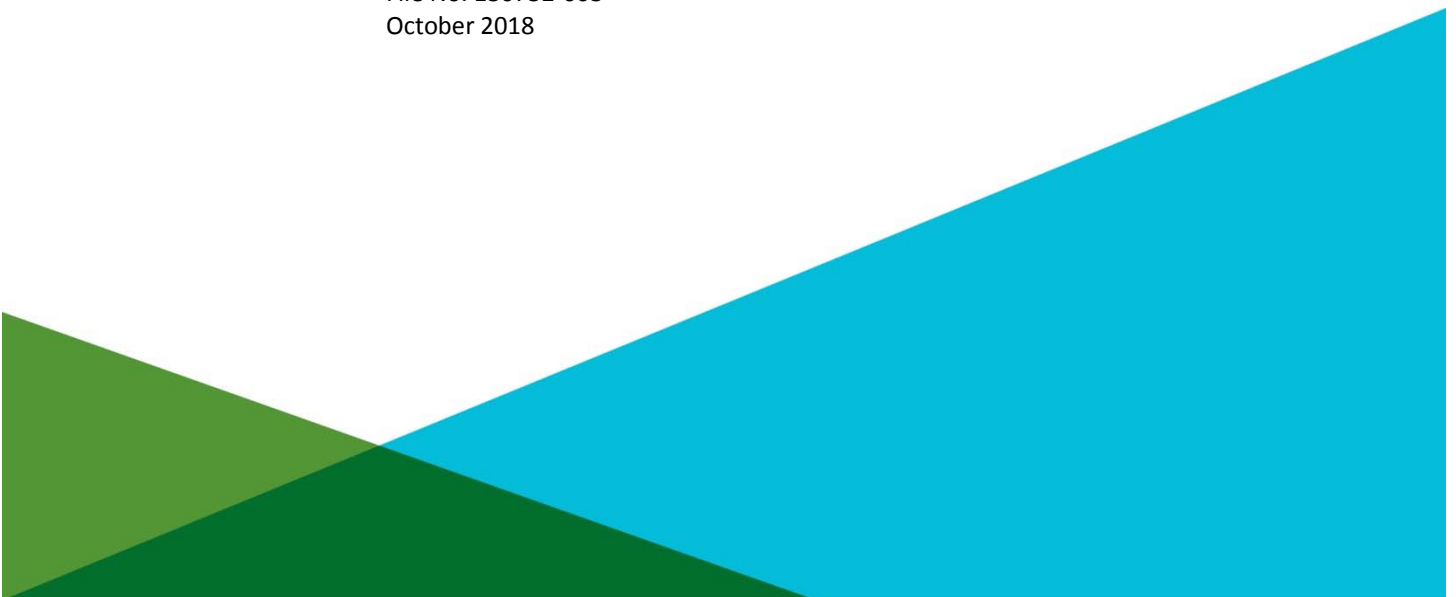


GEOTECHNICAL DESIGN REPORT
GRIST MILL BRIDGE NO. 2334 OVER SOUDABSCOOK STREAM
MAINEDOT WIN 021692.00
HAMPDEN, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Maine Department of Transportation
Augusta, Maine

File No. 130732-003
October 2018





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

Subject: Geotechnical Design Report
Grist Mill Bridge No. 2334 over Souadabscook Stream
MaineDOT WIN 021692.00
Hampden, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Geotechnical Design Report, Grist Mill Bridge No. 2334 over Soudabscook Stream, MaineDOT WIN 021692.00, Hampden, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our mutually agreed upon work scope and in accordance with the provisions of our General Consultant Agreement (GCA) with MaineDOT, No. 20150706000000000010, and with our project-specific assignment letters dated 20 June 2017, 22 February 2018, and 18 July 2018.

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 Please note that the information summarized herein is superseded by the information contained in the Contract Documents (CDs; plans, standard specifications, and special provisions) and that the information contained in the CDs takes precedence over the information provided in this GDR.

Introduction

This GDR presents the results of preliminary and supplemental phase subsurface investigation and laboratory testing programs, engineering evaluations, and geotechnical design recommendations conducted and prepared by Haley & Aldrich, Inc. (Haley & Aldrich) on behalf of the Maine Department of Transportation (MaineDOT) for the proposed replacement of the Grist Mill Bridge in Hampden, Maine (see Figure 1, Project Locus). We coordinated our technical evaluations with T.Y. Lin International (TY Lin; bridge and highway designer) through the design phase of the project in support of the development and submission of the 100 percent plans, specifications and estimate (PSE) package to MaineDOT.

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM AND BASELINE STATIONING

Plan locations of test borings are reported in Table I as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 Central Zone. Elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88). Several baselines were developed by TY Lin for the multiple alignments as summarized below.

- Route 1A/9: Sta. 30+00
- Dudley Road: Sta. 130+00
- Cold Brook Road: Sta. 140+00

Please note that all stationing referenced herein is referenced to the Route 1A/9 baseline.

PROJECT LOCATION AND EXISTING BRIDGE STRUCTURE

The existing Grist Mill Bridge (bridge) carries Route 1A/Route 9 traffic over the Souadabscook Stream (stream) in Hampden, Maine.

Based on our review of the Preliminary Draft Report (PDR) prepared by TY Lin and historic bridge drawings provided by MaineDOT, we understand that the existing bridge was originally constructed in 1924 and was widened on the upstream (northern) side of the structure in 1948. The existing bridge consists of an approximate 51-ft long, 30-ft wide (curb-to-curb), single-span bridge supported on two abutments. The original abutments (constructed in 1924) consist of stacked granite masonry block walls that were concrete-jacketed as part of the 1948 widening project. Wingwalls are either cast-in-place (CIP) concrete or stacked granite blocks. The historic bridge drawings suggest that the existing abutments and wingwalls bear on bedrock. Historic bridge drawings are provided for reference in Appendix C.

PROPOSED BRIDGE STRUCTURE

It is our understanding that during the preliminary design phase, TY Lin evaluated multiple bridge rehabilitation and replacement alternatives. We also understand that Alternative 5, which consists of a 95-ft long, semi-integral, single-span structure was the preferred Alternative and was recommended in the PDR to proceed into final design. The proposed bridge structure will be approximately 40-ft wide and will consist of two, 11-ft wide travel lanes, two, 5-ft wide shoulders and a 5-ft wide sidewalk on the north side of the bridge. The bridge superstructure will be supported by full-height, cantilevered, CIP concrete abutments bearing on bedrock with flared wingwalls. The abutments are located at the stations summarized below.

Substructure	Station at Centerline of Alignment (ft)
Abutment No. 1	33+64.98
Abutment No. 2	34+60.00

Minor modifications to the horizontal and vertical alignments of the bridge approaches will be required (i.e., on-alignment replacement of the existing bridge). It is anticipated that bridge replacement will require a complete bridge closure with two-way traffic maintained on an off-site detour along Route 9, Route 202, and Interstate I-395.

TERRAIN AND STREAM HYDRAULICS

The existing ground surface along the approach roadways consists of a sag curve with the low point approximately 175 ft west of proposed Abutment No. 1 (Sta. 31+90). Based on the progress set drawings, we understand that the proposed approach grades will generally mirror those of the existing grades with only minor modifications (i.e., raises in grade of approximately 6 in. or less).

TY Lin completed hydrology and hydraulic (H&H) evaluations for the project. Headwater elevations in the stream for various discharges, as determined by TY Lin, are summarized below.

Discharge	Headwater (ft, NAVD 88)
Q _{1.1}	El. 9.4
Q ₅₀	El. 14.7
Q ₁₀₀	El. 15.4

As part of their H&H evaluations, TY Lin indicated that a scour analysis may be “deemed necessary given the abutment type and bedrock elevations”. Since the streambed beneath the bridge consists of bedrock and the proposed bridge will be supported by spread footings bearing directly on bedrock or on seal concrete bearing on bedrock, it is our opinion that scour is unlikely to occur.

Geologic Setting

According to the Hampden Quadrangle Map, Open-File No. 13-12 2013, published by the Maine Geological Survey, surficial soil units present and in the vicinity of the bridge consist of glaciomarine silt, clay, and sand of the Presumpscot Formation, which were deposited on the late-glacial sea floor. Surficial soil deposits encountered in the preliminary and supplemental phase test borings completed at the site, as discussed in subsequent sections of this report, generally consisted of fill overlying marine silt/clay, glacial till and bedrock.

According to the Bedrock Geology of the Hampden Quadrangle, Maine Map, Open-File No. 16-10 2016, the bedrock at the site is mapped as Penobscot River Member of the Vassalboro Group, which dates back to the Silurian Age. These types of bedrock are described as medium gray to dark gray, medium-grained to very fine-grained, feldspathic META-WACKE with subordinate amounts of dark gray slate. Bedrock sampled and recovered in the preliminary phase test borings drilled at the site, as discussed in subsequent sections of this report, is generally described as GRANOFELS or SCHIST.

Subsurface Explorations

PRELIMINARY PHASE (JULY 2009)

Haley & Aldrich conducted a preliminary phase subsurface investigation at the site in July 2009 to provide general subsurface information along the existing bridge alignment. Two test borings, designated BB-HSS-101 and BB-HSS-102, were drilled behind each of the existing abutments and on opposite corners of the bridge. The as-drilled location of each test boring was determined in the field by Haley & Aldrich by taping distances from existing site features. The approximate, as-drilled locations of the preliminary phase test borings are shown on Figure 2, Site and Subsurface Exploration Location Plan.

Test borings were drilled by MaineDOT and monitored by a Haley & Aldrich geologist on 22 and 23 July 2009 using a truck-mounted, CME 45C drill rig. Test borings were drilled using “cased-wash” methods to depths ranging from approximately 36 to 38 ft below ground surface (BGS) using 3.0-inch (in.) (NW-size) and 4.0-in. (HW-size) inside diameter (ID) steel casing. Soil samples were collected at standard 5-ft intervals by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., using a calibrated automatic hammer.

The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval (blows per foot, bpf). The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.84 (i.e., 84 percent theoretical hammer efficiency).

Test borings were advanced approximately 10 to 11 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel and bedrock core samples were collected.

Test borings were backfilled with soil cuttings and filter sand upon completion of drilling and were sealed at the roadway surface with cold patch.

All soil and bedrock samples were provided to MaineDOT upon the completion of drilling for review and preparation of the test boring logs (Appendix A) and to assign geotechnical laboratory testing (Appendix B).

SUPPLEMENTAL PHASE (JULY 2018)

Upon completion of the preliminary phase subsurface exploration and submission of the PDR, Haley & Aldrich recommend that additional test borings be drilled at the site so that the top of bedrock surface along the width of each abutment and along the length of the wingwalls could be determined. This information would be used to establish foundation bearing levels that would minimize the amount of bedrock excavation that would be needed to construct the footings during construction. The need for and location of additional test borings was discussed with and agreed upon by MaineDOT and TY Lin during our conference call on 7 March 2018.

As a result, a supplemental phase subsurface exploration program consisting of five test borings, designated BB-HSS-201 through BB-HSS-205, was completed at the site. The test borings were laid out in the field by MaineDOT prior to the start of drilling. As-drilled test boring locations and ground surface elevations were determined by MaineDOT using optical and/or GPS survey equipment upon completion of drilling. The as-drilled coordinates of each test boring were used to determine station and offset distance and direction relative to the Route 1A/9 baseline. Both coordinate location and station/offset information for each test boring is shown on Table I. Only station/offset information is provided on the test boring logs included in Appendix A. The plan locations of the supplemental phase explorations, in addition to the preliminary phase explorations, are shown on Figure 2.

The test borings were drilled by MaineDOT and monitored by a Haley & Aldrich geologist on 23 and 24 July 2018 using a truck-mounted, CME 45C drill rig. Test borings were advanced through the overburden soil and terminated in/on bedrock at depths ranging from approximately 23 to 44 ft BGS using the same drilling means and methods that were used in the preliminary phase investigation except the hammer efficiency factor is equal to 0.928.

Each test boring was advanced up to approximately 4 ft into bedrock using a roller cone. Please note that bedrock samples were not obtained since coring was not completed.

Test borings were backfilled with soil cuttings and filter sand upon completion of drilling and were sealed at the roadway surface with cold patch.

All soil samples are available for review upon request. The available soil samples are currently being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

Generalized Subsurface Conditions

The subsurface conditions encountered at the site generally consist of the following geologic units presented in order of increasing depth BGS: fill, marine silt/clay, glacial till, and bedrock. Refer to Table I for a summary of the soil units and encountered thicknesses at each test boring location. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the test boring logs in Appendix A. Refer to Figure 3 for a graphical representation of the subsurface conditions along the north and south sides of the bridge.

Please note that the soil and bedrock descriptions provided on the test boring logs, shown on Figure 3 and summarized below do not represent actual conditions other than at the specific test boring locations. The actual conditions will vary from those described and shown herein.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Soil/Bedrock Description
Fill	13 to 27	very loose to dense, fine to coarse SAND with variable amounts of clay, silt and gravel; medium stiff to very stiff CLAY with variable amounts of sand and gravel; portions of the fill contained wood, brick, and cinders. <i>(encountered in each test boring)</i>
Marine Deposit	1 to 3	very soft to very stiff SILT to fine sandy SILT, little gravel, trace medium to coarse sand, trace organics. <i>(encountered in test borings BB-HSS-202, -204, and -205)</i>
Glacial Till	3 to 17	medium dense to very dense, fine to coarse SAND to fine GRAVEL, little to some gravel, little to some silt, little coarse sand, weathered bedrock fragments, boulders/cobbles. <i>(encountered in test borings BB-HSS-202, -204, and -205)</i>
Bedrock		<u>Abutment No. 1:</u> Top of bedrock surface encountered at approximately 27 ft BGS (El. 3.4) and generally slopes down from southeast to northwest. <i>(test boring BB-HSS-101)</i>
		<u>Abutment No. 2:</u> Top of bedrock surface encountered at approximately 20 to 26 ft BGS (El. 7 to El. 17) and generally slopes down from northeast to southwest. <i>(test borings BB-HSS-102, -203 and -205).</i>

BEDROCK

As stated previously, approximately 10 to 11 ft of bedrock was sampled in the preliminary phase test borings (i.e., BB-HSS-101 and BB-HSS-102). The sampled and recovered bedrock generally consisted of the following:

- Moderately hard to very hard, fresh to moderately weathered GRANOFELS and SCHIST. Primary joints dip at low to moderate angles and are very close to closely spaced. Secondary joints dipping steeply and are moderately close to widely spaced. Joints are tight to open and some joint surfaces have silt infillings, chlorite coatings, calcite coatings, and are weathered. Some slickensided joint surfaces were noted.

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. A summary of pertinent bedrock information encountered at the proposed substructures is presented below.

Substructure	Test Boring No.	Approximate Top of Bedrock Elevation (ft, NAVD 88)	Rock Quality Designation ¹ (%)	
			Range	Average
Abutment No. 1	BB-HSS-101	3.4	11 to 94	65
	BB-HSS-201	-0.6	--	--
	BB-HSS-202	-12.5	--	--
Abutment No. 2	BB-HSS-102	7.4	0 to 78	39
	BB-HSS-203	9.8	--	--
	BB-HSS-204	12.4	--	--
	BB-HSS-205	16.7	--	--

Notes:

¹ - "--" indicates no bedrock samples obtained and therefore, RQD could not be determined.

The top of bedrock elevation at each test boring location is shown on Figure 2. A photograph of the bedrock core from the preliminary phase test borings is included in Appendix A.

GROUNDWATER

Observation wells were not installed in any of the completed boreholes. As a result, long-term, static water levels at the site were not determined. However, soil samples collected from the preliminary and supplemental phase test borings were visually observed to be "moist" or "wet" at and below the elevations summarized below. In addition, water levels were measured in each test boring upon the completion of drilling and removal of drill casing.

Bridge Approach	Test Boring No.	Water Level Elevation Information		
		Soil Sample Moisture Condition		Measured Water Level
		"Moist"	"Wet"	
West	BB-HSS-201	El. 21.9	El. 11.9	El. 14.9
	BB-HSS-202	El. 25.6	El. 7.6	El. 3.6
	BB-HSS-101	El. 20.5	El. 10.0	El. 6.2
East	BB-HSS-102	El. 23.8	--	El. 29.2
	BB-HSS-203	El. 32.5	El. 30.5	El. 24.5
	BB-HSS-204	El. 34.9	El. 27.9	El. 29.7
	BB-HSS-205	El. 31.3	El. 26.3	El. 32.3

Please note that these water levels were likely impacted by drilling means/methods and may not be representative of actual static water levels at the site.

In general, groundwater levels can be expected to fluctuate, subject to test boring drilling means/methods, seasonal variation, local soil conditions, topography, precipitation and water levels in the stream. Groundwater levels present at the site and encountered during construction may differ from those observed in the test borings.

Geotechnical Laboratory Testing Program

A geotechnical laboratory testing program was undertaken on representative soil and bedrock samples collected during the preliminary and supplemental phase subsurface exploration programs to aid in soil classification and determination of engineering properties of bedrock needed for final design. All laboratory testing was performed in accordance with applicable American Society for Testing Materials (ASTM) testing procedures by MaineDOT (soil) in their laboratory facility in Bangor, Maine and GeoTesting Express (bedrock) in Acton, Massachusetts. The number, type and results of the laboratory testing are shown on the individual test boring logs in Appendix A and are summarized below.

Laboratory Test	ASTM Test Designation	Soil/Bedrock Unit	No. of Tests Completed	Range in Test Results ¹
Grain Size	ASTM D 422	Fill	3	<u>AASHTO Classification:</u> A-4, A-2-4 <u>USCS Classification:</u> SM, SC-SM, CL
Atterberg Limits	ASTM D 4318	Fill	1	WC=14.4%; PL=18%; LL=26%
Compressive Strength and Elastic Moduli of Rock	ASTM D 7012 – Method D	Bedrock	8	<u>Peak Compressive Stress:</u> 9,460 psi to 10,627 psi <u>Young's Modulus:</u> 6,560,000 psi to 9,140,000 psi

Notes:

¹ – WC = moisture content; PL = plastic limit; LL = liquid limit; psi = pounds per square inch.

Complete laboratory test results are provided in Appendix B.

