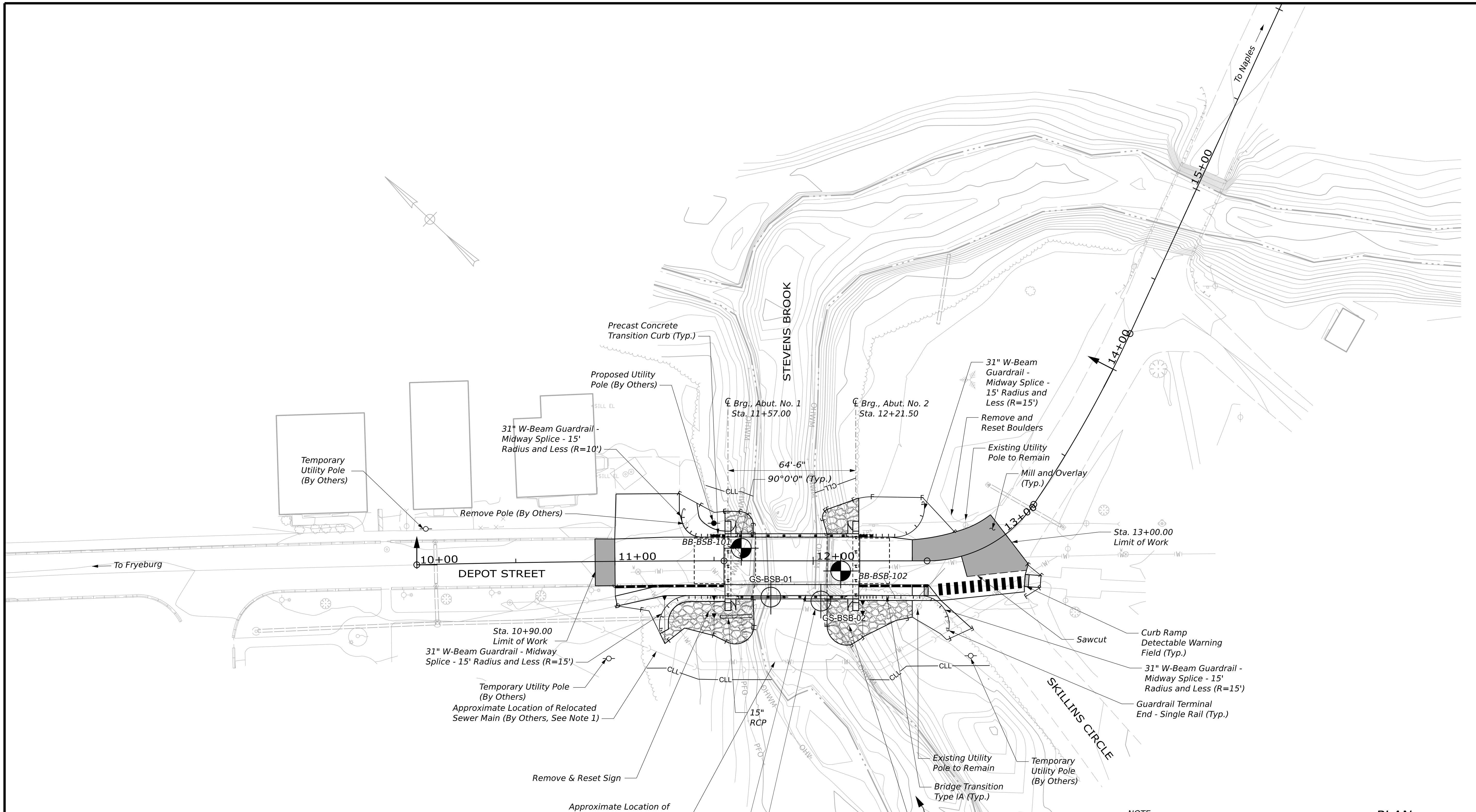
CORNSHOP BRIDGE NO. 0318
DEPOT STREET OVER STEVENS BROOK
BRIDGTON, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 INCH = 3,000 FEET
JANUARY 2026

FIGURE 1

0205731_000_LOCUS HALEYALDRICH\NSHERWOOD



NOTE
1. The sewer force main and water main relocations have been completed by others prior to the bridge construction. Locations shown are approximate and shall be verified in the field.

PLAN

25 0 25 50

Scale of Feet

LEGEND:

BB-BSB-101	Designation and location of test boring drilled by New England Boring Contractors and monitored by Haley & Aldrich, Inc. in December 2022.
GS-BSB-01	Designation and approximate location of grab sample obtained by Haley & Aldrich, Inc. in December 2022. As-sampled locations shown were determined by taping to existing site features.
	Approximate location and orientation of interpretive subsurface profile (see Sheet 3).

Key

BB = Bridge boring

GS = Grab sample

NOTES:

1. Existing site and topographic information and project stationing were taken from electronic files provided by Stantec on 9 May 2023.
2. As-drilled locations of test borings were determined in the field by MaineDOT using GPS survey equipment.
3. Refer to Appendix A for test boring logs.
4. Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

HALEY ALDRICH

CORNSHOP BRIDGE NO. 0318
DEPOT STREET OVER STEVENS BROOK
BRIDGTON, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SCALE: AS SHOWN
JANUARY 2026

FIGURE 2

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2623600

BRIDGE NO. 0318

WIN 26236.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	E. HUNSTEIN	K. POSTOLOWSKI	9/11/24
CHECKED-REVIEWED	N. SHERWOOD	E. FORCE	9/25/25
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

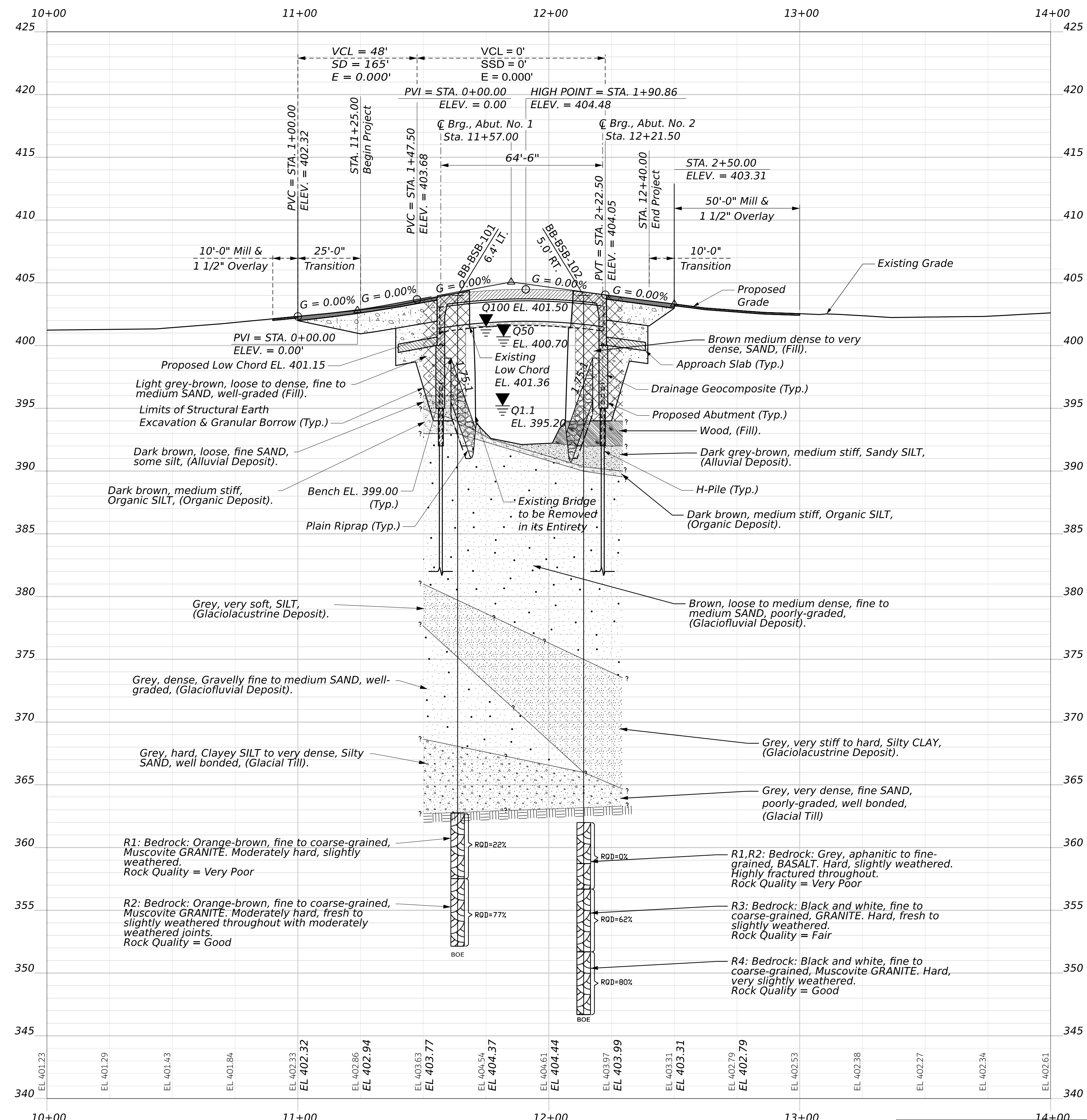
CORNSHOP BRIDGE NO. 0318
CROSSINGS STEVENS BROOK
BRIDGTON

SHEET NUMBER

OF

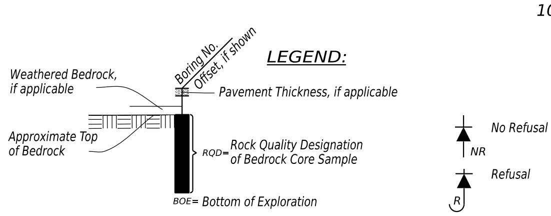
Username: kpostolowski Date: 1/7/2026

Filename: \\haleyaldrich.com\share\CF\Projects\0205731\CAD\0205731_002_Boring Plan.dgn



PROFILE

LEGEND:



NOTES:

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. The existing timber grillage measurements and observations are limited. The lengths and depths of the timbers are unknown.

HALEY ALDRICH

CORNSHOP BRIDGE NO. 0318
DEPOT STREET OVER STEVENS BROOK
BRIDGTON, MAINE

INTERPRETIVE SUBSURFACE PROFILE

SCALE: AS SHOWN
JANUARY 2026

FIGURE 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2623600

WIN 26236.00

BRIDGE PLANS

BRIDGE NO. 0318

	BY	DATE
PROJ. MANAGER	J. BRASK	
DESIGN-DETAILED	N. SHERWOOD	K. POSTOLOWSKI 9/11/24
CHECKED-REVIEWED	N. SHERWOOD	E. FORCE 9/25/25
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

CORNSHOP BRIDGE NO. 0318
CROSSINGS STEVENS BROOK
BRIDGTON

Username: kpostolowski Date: 9/25/2025

Filename: \\haleyaldrich.com\share\CF\Projects\0205731\CAD\0205731_003_ISP.dgn

APPENDIX A

Boring Logs and Bedrock 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: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: BB-BSB-101
WIN: 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	403.4	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-13-2022/12-14-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 11+63.7, 6.4 ft LT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	8 ft (Approx.)
Hammer Efficiency Factor: 0.93		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/10	0.5 - 2.0	15-15-14	29	45		402.9		Bituminous Concrete — 0.5 — Light grey-brown, dry, dense, fine to medium SAND, little coarse sand, little silt, little fine gravel, trace coarse gravel, well-graded, (Fill).	
	2D	24/10	2.0 - 4.0	10-21-11-10	32	50				Light grey-brown, dry, dense, fine to medium SAND, little coarse sand, little silt, little fine gravel, trace coarse gravel, well-graded, (frozen top 4 in.), (Fill).	
	3D	24/8	4.0 - 6.0	8-5-3-2	8	12				Similar to 2D, except medium dense, (Fill).	
5											
	4D	24/6	6.0 - 8.0	2-3-2-2	5	8				Similar to 2D, except loose, (Fill).	#701514 A-1-b, SM
								395.4		8.0	
	5D	24/10	8.0 - 10.0	3-2-1-1	3	5				Dark brown, wet, loose, fine SAND, some silt, trace medium to coarse sand, trace organics/fibers, (Alluvial Deposit).	#701515 A-2-4, SM
10								393.4		10.0	
	6D	24/14	10.0 - 12.0	1-2-3-3	5	8				Dark brown, wet, medium stiff, Organic SILT, some fine to medium sand, (Organic Deposit).	
								391.9		11.5	
	7D	24/12	12.0 - 14.0	4-3-3-3	6	9				6D Bottom: Brown, wet, loose, fine to medium SAND, trace silt, poorly-graded, (Glaciofluvial Deposit). 7D: Brown, wet, loose, fine to medium SAND, little coarse gravel, trace coarse sand, trace fine gravel, poorly-graded, (Glaciofluvial Deposit).	#701516 A-3, SP
	8D	24/0	14.0 - 16.0	3-2-3-3	5	8				8D: No Recovery	
15											
20											
	9D	24/6	20.0 - 22.0	2-2-3-4	5	8				Light grey-brown, wet, loose, fine to medium SAND, trace fine to coarse gravel, trace coarse sand, trace silt, poorly-graded, (Glaciofluvial Deposit).	#17399-01 A-1-b, SP WC=19.4
								379.4		24.0	
25											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: BB-BSB-101
WIN: 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	403.4	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-13-2022/12-14-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 11+63.7, 6.4 ft LT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	8 ft (Approx.)
Hammer Efficiency Factor:	0.93	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	10D	24/16	25.0 - 27.0	WOR-WOR-WOH-2	-					Grey, wet, very soft, SILT, little fine sand, trace clay, non-plastic, (Glaciolacustrine Deposit).	#701517 WC=27
								374.4		29.0	
										Note: Cobbles from 29.3 to 29.8 ft.	
30	11D	24/8	30.0 - 32.0	14-12-8-12	20	31				Grey, wet, dense, Gravelly fine to medium SAND, trace coarse sand, trace silt, well-graded, slightly bonded, (Glaciofluvial Deposit).	#701518 A-1-b, SP-SM
										Note: Cobble from 33.5 to 33.8 ft.	
35	12D	24/22	35.0 - 37.0	16-11-12-17	23	36		367.6		35.8	
										Grey, wet, hard, Clayey SILT, little fine to coarse sand, slightly plastic, well bonded, (Glacial Till).	
40	13D	16/12	40.0 - 41.3	7-21-50(4")	71	110		362.7		40.7	
								362.1		Grey, wet, very dense, Silty fine SAND, trace medium to coarse sand, poorly-graded, well bonded, (Glacial Till).	
										41.3	
	R1	54/40	42.0 - 46.5	RQD = 22%			NX Core			Top of Bedrock at El. 362.1 ft Note: Advance rollerbit to 42.0 ft. Begin NX core at 42.0 ft. R1: Orange-brown with black and tan, fine to coarse-grained, Muscovite GRANITE. Moderately hard, slightly weathered. Primary joints horizontal, close to moderately close, open with no infilling. Secondary joints dipping at steep to vertical angles, moderately close, open, rough, orange-brown color due to weathered feldspar through sample. Rock Quality=Very Poor Recovery=74% R1 Core Times (min:sec): 42.0-43.0' (5:00); 43.0-44.0' (3:00); 44.0-45.0' (10:00); 45.0-46.0' (6:00); 46.0-46.4' (5:00)	
45	R2	65/65	46.5 - 51.9	RQD = 77%						R2: Orange-brown with black and tan, fine to coarse-grained, Muscovite GRANITE. Moderately hard, fresh to slightly weathered throughout with moderately weathered joints with iron-oxide staining. No infilling. Primary joints horizontal to low angles, close to moderate, open, rough. Secondary joints dipping	qp=4,051 ksf (46.04'-46.41')
50											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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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: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: BB-BSB-101
WIN: 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	403.4	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-13-2022/12-14-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 11+63.7, 6.4 ft LT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	8 ft (Approx.)
Hammer Efficiency Factor:	0.93	**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.
50							NX Core	351.5		at steep to vertical angles, moderately close, open, rough. Rock Quality=Good Recovery=100% R2 Core Times (min:sec): 46.5-47.5' (5:00); 47.5-48.5' (7:00); 48.5-49.5' (5:00); 49.5-50.5' (6:00); 50.5-50.9' (5:00) 51.9 **Bottom of Exploration at 51.9 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: BB-BSB-102
WIN: 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	404.1	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-12-2022/12-13-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 12+13.9, 5.0 ft RT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	12 ft (Approx.)
Hammer Efficiency Factor:	0.93	**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
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	18/12	0.5 - 2.0	110-40-23	63	98		403.6		Bituminous Concrete (0.5) Light brown, dry, very dense, fine to medium SAND, little silt, trace coarse sand, trace fine gravel, poorly-graded, (Fill).	
	2D	24/18	2.0 - 4.0	12-13-14-8	27	42				Yellow-brown, dry, dense, fine to medium SAND, little silt, trace coarse sand, trace fine gravel, poorly-graded, (Fill).	
5	3D	24/8	4.0 - 6.0	4-4-3-8	7	11				Similar to 2D, except medium dense, (Fill).	#701519 A-2-4, SM
	4D	24/0	6.0 - 8.0	7-6-5-3	11	17				4D: No Recovery Note: Possibly pushed a cobble.	
	5D	24/6	8.0 - 10.0	4-3-4-7	7	11				Yellow-brown, dry, medium dense, fine to medium SAND, little silt, trace coarse sand, trace fine gravel, poorly-graded, (Fill).	
10	6D	24/10	10.0 - 12.0	5-3-3-3	6	9		393.8		Similar to 5D, except light grey, (Fill). (10.3) Upper 4 in.: Sawdust, wood particles Bottom 2 in.: Wood. Note: Wood present beneath existing bridge abutment.	
	7D	24/10	12.0 - 14.0	5-3-2-2	5	8		392.1		(12.0) Dark grey-brown, wet, medium stiff, Sandy SILT, (Alluvial Deposit).	
								390.6 / 390.1		(13.5) Dark brown, wet, medium stiff, Organic SILT with root fibers, little fine sand, (Organic Deposit). (14.0)	
15	8D	24/6	14.0 - 16.0	4-5-3-3	8	12				Brown-grey, wet, medium dense, fine to medium SAND, little coarse gravel, trace coarse sand, trace silt, trace fine gravel, well-graded, (Glaciofluvial Deposit).	#701520 A-1-b, SP-SM
20	9D	24/10	20.0 - 22.0	5-5-4-8	9	14		384.1		(20.0) Light brown-grey, wet, medium dense, Gravelly medium to coarse SAND, trace fine sand, trace silt, poorly-graded, (Glaciofluvial Deposit).	#17399-02 A-1-a, SP WC=9.9
25											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Cornshop Bridge #0318 carries Depot Street over Stevens Brook **Location:** Bridgton, Maine	**Boring No.:** BB-BSB-102 **WIN:** 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	404.1	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-12-2022/12-13-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 12+13.9, 5.0 ft RT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	12 ft (Approx.)
Hammer Efficiency Factor:	0.93	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	10D	24/12	25.0 - 27.0	4-5-9-10	14	22				Light brown-grey, wet, medium dense, fine SAND, little medium to coarse sand, trace silt, poorly-graded, (Glaciofluvial Deposit).	
								375.1		29.0	
30	11D	24/16	30.0 - 32.0	4-5-6-8	11	17				Grey, wet, very stiff, Silty CLAY, moderate plasticity, trace rock fragments in bottom of sample, (Glaciolacustrine Deposit).	#701521 WC=30 LL=49 PL=22 PI=27
35	12D	24/18	35.0 - 37.0	8-17-15-15	32	50				Grey, wet, hard, Silty CLAY, high plasticity, (Glaciolacustrine Deposit). Pocket Penetrometer=1.5-2.0 tsf	
								366.1		38.0	
40	13D	8/8	40.0 - 40.7	23-50(2")				363.4		Grey, wet, very dense, fine SAND, little silt, trace medium to coarse sand, poorly-graded, well bonded, (Glacial Till). 40.7 Top of Bedrock at El. 363.4 ft Note: Advance rollerbit to 42.0 ft. Begin NX core at 42.0 ft.	
	R1	36/18	42.0 - 45.0	RQD = 0%			NX Core			R1: Grey, aphanitic to fine-grained, BASALT. Hard, slightly weathered. Primary joints dipping at vertical angles, very close, open, calcite coatings on some joint surfaces. Highly fractured throughout. Rock Quality=Very Poor Recovery=50% R1 Core Times (min:sec): 42.0-43.0' (5:00); 43.0.0-44.0' (8:00); 44.0-45.0' (5:00)	
45	R2	24/16	45.0 - 47.0	RQD = 0%						R2: Similar to R1 Rock Quality=Very Poor Recovery=67% R2 Core Times (min:sec): 45.0-46.0' (5:00); 46.0-47.0' (5:00)	
	R3	60/60	47.0 - 52.0	RQD = 62%				356.6		47.5 R3: Black and white, fine to coarse-grained, Muscovite GRANITE. Hard, fresh to slightly weathered. Primary joints horizontal, close to moderately close, open, rough. Rock Quality=Fair	
50											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: BB-BSB-102
WIN: 26236.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	404.1	**Auger ID/OD:**	--
Operator:	S. Shaw	**Datum:**	NAVD 88	**Sampler:**	Split-Spoon (1.375 in. ID)
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-53 Track	**Hammer Wt./Fall:**	140 lb./30 in.
Date Start/Finish:	12-12-2022/12-13-2022	**Drilling Method:**	SSA/HW Cased Wash/Drive	**Core Barrel:**	NX-2 in. ID
Boring Location:	Sta. 12+13.9, 5.0 ft RT	**Casing ID/OD:**	HW-4 in. ID	**Water Level*:**	12 ft (Approx.)
Hammer Efficiency Factor:	0.93	**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.
50							NX Core			Recovery=100% R3 Core Times (min:sec): 47.0-48.0' (5:00); 48.0-49.0' (7:00); 49.0-50.0' (5:00); 50.0-51.0' (5:00); 51.0-52.0' (5:00)	qp=3,956 ksf (50.2'-50.8')
	R4	60/53	52.0 - 57.0	RQD = 80%						R4: Black and white, fine to coarse-grained, Muscovite GRANITE. Hard, very slightly weathered. Primary joints horizontal, close to moderately close, open, rough, slight iron-oxide discoloration on joint surfaces. Rock Quality=Good Recovery=88% R4 Core Times (min:sec): 52.0-53.0' (4:00); 53.0-54.0' (4:00); 54.0-55.0' (4:00); 55.0-56.0' (6:00); 56.0-57.0' (4:00)	
55											qp=3,664 ksf (54.6'-55.3')
								347.1		57.0 **Bottom of Exploration at 57.0 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

1. No elevated photo-ionization detector readings were measured.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: GS-BSB-01
WIN: 26236.00

Driller:	--	**Elevation (ft.)**	392.0	**Auger ID/OD:**	--
Operator:	--	**Datum:**	NAVD 88	**Sampler:**	Grab Sample
Logged By:	H. Hollauer	**Rig Type:**	--	**Hammer Wt./Fall:**	--
Date Start/Finish:	12-14-2022	**Drilling Method:**	--	**Core Barrel:**	--
Boring Location:	11+76.0, 18.2 ft RT	**Casing ID/OD:**	--	**Water Level*:**	--
Hammer Efficiency Factor:	--	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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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	GS1		0.0 - 1.0					391.0		Brown, wet, Gravelly fine to coarse SAND, trace silt, poorly-graded, (Glaciofluvial Deposit). 1.0	#17399-03 A-1-a, SP-SM WC=18.5
										Bottom of Exploration at 1.0 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

1. Grab sample collected at the brook level adjacent to the existing northern abutnment.
2. Existing ground surface elevation estimated based on existing conditions plan provided by Stantec.

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.: GS-BSB-01

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cornshop Bridge #0318 carries Depot Street over Stevens Brook
Location: Bridgton, Maine

Boring No.: GS-BSB-02
WIN: 26236.00

Driller:	--	**Elevation (ft.)**	392.0	**Auger ID/OD:**	--
Operator:	--	**Datum:**	NAVD 88	**Sampler:**	Grab Sample
Logged By:	H. Hollauer	**Rig Type:**	--	**Hammer Wt./Fall:**	--
Date Start/Finish:	12-14-2022	**Drilling Method:**	--	**Core Barrel:**	--
Boring Location:	12+5.0, 20.8 ft RT	**Casing ID/OD:**	--	**Water Level*:**	--
Hammer Efficiency Factor: --		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	GS1		0.0 - 1.0					391.0		Brown, wet, fine to medium SAND, little silt, trace coarse sand, trace fine gravel, poorly-graded, (Glaciofluvial Deposit). 1.0	#17399-04 A-2-4 SM WC=48.3
										Bottom of Exploration at 1.0 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

1. Grab sample collected at the brook level adjacent to the existing southern abutnment.
2. Existing ground surface elevation estimated based on existing conditions plan provided by Stantec.

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.: GS-BSB-02

BEDROCK CORE PHOTOGRAPHS
CORNSHOP BRIDGE NO. 0318
BRIDGTON, MAINE
File No. 0205731-000

Top Row: BB-BSB-101, Run No. 1 42.0 ft (left) to 46.5 ft (right)
Top Middle Row: BB-BSB-101, Run No. 2 46.5 ft (left) to 51.5 ft (right)
Bottom Middle Row: BB-BSB-101, Run No. 2 51.5 ft (left) to 51.9 ft (right)
Bottom Row: Empty

Top Row: BB-BSB-102, Run No. 1 42.0 ft (left) to 45.0 ft (middle-left); Run No. 2 45.0 ft (middle left) to 47.0 ft (right)
Top Middle Row: BB-BSB-102, Run No. 3 47.0 ft (left) to 52.0 ft (right)
Bottom Middle Row: BB-BSB-102, Run No. 4 52.0 ft (left) to 57.0 ft (right)
Bottom Row: Empty

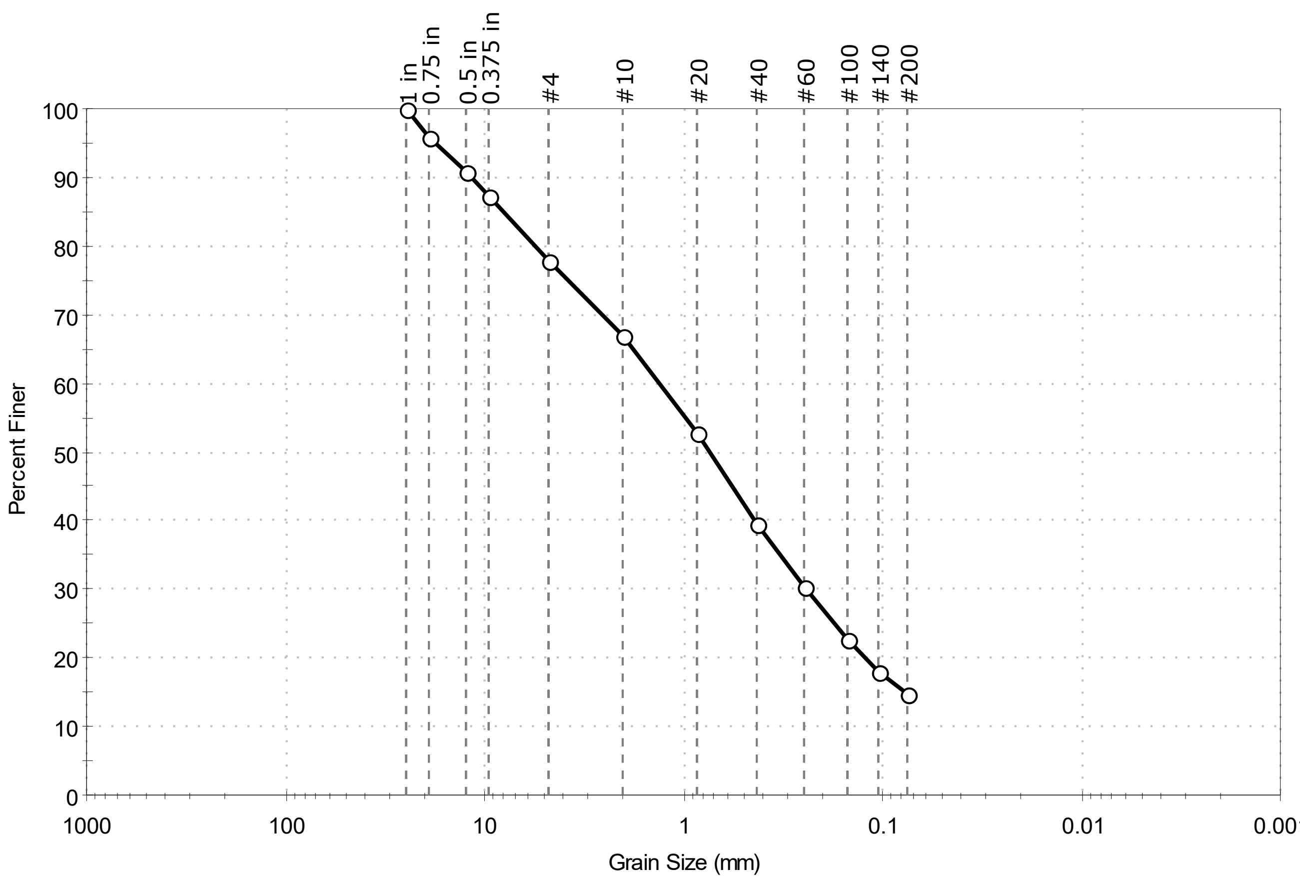
APPENDIX B

Laboratory Test Results

Client: Haley & Aldrich, Inc.			
Project: Cornshop Bridge			
Location: Bridgton, ME		Project No:	GTX-316595
Boring ID: BB-BSB-101	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 01/11/23	Checked By:	jsc
Depth : 6-8'	Test Id: 701514		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	22.1	63.2	14.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	96		
0.5 in	12.50	91		
0.375 in	9.50	87		
#4	4.75	78		
#10	2.00	67		
#20	0.85	53		
#40	0.42	40		
#60	0.25	30		
#100	0.15	23		
#140	0.11	18		
#200	0.075	15		

Coefficients

D_{85} = 7.9942 mm D_{30} = 0.2434 mm
D_{60} = 1.3130 mm D_{15} = 0.0773 mm
D_{50} = 0.7371 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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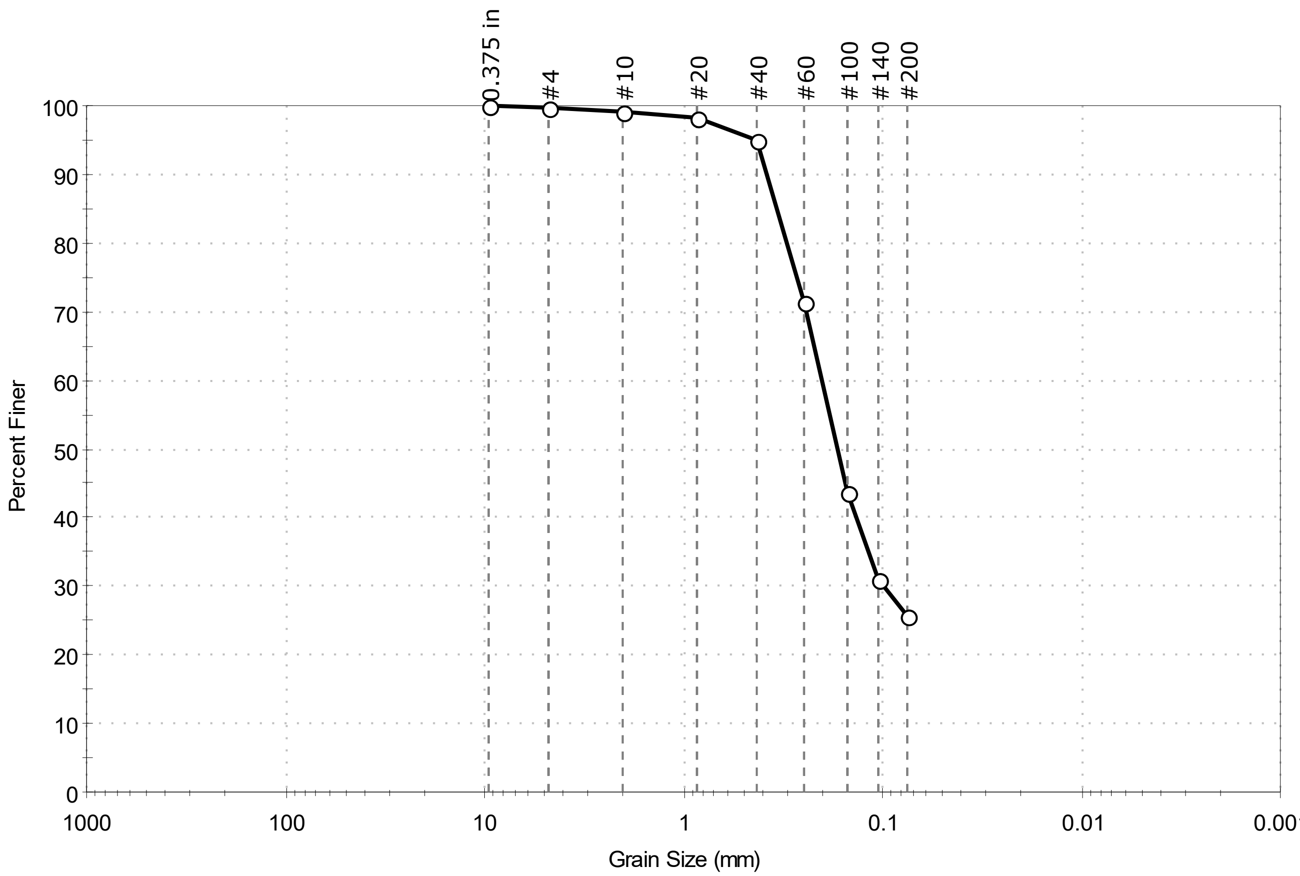
Client:	Haley & Aldrich, Inc.		
Project:	Cornshop Bridge		
Location:	Bridgton, ME	Project No:	GTX-316595
Boring ID: BB-BSB-101	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 01/11/23	Checked By:	jsc
Depth : 8-10'	Test Id: 701515		
Test Comment:	---		
Visual Description:	Moist, very dark brown silty sand		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.5	73.9	25.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	98		
#40	0.42	95		
#60	0.25	71		
#100	0.15	44		
#140	0.11	31		
#200	0.075	26		

Coefficients

D_{85} =0.3401 mm D_{30} =0.0989 mm

D_{60} =0.2030 mm D_{15} =N/A

D_{50} =0.1690 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 : ---

Sand/Gravel Hardness : ---

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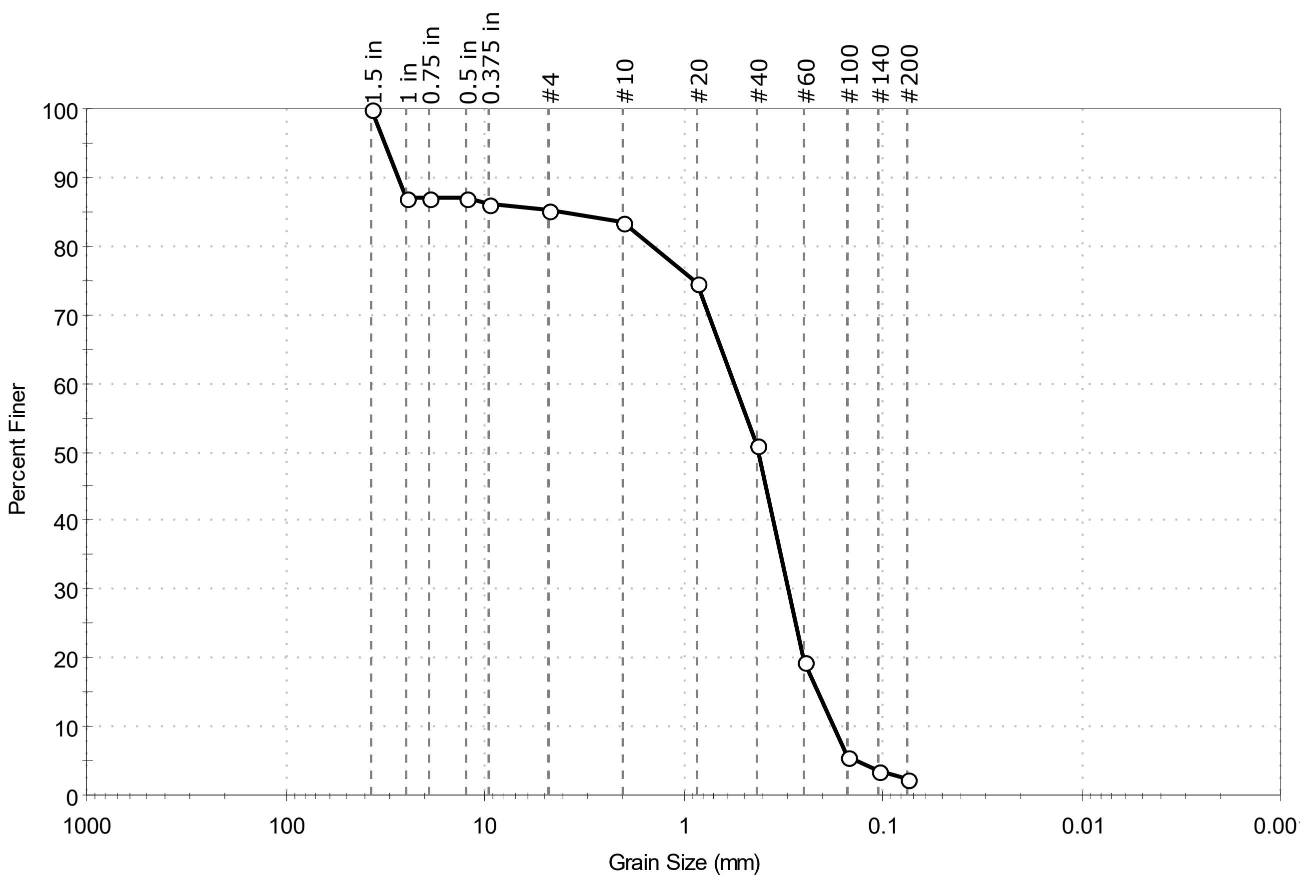
Client: Haley & Aldrich, Inc.			
Project: Cornshop Bridge			
Location: Bridgton, ME		Project No:	GTX-316595
Boring ID: BB-BSB-101	Sample Type: jar	Tested By:	ckg
Sample ID: 7D	Test Date: 01/11/23	Checked By:	jsc
Depth : 12-14'	Test Id: 701516		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.9	82.8	2.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	87		
0.75 in	19.00	87		
0.5 in	12.50	87		
0.375 in	9.50	86		
#4	4.75	85		
#10	2.00	83		
#20	0.85	75		
#40	0.42	51		
#60	0.25	19		
#100	0.15	6		
#140	0.11	3		
#200	0.075	2.3		