Geotechnical Design Recommendations

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

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Eighth Edition, 2017, referred to herein as AASHTO LRFD,
- MaineDOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as Bridge Design Guide.

Engineering calculations that support the recommendations outlined in this section are provided for reference in Appendix D.

APPROACH EMBANKMENT SETTLEMENT AND STABILITY

As stated previously, the vertical alignment will remain virtually unchanged from the existing grades. We recommend that once the existing bridge has been demolished and grubbing within the limits of the fill areas has been completed, the exposed surfaces be compacted/proofrolled prior to fill/backfill placement. After proofrolling has been completed, excavation may be necessary to remove localized areas of weak, disturbed, or otherwise unacceptable soils as determined by the Resident and/or project Geotechnical Engineer. We also recommend that embankment fill consist of Granular Borrow (MaineDOT Standard Specification 703.19). Granular Borrow should be placed and compacted in accordance with MaineDOT Standard Specification 203.

Given the nature and consistency of the fill and naturally-deposited soils that are present at the site as well as the subgrade preparation and embankment material recommendations stated herein, we anticipate that the resulting magnitude of elastic compression will be minimal and will likely occur rapidly during construction, soon after the fill is placed and prior to final paving. As a result, we anticipate post-construction settlement to be negligible (i.e., < ¼ in.). Furthermore, in our experience, conventional earthfill embankments constructed using the material types and methodology recommended herein over a properly prepared subgrade meet the minimum safety factors required by AASHTO LRFD and the BDG.

SEISMIC DESIGN

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

Based on the nature and thickness of the overburden soils and depth to bedrock at the site, as determined from the preliminary and supplemental phase test borings, we recommend the site be considered "Site Class D". Spectral accelerations were determined based on the geographic site location and the recommended "Site Class D" designation using the United States Geological Survey (USGS) software application Seismic Design Parameters v. 2.0, which is based on a seismic event having a 7 percent probability of exceedance in 75 years (approximate 1,000-year return period). The recommended values are summarized below.

Seismic Parameter	Design Value (g)
Mapped Peak Ground Acceleration, PGA =	0.067
Mapped Short (0.2-second) Period, S_s =	0.144
Mapped Long (1.0-second) Period, S_1 =	0.044

Based on the information summarized above we recommend the following:

- Site Factors (F_{pgs} , F_a , and F_v) be determined in accordance with AASHTO LRFD Section 3.10.3.2.
- Horizontal Response Spectral Acceleration Coefficients (S_{DS} and S_{D1}) and the Peak Seismic Ground Acceleration Coefficient (A_s) be determined in accordance with AASHTO LRFD Section 3.10.4.
- The Seismic Performance Zone be determined in accordance with AASHTO LRFD Section 3.10.6 based on the calculated value of S_{D1} .
- The Seismic Performance Category be determined in accordance with the BDG Section 3.7.1.1.
- A complete seismic analysis is completed in accordance with the BDG Section 3.7.2 considering the structures functional importance (BDG Section 3.7.1.3) and major/minor structure designation (BDG Section 3.7.1.4).

Based on our review of the soil conditions encountered in the test borings, it is our opinion that the overall potential for saturated granular soils present at the site to liquefy during the design earthquake event is low. Refer to Appendix D for seismic design evaluations.

FOUNDATIONS

We recommend that the abutments and wingwalls be supported on mass concrete footings founded on intact bedrock or seal concrete founded on intact bedrock. Based on the top of bedrock elevation encountered in the preliminary and design phase test borings and their proximity to the proposed abutment/wingwall locations, we estimate that the bottom of footing or seal elevation at Abutment No. 1 and Abutment No. 2 will be approximately El. 2 and El. 8 and El. 2 and El. 7, respectively. Please note that the top of bedrock elevation is unknown within some portions of the proposed abutment and wingwall limits and the available subsurface information indicates that the surface is variable. Based on this, it is likely that the actual top of bedrock elevation within the limits of the proposed abutments and wingwalls will either be higher or lower than the bottom of footing elevations noted herein. Additional foundation recommendations are summarized below. Please note that the design recommendations presented below assume foundation subgrade preparation is completed in accordance with the guidance provided in the Construction Considerations section of this report.

- Bearing Resistance:
 - For the Service Limit State, mass concrete footings should be designed such that footing contact pressures do not exceed 20 kips per square foot (ksf), which is the presumptive bearing resistance of the bedrock at the site. At this pressure, it is estimated that settlement of footings bearing on competent bedrock will be less than 1 in. as stated in AASHTO LRFD Section 10.6.2.6.1.
 - For the Strength and Extreme Event Limit States, footings should be designed for the following factored bearing resistances, which are based on resistance factors of 0.45 (Strength Limit State) and 0.80 (Extreme Event Limit State) in accordance with AASHTO LRFD Table 10.5.5.2.2-1 and Section C11.5.8, respectively:

- Strength Limit State: 32 ksf
 - Extreme Event Limit State: 57 ksf
- The factored contact pressures should not exceed the factored compressive resistance of the spread footing concrete, which may be taken as 30 percent of the concrete's compressive strength. We recommend that the spread footings be at least 3-ft wide regardless of the applied bearing pressure.
- Bearing Distribution and Eccentricity:
 - Application of permanent and transient loads is specified in AASHTO LRFD Section 11.5.6. We recommend the stress distribution be assumed to be a triangular or trapezoidal distribution over the effective footing base as shown in AASHTO LRFD Figure 11.6.3.2-2.
 - The eccentricity of loading at the Strength Limit State, based on factored loads, should not exceed 0.45 of the spread footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the base width and length.
- Sliding Resistance:
 - In accordance with AASHTO LRFD Tables 3.11.5.3-1 and 10.5.5.2.2-1, sliding computations for the resistance of abutment and wingwall spread footings to lateral loads, for different degrees of saturation present at the time the spread footings are constructed on the competent bedrock subgrade, may be calculated with the design parameters presented below.

Subgrade Saturation Condition During Construction	Coefficient of Friction ($\tan \delta$, dim.)	Interface Friction Angle (δ , deg.)	Strength Limit State Resistance Factor for Sliding (ϕ_t)	Service/Extreme Limit State Resistance Factor for Sliding (ϕ_t)
Prepared in-the-dry	0.7	35	0.8	1.0
Prepared in-the-wet	0.6	29		

- Lateral passive soil resistance in front of the spread footings should be neglected in accordance with requirements of the BDG. "Anchorage" of the spread footing concrete to the concrete seal is required by BDG Section 5.2.2. The dowels should be drilled and grouted into the seal concrete after dewatering and prior to placing the spread footing concrete. "Anchorage" of concrete seals to bedrock may also be required to resist sliding forces and improve stability. If "anchorage" between the concrete seal and bedrock is needed, as determined by TY Lin, additional geotechnical recommendations can be provided.

ABUTMENT AND WINGWALL DESIGN

We recommend that the abutments and wingwalls be designed for all relevant Service, Strength, and Extreme Limit States and load combinations specified in AASHTO LRFD Sections 3.4.1 and 11.5.5. Additional recommendations are provided below.

- Drainage:
 - The abutment design shall include a drainage system behind the abutment to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutment and wingwall spread footings. Drainage behind the structure shall be in accordance with BDG Section 5.4.2.13.
- Lateral Earth Pressures:
 - Recommendations summarized in the table below are based on the following:
 - Abutments and wingwalls are backfilled with a free-draining material (i.e., Soil Type 4, BDG Table 3-3).
 - The abutment and wingwall backwalls are vertical.
 - Adequate drainage is provided, as recommended herein, to eliminate the potential for unbalanced hydrostatic pressures to develop.
 - Horizontal inclination of the backfill surface.

Substructure	Active Lateral Earth Pressure Coefficient (K_a , dim.)	At-Rest Lateral Earth Pressure Coefficient (K_o , dim.)	Passive Lateral Earth Pressure Coefficients (K_p , dim.)		Backfill Total Unit Weight (γ_t , pcf)	Backfill Internal Angle of Friction (ϕ' , deg.)
			Rankine	Coulomb		
Abutment No. 1	0.27	0.47	3.25	8.38	125	32
Abutment No. 2	0.29			10.79		

- In accordance with the BDG, we recommend that cantilever, semi-integral abutments be designed to withstand a maximum applied lateral load equal to the passive earth pressure calculated using either the Rankine or Coulomb passive lateral earth pressure coefficients presented above.

- In accordance with BDG Section 5.4.2.11, “developing full passive earth pressure requires that wall rotation, i.e., the ratio of lateral abutment movement to abutment height (y/H), exceeds 0.005. If the calculated rotation is significantly less than required to develop full passive pressure, the Designer may consider using the Rankine passive earth pressure case, which assumes no wall friction.” In addition, considering that AASHTO LRFD does not provide a load factor for passive earth pressure, we understand that MaineDOT standard practice for integral abutment design considers a load factor equal to 1.5 to calculate factored passive lateral earth pressures.
- Additional lateral earth pressures due to live load surcharge are required in accordance with BDG Section 3.6.8 for abutments if an approach slab is not included. When an approach slab is specified, reduction, not elimination of the surcharge load is permitted in accordance with AASHTO LRFD Section 3.11.6.5. We recommend that the live load surcharge be estimated as a uniform horizontal earth pressure due to an equivalent height of soil that is related to the abutment and wingwall heights, as presented to BDG Table 3-4.
- If determined applicable by T.Y. Lin, the abutments and wingwalls should be designed for a uniform lateral load to account for seismic soil loading in accordance with AASHTO LRFD Article A.11.1. Based on the seismic site class (Site Class “D”), we recommend a uniform lateral load (in psf) equal to 20 times the wall height (in feet) be used for design. Note that this soil pressure includes both the static and seismic lateral earth loads.

SCOUR AND SLOPE PROTECTION

We recommend that scour protection of the abutment and wingwall foundations include the following:

- Placement of spread footings or concrete seals directly on bedrock surfaces that are cleaned of all weathered, loose, and potentially erodible or scourable rock as determined by the Resident or project Geotechnical Engineer.
- Construction of slopes no steeper than 1.75H:1V in front of abutments and wingwalls using riprap and erosion control geotextile in accordance with MaineDOT Standard Detail 610(03).

FROST PROTECTION

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

Construction Considerations

TEMPORARY EARTH SUPPORT

Based on the proposed elevation of the bottom of the foundations relative to existing site grades and the potential need to temporarily support existing utilities that need to remain in service or to prevent impacts to adjacent properties or the stream, temporary earth support systems (e.g. steel sheetpile cofferdams) will likely be needed to prepare and construct the abutment and wingwall foundations in-the-dry.

In general, design of temporary earth support systems is the responsibility of the Contractor and should be conducted by a Licensed Professional Engineer in the State of Maine. We recommend that temporary earth support systems be designed to support all appropriate combinations of earth, water, and surcharge loads (from traffic, construction equipment, material stockpiles and other sources) imposed on the system(s) during all phases of the construction period. The Contractor is responsible for choosing an applicable factor of safety for the earth support system(s). The Contractor's design should also consider the means and methods and construction sequencing proposed. We recommend that design calculations and shop drawings be prepared by the Contractor, stamped by a Licensed Professional Engineer in the State of Maine and submitted to MaineDOT for review prior to construction.

BEDROCK SUBGRADE PREPARATION

The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the abutments and wingwalls are completed. Construction activities should not be permitted to disturb the bedrock mass or to create any rock falls or any open fissures. The bedrock surface shall be cleared of all loose fractured bedrock, loose, decomposed bedrock, and soil (i.e., unsuitable material). If excavation of unsuitable material extends below the bearing level of the footings, we recommend that Class S concrete be placed from the bottom of excavation up to the proposed footing bearing level, after the surface has been examined in the field by the Resident and/or project Geotechnical Engineer, as discussed below.

- Foundation bearing areas should be level. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level. Smooth bedrock, if present, should be roughened or serrated prior to placing concrete to enhance sliding stability.
- In-the-dry or underwater excavation of steeply sloping and/or loose, fractured bedrock may be done using conventional excavation methods. Based on the proposed bearing level of the abutment and wingwall footings and the top of bedrock surface encountered in the preliminary and supplemental phase test borings we do not anticipate the need for bedrock removal using drilling and blasting techniques.

- Depending on the actual water level in the stream during foundation construction, portions, if not all, of the abutment and wingwall excavations may be submerged. In accordance with MaineDOT Standard Specifications Section 511 – Cofferdams, if the foundation excavations are submerged, the Contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Resident and project Geotechnical Engineer for approval. We recommend that the final bedrock surface be approved by the Resident and/or project Geotechnical Engineer prior to placement of the concrete seals or footing reinforcing steel and concrete.
- It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps so that subgrade preparation and foundation construction is completed in-the-dry. We recommend that dewatering effluent be managed in accordance with all local, state and/or federal regulations.

REUSE OF EXCAVATED ON-SITE SOILS

Based on the nature and consistency of the fill and naturally-deposited soils that are present at the site as well as the results of laboratory grain size analyses, we recommend that reuse of excavated soils as structural backfill behind abutments and wingwalls not be permitted. The excavated soils may be used as Common Borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate existing base, subbase and potentially some amount of subgrade soils along the approach roadways. We recommend that these materials not be reused as base or subbase course materials. We anticipate that these materials could be reused as fill below subgrade level where raises in grade are anticipated provided that all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

The excavated soils may be saturated and may require a laydown/stockpile area that can be used by the Contractor to moisture-condition (e.g., air dry, mix, harrow, etc.) the excavated soil to achieve a moisture content suitable for placement and adequate compaction prior to reuse.

SUBMITTAL REVIEW

The CDs should be written so that the requirements of the documents are consistent with the design intent of the geotechnical recommendations outlined herein. The CDs require that the Contractor and the Contractor's Engineer perform analyses and submit results to MaineDOT for review. We recommend that Haley & Aldrich be allowed to review the geotechnical-related submittals to ensure that the Contractor's analyses/submittals are in accordance with the intent of the design as summarized herein. This will enable us to observe compliance with the design concepts, assumptions and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

CONSTRUCTION MONITORING

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction activities is required to enable the project 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.

- Preparation of abutment and wingwall foundation bearing surfaces.
- Placement and compaction of compacted fills.

Limitations and Closure

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

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

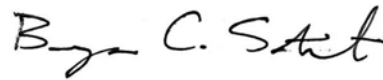
We understand that this report will be included as a reference document in the package that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the Contract Documents (CDs; plans and specifications)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

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

Sincerely yours,
HALEY & ALDRICH, INC.



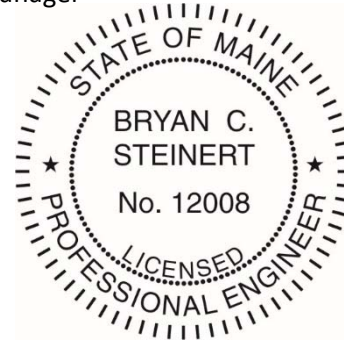
Nathan A. Sherwood, P.E.
Senior Geotechnical Engineer



Bryan C. Steinert, P.E.
Project Manager



Wayne A. Chadbourne, P.E.
Senior Associate



Enclosures:

- Table 1 – Summary of Subsurface Explorations
- Figure 1 – Project Locus
- Figure 2 – Boring Location Plan
- Figure 3 – Interpretive Subsurface Profiles
- Appendix A – Test Boring Logs and Rock Core Photographs
- Appendix B – Laboratory Test Results
- Appendix C – Historic Bridge Drawings
- Appendix D – Calculations

TABLE I**Subsurface Exploration Location and Subsurface Data**

Grist Mill Bridge No. 2334 over Soudabscook Stream

MaineDOT WIN 021692.00

Hampden, Maine

Haley & Aldrich, Inc. File No.: 130732-003

LOCATION DATA:

Drilling Phase	Test Boring No. ¹	Coordinates ²		Station and Offset Distance/Direction ⁴ (ft)		
		Northing	Easting	Station	Offset Distance	Offset Direction
Prelim.	BB-HSS-101	455,361.00	1,716,391.52	33 + 64.7	11.1	RT
	BB-HSS-102	455,416.51	1,716,453.61	34 + 44.9	11.3	LT
Supplemental	BB-HSS-201	455,334.26	1,716,324.22	32 + 92.7	9.2	RT
	BB-HSS-202	455,362.39	1,716,346.66	33 + 24.2	8.1	LT
	BB-HSS-203	455,395.11	1,716,471.80	34 + 50.9	16.1	RT
	BB-HSS-204	455,379.58	1,716,524.08	34 + 88.1	55.1	RT
	BB-HSS-205	455,436.76	1,716,489.47	34 + 86.5	11.8	LT

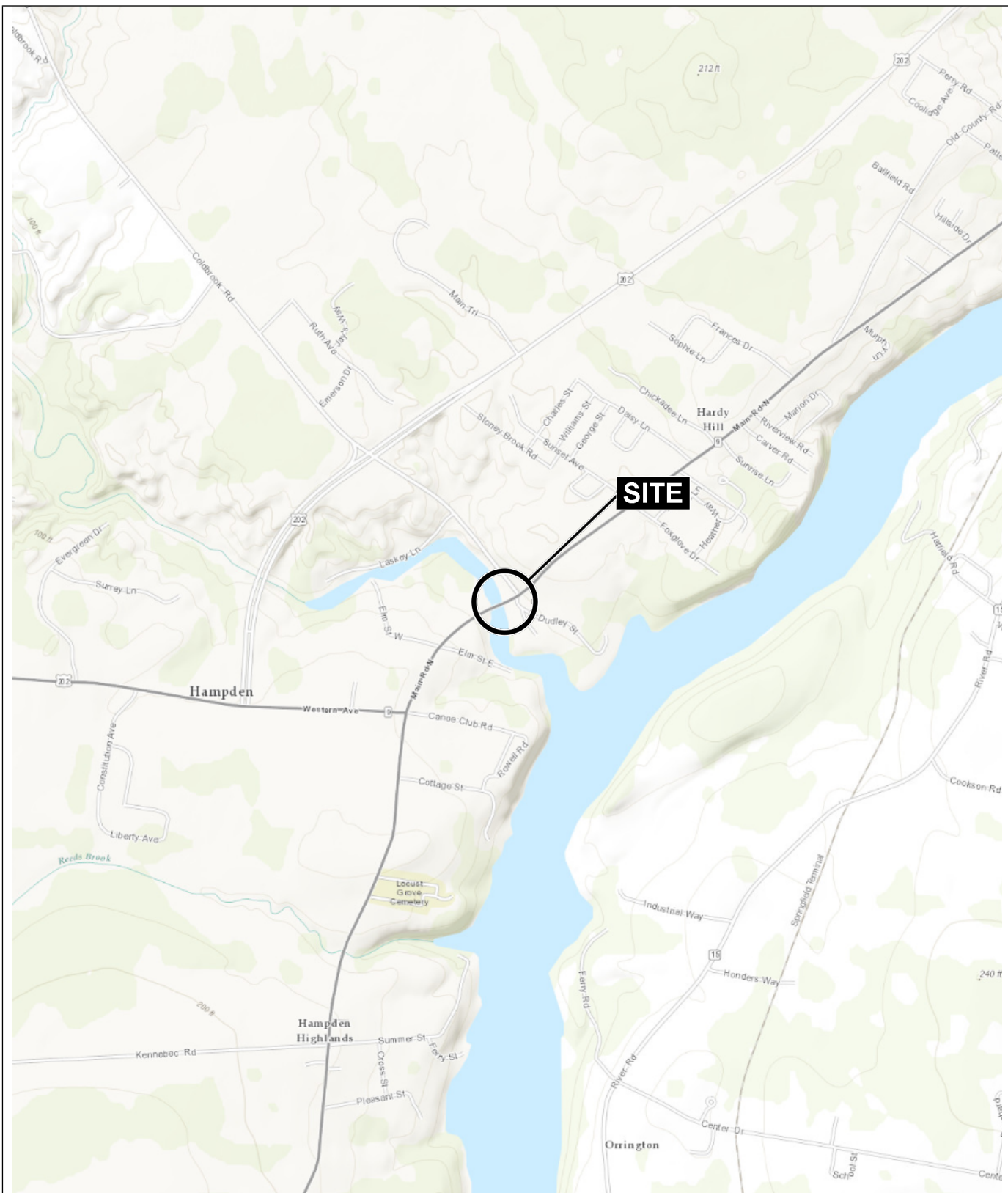
SUBSURFACE DATA:

Drilling Phase	Test Boring No. ¹	Ground Surface Elevation ³	Approximate Strata Thickness ⁵ (ft)				Approximate Elevation of Top of Bedrock ³	Approximate Elevation of Bottom of Exploration ³
			Bituminous Concrete	Fill	Marine Deposit	Glacial Till		
Prelim.	BB-HSS-101	30.5	1.0	26.1	NE	NE	3.4	-7.5
	BB-HSS-102	33.8	1.0	25.4	NE	NE	7.4	-2.1
Supplemental	BB-HSS-201	26.9	0.8	26.7	NE	NE	-0.6	-2.1
	BB-HSS-202	27.6	1.8	18.2	3.0	17.1	-12.5	-16.4
	BB-HSS-203	35.5	1.2	24.5	NE	NE	9.8	7.3
	BB-HSS-204	37.9	0.9	14.1	0.6	9.9	12.4	8.9
	BB-HSS-205	36.3	1.9	13.1	2.0	2.6	16.7	13.3

Notes:

- 1 Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
- 2 As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet, and reference NAD83, Maine 2000 Central Zone coordinate system.
- 3 Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment and reference the North American Vertical Datum of 1988 (NAVD 88).
- 4 Station and offset information determined by MaineDOT and provided to Haley & Aldrich.
- 5 "NE" indicates stratum was not encountered in test boring.

Prepared By:	NAS	8/22/2018
Checked By:	BCS	8/29/2018
Reviewed By:	WAC	9/27/2018



MAP SOURCE: ESRI

SITE COORDINATES: 44°44'56"N, 68°49'57"W



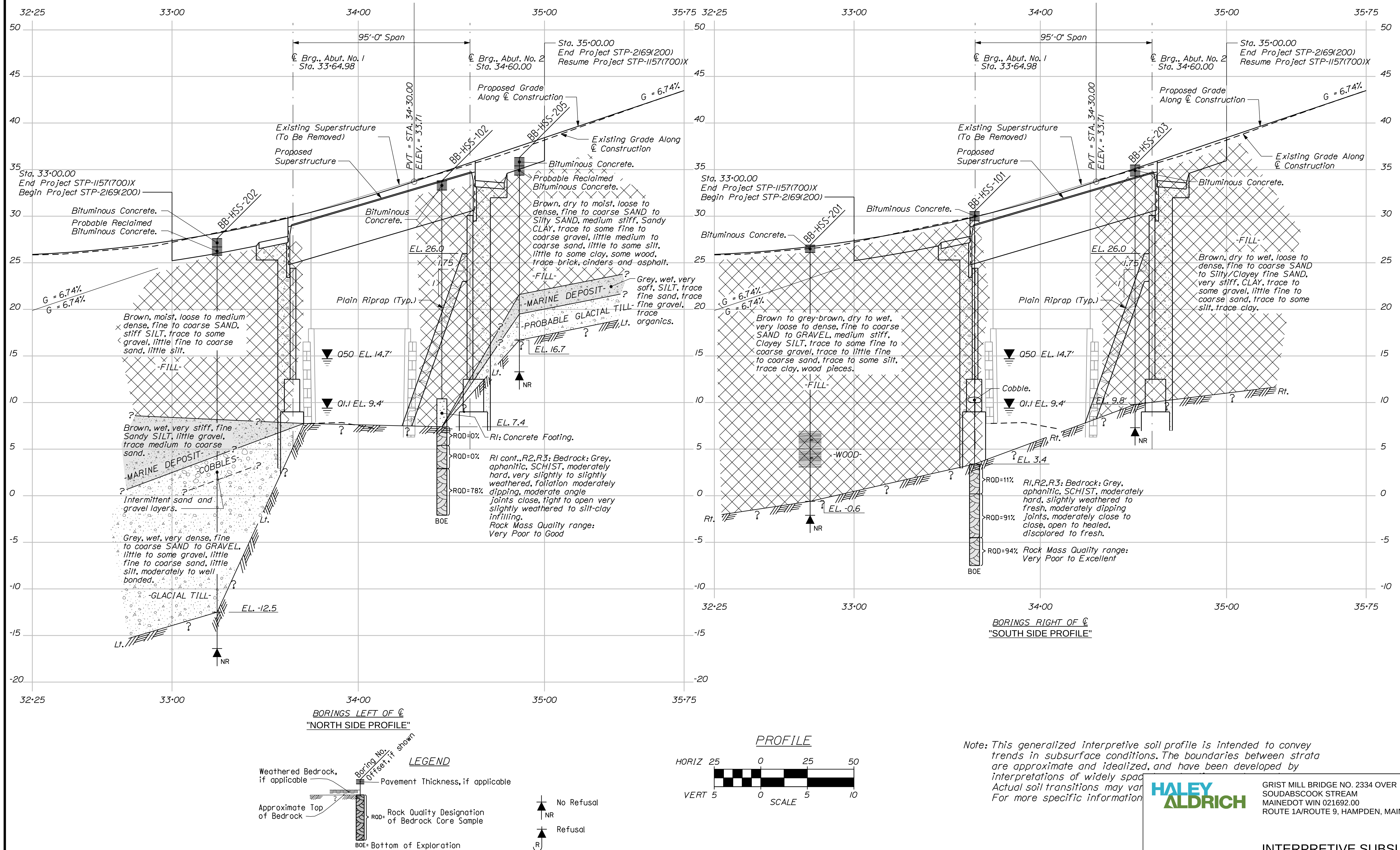
**HALEY
ALDRICH**

GRIST MILL BRIDGE NO. 2334 OVER
SQUADABSCOOK STREAM
MAINEDOT WIN 021692.00
HAMPDEN, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
OCTOBER 2018

FIGURE 1



Note: This generalized interpretive soil 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 data. Actual soil transitions may vary. For more specific information, contact the project engineer.

**HALEY
ALDRICH**

GRIST MILL BRIDGE NO. 2334 OVER
SOUDABSCOOK STREAM
MAINEDOT WIN 021692.00
ROUTE 1A/ROUTE 9, HAMPDEN, MAINE

INTERPRETIVE SUBSURFACE PROFILES

SCALE: AS SHOWN
SEPTEMBER 2018

FIGURE 3

APPENDIX A

Test Boring Logs and Rock Core Photographs

Preliminary Phase (2009)
Test Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 carries Route 1A/9 over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-101 WIN: 21692.00			
Driller: MaineDOT		Elevation (ft.) 30.5		Auger ID/OD: Hollow Stem Auger							
Operator: E. Giguere/C. Giles		Datum: NAVD 88		Sampler: Standard Split Spoon							
Logged By: B. Babcock		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"							
Date Start/Finish: 7/23/09-7/23/09		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"							
Boring Location: STA.33+64.7; 11.1 RT		Casing ID/OD: HW		Water Level*: 24.3' bgs.							
Hammer Efficiency Factor: 0.84		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt		R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person		S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected		T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test					
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
0							HSA	29.5	ASPHALT.	G#212329 A-2-4, SM WC=9.6%	
	1D	24/12	1.00 - 3.00	14/11/9/7	20	28			Brown, dry, medium dense, fine to coarse SAND, little fine to coarse gravel, trace silt, (Fill). Augered to 15.0' bgs, then Hydraulic Push HW Casing to 11.0' bgs.		
5	2D	24/14	5.00 - 7.00	3/4/2/3	6	8			Brown, dry, loose, fine to coarse SAND, some silt, little fine to coarse gravel.		
10	3D	24/6	10.00 - 12.00	WOH/WOH/3/6	3	4			Brown, moist, very loose, fine to coarse SAND, little silt, trace fine gravel, trace clay.		
								9			
								12			
								27			
								34			
15	4D	24/10	15.00 - 17.00	4/12/9/15	21	29	65		Brown, moist, medium dense, fine to coarse SAND, little silt, little fine to coarse gravel, 3" of coarse gravel in bottom of recovery.		
								96			
								96			
								93			
								115			
20	5D	24/4	20.50 - 22.50	9/6/6/15	12	17	35		Cobble from 19.9-20.5' bgs. Grey with brown, wet, medium dense, fine to coarse GRAVEL, little silt, trace fine sand.		
								52			
								110			
								108			
25								107			
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-HSS-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 carries Route 1A/9 over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-101 WIN: 21692.00			
Driller: MaineDOT				Elevation (ft.): 30.5				Auger ID/OD: Hollow Stem Auger			
Operator: E. Giguere/C. Giles				Datum: NAVD 88				Sampler: Standard Split Spoon			
Logged By: B. Babcock				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 7/23/09-7/23/09				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: STA.33+64.7; 11.1 RT				Casing ID/OD: HW				Water Level*: 24.3' bgs.			
Hammer Efficiency Factor: 0.84				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 _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	6D	24/8	25.00 - 27.00	4/12/13/27	25	35	a225 b97 WASH AHEAD	3.4		a225 blows for 10". Brown, wet, medium dense, coarse SAND, little fine to medium sand, some coarse to fine gravel. b97 blows for 8". Washed ahead to 27.1' with Roller Bit, then drove Casing to 26.5' bgs, then began coring at 27.1' bgs. Top of Bedrock at Elev. 3.4'. R1: Grey, aphanitic, PHYLLITE, moderately hard, slightly weathered, moderately dipping with occasional low angle joints, very close to close, open to healed, discolored, slightly weathered, secondary vertical joint approximately 29.6'-30.0', open, discolored, slightly weathered, occasional calcite veins typically 1/4" thickness, zone of lost recovery probably between 28.0'-29.0' bgs. Highly fractured zone approximately 29.0'-30.0' -BANGOR FORMATION- Rock Mass Quality= Very Poor R1: Core Times (min:sec) 27.1-28.1' (4:13) 28.1-29.1' (3:02) 29.1-30.1' (3:58) 30.1-30.3' (4:00) 79% Recovery Core barrel jammed, probable cause vertical fracturing. 100% water loss during core. R2: Grey, aphanitic, PHYLLITE, moderately hard, very slightly weathered, low angle to moderately dipping joints, moderately close, open, discolored to moderately weathered, slightly to moderately weathered joint at 34.0' bgs. -BANGOR FORMATION- Rock Mass Quality= Excellent R2: Core Times (min:sec) 30.3-31.3' (3:01) 31.3-32.3' (4:17) 32.3-33.3' (4:38) 33.3-34.3' (4:26) 34.3-35.0' (4:25) 100% Recovery Core barrel jammed. 100% water loss during core. R3: Grey, aphanitic, PHYLLITE, moderately hard, fresh, moderately dipping joints, close, open, discolored to fresh. -BANGOR FORMATION- Rock Mass Quality= Excellent R3: Core Times (min:sec) 35.0-36.0' (4:03) 36.0-37.0' (4:09) 37.0-38.0' (3:57) 100% Recovery Bottom of Exploration at 38.0 feet below ground surface.	qp=10,627 psi (30.64'-31.0')
	R1	38.4/30	27.10 - 30.30	RQD = 11%			NQ-2				
30											
	R2	56.4/56.4	30.30 - 35.00	RQD = 91%							
35											
	R3	36/36	35.00 - 38.00	RQD = 94%							
40											
45											
50											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 2 of 2 Boring No.: BB-HSS-101	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 carries Route 1A/9 over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-102 WIN: 21692.00					
Driller: MaineDOT				Elevation (ft.): 33.8				Auger ID/OD: Hollow Stem Auger					
Operator: E. Giguere/C. Giles				Datum: NAVD 88				Sampler: Standard Split Spoon					
Logged By: B. Babcock				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"					
Date Start/Finish: 7/22/09-7/22/09				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"					
Boring Location: STA.34+44.9; 11.3 LT				Casing ID/OD: HW & NW				Water Level*: 4.6' bgs.					
Hammer Efficiency Factor: 0.84				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐									
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test													
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows						
0							HSA	32.8		ASPHALT.	G#212330 A-1-a, SM WC=3.5%		
	1D	24/12	1.00 - 3.00	48/23/13/10	36	50				1.0'		Brown, dry, dense, fine to coarse SAND, some fine to coarse gravel, little silt, trace asphalt, (Fill).	
5													
	2D	24/6	5.00 - 7.00	6/5/6/4	11	15						Brown, dry, medium dense, fine SAND, some silt, little medium to coarse sand, trace fine to coarse gravel.	
10													
	3D	24/12	10.00 - 12.00	4/3/3/3	6	8						Brown, moist, loose, fine to coarse SAND, some clay, little fine to coarse gravel, trace brick, cinders.	
												G#212331 A-2-4, SC-SM WC=14.4% LL=26 PL=18 PI=8	
15													
	4D	24/6	15.00 - 17.00	3/2/3/3	5	7	100					Grey, moist, medium stiff, sandy CLAY, some wood, trace fine gravel, trace brick.	
20													
	5D	24/43	19.00 - 21.00	6/3/2/2	5	7	72				aNo Recovery, re-drove spoon through sample interval to 21.0' bgs and recovered 3" of wood and gravel. Drove HW Casing to Refusal at 23.4' bgs.		
	R1	60/60	23.40 - 28.40	RQD = 0%			b154	10.4			b154 blows for 4.8".		
25							NQ-2				R1: Concrete Footing from 23.4-26.4' bgs. R1:Core Times (min:sec)		
Remarks:													
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Page 1 of 2 Boring No.: BB-HSS-102			

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> <u>US CUSTOMARY UNITS</u>				Project: Grist Mill Bridge #2334 carries Route 1A/ 9 over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-102 WIN: 21692.00																									
Driller: MaineDOT				Elevation (ft.) 33.8				Auger ID/OD: Hollow Stem Auger																									
Operator: E. Giguere/C. Giles				Datum: NAVD 88				Sampler: Standard Split Spoon																									
Logged By: B. Babcock				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"																									
Date Start/Finish: 7/22/09-7/22/09				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"																									
Boring Location: STA.34+44.9; 11.3 LT				Casing ID/OD: HW & NW				Water Level*: 4.6' bgs.																									
Hammer Efficiency Factor: 0.84				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																													
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.																					
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25										7.4		23.4-24.4' (4:21) 24.4-25.4' (3:59) 25.4-26.4' (3:06) 100% Recovery Note: Concrete not included in RQD Recovery Calculations. Top of Bedrock at Elev. 7.4'. R1:Grey, aphanitic PHYLLITE, moderately hard, very slightly weathered, moderately dipping joints parallel to foliation, close, tight to open, very slightly weathered, vertical joint at 28.1'-28.7' open, discolored with secondary pyrite mineralization present. -BANGOR FORMATION- Rock Mass Quality= Very Poor R1:Core Times (min:sec) cont. 26.4-27.4' (3:55) 27.4-28.4' (5:23) 100% Recovery R2:Grey aphanitic PHYLLITE, moderately hard, slightly weathered, joints moderately dipping, close, open, slightly weathered with occasional silty-clay thin joint coatings, secondary high angle to vertical joints, very close to moderately close, open, discolored. Occasional 1/16" to 1/4" calcite veins. Highly fractured zone approximately 30.1'-30.4' -BANGOR FORMATION- Rock Mass Quality= Very Poor R2:Core Times (min:sec) 28.4-29.4' (5:46) 29.4-30.4' (6:07) 30.4-30.9' (12:31) 100% Recovery Barrel Jammed at 30.9' bgs. R3:Grey, aphanitic PHYLLITE, moderately hard, slightly weathered, moderate angle joints dipping parallel to foliation, close to moderately close, open, discolored to slightly weathered. Healed vertical joint at 32.6' bgs. -BANGOR FORMATION- Rock Mass Quality= Good R3:CoreTimes (min:sec) 30.9-31.9' (5:11) 31.9-32.9' (5:06) 32.9-33.9' (5:11) 33.9-34.9' (4:50) 34.9-35.9' (3:40) 95% Recovery Bottom of Exploration at 35.9 feet below ground surface.	qp=9,460 psi (34.5'-34.9')																				
30																																	
35																																	
40																																	
45																																	
50																																	

Remarks:												
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.												
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.												
Page 2 of 2												Boring No.: BB-HSS-102

Supplemental Phase (2018)
Test Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-201 WIN: 21692.00	
Driller: Maine Department of Transportation				Elevation (ft.): 26.9		Auger ID/OD: --			
Operator: T. Daggett				Datum: NAVD 88		Sampler: Split-Spoon 1.375 in.			
Logged By: M. Snow				Rig Type: CME 45C		Hammer Wt./Fall: SS-140#/30; NW 140#/30			
Date Start/Finish: 7-23-18/7-23-18				Drilling Method: SSA/NW Drive		Core Barrel: --			
Boring Location: STA.32+92.7; 9.2 RT				Casing ID/OD: NW-3.0 in. ID		Water Level*: 12.0 ft			
Hammer Efficiency Factor: 0.928				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test									

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								26.1		-BITUMINOUS CONCRETE-		
	1D	24/14	1.00 - 3.00	23/21/20/19	41	63	SSA			0.8	Brown, dry, medium dense, fine to coarse SAND, some fine to coarse gravel, trace silt -FILL-(SW) PID=7ppm	
	2D	24/18	3.00 - 5.00	10/14/12/12	26	40				3-in. layer clayey SILT with sand and gravel, petroleum-like odor Brown, dry, medium dense, fine to coarse SAND, some fine to coarse gravel, trace silt -FILL-(SW) PID=18ppm		
5	3D	24/12	5.00 - 7.00	8/5/2/1	7	11				Brown, moist, loose, fine to medium SAND, some silt, little coarse sand, trace gravel, petroleum-like odor -FILL-(SM) PID=15ppm		
10	4D	24/15	10.00 - 12.00	1/3/4/10	7	11	3			8.5	Brown, moist, medium stiff, clayey SILT, little fine to medium sand, little gravel, trace coarse sand, slight petroleum-like odor -FILL-(ML) PID=15ppm	
							9					
							25					
							20					
							23					
15	5D	24/6	15.00 - 17.00	3/2/4/4	6	9	5			13.5	Brown, wet, loose, fine to coarse SAND, some gravel, little silt, slight petroleum-like odor -FILL-(SM) PID=12ppm	
							15					
							27					
							30					
							27					
20	6D	24/12	20.00 - 22.00	4/2/2/2	4	6	15		6.9	20.0	Brown, wet, loose WOOD, trace fine to medium sand, faint petroleum-like odor -WOOD- PID=17ppm	
							21					
							22					
							37					
25							67		2.9	24.0		

Remarks:
 *Water level taken after completion of boring with casing removed.
 Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.