Coefficients

D_{85} =4.4525 mm
D_{30} =0.2991 mm
D_{60} =0.5539 mm
D_{15} =0.2126 mm
D_{50} =0.4185 mm
D_{10} =0.1763 mm
C_u =3.142
C_c =0.916

Classification

ASTM Poorly graded SAND (SP)

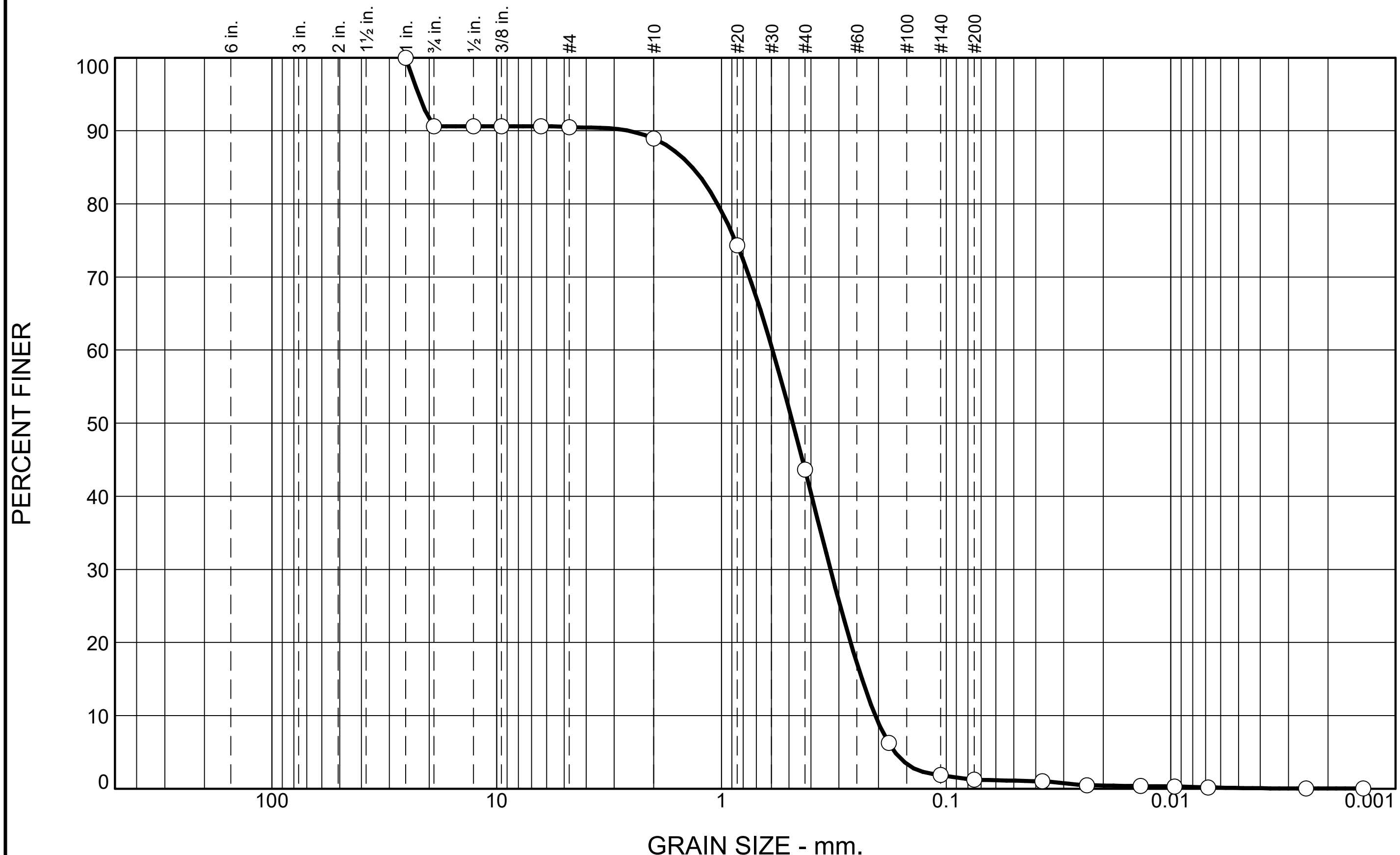
AASHTO Fine Sand (A-3 (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.4	0.1	1.6	45.3	42.4	1.2	0.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	90.6		
1/2"	90.6		
3/8"	90.6		
1/4"	90.6		
#4	90.5		
#10	88.9		
#20	74.3		
#40	43.6		
#80	6.2		
#140	1.9		
#200	1.2		
0.0373 mm.	1.0		
0.0237 mm.	0.5		
0.0137 mm.	0.4		
0.0096 mm.	0.3		
0.0068 mm.	0.2		
0.0025 mm.	0.0		
0.0014 mm.	0.0		

* (no specification provided)

Soil Description

poorly graded sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.5946 D_{85}= 1.3499 D_{60}= 0.5920
D_{50}= 0.4811 D_{30}= 0.3272 D_{15}= 0.2371
D_{10}= 0.2066 C_u= 2.87 C_c= 0.88

Classification

USCS= SP AASHTO= A-1-b

Remarks

Moisture Content: 19.4%

Location: BB-BSB-101
Sample Number: 9D **Depth:** 20-22' **Date:** 02/15/2023

R.W. Gillespie & Associates, Inc. Biddeford, Maine

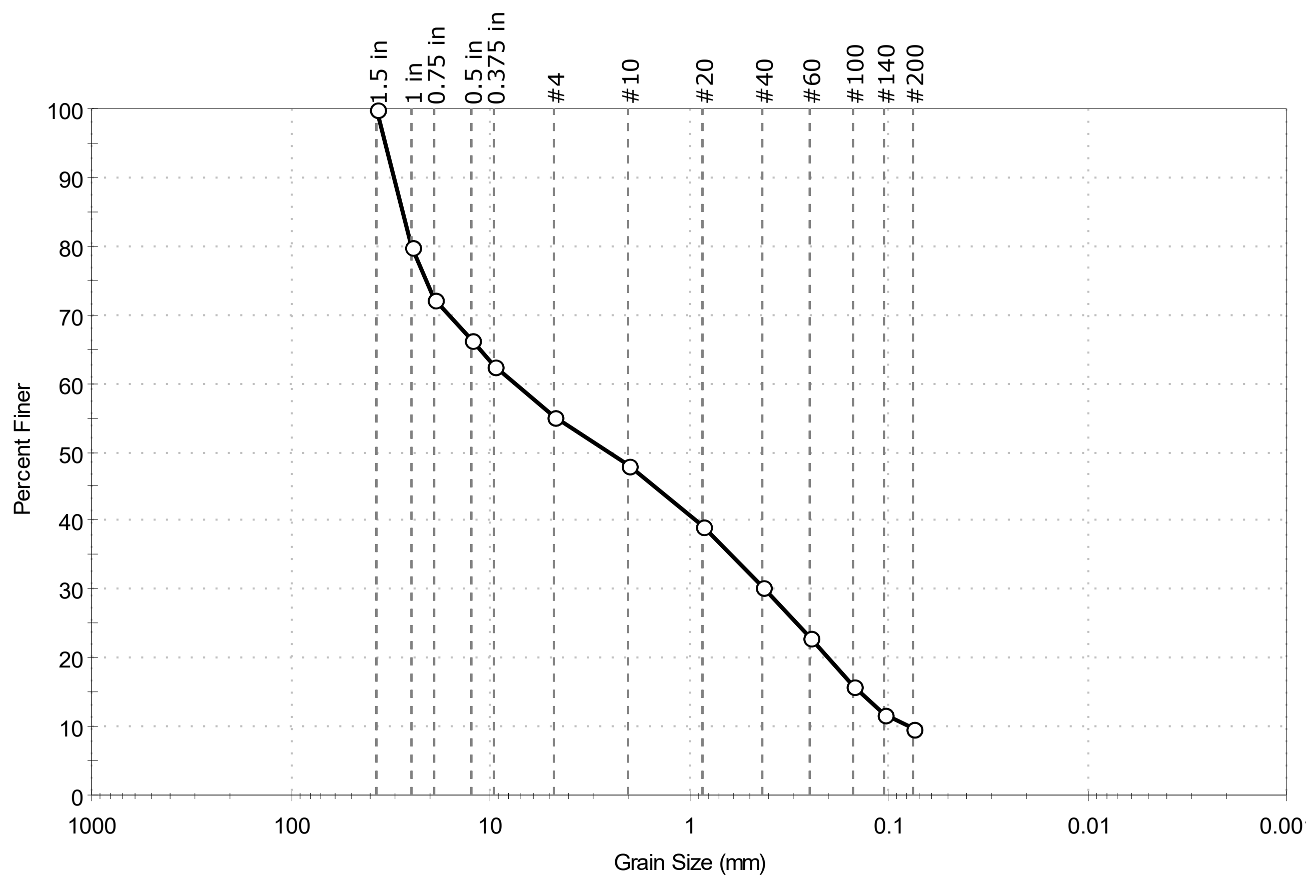
Client: Haley & Aldrich, Inc.
Project: Bridgton Cornship Bridge #0205731-000
Bridgton, ME
Project No: 0956-013BG10 **Lab No.** 17399-01

Tested By: JMT/CAG **Checked By:** MTG

MTG

Client:	Haley & Aldrich, Inc.				
Project:	Cornshop Bridge				
Location:	Bridgton, ME			Project No:	GTX-316595
Boring ID:	BB-BSB-101	Sample Type:	jar	Tested By:	ckg
Sample ID:	11D	Test Date:	01/11/23	Checked By:	jsc
Depth :	30-32'	Test Id:	701518		
Test Comment:	---				
Visual Description:	Moist, gray sand with silt and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	44.9	45.2	9.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	80		
0.75 in	19.00	72		
0.5 in	12.50	66		
0.375 in	9.50	63		
#4	4.75	55		
#10	2.00	48		
#20	0.85	39		
#40	0.42	30		
#60	0.25	23		
#100	0.15	16		
#140	0.11	12		
#200	0.075	9.9		

Coefficients

D_{85} =27.6837 mm	D_{30} =0.4174 mm
D_{60} =7.4981 mm	D_{15} =0.1391 mm
D_{50} =2.5228 mm	D_{10} =0.0767 mm
C_u =97.759	C_c =0.303

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

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Client:	Haley & Aldrich, Inc.		
Project:	Cornshop Bridge		
Location:	Bridgton, ME	Project No:	GTX-316595

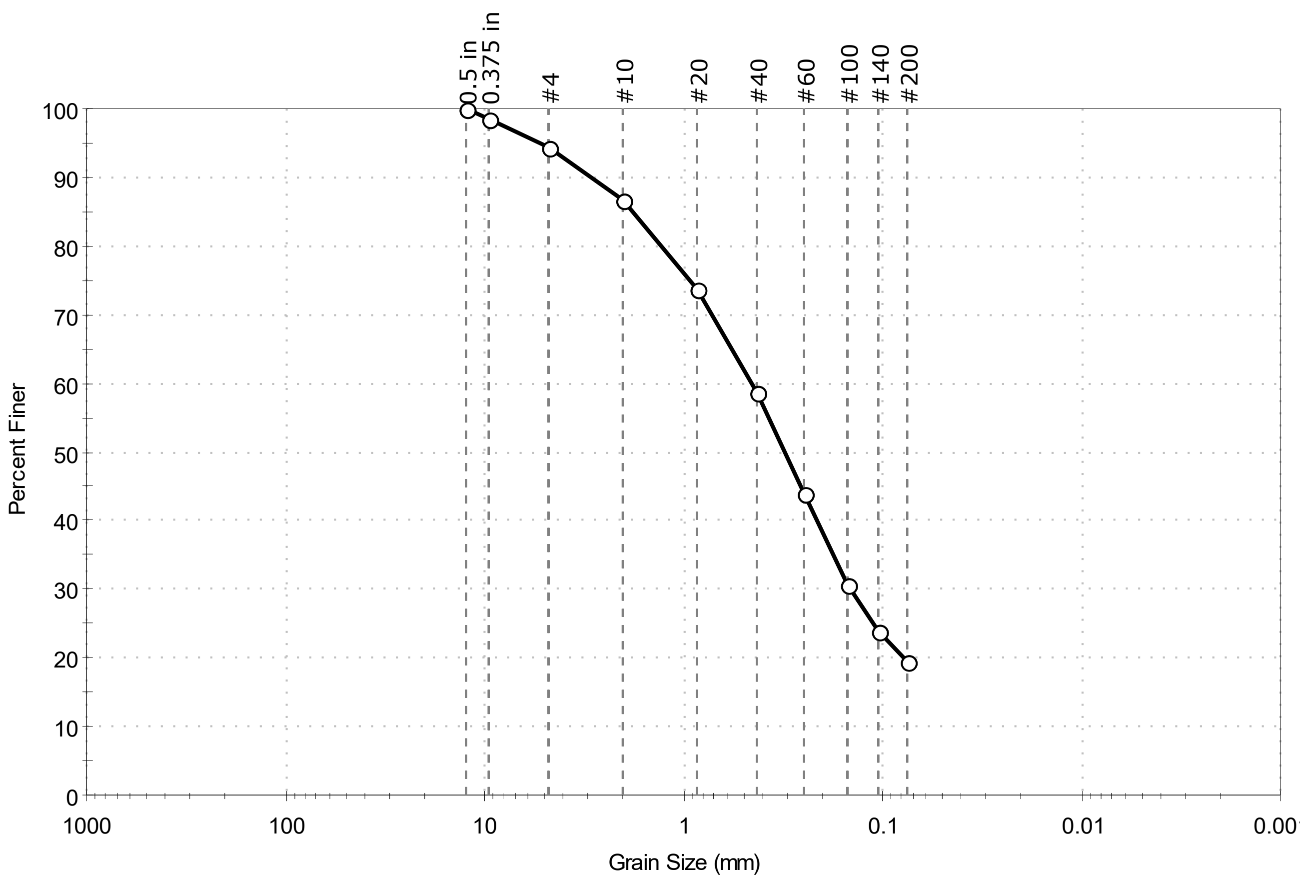
Boring ID:	BB-BSB-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	3D	Test Date:	01/11/23	Checked By:	jsc
Depth :	4-6'	Test Id:	701519		

Test Comment: ---

Visual Description: Moist, brown silty sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.7	74.8	19.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	94		
#10	2.00	87		
#20	0.85	74		
#40	0.42	59		
#60	0.25	44		
#100	0.15	31		
#140	0.11	24		
#200	0.075	19		

Coefficients

D_{85} =1.7729 mm

D_{30} =0.1452 mm

D_{60} =0.4489 mm

D_{15} =N/A

D_{50} =0.3112 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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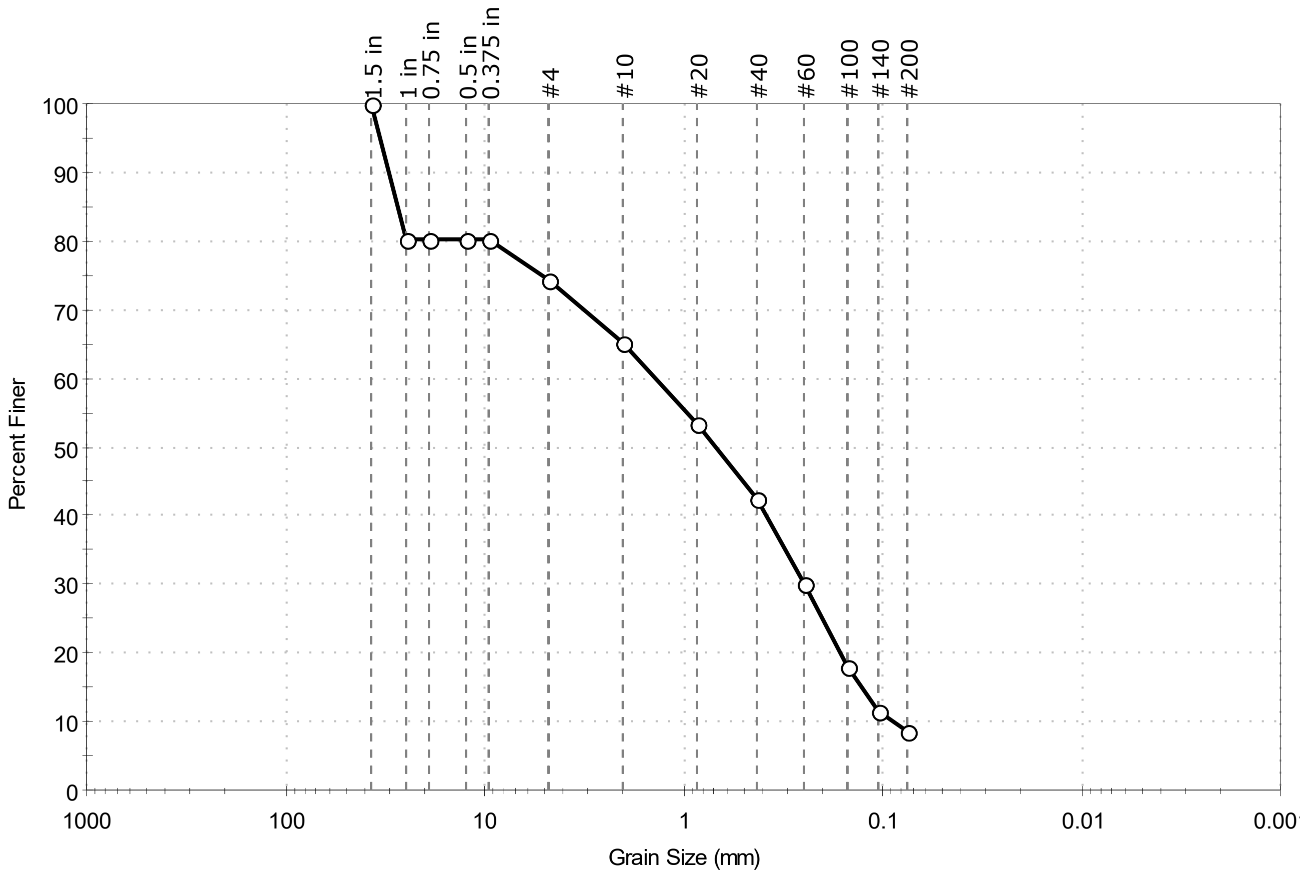
Client:	Haley & Aldrich, Inc.		
Project:	Cornshop Bridge		
Location:	Bridgton, ME	Project No:	GTX-316595
Boring ID:	BB-BSB-102	Sample Type: jar	Tested By: ckg
Sample ID:	8D	Test Date: 01/11/23	Checked By: jsc
Depth :	14-16'	Test Id: 701520	
Test Comment:	---		
Visual Description:	Moist, grayish brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.5	65.9	8.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	80		
0.75 in	19.00	80		
0.5 in	12.50	80		
0.375 in	9.50	80		
#4	4.75	74		
#10	2.00	65		
#20	0.85	53		
#40	0.42	42		
#60	0.25	30		
#100	0.15	18		
#140	0.11	11		
#200	0.075	8.6		

Coefficients

D_{85} =27.5425 mm
D_{30} =0.2495 mm
D_{60} =1.3797 mm
D_{15} =0.1274 mm
D_{50} =0.6895 mm
D_{10} =0.0888 mm
C_u =15.537
C_c =0.508

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

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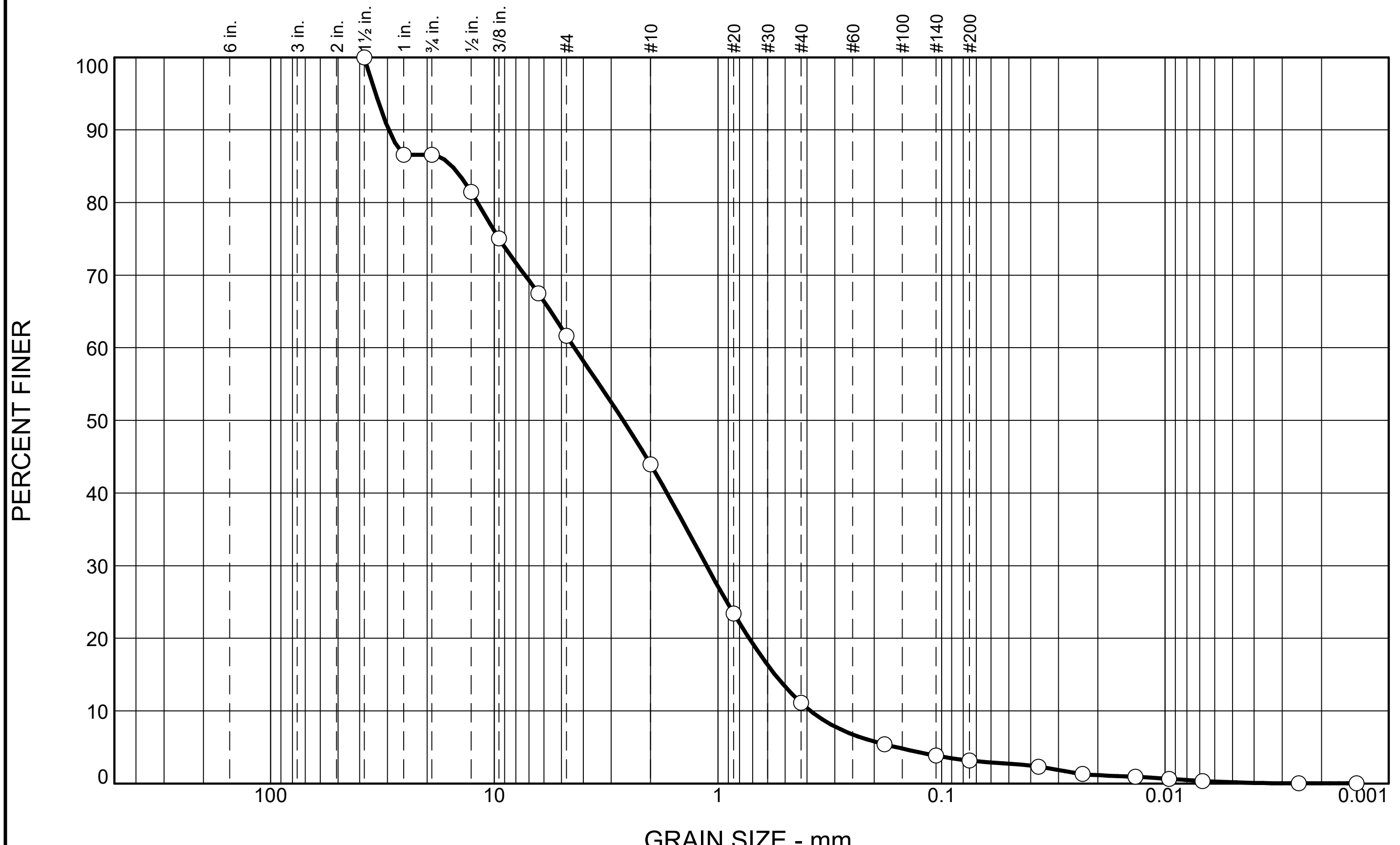
Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.4	25.0	17.7	32.8	7.9	3.2	0.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	86.6		
3/4"	86.6		
1/2"	81.5		
3/8"	75.0		
1/4"	67.5		
#4	61.6		
#10	43.9		
#20	23.4		
#40	11.1		
#80	5.4		
#140	3.9		
#200	3.2		
0.0368 mm.	2.3		
0.0234 mm.	1.3		
0.0136 mm.	1.0		
0.0096 mm.	0.6		
0.0068 mm.	0.3		
0.0025 mm.	0.0		
0.0014 mm.	0.0		

* (no specification provided)

Soil Description

poorly graded sand with gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 29.6160 D_{85}= 15.4372 D_{60}= 4.3815
D_{50}= 2.6529 D_{30}= 1.1239 D_{15}= 0.5549
D_{10}= 0.3852 C_u= 11.37 C_c= 0.75

Classification

USCS= SP AASHTO= A-1-a

Remarks

Moisture Content- 9.9%

Location: BB-BSB-102
Sample Number: 9D **Depth:** 20-22' **Date:** 02/15/2023

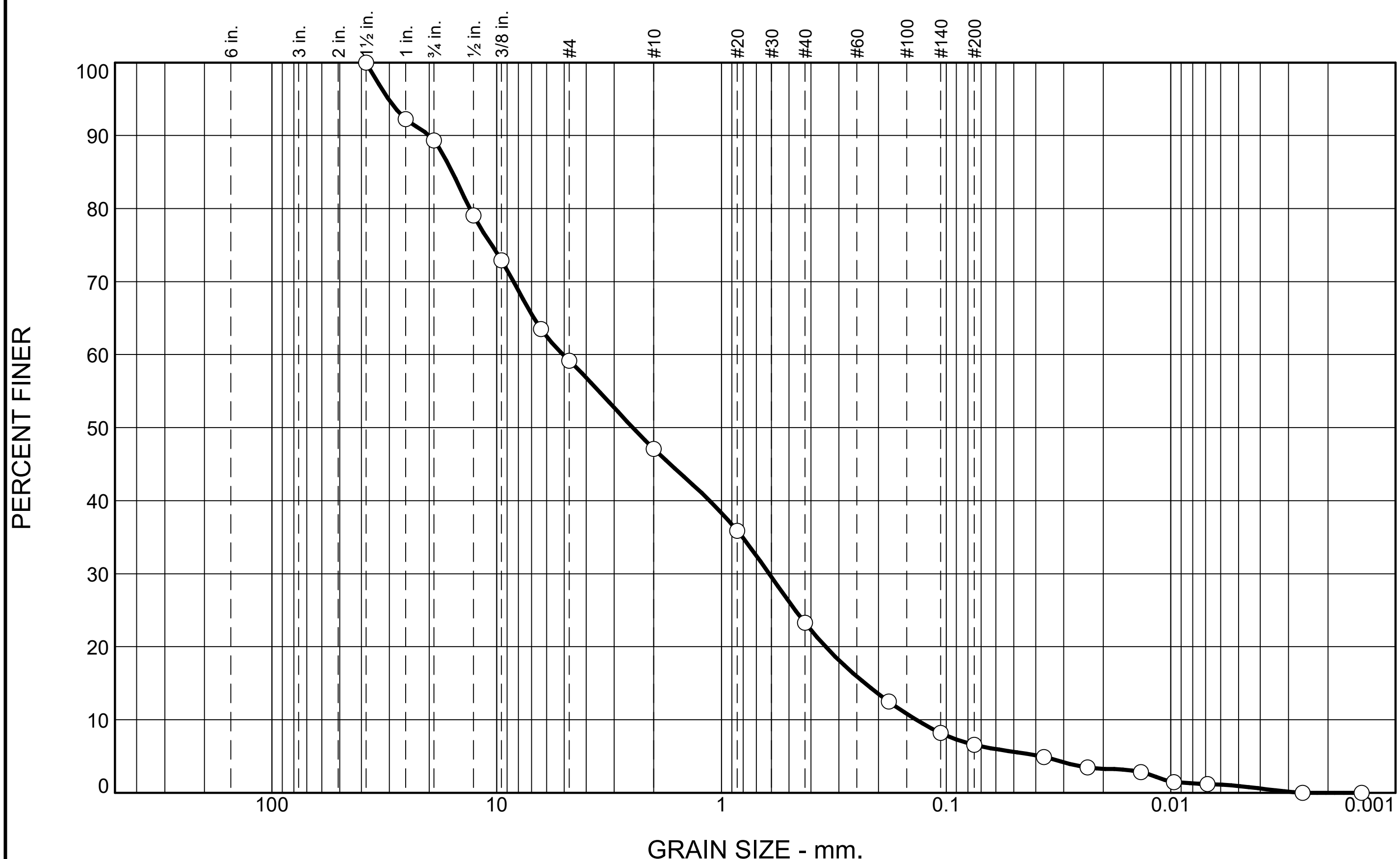
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Bridgton Cornship Bridge #0205731-000
Bridgton, ME
Project No: 0956-013BG10 **Lab No.** 17399-02

Tested By: JMT/CAG **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.7	30.1	12.1	23.8	16.7	6.6	0.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	92.2		
3/4"	89.3		
1/2"	79.1		
3/8"	72.9		
1/4"	63.5		
#4	59.2		
#10	47.1		
#20	35.9		
#40	23.3		
#80	12.5		
#140	8.2		
#200	6.6		
0.0367 mm.	4.9		
0.0235 mm.	3.5		
0.0136 mm.	2.8		
0.0097 mm.	1.4		
0.0069 mm.	1.2		
0.0026 mm.	0.0		
0.0014 mm.	0.0		

* (no specification provided)

Soil Description

poorly graded sand with silt and gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 19.9739 D_{85}= 15.7680 D_{60}= 5.0604
D_{50}= 2.4792 D_{30}= 0.6132 D_{15}= 0.2299
D_{10}= 0.1360 C_u= 37.22 C_c= 0.55

Classification

USCS= SP-SM AASHTO= A-1-a

Remarks

Moisture Content: 18.5%

Location: GS-BSB-01
Sample Number: Grab **Depth:** 0-1' **Date:** 02/15/2023

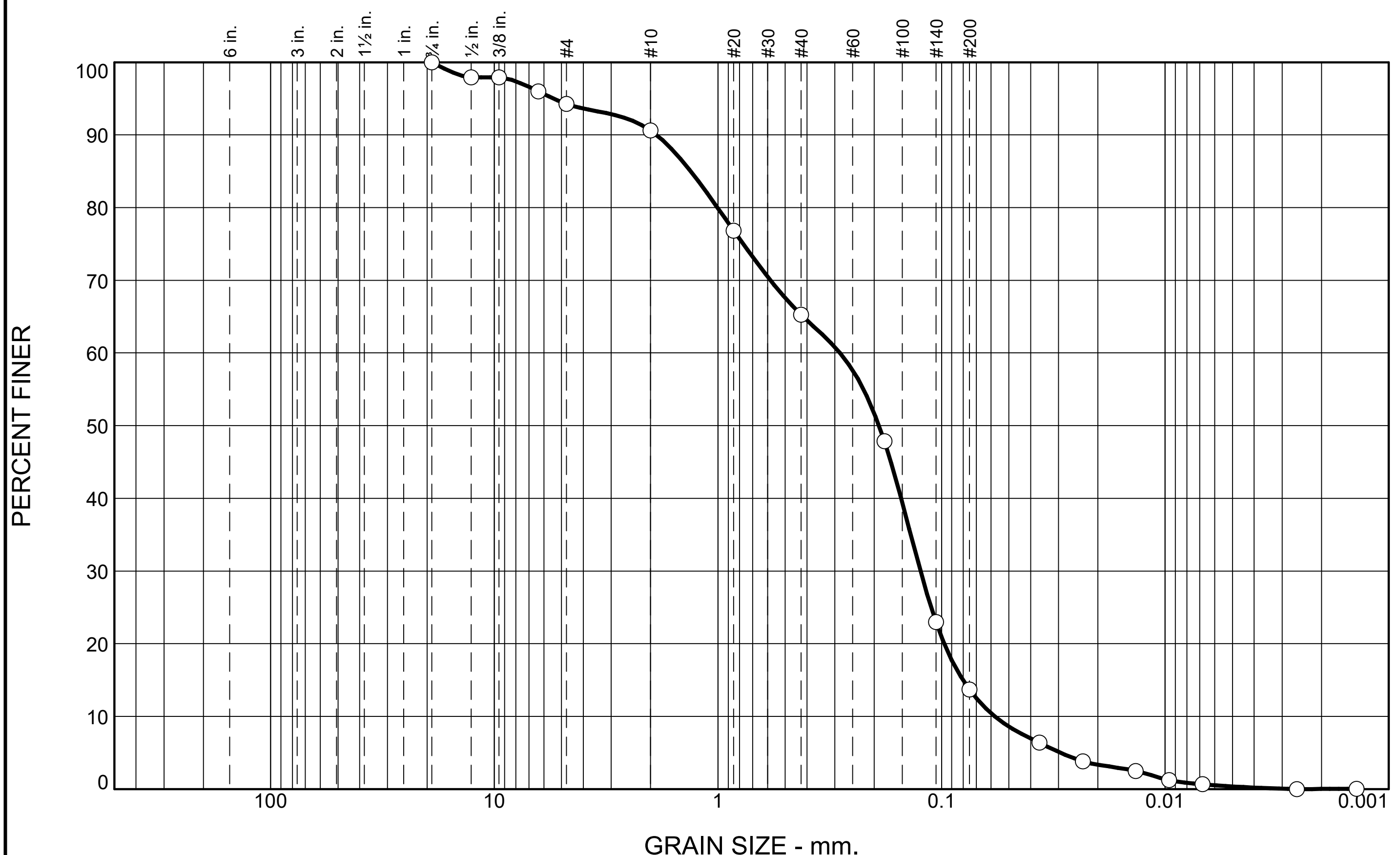
R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Bridgton Cornship Bridge #0205731-000
Bridgton, ME
Project No: 0956-013BG10 **Lab No.** 17399-03

Tested By: JMT/CAG **Checked By:** MTG

MTG

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.8	3.6	25.3	51.6	13.7	0.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	97.9		
3/8"	97.9		
1/4"	96.0		
#4	94.2		
#10	90.6		
#20	76.8		
#40	65.3		
#80	47.9		
#140	23.0		
#200	13.7		
0.0365 mm.	6.4		
0.0233 mm.	3.8		
0.0135 mm.	2.5		
0.0096 mm.	1.2		
0.0068 mm.	0.7		
0.0026 mm.	0.0		
0.0014 mm.	0.0		

* (no specification provided)

Soil Description

silty sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.8846 D_{85}= 1.3247 D_{60}= 0.2846
D_{50}= 0.1903 D_{30}= 0.1242 D_{15}= 0.0802
D_{10}= 0.0578 C_u= 4.92 C_c= 0.94

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Moisture Content: 48.3%

Location: GS-BSB-02
Sample Number: Grab **Depth:** 0-1' **Date:** 02/15/2023

R.W. Gillespie & Associates, Inc. Biddeford, Maine

Client: Haley & Aldrich, Inc.
Project: Bridgton Cornship Bridge #0205731-000
Bridgton, ME
Project No: 0956-013BG10 **Lab No.** 17399-04

Tested By: JMT/CAG **Checked By:** MTG

MTG

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project:	Cornshop Bridge
Location:	Bridgton, ME
Project No:	GTX-316595
Boring ID:	BB-BSB-101
Sample Type:	jar
Tested By:	cam
Sample ID:	10D
Test Date:	01/10/23
Checked By:	jsc
Depth :	25-27'
Test Id:	701517
Test Comment:	---
Visual Description:	Moist, gray silt
Sample Comment:	---

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	10D	B-BSB-10	25-27'	27	n/a	n/a	n/a	n/a	