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

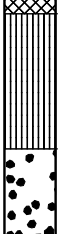
 * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 2

Boring No.: BB-HSS-201

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-201 WIN: 21692.00																																																																																																																																																																																																																												
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Operator: T. Daggett		Datum: NAVD 88		Sampler: Split-Spoon 1.375 in.																																																																																																																																																																																																																																
Logged By: M. Snow		Rig Type: CME 45C		Hammer Wt./Fall: SS-140#/30; NW 140#/30																																																																																																																																																																																																																																
Date Start/Finish: 7-23-18/7-23-18		Drilling Method: SSA/NW Drive		Core Barrel: --																																																																																																																																																																																																																																
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine		Boring No.: BB-HSS-202 WIN: 21692.00				
Driller: Maine Department of Transportation			Elevation (ft.): 27.6		Auger ID/OD: --					
Operator: T. Daggett			Datum: NAVD 88		Sampler: Split-Spoon 1.375 in.					
Logged By: M. Snow			Rig Type: CME 45C		Hammer Wt./Fall: SS-140#/30; NW 140#/30					
Date Start/Finish: 7-24-18/7-24-18			Drilling Method: SSA/NW Drive		Core Barrel: --					
Boring Location: STA.33+24.2; 8.1 LT			Casing ID/OD: NW-3.0 in. ID		Water Level*: 24.0 ft					
Hammer Efficiency Factor: 0.928			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt								R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person	S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected	T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	26.7		-BITUMINOUS CONCRETE-	
								25.8		-PROBABLE RECLAIMED BITUMINOUS CONCRETE-	
	1D	24/16	2.00 - 4.00	11/14/9/9	23	36				Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt -FILL-(SW-SM) PID=0ppm	
								23.1		-----4.5	
5	2D	24/16	5.00 - 7.00	3/5/5/6	10	15				Brown, moist, stiff SILT, little fine to coarse sand, trace gravel -FILL-(ML) PID=0ppm	
								19.1		-----8.5	
10	3D	24/3	10.00 - 12.00	2/3/2/3	5	8	6			Brown, moist, loose, fine to medium SAND, little coarse sand, little silt, trace gravel, slight petroleum-like odor -FILL-(SP-SM) PID=4ppm	
							8				
	3DA*	24/3	12.00 - 14.00				14			*Redrove sampler Brown, moist, fine to coarse SAND, little silt, trace gravel -FILL-(SW-SM)	
							24				
							38				
15	4MD	24/0	15.00 - 17.00	18/20/18/8	38	59	30		Note: Missed sample, redrove sampler, pushing on rock, no recovery -FILL-		
								26			
								40			
								93			
								83			
20	5D	24/4	20.00 - 22.00	8/10/8/6	18	28	29	7.6		Brown, wet, very stiff, fine sandy SILT, little gravel, trace medium to coarse sand -MARINE DEPOSIT-(ML) PID=0ppm	
							40				
							45				
							163				
25							191	4.6		COBBLES encountered from approximately 23.0 to 25.5 ft with intermittent sand and gravel layers approximately 2 to 3-in. thick based on drill action -GLACIAL TILL-	

Remarks:
 *Water level taken after completion of boring; casing at 33.1 ft below ground surface.
 Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.

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

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

Page 1 of 2

Boring No.: BB-HSS-202

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-202 WIN: 21692.00			
Driller: Maine Department of Transportation				Elevation (ft.): 27.6				Auger ID/OD: --			
Operator: T. Daggett				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in.			
Logged By: M. Snow				Rig Type: CME 45C				Hammer Wt./Fall: SS-140#/30; NW 140#/30			
Date Start/Finish: 7-24-18/7-24-18				Drilling Method: SSA/NW Drive				Core Barrel: --			
Boring Location: STA.33+24.2; 8.1 LT				Casing ID/OD: NW-3.0 in. ID				Water Level*: 24.0 ft			
Hammer Efficiency Factor: 0.928				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 _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25							72	2.1		25.5 Gray, wet, very dense, fine to coarse GRAVEL, little fine to coarse sand, little silt, moderately bonded -GLACIAL TILL-(GW-GM) PID=0ppm -1.4 29.0 Gray, wet, very dense, fine to medium SAND, little coarse sand, little silt, little gravel, well bonded -GLACIAL TILL-(SM) PID=0ppm Note: Casing refusal at 33.1 ft. Advanced roller bit open hole. 35 8D 2/2 35.00 - 35.17 125(2") 40 9D 0/0 40.00 - 40.00 80(0") RC -12.5 Note: Split-spoon refusal at 40.0 ft 40.1 Top of Probable Bedrock at 40.1 ft Note: Advanced roller bit to 44.0 ft. Recovered bedrock chips in wash water. -PROBABLE BEDROCK- -16.4 44.0 Bottom of Exploration at 44.0 feet below ground surface.	
	6D	24/5	26.00 - 28.00	43/73/28/21	101	156	67				
							87				
							93				
							98				
30	7D	9/7	30.00 - 30.75	30/92(3")			17				
							73				
							271				
							187/1"				
							OPEN				
35	8D	2/2	35.00 - 35.17	125(2")							
40	9D	0/0	40.00 - 40.00	80(0")			RC				
45											
50											

Remarks:
 *Water level taken after completion of boring; casing at 33.1 ft below ground surface.
 Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.

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

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

Page 2 of 2

Boring No.: BB-HSS-202

Maine Department of Transportation				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream		Boring No.: BB-HSS-203				
Soil/Rock Exploration Log US CUSTOMARY UNITS				Location: Hampden, Maine		WIN: 21692.00				
Driller: Maine Department of Transportation		Elevation (ft.): 35.5		Auger ID/OD: --						
Operator: T. Daggett		Datum: NAVD 88		Sampler: Split-Spoon 1.375 in.						
Logged By: M. Snow		Rig Type: CME 45C		Hammer Wt./Fall: SS-140#/30; NW 140#/30						
Date Start/Finish: 7-23-18/7-23-18		Drilling Method: SSA/NW Drive		Core Barrel: --						
Boring Location: STA.34+50.9; 16.1 RT		Casing ID/OD: NW-3.0 in. ID		Water Level*: 11.0 ft						
Hammer Efficiency Factor: 0.928		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test										
Depth (ft.)	Sample Information							Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows			
0							SSA	34.3	-BITUMINOUS CONCRETE-	
	1D	24/6	1.00 - 3.00	4/4/4/5	8	12			Brown, dry, loose, fine to coarse SAND, trace silt, trace clay, trace gravel -FILL-(SW-SM) PID=3.8ppm	1.2
	2D	24/10	3.00 - 5.00	5/3/2/3	5	8			Brown, moist, loose, fine to coarse SAND, little silt, little gravel -FILL-(SW-SM) PID=3.7ppm	
5	3D	24/5	5.00 - 7.00	4/4/3/2	7	11			Brown, wet, loose, fine to coarse SAND, some silt, little gravel -FILL-(SM) PID=1ppm	
10	4D*	24/19	10.20 - 12.20	6/13/10/11	23	36	16		Brown, wet, medium dense, clayey fine SAND, trace medium to coarse sand, trace gravel -FILL-(SC) PID=0ppm *Note: Casing advanced below sample 4D depth. Sample recovered inside casing prior to washing out.	
							10			
							16			
							21			
							30			
15	5D	24/4	15.00 - 17.00	13/7/5/4	12	19	22		Brown, wet, medium dense, clayey fine to coarse SAND, 2-in. thick layer of gravel (crushed rock) -FILL-(SC) PID=1.5ppm	
							22			
							20			
							21			
							30			
20	6D	24/9	20.00 - 22.00	8/8/11/74	19	29	38		Brown, wet, very stiff CLAY, some gravel, little fine to coarse sand, contains brick -FILL-(CL) PID=1ppm	
							34			
							74		Note: Washed ahead of casing from 20.0 to 25.0 ft. Drill action and wash water contents indicate gravel/cobbles from approximately 21.5 to 25.0 ft.	21.5
							174			
25							173			
Remarks: *Water level taken after completion of boring with casing removed. Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.										
Stratification lines represent approximate boundaries between soil types; transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.									Page 1 of 2 Boring No.: BB-HSS-203	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-203 WIN: 21692.00			
Driller: Maine Department of Transportation				Elevation (ft.): 35.5				Auger ID/OD: --			
Operator: T. Daggett				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in.			
Logged By: M. Snow				Rig Type: CME 45C				Hammer Wt./Fall: SS-140#/30; NW 140#/30			
Date Start/Finish: 7-23-18/7-23-18				Drilling Method: SSA/NW Drive				Core Barrel: --			
Boring Location: STA.34+50.9; 16.1 RT				Casing ID/OD: NW-3.0 in. ID				Water Level*: 11.0 ft			
Hammer Efficiency Factor: 0.928				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_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25	7D	6/3	25.00 - 25.50	13/20(0")				9.8		Brown to rust-brown, wet, dense, silty fine to coarse SAND, little gravel (rusty weathered rock pieces) -FILL-(SM) PID=2.5ppm Note: Split-spoon refusal at 25.5 ft. Advanced roller bit to 25.7 ft.	
								7.3		Top of Probable Bedrock at 25.7 ft Note: Advanced roller bit to 28.2 ft, occasional weathered bedrock seams (2-in. thick) -PROBABLE BEDROCK-	
30											
35											
40											
45											
50											
Remarks: *Water level taken after completion of boring with casing removed. Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-HSS-203	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-204 WIN: 21692.00			
Driller: Maine Department of Transportation				Elevation (ft.): 37.9				Auger ID/OD: --			
Operator: T. Daggett				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in.			
Logged By: M. Snow				Rig Type: CME 45C				Hammer Wt./Fall: SS-140#/30; NW 140#/30			
Date Start/Finish: 7-23-18/7-24-18				Drilling Method: SSA/NW Drive				Core Barrel: --			
Boring Location: STA.34+88.1; 55.1 RT				Casing ID/OD: NW-3.0 in. ID				Water Level*: 8.2 ft			
Hammer Efficiency Factor: 0.928				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	37.0		-BITUMINOUS CONCRETE-	
	1D	24/16	1.00 - 3.00	4/4/6/5	10	15				Brown, damp, loose, fine to coarse SAND, little silt, little gravel -FILL-(SW-SM) PID=3ppm	0.9
	2D	24/11	3.00 - 5.00	3/2/2/2	4	6				Brown, moist, very loose, clayey fine to coarse SAND, little gravel, slight petroleum-like odor -FILL-(SC) PID=7ppm	3.0
5	3D	24/5	5.00 - 7.00	2/2/2/2	4	6				Brown, moist, very loose, fine to medium SAND, some silt, little coarse sand, trace gravel, petroleum-like odor -FILL-(SM) PID=3.5ppm	5.0
								29.4			8.5
10	4D	24/16	10.00 - 12.00	3/6/8/3	14	22	5			Brown, moist to wet, medium dense, clayey fine SAND, little coarse to medium sand, trace gravel, trace brick, petroleum-like odor -FILL-(SC) PID=16ppm	
							24				
							26				
							15				
							23				
15	5D/A	24/13	15.00 - 17.00	WOH/6/8/28	14	22	8			Gray, wet, very soft SILT, trace fine sand, slight organic odor -MARINE DEPOSIT-(ML) PID=1ppm	15.0
							9				15.6
							116			Brown, wet, medium dense, fine to coarse SAND, some silt, little gravel, well bonded -GLACIAL TILL-(SM) PID=13ppm	20.6
							70				20.0
							64			-COBBLE-	17.3
20	6D	6/6	20.00 - 20.50	125(6")			NW			Note: Washed ahead of casing from 17.0 to 20.0 ft, and 20.0 to 25.0 ft, casing blows not recorded. Brown to rust-brown, wet, very dense, coarse SAND and fine GRAVEL, some silt, little fine to medium sand, weathered rock fragments, well bonded -GLACIAL TILL-(SM/GM) PID=5ppm	17.9
											17.4
											14.9
										-BOULDER/COBBLES-	20.5
25											23.0
Remarks: *Water level taken after completion of boring with casing removed. Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-HSS-204	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Soudabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-204 WIN: 21692.00																																																																																														
Driller: Maine Department of Transportation				Elevation (ft.) 37.9				Auger ID/OD: --																																																																																														
Operator: T. Daggett				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in.																																																																																														
Logged By: M. Snow				Rig Type: CME 45C				Hammer Wt./Fall: SS-140#/30; NW 140#/30																																																																																														
Date Start/Finish: 7-23-18/7-24-18				Drilling Method: SSA/NW Drive				Core Barrel: --																																																																																														
Boring Location: STA.34+88.1; 55.1 RT				Casing ID/OD: NW-3.0 in. ID				Water Level*: 8.2 ft																																																																																														
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25	7D	2/2	25.00 - 25.17	20(2'')/50(0'')			RC	12.4		Brown to rust-brown, wet, very dense, coarse SAND and fine GRAVEL, little silt, rock lodged in spoon tip -GLACIAL TILL-(SP-SM/GP-GM) PID=0ppm Top of Probable Bedrock at 25.5 ft Note: Advanced roller bit to 29.0 ft. Recovered bedrock chips in wash water. -PROBABLE BEDROCK-																																																																																												
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Grist Mill Bridge #2334 Route 1A/9 Over Souadabscook Stream Location: Hampden, Maine				Boring No.: BB-HSS-205 WIN: 21692.00																							
Driller: Maine Department of Transportation				Elevation (ft.): 36.3				Auger ID/OD: --																							
Operator: T. Daggett				Datum: NAVD 88				Sampler: Split-Spoon 1.375 in.																							
Logged By: M. Snow				Rig Type: CME 45C				Hammer Wt./Fall: SS-140#/30; NW 140#/30																							
Date Start/Finish: 7-24-18/7-24-18				Drilling Method: SSA/NW Drive				Core Barrel: --																							
Boring Location: STA.34+86.5; 11.8 LT				Casing ID/OD: NW-3.0 in. ID				Water Level*: 4.0 ft																							
Hammer Efficiency Factor: 0.928				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				Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected																							
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows																								
0							SSA	35.4		-BITUMINOUS CONCRETE-																					
								34.4		-PROBABLE RECLAIMED BITUMINOUS CONCRETE-	0.9																				
	1D	24/14	2.20 - 4.20	9/10/9/10	19	29				Brown, dry, medium dense, fine to coarse SAND, little silt, little gravel -FILL-(SW-SM) PID=0ppm	1.9																				
5	2D	24/13	5.00 - 7.00	4/3/3/3	6	9				Brown, moist, loose, silty fine to coarse SAND, little gravel, occasional clayey sand layers, trace clay, strong petroleum-like odor -FILL-(SM) PID=0ppm	4.6																				
10	3D	24/12	10.00 - 12.00	2/2/3/4	5	8	6			Brown, wet, loose, silty fine to coarse SAND, little gravel, contains wood -FILL-(SM) PID=0ppm																					
15	4D	24/21	15.40 - 17.40	WOH/WOH/3/2	3	5	9			Gray, wet, very soft, SILT, trace fine sand, trace fine gravel, trace organics -MARINE DEPOSIT-(ML) PID=0ppm	15.0																				
										Note: Probable Glacial Till at 17.0 ft based on casing blows and drill action. -PROBABLE GLACIAL TILL-	17.0																				
20							165/6" RC	16.7		Top of Probable Bedrock at 19.6 ft Note: Advanced roller bit from 19.6 to 23.0 ft. Recovered bedrock chips in wash water, consistent penetration with no weathered zones observed. -PROBABLE BEDROCK-	19.6																				
25								13.3		Bottom of Exploration at 23.0 feet below ground surface.	23.0																				
Remarks: *Water level taken after completion of boring with casing removed. Jar samples screened with a photoionization detector (PID) by Haley & Aldrich upon completion of the subsurface exploration program. Screening was not completed in the field.																															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 1																					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-HSS-205																					

Rock Core Photographs

ROCK CORE PHOTOGRAPHS
GRIST MILL BRIDGE NO. 2334 OVER SOUDABSCOOK STREAM
MAINEDOT WIN 021692.00
HAMPDEN, MAINE



Top Row: BB-HSS-102, Run No. R1, 23.4 (left) to 28.4 (right)
Top Middle Row: BB-HSS-102, Run No. R2, 28.4 (left) to 30.9 (middle); Run No. R3, 30.9 (middle) to 35.9 (right)
Bottom Middle Row: BB-HSS-101, Run No. R1, 27.1 (left) to 30.3 (middle); Run No. R2, 30.3 (middle) to 35.0 (right)
Bottom Row: BB-HSS-101, Run No. R3, 35.0 (left) to 38.0 (right)

APPENDIX B

Laboratory Test Results

Laboratory Testing Summary Sheet

Town(s): Hampden

Project Number: 21692.00

[illegible]

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible).