Dry Strength: LOW

Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

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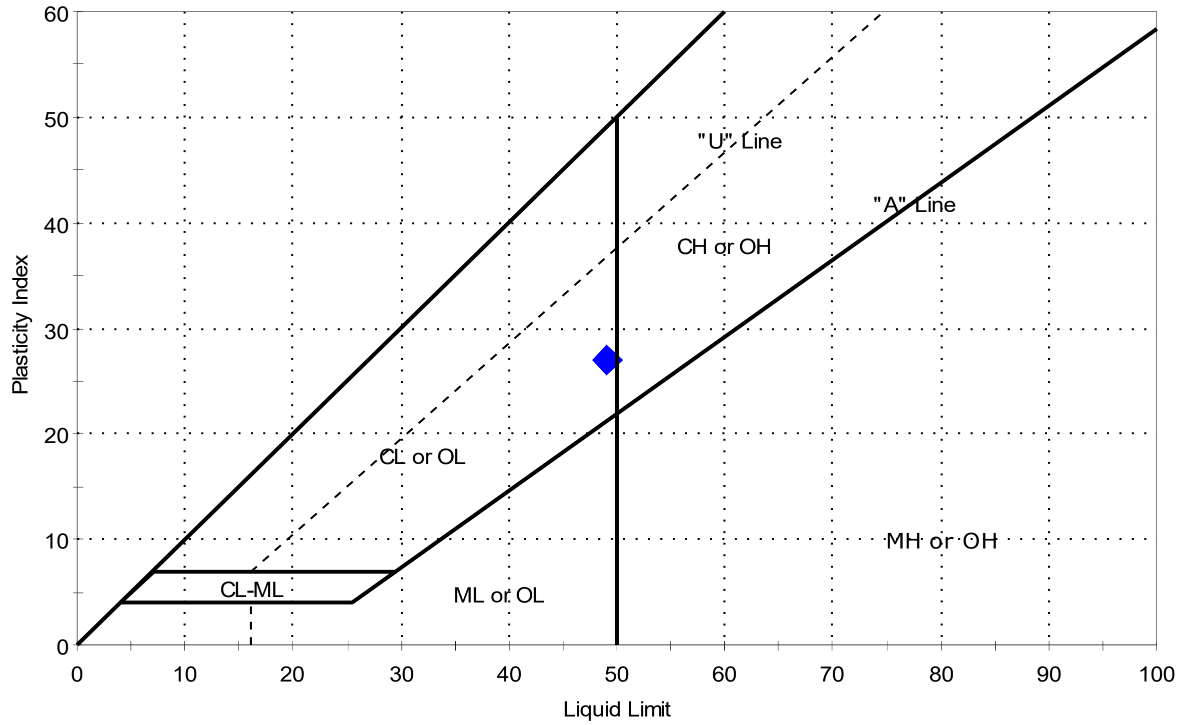
Client:	Haley & Aldrich, Inc.				
Project:	Cornshop Bridge				
Location:	Bridgton, ME			Project No:	GTX-316595
Boring ID:	BB-BSB-102	Sample Type:	jar	Tested By:	cam
Sample ID:	11D	Test Date:	01/12/23	Checked By:	jsc
Depth :	30-32'	Test Id:	701521		

Test Comment: ---

Visual Description: Moist, dark gray clay

Sample Comment: ---

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	11D	B-BSB-10	30-32'	30	49	22	27	0.3	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

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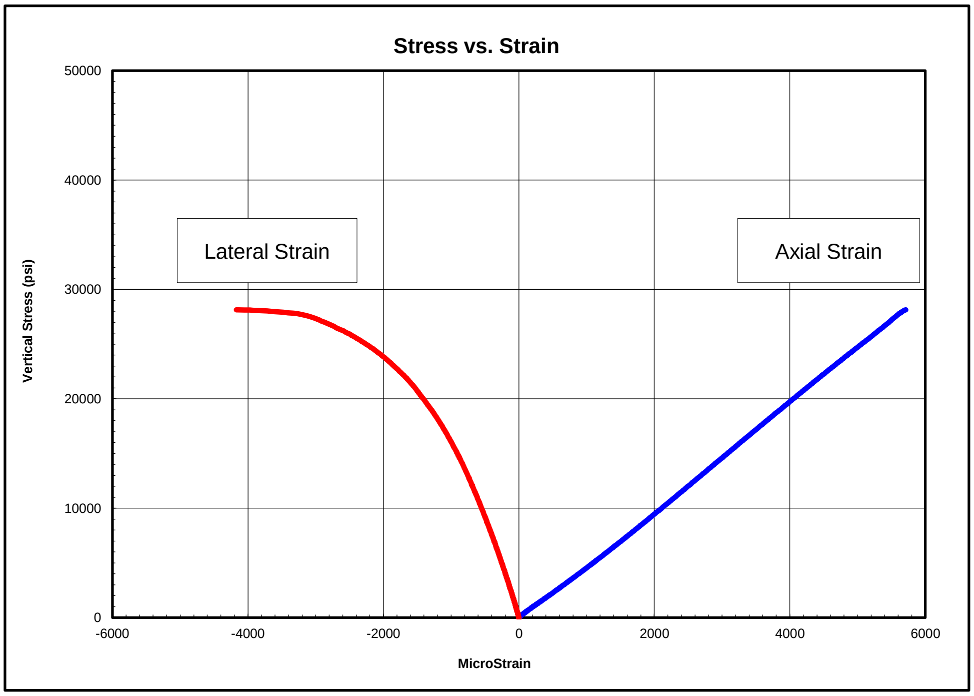
Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-BSB-101
Sample ID:	R1
Depth, ft:	46.04-46.41
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 28,135 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
2800-10300	4,850,000	0.28
10300-17800	5,160,000	0.40
17800-25300	5,010,000	---

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

Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Cornshop Bridge	Tested By:	bp/te
Project Location:	Bridgton, ME	Checked By:	smd
GTX #:	316595		
Boring ID:	BB-BSB-101		
Sample ID:	R1		
Depth:	46.04-46.41 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:	4.27	4.27	4.27
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	565.15		
Bulk Density, lb/ft^3	162	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

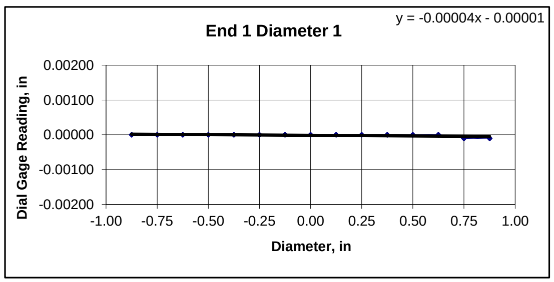
END FLATNESS AND PARALLELISM (Procedure FP1)

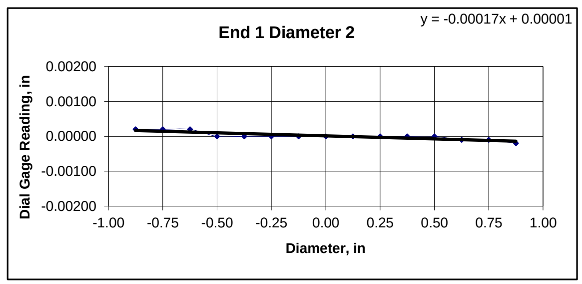
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00000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

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	-0.00020
Diameter 2, in (rotated 90°)	-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	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0002 90° = 0.0002
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020
Flatness Tolerance Met? **YES**





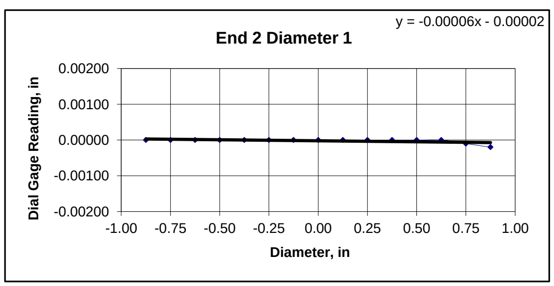
DIAMETER 1

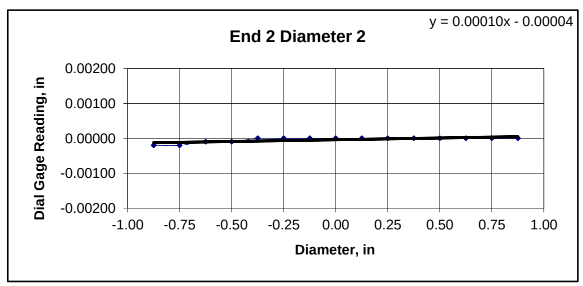
End 1:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00213

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

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00999

End 2:
Slope of Best Fit Line 0.00010
Angle of Best Fit Line: 0.00573

Maximum Angular Difference: 0.00426

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	1.990	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-BSB-101
Sample ID:	R1
Depth, ft:	46.04-46.41

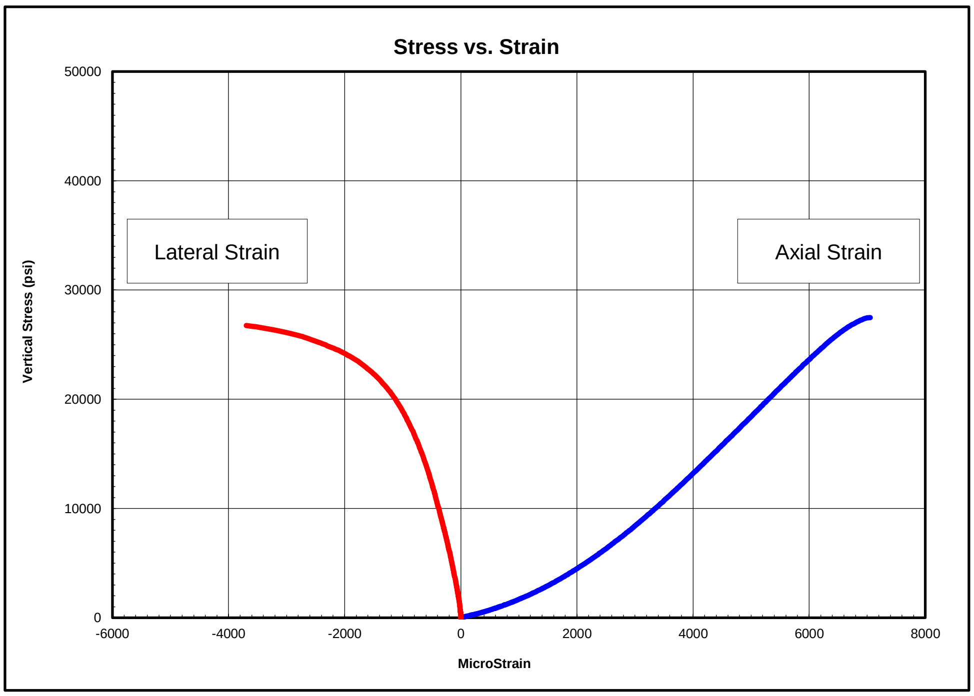
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-BSB-102
Sample ID:	R3
Depth, ft:	~~45.25-45.63~~ 50.2-50.8
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: 27,474 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
2700-10100	3,790,000	0.16
10100-17400	5,090,000	0.32
17400-24700	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.

GeoTesting EXPRESS

Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Cornshop Bridge	Tested By:	bp/te
Project Location:	Bridgton, ME	Checked By:	smd
GTX #:	316595		
Boring ID:	BB-BSB-102		
Sample ID:	R3		
Depth:	~~45.25-45.63 ft~~ 50.2-50.8 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:	4.34	4.34	4.34	
Specimen Diameter, in:	1.97	1.97	1.97	
Specimen Mass, g:	567.19			
Bulk Density, lb/ft^3	163	**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)

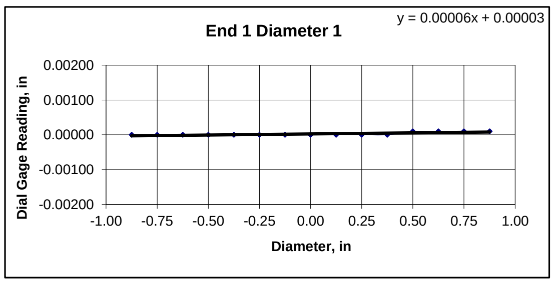
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000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

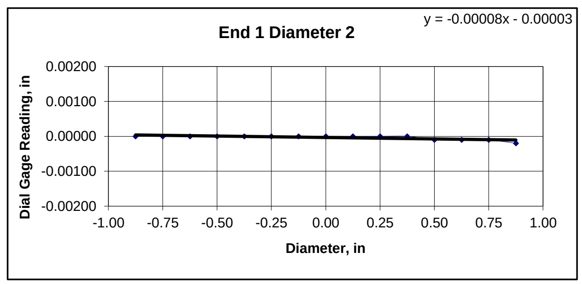
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00020

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

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

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00015
Flatness Tolerance Met? **YES**





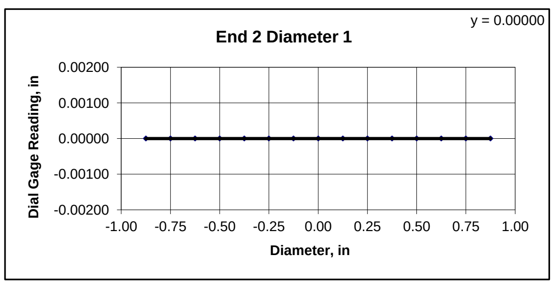
DIAMETER 1

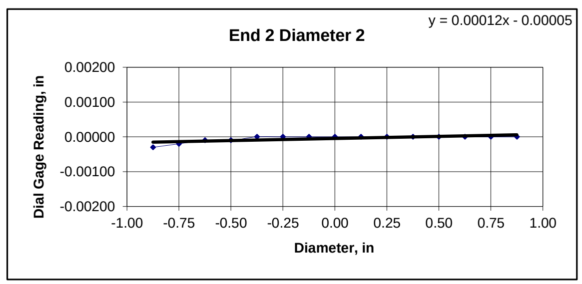
End 1:
Slope of Best Fit Line 0.00006
Angle of Best Fit Line: 0.00360

End 2:
Slope of Best Fit Line 0.00000
Angle of Best Fit Line: 0.00000

Maximum Angular Difference: 0.00360

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00008
Angle of Best Fit Line: 0.00475

End 2:
Slope of Best Fit Line 0.00012
Angle of Best Fit Line: 0.00688

Maximum Angular Difference: 0.00213

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	1.970	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00020	1.970	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00000	1.970	0.00000	0.000	YES
Diameter 2, in (rotated 90°)	0.00030	1.970	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? **YES**

GeoTesting EXPRESS

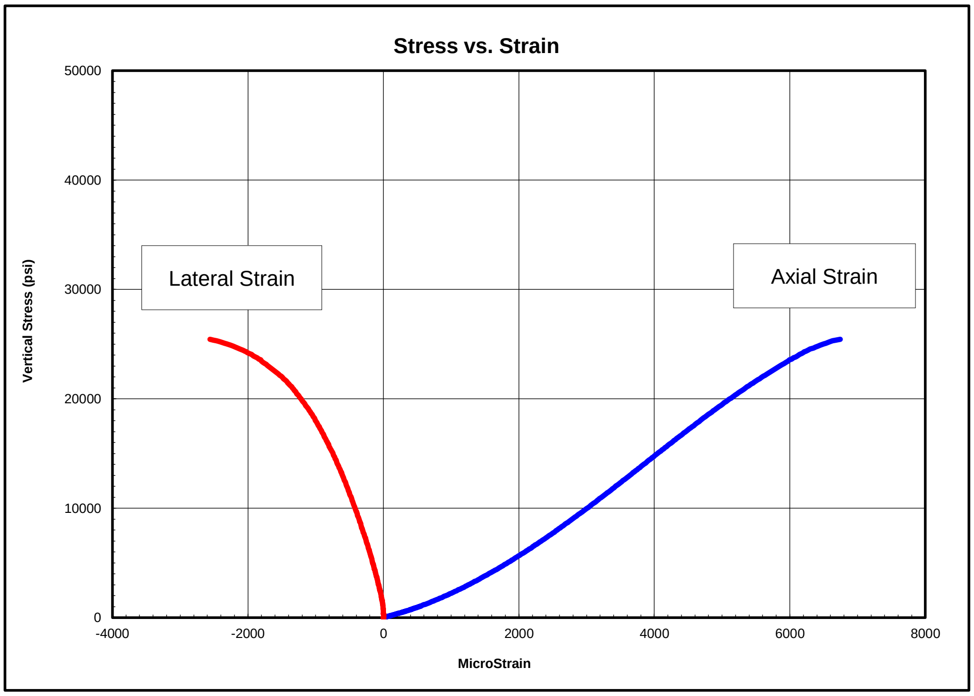
Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-BSB-102
Sample ID:	R3
Depth, ft:	~~45.25-45.63~~ 50.2-50.8 ft

After cutting and grinding

After break

Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-BSB-102
Sample ID:	R4
Depth, ft:	~~50.24-50.62~~ 54.6-55.3 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: 25,442 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
2500-9300	3,790,000	0.18
9300-16100	4,810,000	0.32
16100-22900	4,430,000	---

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

Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Cornshop Bridge	Tested By:	bp/te
Project Location:	Bridgton, ME	Checked By:	smd
GTX #:	316595		
Boring ID:	BB-BSB-102		
Sample ID:	R4		
Depth:	~~50.24-50.62 ft~~ 54.6-55.3 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:	4.30	4.30	4.30	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	567.62			
Bulk Density, lb/ft^3	163	**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	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010

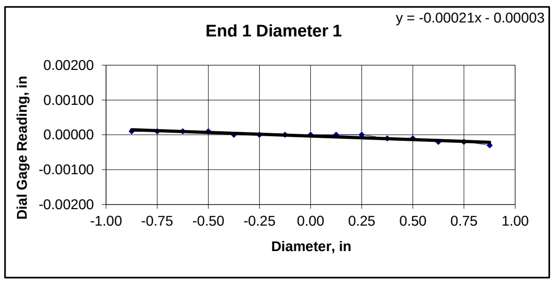
Difference between max and min readings, in:
0° = 0.00040 90° = 0.00030

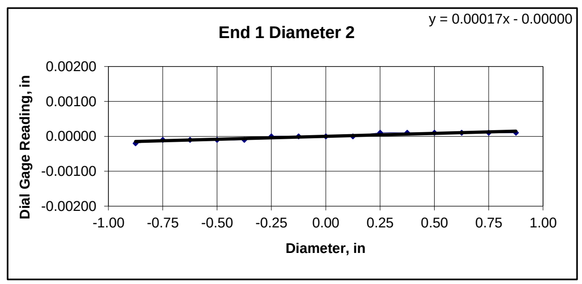
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00020	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010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

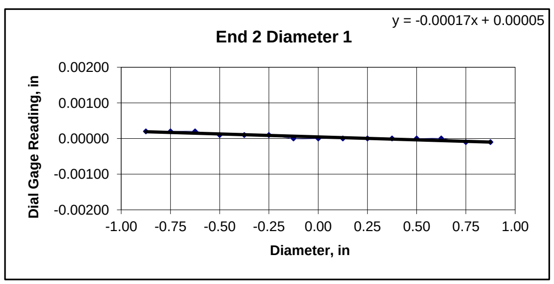
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0002

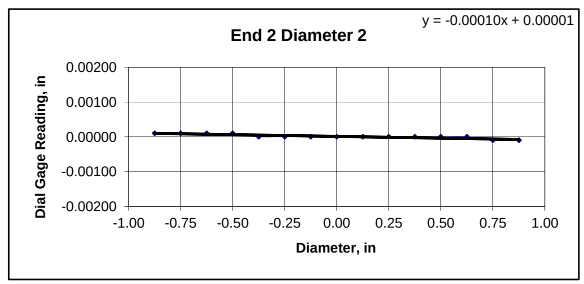
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00021
Angle of Best Fit Line: 0.01179

End 2:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00949

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00017
Angle of Best Fit Line: 0.00966

End 2:
Slope of Best Fit Line 0.00010
Angle of Best Fit Line: 0.00573

Maximum Angular Difference: 0.00393

Parallelism Tolerance Met? **YES**
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00040	1.980	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00030	1.980	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Haley & Aldrich, Inc.
Project Name:	Cornshop Bridge
Project Location:	Bridgton, ME
GTX #:	316595
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-BSB-102
Sample ID:	R4
Depth, ft:	~~50.24-50.62~~ 54.6-55.3 ft

After cutting and grinding

After break

APPENDIX C

Annotated Existing Bridge Drawings and Site Visit Measurements and Photographs

BOLT SCHEDULE

MARK	SIZE	NO	LENGTH	REMARKS
B1	3/4"φ	12	1'-2"	Carriage Bolts with cut washers. Posts to Curb
B2	3/4"φ	24	1'-0"	" " " " " 1 1/16" Diam. Holes
P1	1"φ	12	0'-7 1/2"	1"φ metal galvanized pipe Curbs
H1	5/8"φ	24		Standard Guard rail hook bolts.