The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

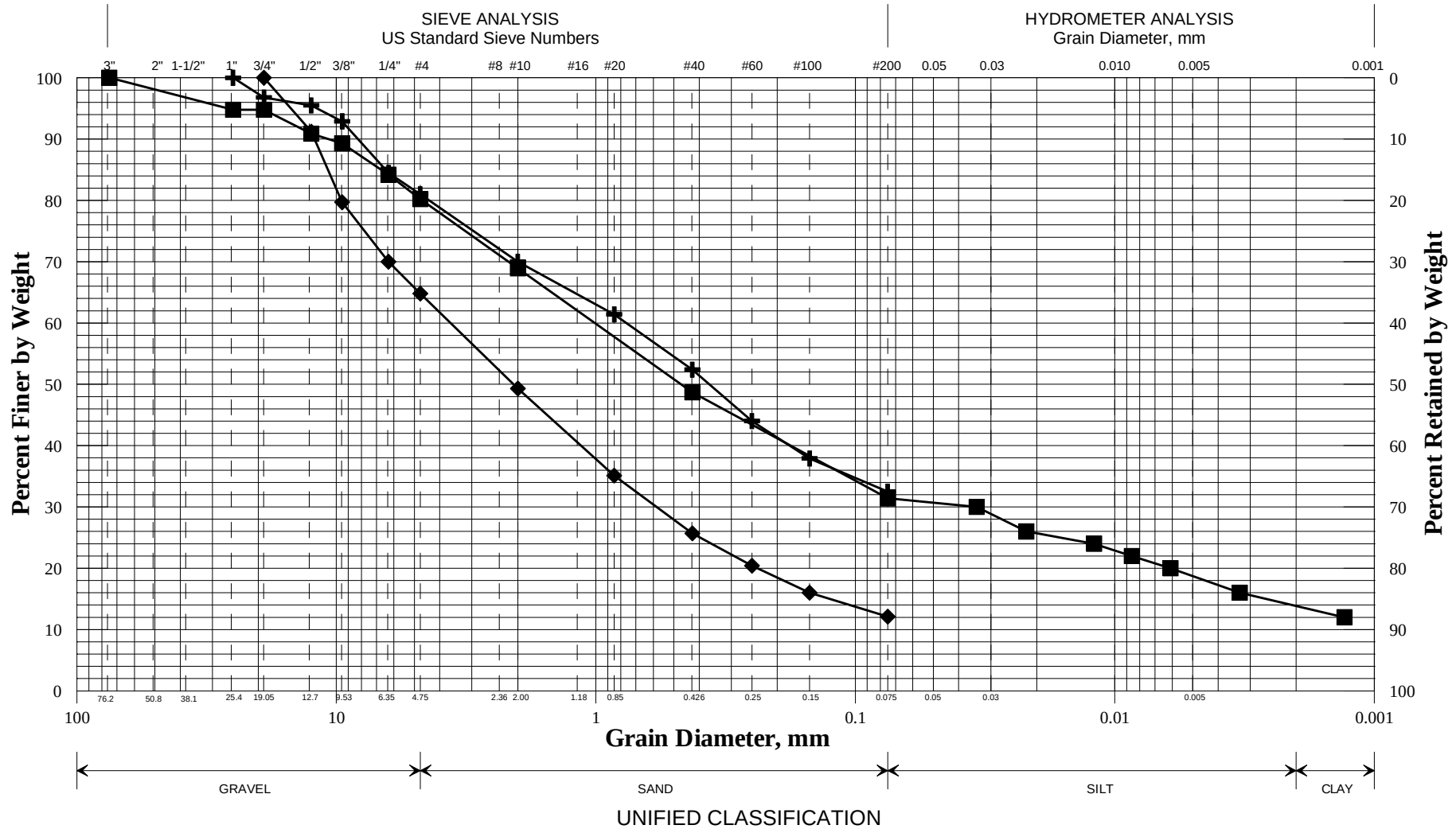
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-HSS-101/2D	33+64.7	11.1 RT	5.0-7.0	SAND, some silt, little gravel.	9.6			
◆	BB-HSS-102/1D	34+44.9	11.3 LT	1.0-3.0	SAND, some gravel, little silt.	3.5			
■	BB-HSS-102/3D	34+44.9	11.3 LT	10.0-12.0	SAND, little gravel, little silt, little clay.	14.4	26	18	8
●									
▲									
×									

WIN	
021692.00	
Town	
Hampden	
Reported by/Date	
WHITE, TERRY A	11/15/2016



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **212329** Boring No./Sample No. **BB-HSS-101/2D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **7/23/2009** Received **7/24/2009**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **33+64.7** Offset, ft: **11.1** RT Dbfg, ft: **5.0-7.0**

WIN/Town **021692.00 - HAMPDEN** Sampler: **ERVIN GIGUERE**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	96.8
½ in. [12.5 mm]	95.5
⅜ in. [9.5 mm]	92.9
¼ in. [6.3 mm]	84.5
No. 4 [4.75 mm]	81.0
No. 10 [2.00 mm]	70.0
No. 20 [0.850 mm]	61.4
No. 40 [0.425 mm]	52.4
No. 60 [0.250 mm]	44.0
No. 100 [0.150 mm]	37.9
No. 200 [0.075 mm]	32.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.6

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

Westbrook project.

AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG**

Date Reported: **7/30/2009**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **212330** Boring No./Sample No. **BB-HSS-102/1D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **7/22/2009** Received **7/24/2009**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **34+44.9** Offset, ft: **11.3** LT Dbfg, ft: **1.0-3.0**

WIN/Town **021692.00 - HAMPDEN** Sampler: **ERVIN GIGUERE**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	91.2
⅜ in. [9.5 mm]	79.7
¼ in. [6.3 mm]	70.0
No. 4 [4.75 mm]	64.8
No. 10 [2.00 mm]	49.3
No. 20 [0.850 mm]	35.1
No. 40 [0.425 mm]	25.7
No. 60 [0.250 mm]	20.4
No. 100 [0.150 mm]	16.0
No. 200 [0.075 mm]	12.1

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	3.5

Consolidation (T 216)

Trimmings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

Asphalt through out sample. Westbrook project.

AUTHORIZATION AND DISTRIBUTION

Reported by: **BRIAN FOGG**

Date Reported: **7/30/2009**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No. **212331** Boring No./Sample No. **BB-HSS-102/3D** Sample Description **GEOTECHNICAL (DISTURBED)** Sampled **7/22/2009** Received **7/24/2009**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **34+44.9** Offset, ft: **11.3** LT Dbfg, ft: **10.0-12.0**

WIN/Town **021692.00 - HAMPDEN** Sampler: **ERVIN GIGUERE**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	94.8
¾ in. [19.0 mm]	94.8
½ in. [12.5 mm]	90.9
⅜ in. [9.5 mm]	89.3
¼ in. [6.3 mm]	84.2
No. 4 [4.75 mm]	80.2
No. 10 [2.00 mm]	68.9
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	48.7
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	31.4
[0.0340 mm]	30.0
[0.0219 mm]	26.0
[0.0120 mm]	24.0
[0.0086 mm]	22.0
[0.0061 mm]	20.0
[0.0033 mm]	16.0
[0.0013 mm]	12.0

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	26
Plastic Limit (T 90), %	18
Plasticity Index (T 90), %	8
Specific Gravity, Corrected to 20°C (T 100)	2.69
Loss on Ignition, % (T 267)	
Water Content (T 265), %	14.4

Consolidation (T 216)

Trimming, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

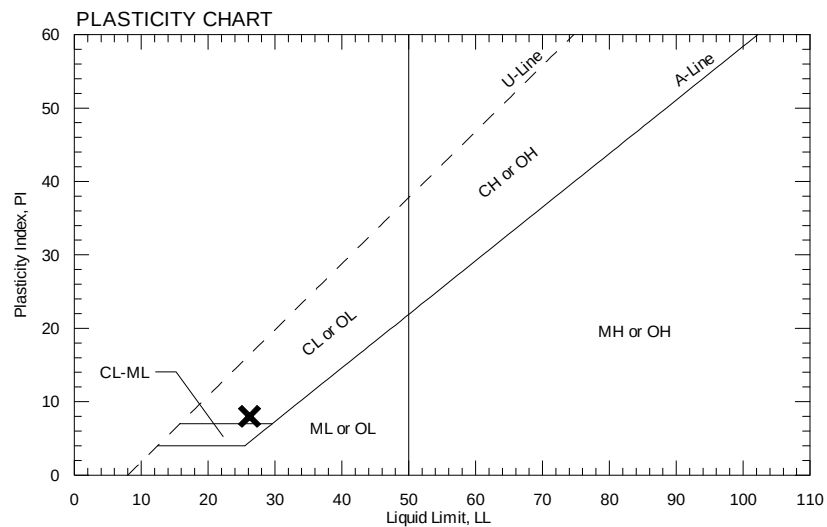
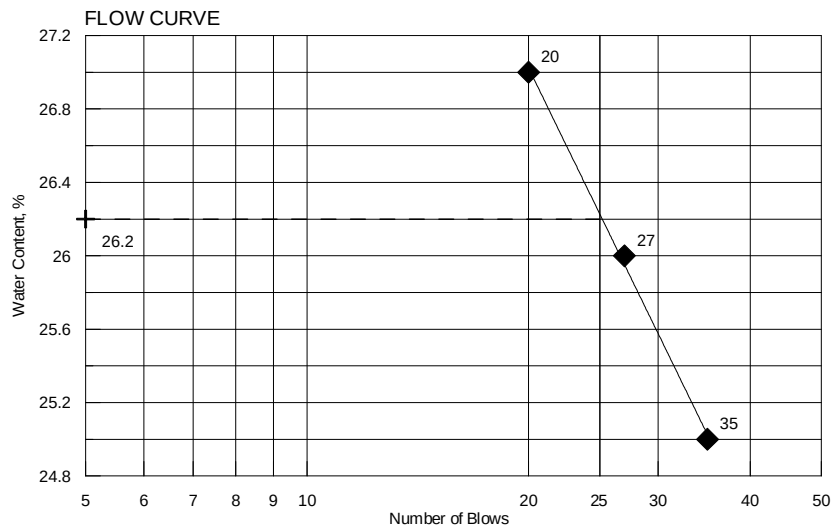
Reported by: **BRIAN FOGG**

Date Reported: **8/4/2009**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Hampden	Reference No.	212331
WIN	021692.00	Water Content, %	14.4
Sampled	7/22/2009	Liquid Limit @ 25 blows (T 89), %	26
Boring No./Sample No.	BB-HSS-102/3D	Plastic Limit (T 90), %	18
Station	34+44.9	Plasticity Index (T 90), %	8
Depth	10.0-12.0	Tested By	BBURR

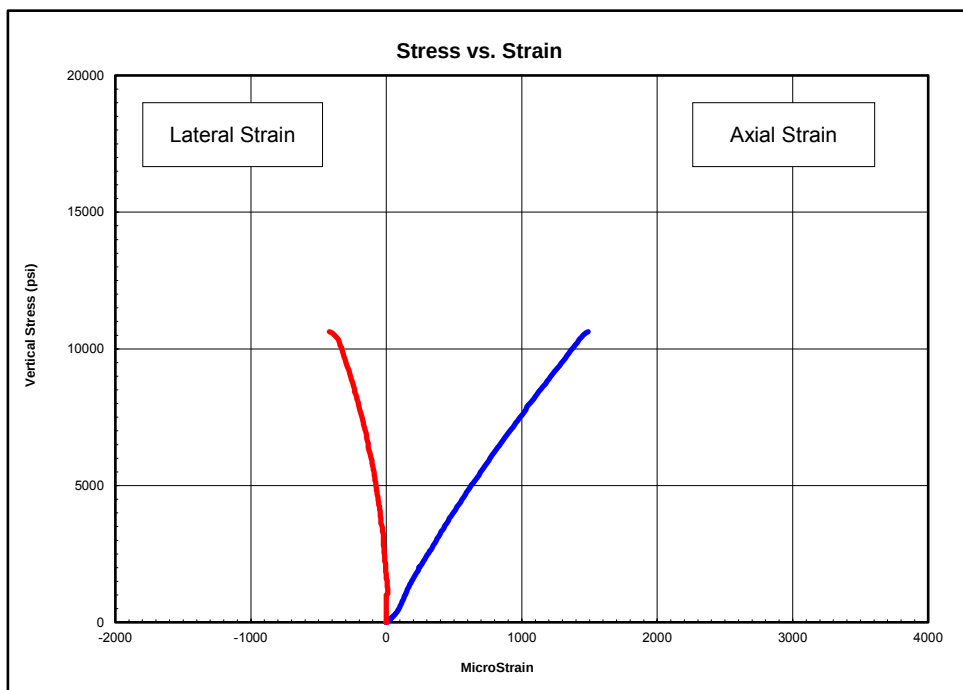
State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet





Client:	Maine Department of Transportation
Project Name:	Grist Mill Bridge #2334
Project Location:	Hampden, ME
GTX #:	307909
Test Date:	4/10/2018
Tested By:	pas/trm
Checked By:	jsc
Boring ID:	BB-HSS-101
Sample ID:	R2
Depth, ft:	30.64-31.0
Sample Type:	rock core
Sample Description:	See photographs Intact material and Discontinuity failure

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



Peak Compressive Stress: 10,627 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1100-3900	8,300,000	0.16
3900-6700	7,250,000	0.26
6700-9600	6,560,000	0.37

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.

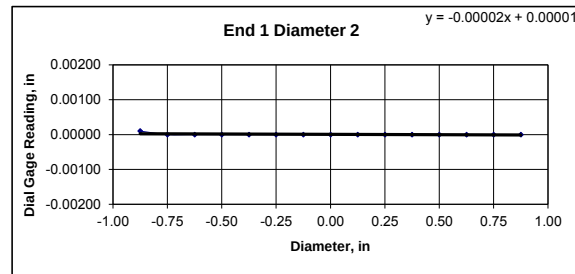
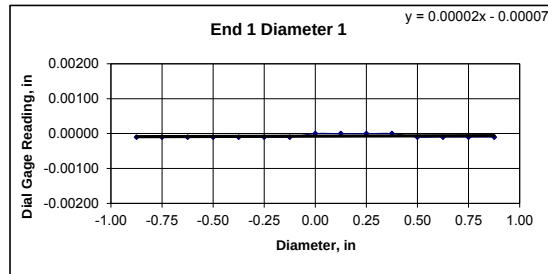


Client:	Maine Department of Transportation	Test Date:	4/6/2018
Project Name:	Grist Mill Bridge #2334	Tested By:	pas/trn
Project Location:	Hampden, ME	Checked By:	jsc
GTx #:	307909		
Boring ID:	BB-HSS-101		
Sample ID:	R2		
Depth:	30.64-31.0 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.10	4.11	4.11	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.98	1.97	1.98	Straightness Tolerance Met? YES	
Specimen Mass, g:	558.11				
Bulk Density, lb/ft ³ :	169				
Length to Diameter Ratio:	2.1				
		Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

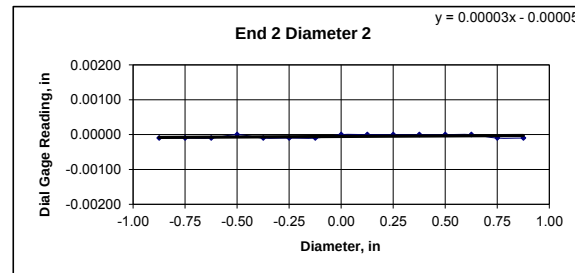
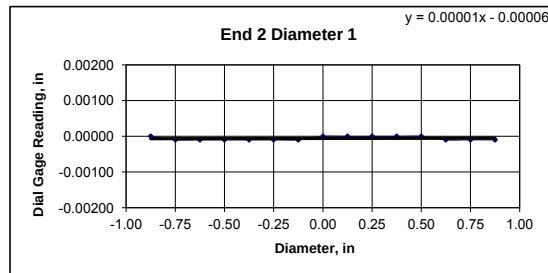
END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	0.00000	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Difference between max and min readings, in:															
0° = 0.0001 90° = 0.0001															
Maximum difference must be < 0.0020 in. Difference = ± 0.00005															
Flatness Tolerance Met? YES															



DIAMETER 1

End 1:		
Slope of Best Fit Line	0.00002	
Angle of Best Fit Line:	0.00098	
End 2:		
Slope of Best Fit Line	0.00001	
Angle of Best Fit Line:	0.00049	
Maximum Angular Difference:	0.00049	

Parallelism Tolerance Met? YES
Spherically Seated



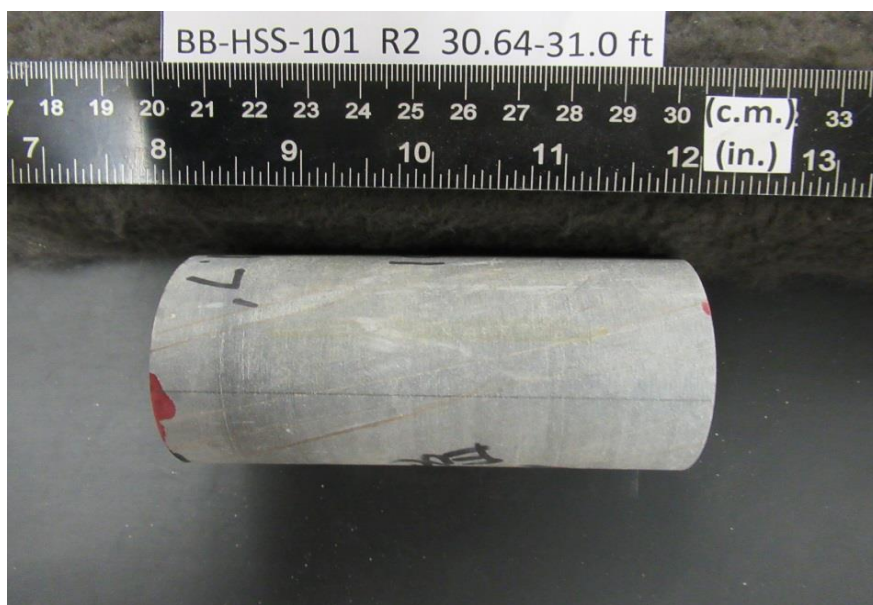
DIAMETER 2

End 1:		
Slope of Best Fit Line	0.00002	
Angle of Best Fit Line:	0.00115	
End 2:		
Slope of Best Fit Line	0.00003	
Angle of Best Fit Line:	0.00180	
Maximum Angular Difference:	0.00065	