STRAIGHT BARS

MARK	SIZE	NO	LENGTH	LOCATION
F3	5/8"φ	92	21'-9"	Slab (spliced)
C1	1/2"φ	8	22'-0"	Curbs (spliced)
C2	1/2"φ	8	4'-3"	Wings (Top)
B	3/4"φ	24	13'-0"	End Diaphragms (Spliced)
A1	3/4"φ	16	24'-6"	Bridge Seat Caps.
A2	1/2"φ	36	1'-9"	" " "
A3	3/4"φ	12	6'-3"	Wings (Bottom)

All dimensions to ℄ of bars.

TIMBER SCHEDULE

MARK	SIZE	NO	DRESS	FIN. SIZE	LENGTH	LOCATION
T1	6"x6"	6	D4S	5 1/2"x5 1/2"	12'-0"	Posts
T2	6"x6"	2	D4S	4 1/4"x5 1/2"	12'-0"	Filler blocks

PLAN A.B.P.
CHECKED G.W.S.
TOWN BRIDGE

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CORNSHOP BRIDGE
OVER
STEVENS BROOK
IN THE TOWN OF
BRIDGTON-CUMBERLAND CO.

SUPERSTRUCTURE DETAILS
SHEET 1 OF 2 AUGUSTA, ME. APRIL 1946

45-151

Photo 1: Facing south to existing southern abutment (exposed timber piles indicated by red arrows)

Photo 2: Facing west (upstream) along bottom of existing southern abutment

Photo 3: Facing east (downstream) along bottom of existing southern abutment (exposed timber piles indicated by red arrows)

Photo 4: Facing west (upstream) along bottom of existing southern abutment with exposed timber piles

Photo 5: Facing north to existing northern abutment (exposed timber piles indicated by red arrows)

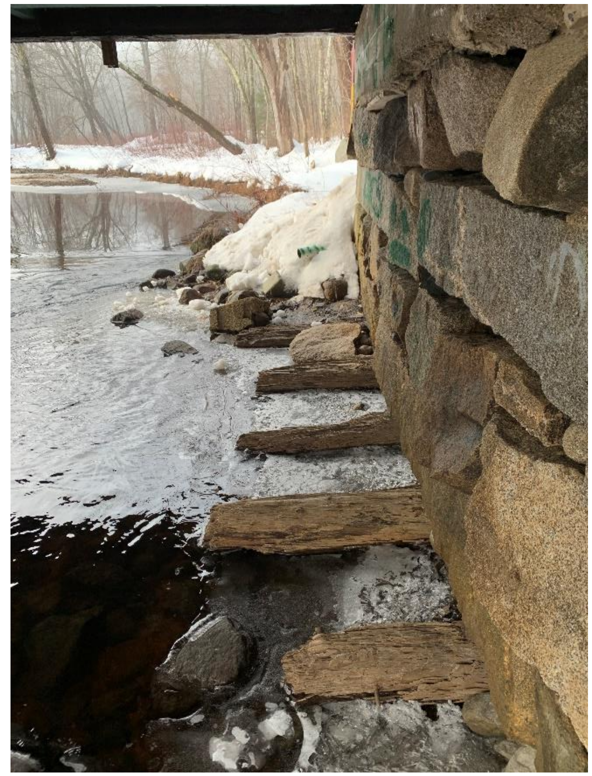


Photo 6: Facing west (upstream) along bottom of existing northern abutment with exposed timber piles

Photo 7: Facing east (downstream) along bottom of existing northern abutment with exposed timber piles

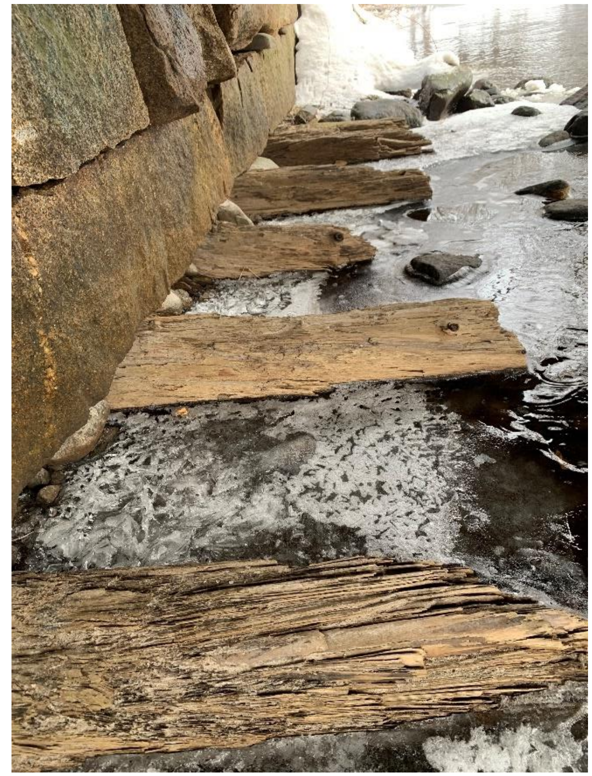


Photo 8: Facing east (downstream) along bottom of existing northern abutment with exposed timber piles

APPENDIX D

Geotechnical Calculations

PROBLEM STATEMENT & OBJECTIVE

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

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered in the borings, we recommend Seismic Site Classes C, C/D, and D.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.
2. Seed. H.B., Wong, R.T., Idriss, I.M. and Tokimatsu, K. (1984), "Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils," Report No. UCB/EERC-84/14, Earthquake Engineering Research Center, University of California, Berkeley, California.
3. Dickenson, SE (1994). Dynamic Response of Soft and Deep Cohesive Soils during the Loma Prieta Earthquake of October 17, 1989, PhD thesis, Dept. of Civil and Enviro. Eng., University of California, Berkeley, CA.
4. Sykora, DE, and KH Stokoe (1983). Correlations of in-situ measurements in sands of shear wave velocity, Soil Dyn. Earthq. Eng., 20:125–36.

AVAILABLE INFORMATION

1. Borings BB-BSB-101 and -102.
2. Elevations reference the NAVD 88 datum.

ASSUMPTIONS

1. Where shear wave velocity was not measured, correlations were used to determine shear wave velocity (V_s) from standard penetration test blow counts (SPT N) and undrained shear strength (S_u) measured from field vane tests.
2. Where V_s data was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The V_s for the extended profile was then assumed based on the available information.
3. Ground water elevations are from the time of drilling and assumed to be at the elevation of the first "wet" soil sample.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.10.3.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. If shear wave velocity was not measured, perform correlations to determine estimated shear wave velocity.
For cohesionless soils use Seed, et al. 1984

$$v_s = \sqrt{\frac{G_{max}}{\rho}}$$

$$G_{max} = 1000(K_2)_{max} * {\sigma'_m}^{0.5}$$

$$(K_2)_{max} = 20[(N_1)_{60}]^{0.33}$$

$$\sigma'_m = \left(\frac{1 + 2 * K_0}{3}\right) * \sigma'_v$$

PROCEDURE

For cohesive soils with vane shear data use Dickerson 1994

$$v_s(fps) = 18{S_u}^{0.475}$$

For cohesive soils without vane shear data use Sykora 1983

195 = Factor for cohesive soils
250 = Factor for cohesionless soils
N = uncorrected SPT N-vale
d = sample depth

$$v_s(fps) = Factor(N^{0.17})(d^{0.2})$$

3. Categorize the site using the average shear wave velocity in the upper 100 ft.

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.

4. Where measured shear wave velocity data is not available, categorize the site using each of the following factors applied to the average shear wave velocity.

$$\overline{V_s} * 1.0$$
$$\overline{V_s} * 1.3$$
$$\overline{V_s}/1.3$$

SITE CLASS DEFINITIONS

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile	$\overline{v}_s$ Calculated or Estimated
A	Hard rock	$\overline{v}_s > 5{,}000$ ft/s
B	Medium hard rock	3,000 ft/s < $\overline{v}_s \leq 5{,}000$ ft/s
B/C	Soft rock	2,100 ft/s < $\overline{v}_s \leq 3{,}000$ ft/s
C	Very dense sand or hard clay	1,450 ft/s < $\overline{v}_s \leq 2{,}100$ ft/s
C/D	Dense sand or very stiff clay	1,000 ft/s < $\overline{v}_s \leq 1{,}450$ ft/s
D	Medium dense sand or stiff clay	700 ft/s < $\overline{v}_s \leq 1{,}000$ ft/s
D/E	Loose sand or medium stiff clay	500 ft/s < $\overline{v}_s \leq 700$ ft/s
E	Very loose sand or soft clay	$\overline{v}_s \leq 500$ ft/s
F	Soils requiring site-specific ground response 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 Owner or those having jurisdiction determines that Site Classes E or F could be present at the site or in the event that Site Classes E or F are established by geotechnical data.

where:

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

Boring No.:	BB-BSB-101
Ground Surface El.:	403.4
Ground Water Depth (ft)	8.0

Hammer Efficiency Factor: 0.93

Sample Number	Depth (ft)	Elevation (ft)	Soil Description	d (ft)	Cohesive or Cohesionless	Unit Weight	σ_v psf	σ'_v psf	SPT N (blows/ft)	N_{60} (blows/ft)	C_N	$(N_1)_{60}$ (blows/ft)	S_u psf	$(K_2)_{max}$	V_s (Seed et al.) (ft/s)	V_s (Dickenson) (ft/s)	Sykora Factor	V_s (Sykora) (ft/s)	V_s (ft/s)	d/V_s
1D	1.3	402.2	Sand	2.0	Cohesionless	115	144	144	29	45	1.7	76	#N/A	85	469	#N/A	250	463	463	0.004
2D	3.0	400.4	Sand	2.0	Cohesionless	115	345	345	32	50	1.7	84	#N/A	88	594	#N/A	250	561	561	0.004
3D	5.0	398.4	Sand	2.0	Cohesionless	115	575	575	8	12	1.7	21	#N/A	55	536	#N/A	250	491	491	0.004
4D	7.0	396.4	Sand	2.0	Cohesionless	115	805	805	5	8	1.6	12	#N/A	46	534	#N/A	250	485	485	0.004
5D	9.0	394.4	Sand	2.0	Cohesionless	120	1,040	978	3	5	1.5	7	#N/A	38	496	#N/A	250	468	468	0.004
6D	11.0	392.4	Silt	1.5	Cohesive	100	1,260	1,073	#N/A	#REF!	1.4	#REF!	500	#N/A	#N/A	345	195	#REF!	345	0.004
7D	13.0	390.4	Sand	2.5	Cohesionless	120	1,480	1,168	6	9	1.3	12	#N/A	46	573	#N/A	250	566	566	0.004
8D	15.0	388.4	Sand	6.0	Cohesionless	120	1,720	1,283	5	8	1.3	10	#N/A	43	565	#N/A	250	565	565	0.011
9D	21.0	382.4	Sand	4.0	Cohesionless	120	2,440	1,629	5	8	1.1	9	#N/A	41	588	#N/A	250	604	588	0.007
10D	26.0	377.4	Silt	5.0	Cohesive	120	3,040	1,917	#N/A	#N/A	1.0	#N/A	250	#N/A	#N/A	248	195	#N/A	248	0.020
11D	31.0	372.4	Sand	6.8	Cohesionless	120	3,640	2,205	20	31	1.0	30	#N/A	62	779	#N/A	250	827	779	0.009
12D/13D	36.0	367.4	Clayey Silt	5.5	Cohesive	130	4,265	2,518	#N/A	#N/A	0.9	#N/A	3,000	#N/A	#N/A	807	195	#N/A	807	0.007
R1/R2	41.3	362.1	Bedrock	58.7	Bedrock	150	5,007	2,929	#N/A	#N/A	0.8	#N/A	#N/A	#N/A	#N/A	#N/A	250	#N/A	3,000	0.020
			Totals =	100.0															Totals =	0.1
																			vs-bar (ft/s) =	982.1

Factor	V_s-bar (ft/s)	Site Class
V_s-bar*1.3	1,277	C/D
V_s-bar*1.0	982	D
V_s-bar/1.3	755	D

Boring No.:	BB-BSB-102
Ground Surface El.:	404.1
Ground Water Depth (ft)	12.0

Hammer Efficiency Factor: 0.93

Sample Number	Depth (ft)	Elevation (ft)	Soil Description	d (ft)	Cohesive or Cohesionless	Unit Weight	σ_v psf	σ'_v psf	SPT N (blows/ft)	N_{60} (blows/ft)	C_N	$(N_1)_{60}$ (blows/ft)	S_u psf	$(K_2)_{max}$	V_s (Seed et al.) (ft/s)	V_s (Dickenson) (ft/s)	Sykora Factor	V_s (Sykora) (ft/s)	V_s (ft/s)	d/V_s
1D	1.3	402.9	Sand	2.0	Cohesionless	115	144	144	63	98	1.7	166	#N/A	110	534	#N/A	250	529	529	0.004
2D	3.0	401.1	Sand	2.0	Cohesionless	115	345	345	27	42	1.7	71	#N/A	83	577	#N/A	250	545	545	0.004
3D	5.0	399.1	Sand	2.0	Cohesionless	115	575	575	7	11	1.7	18	#N/A	53	524	#N/A	250	480	480	0.004
4D	7.0	397.1	Sand	2.0	Cohesionless	115	805	805	11	17	1.6	27	#N/A	60	609	#N/A	250	555	555	0.004
5D/6D	9.0	395.1	Sand	4.0	Cohesionless	115	1,035	1,035	7	11	1.4	15	#N/A	50	589	#N/A	250	540	540	0.007
7D	13.0	391.1	Sandy Silt	2.0	Cohesive	100	1,465	1,403	#N/A	#REF!	1.2	#REF!	500	#N/A	#N/A	345	195	#REF!	345	0.006
8D	15.0	389.1	Sand	6.0	Cohesionless	120	1,685	1,498	8	12	1.2	15	#N/A	49	627	#N/A	250	612	612	0.010
9D	21.0	383.1	Sand	5.0	Cohesionless	120	2,405	1,843	9	14	1.1	15	#N/A	49	662	#N/A	250	668	662	0.008
10D	26.0	378.1	Sand	4.0	Cohesionless	120	3,005	2,131	14	22	1.0	21	#N/A	56	730	#N/A	250	751	730	0.005
11D	31.0	373.1	Silty Clay	6.0	Cohesive	120	3,605	2,419	#N/A	#N/A	0.9	#N/A	2,000	#N/A	#N/A	666	195	#N/A	666	0.009
12D	36.0	368.1	Silty Clay	3.0	Cohesive	120	4,205	2,707	#N/A	#N/A	0.9	#N/A	4,000	#N/A	#N/A	925	195	#N/A	925	0.003
13D	40.4	363.75	Sand	2.7	Cohesionless	130	4,749	2,980	100	155	0.8	130	#N/A	101	1029	#N/A	250	1146	1,029	0.003
R1-R4	40.7	363.4	Bedrock	59.3	Bedrock	150	4,798	3,007	#N/A	#N/A	0.8	#N/A	#N/A	#N/A	#N/A	#N/A	250	#N/A	3,000	0.020
			Totals =	100.0															Totals =	0.1
																			vs-bar (ft/s) =	1163.9

Factor	V_s-bar (ft/s)	Site Class
V_s-bar*1.3	1,513	C
V_s-bar*1.0	1,164	C/D
V_s-bar/1.3	895	D

RESULTS SUMMARY

Boring No.	**Seismic Site Classes**
BB-BSB-101	**C/D, D**
BB-BSB-102	**C, C/D, D**

From USGS Hazard Tool for site location:

Period (s)	**Spectral Acceleration (g)**		
	Site Class C	Site Class C/D	Site Class D
0	0.131	0.132	0.130
0.01	0.145	0.145	0.144
0.02	0.213	0.207	0.208
0.03	0.256	0.242	0.242
0.05	0.317	0.294	0.292
0.075	0.337	0.313	0.307
0.1	0.331	0.320	0.311
0.15	0.292	0.292	0.288
0.2	0.227	0.221	0.257
0.25	0.188	0.253	0.232
0.3	0.161	0.221	0.207
0.4	0.128	0.197	0.175
0.5	0.107	0.164	0.156
0.75	0.075	0.141	0.116
1	0.055	0.101	0.090
1.5	0.035	0.076	0.055
2	0.025	0.047	0.037
3	0.014	0.034	0.020
4	0.009	0.019	0.013
5	0.007	0.012	0.009
7.5	0.004	0.009	0.005
10	0.002	0.005	0.003

PGA = Spectral acceleration at period = 0 sec. Therefore, use PGA = 0.132g (Site Class C/D) for liquefaction analysis.