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq$ 0.25°
Diameter 1, in	0.00010		1.975	0.00005	0.003	YES	
Diameter 2, in (rotated 90°)	0.00010		1.975	0.00005	0.003	YES	Perpendicularity Tolerance Met? YES
END 2							
Diameter 1, in	0.00010		1.975	0.00005	0.003	YES	
Diameter 2, in (rotated 90°)	0.00010		1.975	0.00005	0.003	YES	

Client:	Maine Department of Transportation
Project Name:	Grist Mill Bridge #2334
Project Location:	Hampden, ME
GTX #:	307909
Test Date:	4/6/2018
Tested By:	pas/trm
Checked By:	jsc
Boring ID:	BB-HSS-101
Sample ID:	R2
Depth, ft:	30.64-31.0 ft



After cutting and grinding

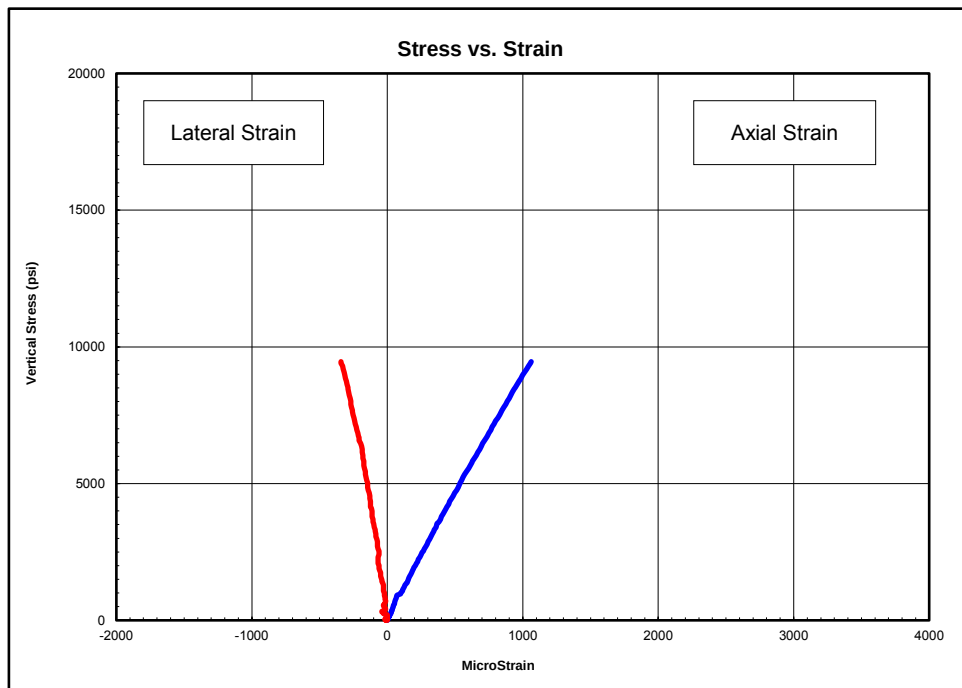


After break



Client:	Maine Department of Transportation
Project Name:	Grist Mill Bridge #2334
Project Location:	Hampden, ME
GTX #:	307909
Test Date:	4/10/2018
Tested By:	pas/trm
Checked By:	jsc
Boring ID:	BB-HSS-102
Sample ID:	R3
Depth, ft:	34.5-34.9
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

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



Peak Compressive Stress: 9,460 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
900-3500	9,140,000	0.27
3500-6000	8,860,000	0.29
6000-8500	8,590,000	0.41

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.

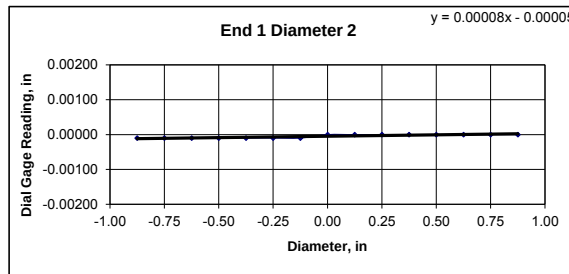
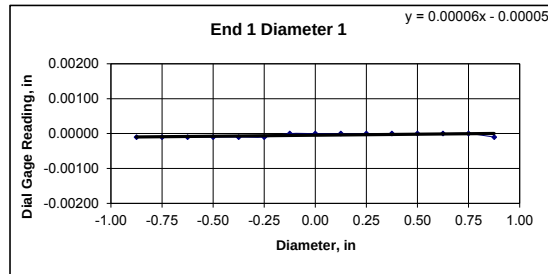


Client:	Maine Department of Transportation	Test Date:	4/6/2018
Project Name:	Grist Mill Bridge #2334	Tested By:	pas/trm
Project Location:	Hampden, ME	Checked By:	jsc
GTX #:	307909		
Boring ID:	BB-HSS-102		
Sample ID:	R3		
Depth:	34.5-34.9 ft		
Visual Description:	See photographs		

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

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.14	4.15	4.15	Maximum difference must be < 0.020 in.	
Specimen Diameter, in:	1.98	1.98	1.98	Straightness Tolerance Met? YES	
Specimen Mass, g:	556.71				
Bulk Density, lb/ft ³ :	166				
Length to Diameter Ratio:	2.1				
		Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

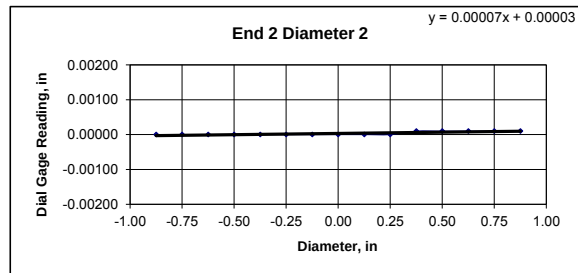
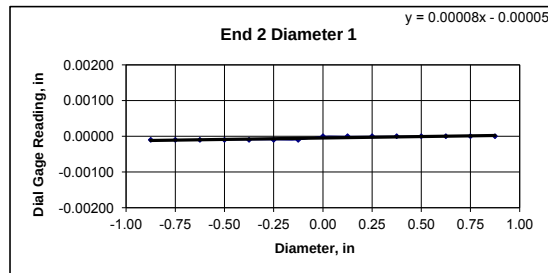
END FLATNESS AND PARALLELISM (Procedure FP1)														
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:														
0° = 0.00010 90° = 0.00010														
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010
Difference between max and min readings, in:														
0° = 0.0001 90° = 0.0001														
Maximum difference must be < 0.0020 in. Difference = ± 0.00005														
Flatness Tolerance Met? YES														



DIAMETER 1

End 1:		
Slope of Best Fit Line	0.00006	
Angle of Best Fit Line:	0.00327	
End 2:		
Slope of Best Fit Line	0.00008	
Angle of Best Fit Line:	0.00458	
Maximum Angular Difference:	0.00131	

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:		
Slope of Best Fit Line	0.00008	
Angle of Best Fit Line:	0.00458	
End 2:		
Slope of Best Fit Line	0.00007	
Angle of Best Fit Line:	0.00409	
Maximum Angular Difference:	0.00049	

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1)						(Calculated from End Flatness and Parallelism measurements above)	
END 1		Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?	Maximum angle of departure must be $\leq 0.25^\circ$
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES		
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES		
END 2							
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES		
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES		
						Perpendicularity Tolerance Met?	YES

Client:	Maine Department of Transportation
Project Name:	Grist Mill Bridge #2334
Project Location:	Hampden, ME
GTX #:	307909
Test Date:	4/6/2018
Tested By:	pas/trm
Checked By:	jsc
Boring ID:	BB-HSS-102
Sample ID:	R3
Depth, ft:	34.5-34.9



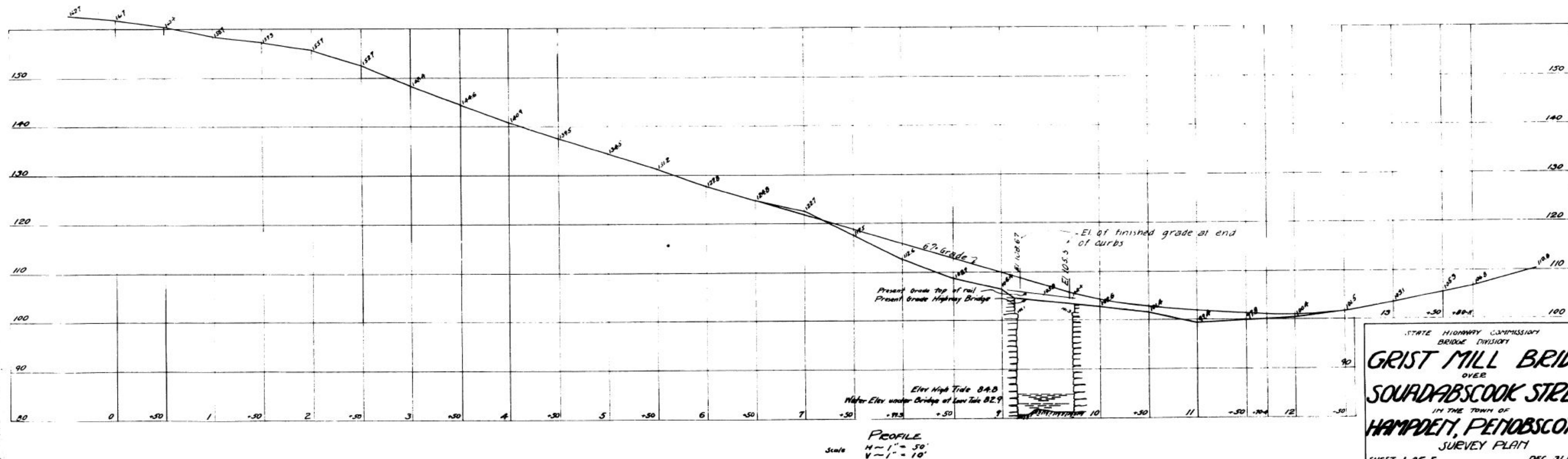
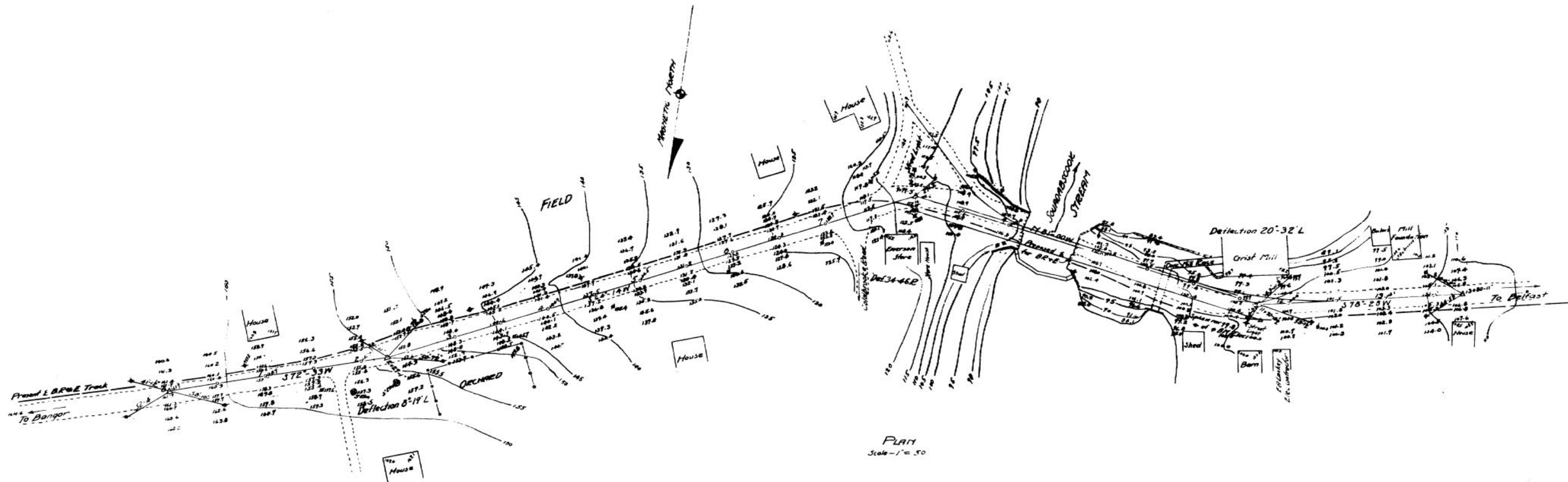
After cutting and grinding



After break

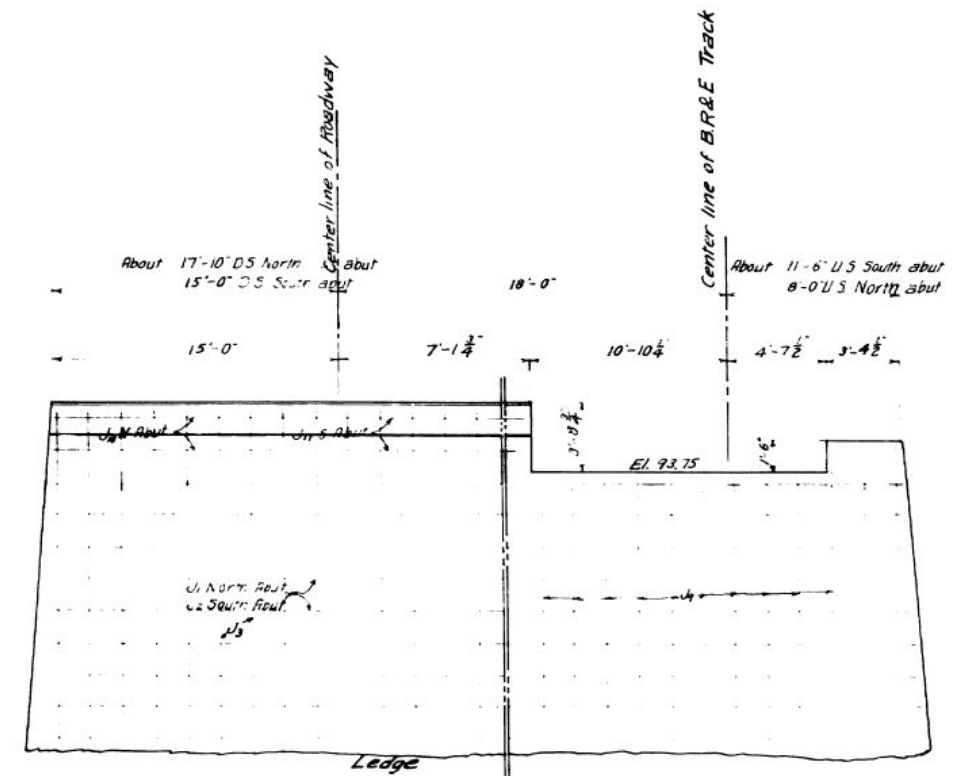
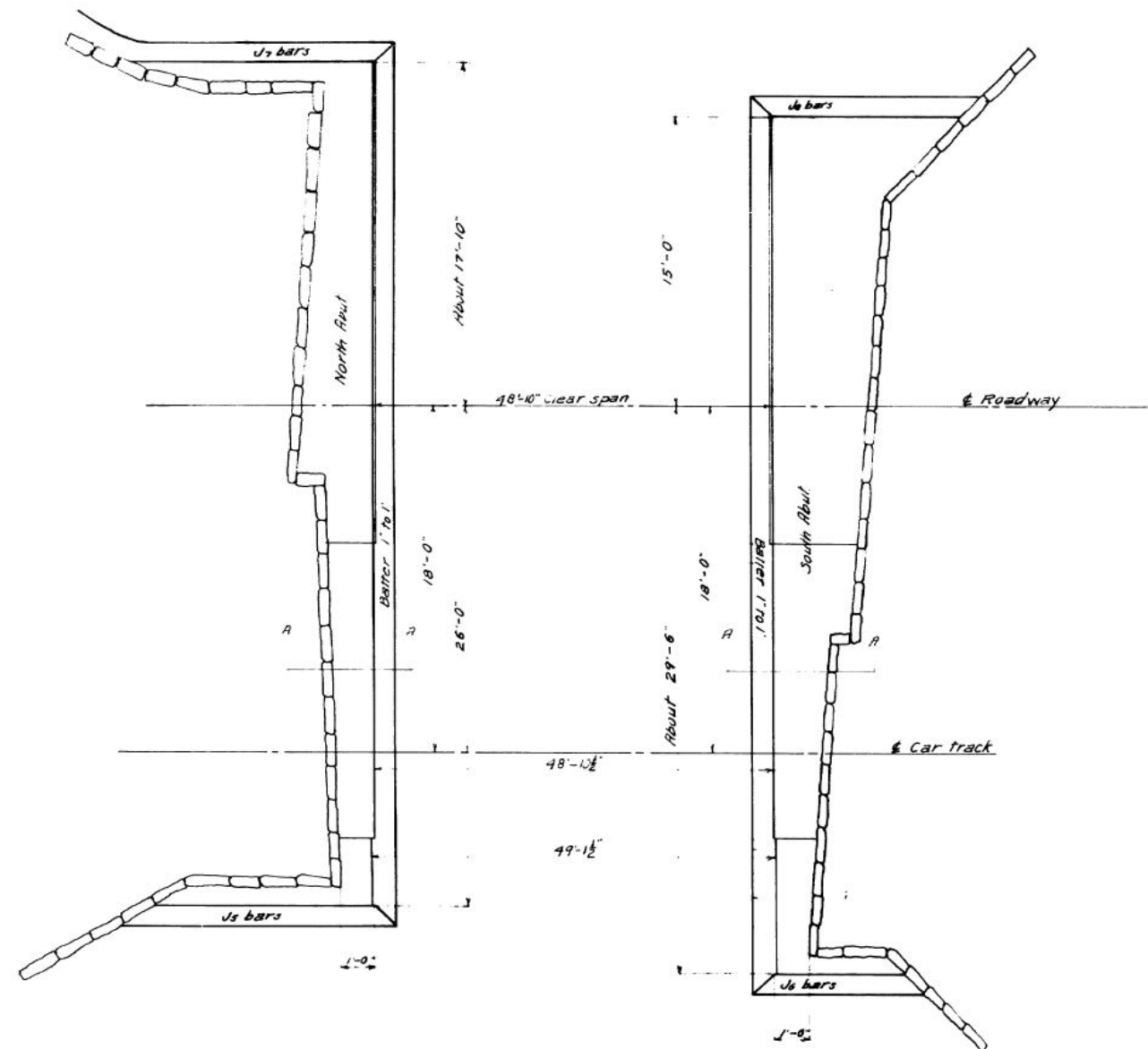
APPENDIX C

Historic Bridge Drawings



STATE HIGHWAY COMMISSION
BRIDGE DIVISION
GRIST MILL BRIDGE
OVER
Soudabscook Stream
IN THE TOWN OF
HAMPDEN, PENOBSCOT CO.
SURVEY PLAN
SHEET 1 OF 5
DEC. 31/1924

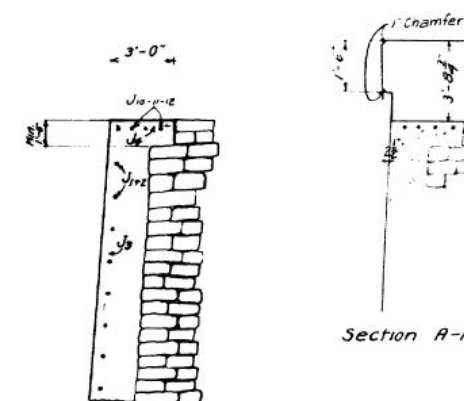
Note Jacketing of wall to extend along the face of wall to a location where the $1\frac{1}{2}:1$ slope of the approach can be economically constructed



J bars to be spaced 18" str.-ctr. and 2" from the form

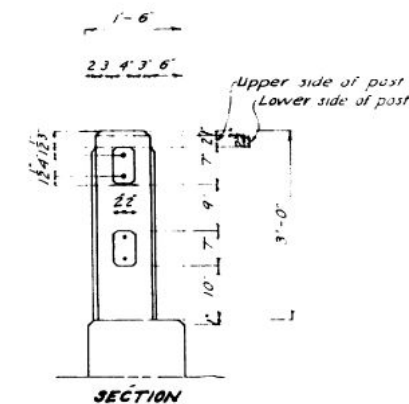
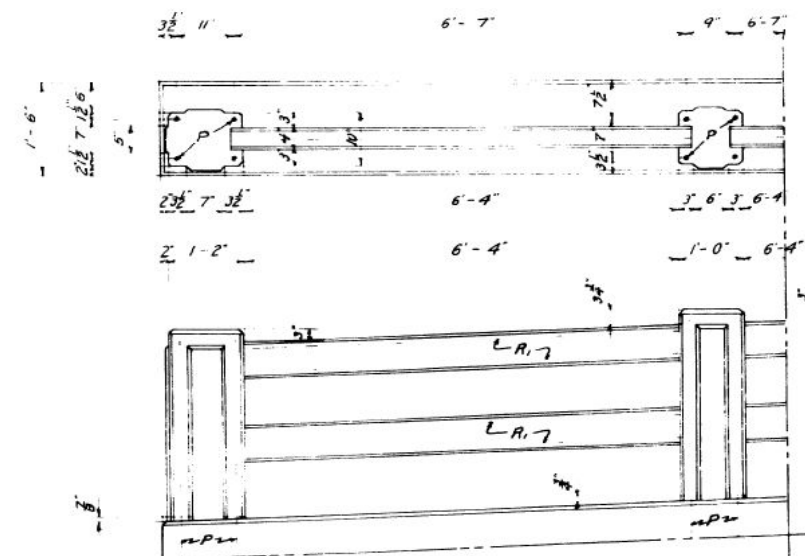
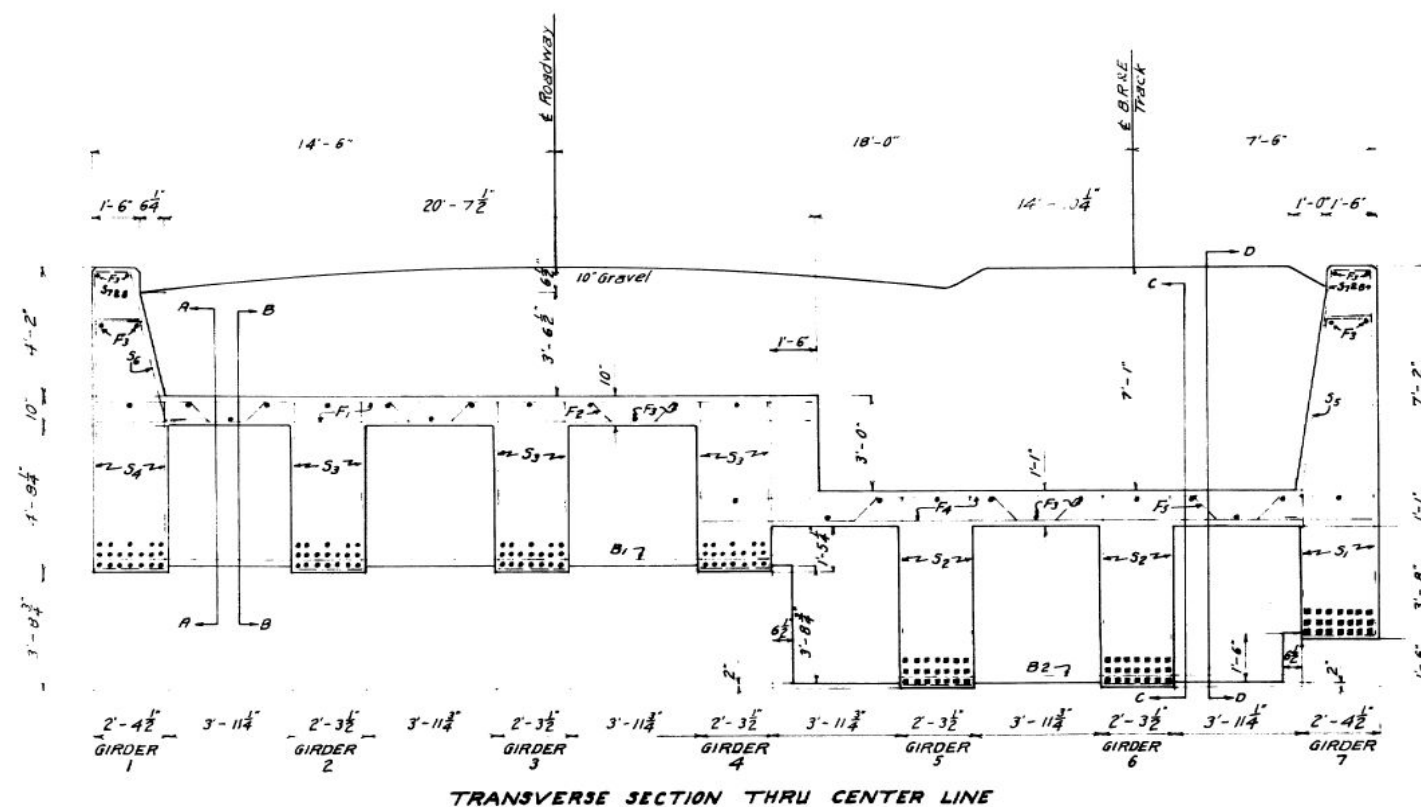
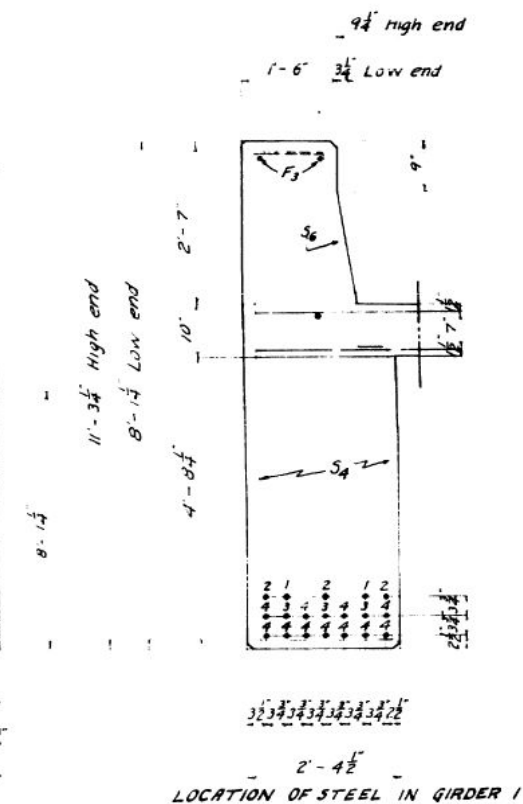
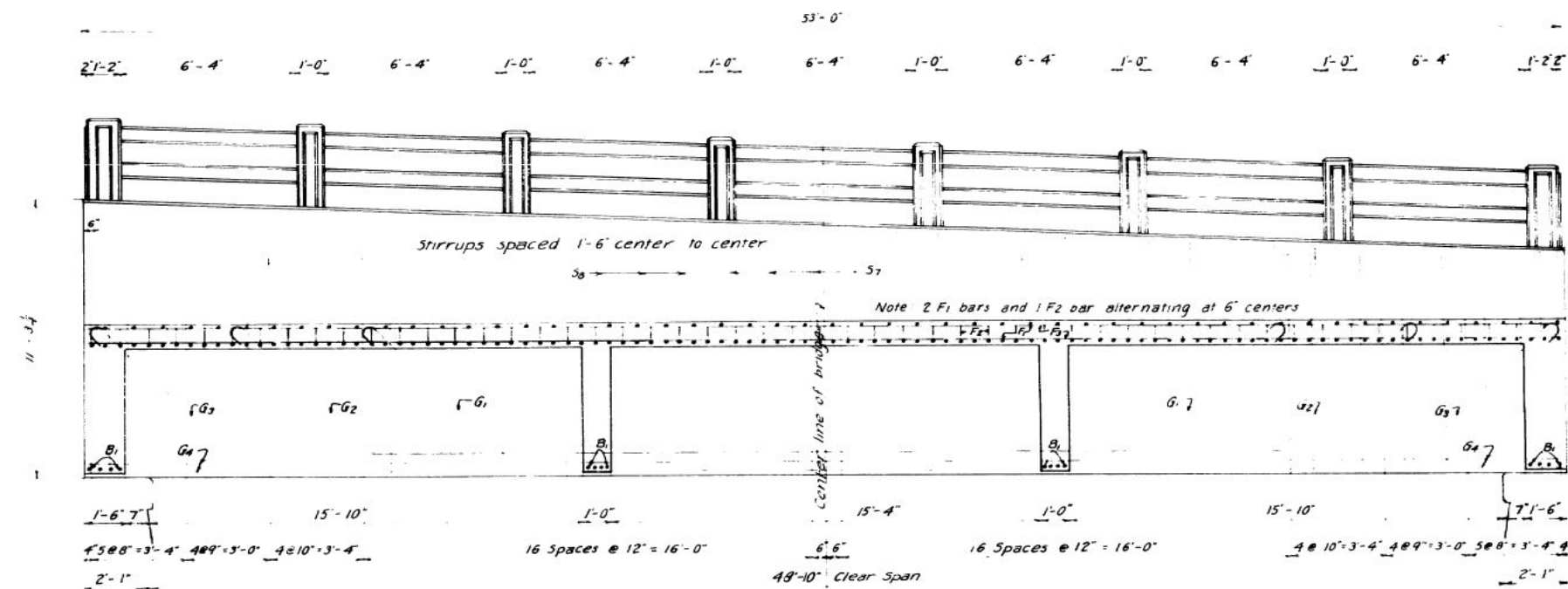
Mark	Size	No. Rebar	Length	Remarks
J1	5/8"	20	22'-10"	Straight
J2	"	20	23'-2"	"
J3	"	50	16'-5"	"
J4	"	50	4'-0"	Bent in field
J5	"	11	12'-0"	" (2'-0" to be bent around to tie on to steel jacket
J6	"	11	9'-0"	"
J7	"	13	14'-0"	"
J8	"	13	11'-0"	"
J9	"	20	12'-8"	Straight
J10	"	6	22'-0"	"
J11	"	6	24'-10"	"
J12	"	6	17'-0"	"

Steel to be plain rounds structural grade



MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
GRIST MILL BRIDGE
OVER
SQUADABSCOOK STREAM
IN THE TOWN OF
HAMPDEN, PENOBSCOT, CO.

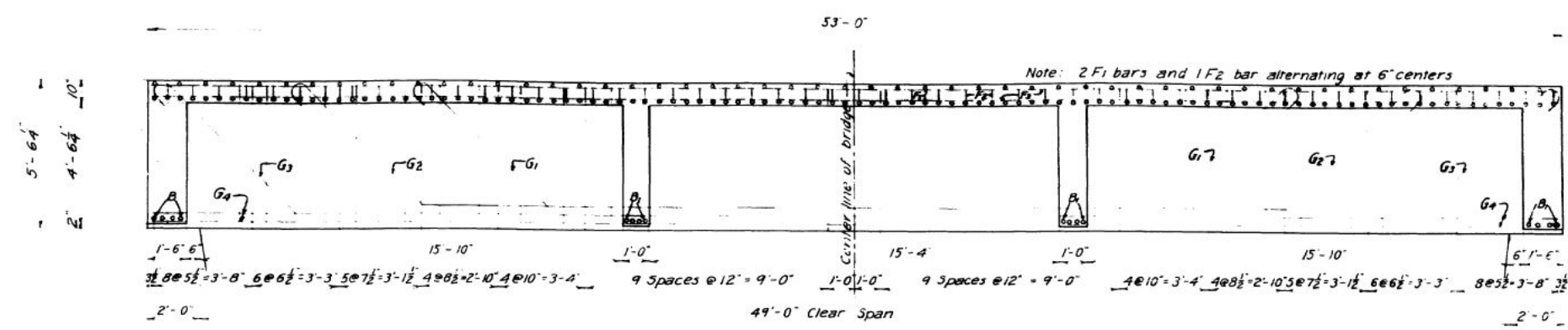
SHEET 2 of 5 AUGUSTA, ME. JAN. 8, 1928.



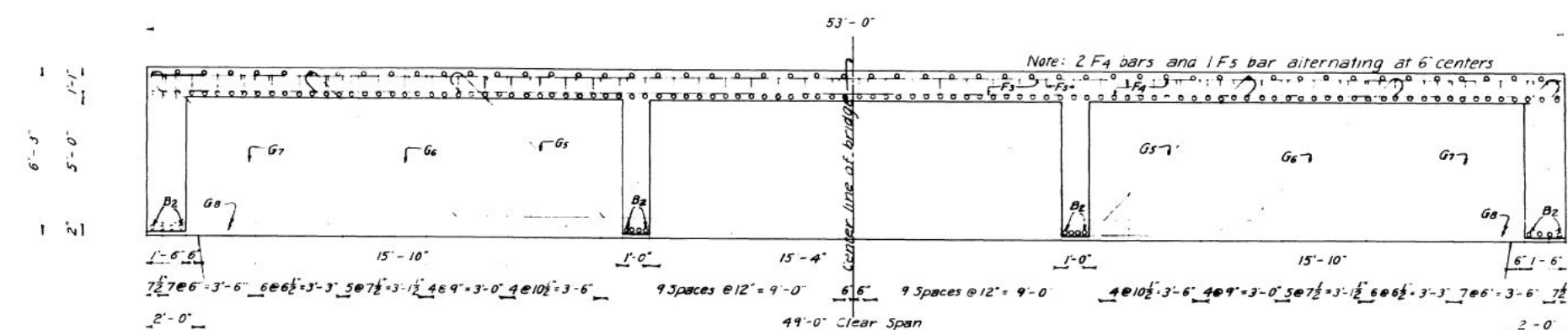
RAIL DETAIL
RAIL CONSTRUCTION

Curb and slab to be cast together. Steel for posts to be set in curb. Precast rail bars in lengths of 6'-9". Place rail bars in position with ends projecting into post forms 25". Wrap end 6" with 2 thicknesses of 2 ply roofing felt. Fold in ends and when post forms are removed cut away all roofing felt left exposed. Panels on posts to be $\frac{5}{8}$ " thick. Chamfer all exposed edges of concrete $\frac{3}{4}$ " unless otherwise indicated.