CONCLUSIONS & RECOMMENDATIONS

Based on the subsurface conditions encountered at the borings, we recommend Seismic Site Classes C, C/D, and D.

PROBLEM STATEMENT & OBJECTIVE

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

EXECUTIVE SUMMARY

The minimum F.S. against liquefaction based on the available information is 1.8; therefore, site soils are not liquefaction susceptible.

REFERENCES

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

AVAILABLE INFORMATION

1. Boring logs BB-BSB-101 and BB-BSB-102.
2. The site-modified peak ground acceleration, A_s (F_{PGA}*PGA), is 0.132g, based on Seismic Site Class C/D.
3. Deaggregation plot of the site from the United States Geological Survey (USGS) Earthquake Hazards website.
4. Assumed groundwater for analysis at El. 395.2 (highest groundwater encountered in borings and Q1.1 elevation).
5. Geotechnical laboratory test results for sieve gradation by GeoTesting Express.

ASSUMPTIONS

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

Boring No.	Sample No.	Depth (ft)	% Fines
BB-BSB-101	4D	6-8	14.7
BB-BSB-101	5D	8-10	25.6
BB-BSB-101	7D	12-14	2.3
BB-BSB-101	9D	20-22	1.2
BB-BSB-101	11D	30-32	9.9
BB-BSB-102	3D	4-6	19.5
BB-BSB-102	8D	14-16	8.6
BB-BSB-102	9D	20-22	3.2

4. Total unit weights used in the analysis were estimated as follows: Fill = 115 pcf, Organic Deposit = 100 pcf, Alluvial/Glaciofluvial/Glaciolacustrine Deposits = 120 pcf, and Glacial Till = 130 pcf.
5. SPT N_{60} values were provided on the boring logs. The SPT N_{60} values were used in the analysis.
6. An SPT value of "1" was assigned to samples with WOR or WOH. An SPT value of "100" was assigned to samples that encountered refusal.
7. Groundwater is at El. 395.2.
8. Magnitude M = 5.58 based on deaggregation plot and corresponding MSF = 2.13 based on equation (24) in Youd & Idriss (2001).

PROCEDURE

1. Evaluate the Cyclic Stress Ratio:

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

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

2. Evaluate the Cyclic Resistance Ratio:

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

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

$(N_1)_{60} = C_N$*N60

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

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

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

α = 0 for FC ≤ 5%

α = exp[1.76-(190/FC^2)] for 5% < FC < 35%

α = 5.0 for FC ≥ 35%

β = 1.0 for FC ≤ 5%

β = [0.99 + (FC$^{1.5}$/1000)] for 5% < FC < 35%

β = 1.2 for FC ≥ 35%

3. Evaluate the Factor of Safety Against Liquefaction:

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

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

CALCULATIONS

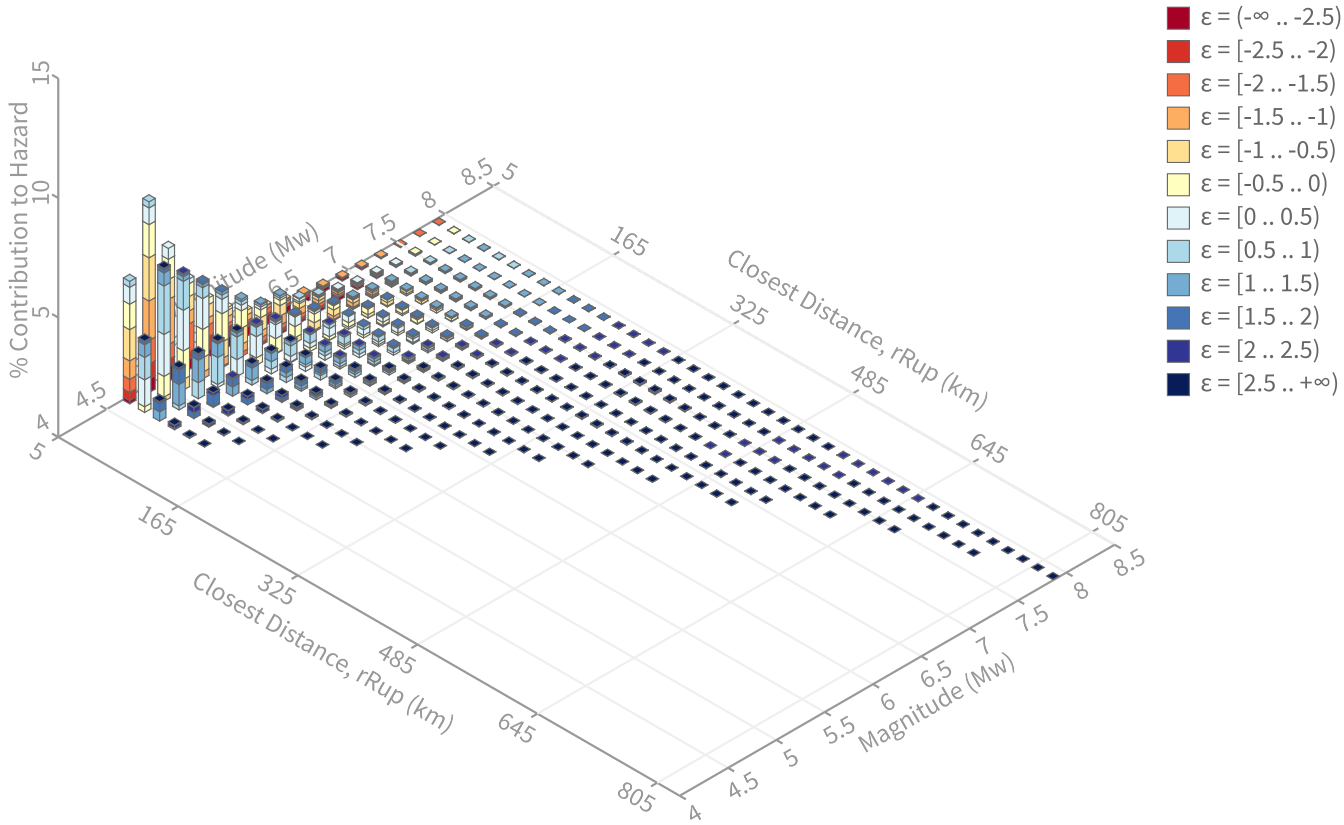
Boring No.	BB-BSB-101	
Estimated Ground Surface Elevation	403.4	ft, NAVD 88
Groundwater Elevation	395.2	ft, NAVD 89
Depth of Groundwater	8.2	ft
Unit Weight of Water	62.4	pcf
A_s (F_{PGA}*PGA)	0.132	g (Seismic Site Class C/D; greatest out of three site classes)
Magnitude (based on mean magnitude from deaggregation plot)	5.58	
Magnitude Scaling Factor	2.13	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N_{60}	γ_t (pcf)	σ_v (psf)	σ_w (psf)	σ'_v (psf)	C_N	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60\text{-}CS}$	$CRR_{7.5}$	r_d	CSR	**FS**
1D	1.3	402.2	FILL	SM	45	115	144	0	144	1.70	77	14.7	2.41	1.05	82	NL	1.00	0.086	**NL**
2D	3.0	400.4	FILL	SM	50	115	345	0	345	1.70	85	14.7	2.41	1.05	91	NL	0.99	0.085	**NL**
3D	5.0	398.4	FILL	SM	12	115	575	0	575	1.70	20	14.7	2.41	1.05	24	0.269	0.99	0.085	**6.8**
4D	7.0	396.4	FILL	SM	8	115	805	0	805	1.61	13	14.7	2.41	1.05	16	0.169	0.98	0.084	**4.3**
5D	9.0	394.4	ALLUVIAL DEPOSIT	SM	5	120	1,080	50	1,030	1.42	7	25.6	4.35	1.12	12	0.134	0.98	0.088	**3.2**
6D	11.0	392.4	ORGANIC DEPOSIT	OL	8	100	1,100	175	925	1.50	12	40.0	5.00	1.20	19	0.208	0.97	0.099	**4.5**
7D	13.0	390.4	GLACIOFLUVIAL DEPOSIT	SP	9	120	1,560	300	1,260	1.29	12	2.3	0.00	1.00	12	0.127	0.97	0.103	**2.6**
8D	15.0	388.4	GLACIOFLUVIAL DEPOSIT	SP	8	120	1,800	424	1,376	1.23	10	1.2	0.00	1.00	10	0.112	0.97	0.108	**2.2**
9D	21.0	382.4	GLACIOFLUVIAL DEPOSIT	SP	8	120	2,520	799	1,721	1.10	9	1.2	0.00	1.00	9	0.103	0.95	0.119	**1.8**
10D	26.0	377.4	GLACIOLACUSTRINE DEPOSIT	ML	1	120	3,120	1,111	2,009	1.02	1	95.0	5.00	1.20	6	0.081	0.94	0.125	**NL**
11D	31.0	372.4	GLACIOFLUVIAL DEPOSIT	SP-SM	31	120	3,720	1,423	2,297	0.95	30	9.9	0.84	1.02	31	NL	0.92	0.128	**NL**
12D	36.0	367.4	GLACIAL TILL	ML	36	130	4,680	1,735	2,945	0.84	30	85.0	5.00	1.20	41	NL	0.88	0.120	**NL**
13D	40.7	362.8	GLACIAL TILL	SM	110	130	5,285	2,025	3,260	0.80	88	20.0	3.61	1.08	99	NL	0.84	0.117	**NL**

Boring No.	BB-BSB-102	
Estimated Ground Surface Elevation	404.1	ft, NAVD 88
Groundwater Elevation	395.2	ft, NAVD 89
Depth of Groundwater	8.9	ft
Unit Weight of Water	62.4	pcf
A_s (F_{PGA}*PGA)	0.132	g (Seismic Site Class C/D; greatest out of three site classes)
Magnitude (based on mean magnitude from deaggregation plot)	5.58	
Magnitude Scaling Factor	2.13	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N_{60}	γ_t (pcf)	σ_v (psf)	σ_w (psf)	σ'_v (psf)	C_N	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60\text{-}CS}$	$CRR_{7.5}$	r_d	CSR	**FS**
1D	1.3	402.2	FILL	SM	98	115	144	0	144	1.70	167	19.5	3.53	1.08	183	NL	1.00	0.086	**NL**
2D	3.0	400.4	FILL	SM	42	115	345	0	345	1.70	71	19.5	3.53	1.08	80	NL	0.99	0.085	**NL**
3D	5.0	398.4	FILL	SM	11	115	575	0	575	1.70	19	19.5	3.53	1.08	24	0.267	0.99	0.085	**6.7**
4D	7.0	396.4	FILL	SM	17	115	805	0	805	1.61	27	19.5	3.53	1.08	33	NL	0.98	0.084	**NL**
5D	9.0	394.4	FILL	SM	11	115	1,035	50	985	1.46	16	19.5	3.53	1.08	21	0.225	0.98	0.088	**5.4**
6D	11.0	392.4	FILL	SM	9	115	1,265	175	1,090	1.38	12	20.0	3.61	1.08	17	0.181	0.97	0.097	**4.0**
7D	13.0	390.4	ALLUVIAL DEPOSIT	ML	8	120	1,560	300	1,260	1.29	10	70.0	5.00	1.20	17	0.185	0.97	0.103	**3.8**
8D	15.0	388.4	GLACIOFLUVIAL DEPOSIT	SP-SM	12	120	1,800	424	1,376	1.23	15	8.6	0.45	1.02	15	0.165	0.97	0.108	**3.2**
9D	21.0	382.4	GLACIOFLUVIAL DEPOSIT	SP	14	120	2,520	799	1,721	1.10	15	3.2	0.00	1.00	15	0.164	0.95	0.119	**2.9**
10D	26.0	377.4	GLACIOFLUVIAL DEPOSIT	SP	22	120	3,120	1,111	2,009	1.02	22	3.2	0.00	1.00	22	0.248	0.94	0.125	**4.2**
11D	31.0	372.4	GLACIOLACUSTRINE DEPOSIT	CL	17	120	3,720	1,423	2,297	0.95	16	70.0	5.00	1.20	24	0.281	0.92	0.128	**4.7**
12D	36.0	367.4	GLACIOLACUSTRINE DEPOSIT	CL	50	120	4,320	1,735	2,585	0.90	45	70.0	5.00	1.20	59	NL	0.88	0.126	**NL**
13D	40.4	363.1	GLACIAL TILL	SP	100	130	5,246	2,006	3,239	0.80	80	5.0	0.00	1.00	80	NL	0.85	0.117	**NL**

NL = Not Liquefiable


ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)
% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)
165
325
485
645
805
4
4.5
5
5.5
6
6.5
7
7.5
8
8.5
10
15

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 1000 yrs
Exceedance rate: 0.001 yr^{-1}
PGA ground motion: 0.093232027 g

Recovered targets

Return period: 999.71819 yrs
Exceedance rate: 0.0010002819 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 1.67 %

Mean (over all sources)

m: 5.58 MOMENT MAGNITUDE FOR LIQUEFACTION ANALYSES
r: 49.38 km
ε_0: -0.13 σ

Mode (largest m-r bin)

m: 4.9
r: 13.66 km
ε_0: -1 σ
Contribution: 7.98 %

Mode (largest m-r-ε_0 bin)

m: 4.89
r: 32.54 km
ε_0: 0.71 σ
Contribution: 2.02 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

PROBLEM STATEMENT AND OBJECTIVE

Calculate the structural axial pile resistances for Abutments 1 and 2.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.

ASSUMPTIONS

1. One size of steel H-pile (HP14x89) was considered for the structural evaluations.
2. A pile yield stress of 50 ksi was considered for the structural resistance evaluations.
3. The pile axial structural resistance was evaluated assuming a 1/16 in. of corrosion.
4. The pile unbraced length is 0 ft (fully embedded pile) for the structural axial resistance calculations.
5. Piles will be driven to the top of bedrock. The structural resistance of the pile is assumed to govern the design based on AASHTO LRFD Section 10.7.3.2.
6. A resistance factor of 0.5 for the pile axial compression at the Strength Limit State was selected based on AASHTO LRFD Section 6.5.4.2 (H-Piles subjected to severe driving conditions where use of a pile tip is necessary).
7. A resistance factor of 1.0 for the pile compression axial resistance at the Extreme Event Limit State was selected based on AASHTO LRFD Section 10.5.5.3.2.

STRUCTURAL RESISTANCE - AASHTO LRFD

Note: This calculation only addresses axial resistance, if the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others. Refer to Section 6.15.

10.7.3.2 - Point Bearing Piles on Rock
10.7.3.2.3 Piles Driven to Hard Rock

The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal resistance shall not exceed the values obtained from Section 6.9.4.1 with the resustance factors specified in Section 6.5.4.2 and 6.15 for severe driving conditions.

10.7.3.13 - Pile Structural Resistance
10.7.3.12.1 - Steel Piles
The nomial axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for non composite piles. The resistance factors for the compression limit state are specified in Article 6.5.4.2.

6.9.4 - Noncomposite Members
6.9.4.1 - Nominal Compressive Resistance

If $\frac{P_0}{P_e} \leq 2.25$, then $P_n = P_0 * (0.658^{\frac{P_0}{P_e}})$ 6.9.4.1.1-1

Otherise: $P_n = 0.877 P_e$ 6.9.4.1.1-2

where
A_g = gross cross-sectional area of the member
F_y = specified minimum yield strength
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional buckling or flexural-torisonal buckling as applicable
P_0 = nominal yield resistance = $F_y A_g$

6.9.4.1.2 - Elastic Flexural Buckling Resistance

The elastic critical buckling resistance, P_e, based on flexural buckling shall be taken as

$$P_e = \frac{\pi^2 E}{(\frac{Kl}{r_s})^2} A_g$$ 6.9.4.1.2-1

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

STRUCTURAL AXIAL RESISTANCE CALCULATIONS

Pile Section	Gross Section Area A_g (in^2)	Corrosion Allowance (in)	Effective Gross Area A_{eff} (in^2)	Effective Length Factor K	Unbraced Length l (ft)	Yield Stress fy (ksi)	Elastic Modulus E (ksi)	Radius of Gyration r_{weak} (in)	P_0 (kips)	P_e (kips)	P_0/P_e	P_n (kips)	ϕ (Strength Limit State)	$ϕP_n$ (kips)
HP14x89	26.1	0.0625	20.5	2	0	50	29,000	3.51	1026	1.3E+15	8.2E-13	1026	0.5	513

Note: The resistance factor of 0.5 for the Strength Limit State was selected based on Section 6.5.4.2 for H-piles subjected to severe driving conditions where the use of a pile tip is necessary.

SUMMARY

Steel H-pile Section	Pile Nominal Structural Resistance in Axial Compression (kips)	Factored Structural Resistance (kips)		
		Service Limit State (ϕ=1.0)	Strength Limit State (ϕ=0.5)	Extreme Limit State (ϕ=1.0)
HP14x89	1,026	1,026	513	1,026

Note: Piles are driven to hard rock. Therefore piles are governed by the structural resistance based on Section 10.7.3.2.3.
The structural resistance was calculated using Section 6.9.4.1 as referenced from Section 10.7.3.13.
Note: The calculated resistance presented above only addresses axial resistance.
If the piles are subjected to lateral loads, the structural resistance under combined axial load and flexure should also be evaluated by others.

	CALCULATIONS	File No.:	0205731-000
		Sheet:	1 of 1
Client:	Stantec	Date:	10/2/2025
Project:	Cornship Bridge No. 0318, Bridgton, Maine	Computed by:	EMH
Subject:	GRL WEAP Pile Driving Analyses	Checked by:	NAS

PROBLEM STATEMENT & OBJECTIVE

Perform impact hammer drivability analyses for HP14x89 steel H-piles.

EXECUTIVE SUMMARY

HP14x89 steel H-piles can be installed to a nominal resistance equal to 750 kips (factored resist. = 488 kips) with a Delmag D36x32 OED hammer operating at 60% pressure at a penetration resistance equal to 15 blows per inch (bpi) and with a maximum compressive stress during driving equal to 42 ksi.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2024.
2. MaineDOT standard specifications.
3. AISC Steel design manual.
4. GRL WEAP 2014 software manual.

AVAILABLE INFORMATION

1. Borings logs: BB-BSB-101 (Abutment 1) and BB-BSB-102 (Abutment 2)
2. Design drawings prepared and provided by Stantec.

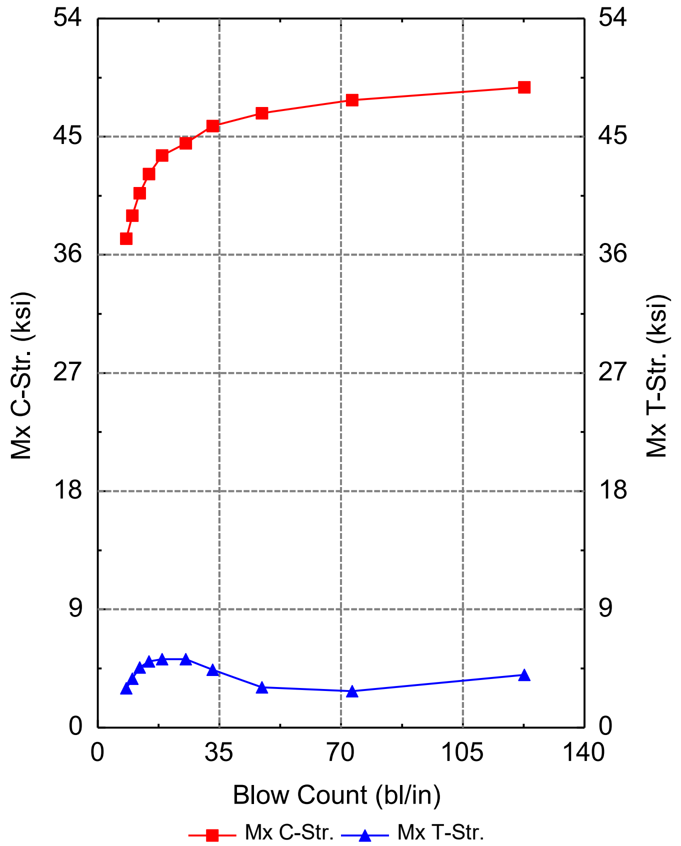
ASSUMPTIONS

1. Elevation Units and Datum: feet, North American Vertical Datum of 1988 (NAVD88).
2. Soil conditions at each substructure are based on the borings indicated above.
3. Piles will be driven approximately 3 ft into rock. One pile section will be analyzed (HP14x89).
4. Top of piles are at approximately El. 397, based on draft PS&E plans.
5. Top of bedrock at each substructure location is based on the nearest boring.
6. Groundwater assumed based on conditions encountered in nearest boring.
7. Pile embedment is based on est. depth from bottom of proposed abutment (El. 395) to 3 ft below top of rock at each location. Pile length is based on pile embedment plus 5 ft. Piles are assumed to be plumb (vertical).
8. Factored resistance is taken as the nominal resistance multiplied by 0.65 (res. factor for CAPWAP dynamic testing).
9. Shaft and toe quake and damping values used are the suggested values in WEAP.
10. Acceptable penetration resistance according to MEDOT standard specifications is 3 to 15 bl/in.
11. Limit pile compressive stress is assumed to be 0.9fy or 45 ksi for fy=50 ksi steel.

CALCULATIONS AND RESULTS

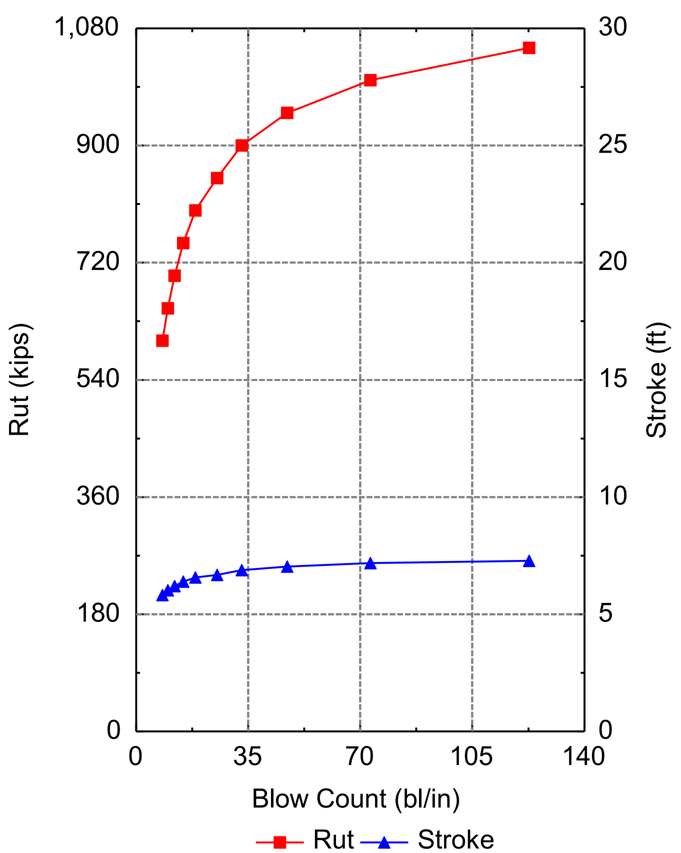
Substructure	Boring	Hammer Pressure Setting	Pile Type	Nominal Resistance (kip)	Factored Resistance (kip)	Max. Comp. Stress (ksi)	Blow Count (blows/in.)
Abutment 1 (North)	BB-BSB-101	60%	HP14x89	750	488	42.42	14.6
Abutment 2 (South)	BB-BSB-102	60%	HP14x89	750	488	42.15	14.7

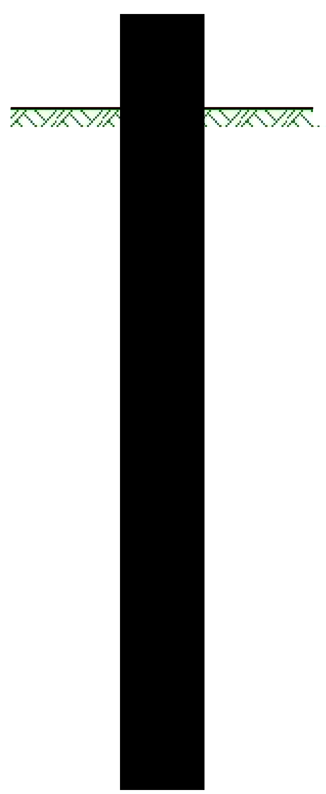
Recommended factored resistance: 488 kips for both abutments



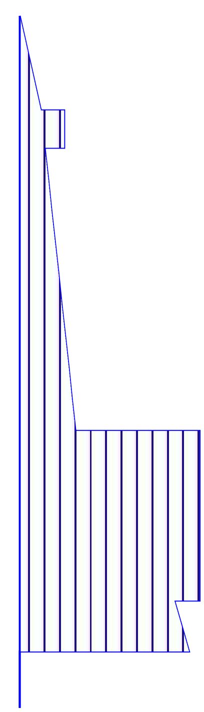
DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	900.0 (60%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.149	s/ft
Pile Length	41.700	ft
Pile Penetration	36.700	ft
Pile Top Area	26.100	in²
RSA	No	





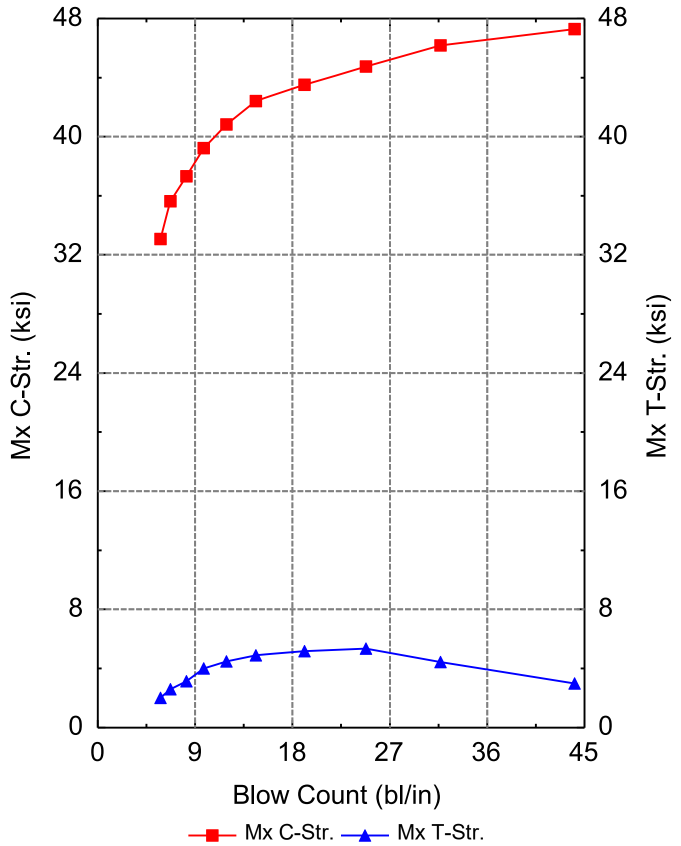
Pile Model



Shaft=10%
(Prop., ST)

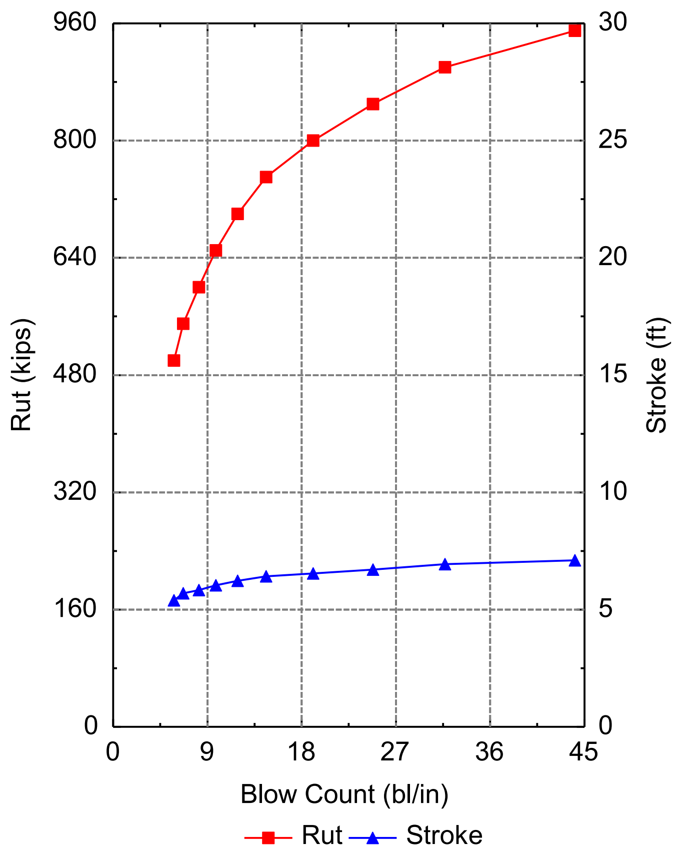
Bearing Graph Summary — DELMAG D 36-32

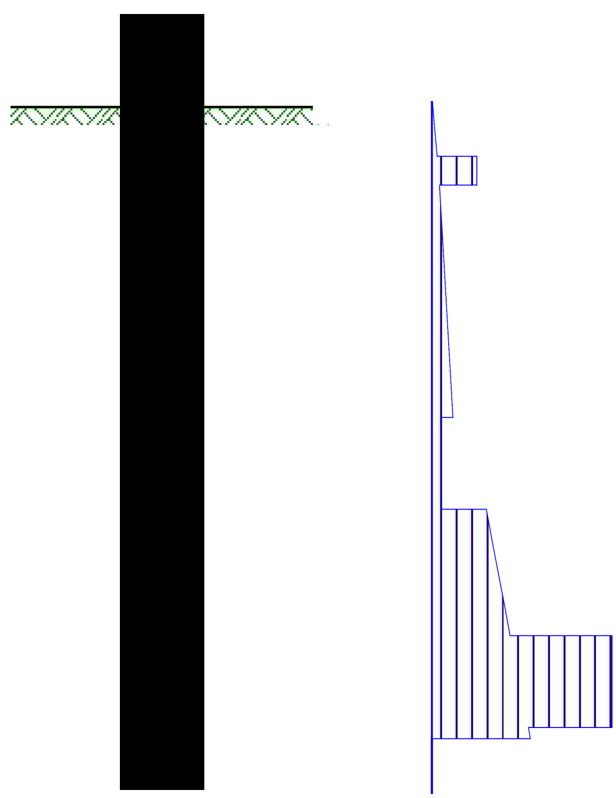
Rut	Mx C-Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer
kips	ksi	ksi	bl/in	ft	kip-ft	-
600.0	37.22	2.98	8.2	5.82	20.12	D 36-32
650.0	39.00	3.73	9.9	6.02	20.91	D 36-32
700.0	40.69	4.56	12.0	6.21	21.66	D 36-32
750.0	42.15	5.03	14.7	6.40	22.38	D 36-32
800.0	43.57	5.22	18.5	6.58	23.08	D 36-32
850.0	44.50	5.22	25.3	6.68	23.35	D 36-32
900.0	45.81	4.40	33.0	6.88	24.18	D 36-32
950.0	46.80	3.06	47.2	7.04	24.75	D 36-32
1000.0	47.79	2.77	73.1	7.18	25.25	D 36-32
1050.0	48.76	4.01	122.7	7.28	25.70	D 36-32



DELMAG D 36-32

Ram Weight	7.93	kips
Efficiency	0.800	
Pressure	900.0 (60%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.149	s/ft
Pile Length	42.300	ft
Pile Penetration	37.300	ft
Pile Top Area	26.100	in²
RSA	No	





Pile Model

Shaft=10%
(Prop., ST)

Bearing Graph Summary — DELMAG D 36-32

Rut	Mx C-Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Hammer
kips	ksi	ksi	bl/in	ft	kip-ft	-
500.0	33.07	2.01	5.8	5.40	18.50	D 36-32
550.0	35.62	2.59	6.7	5.69	19.71	D 36-32
600.0	37.33	3.14	8.2	5.84	20.22	D 36-32
650.0	39.21	4.01	9.8	6.04	21.03	D 36-32
700.0	40.83	4.49	11.9	6.24	21.77	D 36-32
750.0	42.42	4.90	14.6	6.42	22.50	D 36-32
800.0	43.52	5.17	19.1	6.54	22.86	D 36-32
850.0	44.74	5.34	24.8	6.71	23.51	D 36-32
900.0	46.18	4.43	31.7	6.94	24.43	D 36-32
950.0	47.29	2.99	44.1	7.10	25.06	D 36-32

CALCULATIONS		File No.:	0205731-000
		Sheet:	1 of 1
Client:	Stantec	Date:	10/9/2025
Project:	Cornshop Bridge No. 0318 Over Stevens Brook - MaineDOT WIN 026236.00	Computed by:	NAS
Subject:	Lateral Passive Earth Pressure Coefficient	Checked by:	EAF

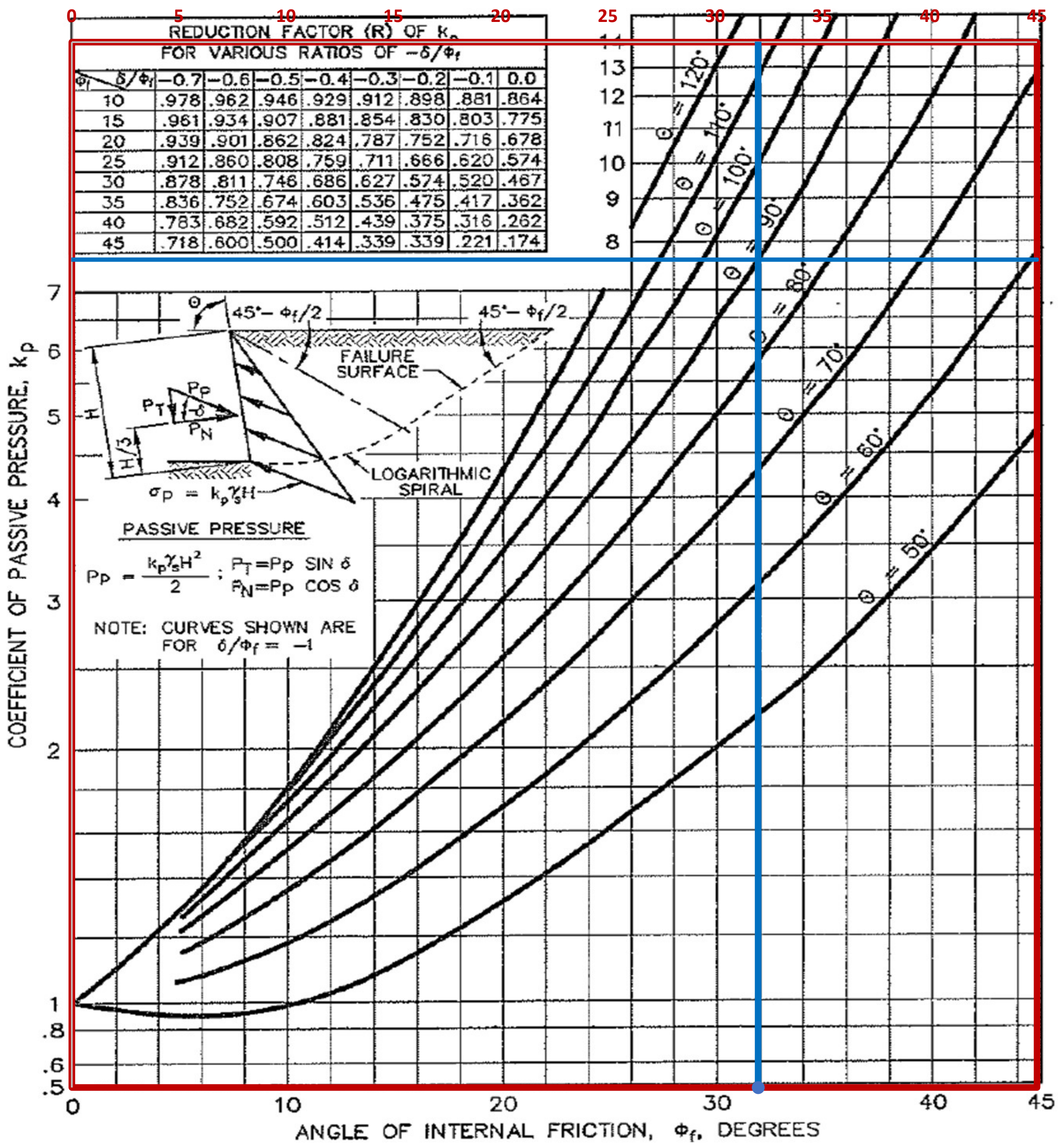
LATERAL PASSIVE LATERAL EARTH PRESSURE COEFFICIENT

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2024 with interim revisions. See plot below.

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

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



ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

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