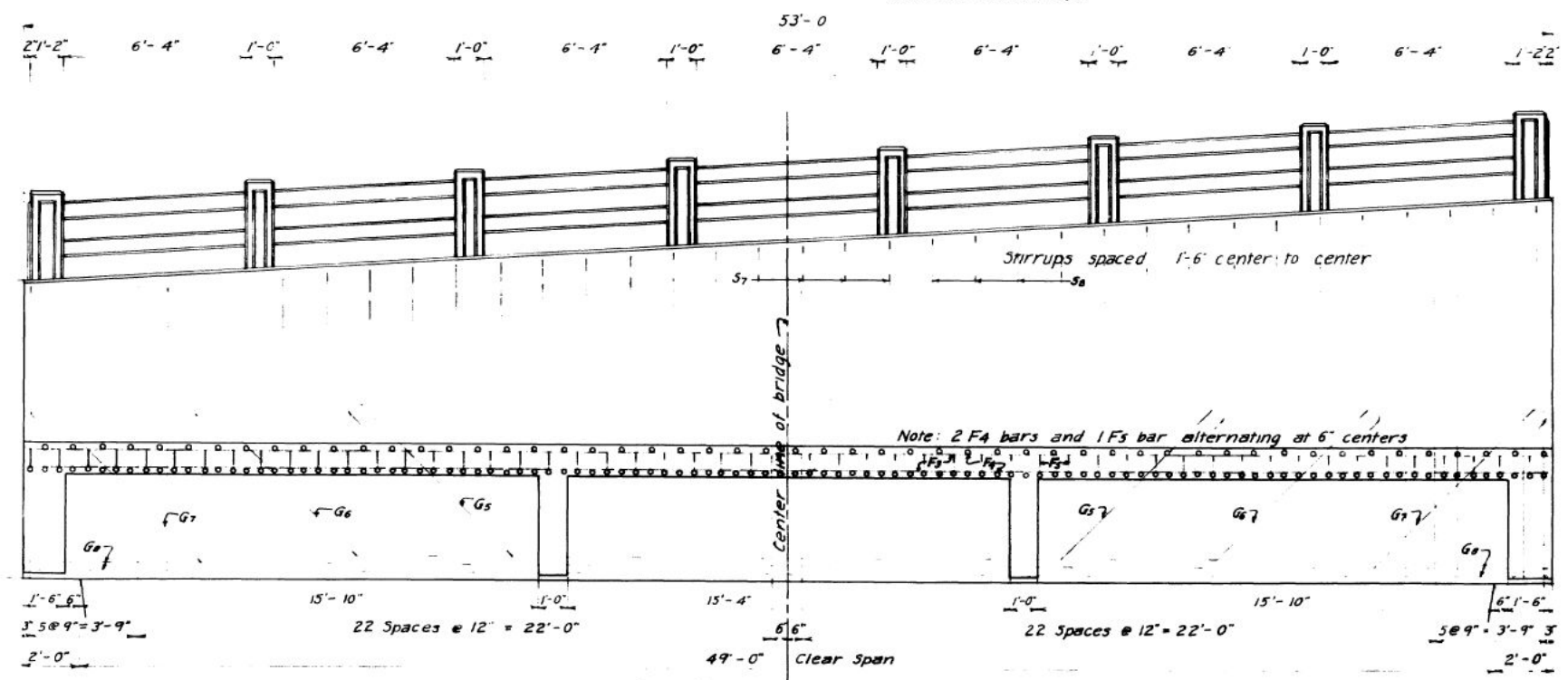
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
GRIST MILL BRIDGE
OVER
SOUADABSCOOK STREAM
IN THE TOWN OF
HAMPDEN, PENOBSCOT, CO.
SHEET 3 of 5 AUGUSTA, MAINE Jan 20, 1925



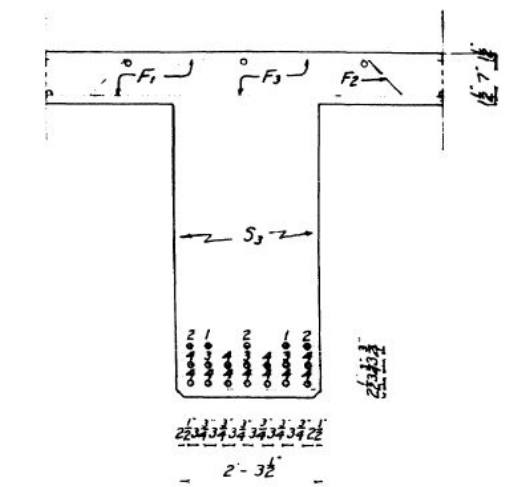
LONGITUDINAL SECTION B-B
SHOWING GIRDERS 2, 3, 4



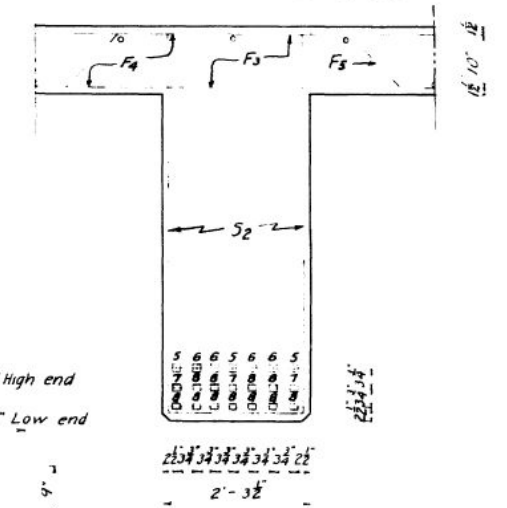
LONGITUDINAL SECTION C-C
SHOWING GIRDERS 5, 6



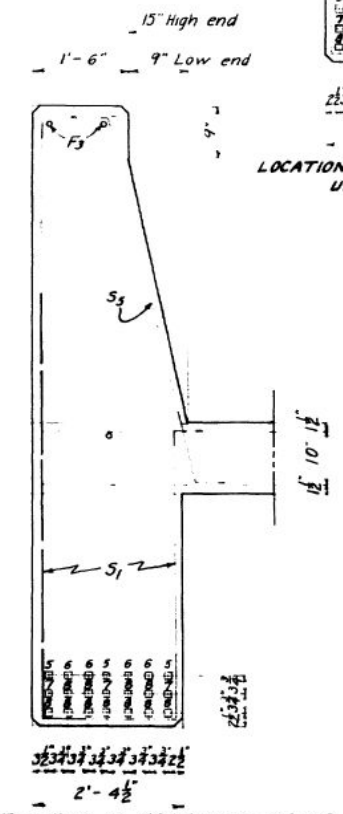
LONGITUDINAL SECTION D-D
SHOWING GIRDER 7



LOCATION OF STEEL IN INTERIOR GIRDERS 2, 3, 4
DOWNSTREAM SIDE OF BRIDGE

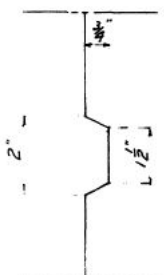


LOCATION OF STEEL IN INTERIOR GIRDERS 5, 6
UPSTREAM SIDE OF BRIDGE

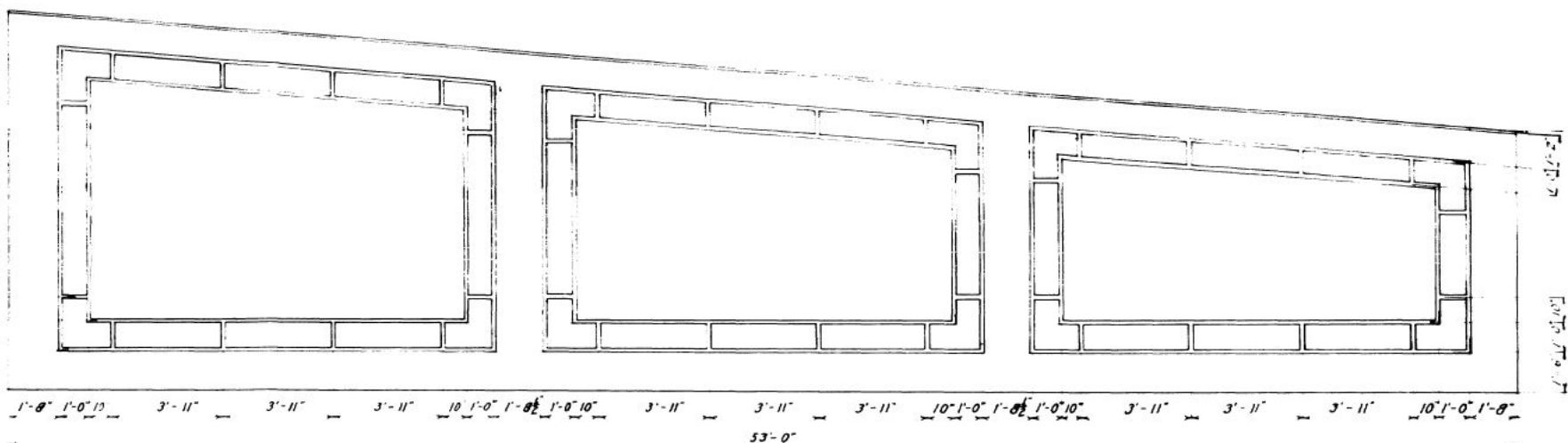


LOCATION OF STEEL IN UPSTREAM GIRDER 7

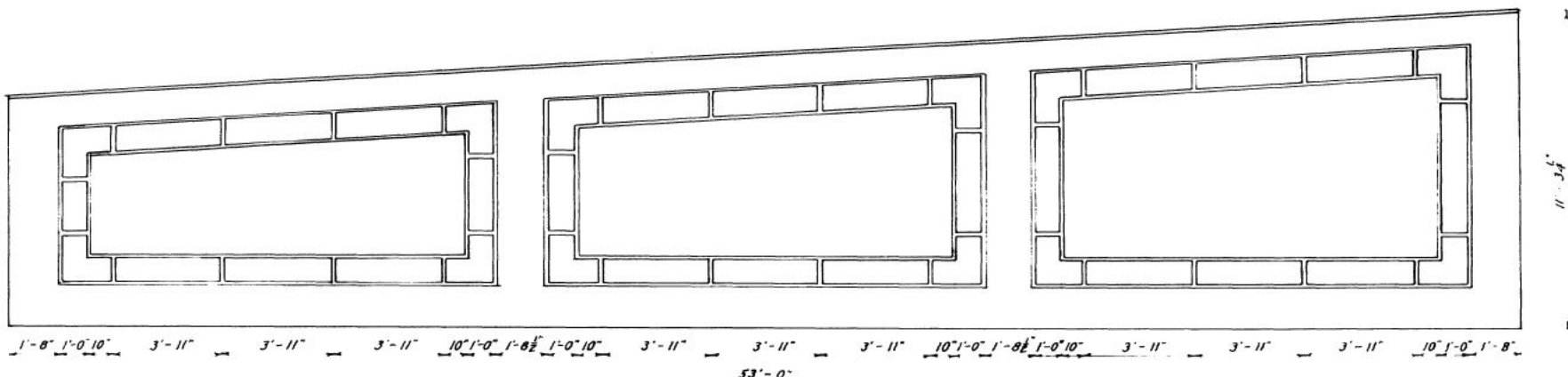
MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
GRIST MILL BRIDGE
OVER
SQUADABSCOOK STREAM
IN THE TOWN OF
HAMPDEN, PENOBSCOT, CO.
SHEET 4 OF 5 AUGUSTA, MAINE Jan. 23, 1925



PANEL DETAIL



PANEL DETAIL UPSTREAM SIDE OF BRIDGE



PANEL DETAIL DOWNSTREAM SIDE OF BRIDGE

BENT BARS

Mark	Size	No Req'd	A	B	C	D	Total length	Location
G ₁	1 1/4"	8	23'-11 1/2"	4'-7"	4'-2 1/2"	6'-2 1/2"	38'-11 1/2"	D.S. Girders 1, 2, 3, 4
G ₂	1 1/4"	12	33'-1 1/2"	4'-7"	4'-2 1/2"	6'-2 1/2"	48'-1 1/2"	"
G ₃	1 1/4"	12	42'-3 1/2"	4'-10 1/2"	4'-6 1/2"	6'-8"	58'-2 1/2"	"
G ₅	1 1/4"	9	19'-7"	5'-3 1/2"	4'-11 1/2"	7'-3 1/2"	36'-0 1/2"	U.S. Girders 5, 6, 7
G ₆	1 1/4"	12	30'-2 1/2"	5'-3 1/2"	4'-11 1/2"	7'-3 1/2"	47'-3 1/2"	"
G ₇	1 1/4"	9	40'-10"	5'-7 1/2"	5'-3 1/2"	7'-8 1/2"	58'-9 1/2"	"

2'-10 1/2" 7'-1 1/2" 3'-8 1/2" 7'-1 1/2" 3'-8 1/2" 7'-1 1/2" 4'-4"

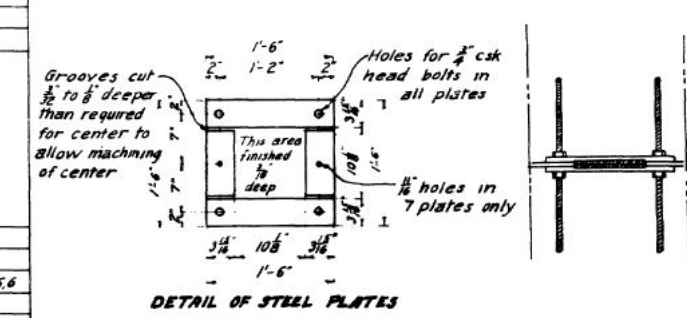
Mark	Size	No. Req'd	Total Length	Location
F ₂	3/8"	53	23'-8 1/2"	Downstream Floor Slab
4'-0 1/2" 10" 3'-5 1/2" 10'-1 1/2" 10" 3'-5 1/2" 10'-1 1/2" 10" 2'-8 1/2"				

Mark	Size	No. Req'd	Total Length	Location
F ₅	3/8"	53	21'-9 1/2"	Upstream Floor Slab

STIRRUPS									
Mark	Size	No. Req'd	Total Length		Location				
S3	3/8"	225	13'-7 1/2"		Downstream Interior Girders 2, 3, 4				
S4	3/8"	60	16'-10 1/2"		Downstream Girder 1				
S6	3/8"	60	5'-0 1/2"						
S7	3/8"	30	7'-3"		Girders 1 and 7				
Mark	Size	No. Req'd	Total Length		Location				
S1	3/4"	56	18'-3 1/2"		Upstream Girder 7				
S2	3/4"	144	15'-0 1/2"		Upstream Interior Girders 5, 6				
S5	3/4"	56	8'-4"		Upstream Girder 7				
S8	1/2"	30	9'-3"		Girders 1 and 7				

STRAIGHT BARS									
Mark	Size	No. Req'd	Length		Location				
G4	1 1/4"	44	52'-7"		Downstream Girders 1, 2, 3, 4				
G8	1 1/4"	33	52'-7"		Upstream Girders 5 & 6, 7				
F1	3/8"	106	22'-3 1/2"		Downstream Floor Slab				
F3	3/8"	33	52'-9"		Floor Slabs and Curbs				
F4	3/8"	106	19'-11 1/2"		Upstream Floor Slab				
B1	3/8"	16	22'-3"		Downstream Cross Beams				
B2	3/8"	16	15'-2"		Upstream Cross Beams				
P1	3/8"	64	4'-0"		Posts				
R1	3/8"	56	6'-8"		Railing				

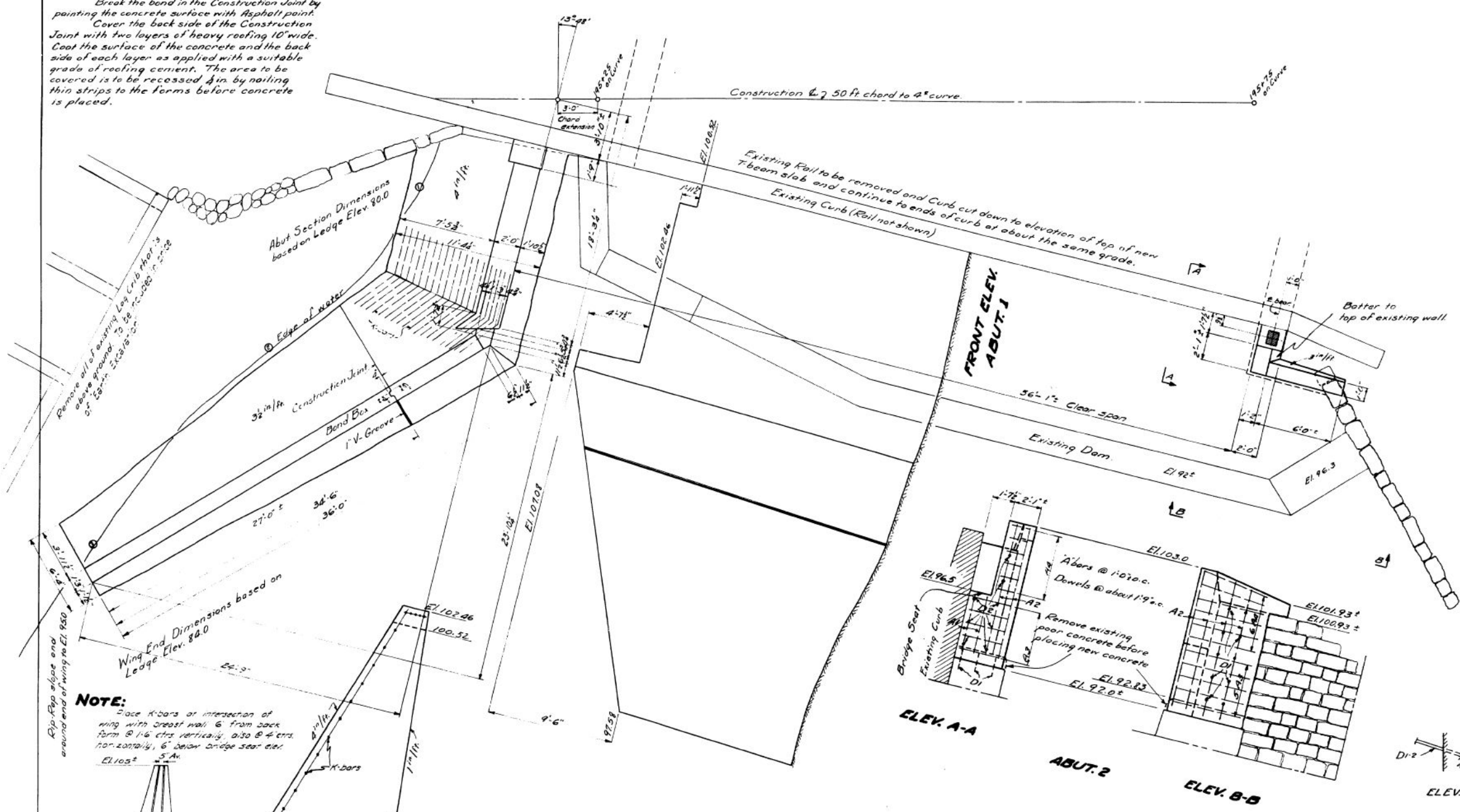
Note: All steel dimensions are to center line of bar.
All steel to be structural grade.
Bars G5, G6, G7 and G8 are plain square bars.
All other bars are plain rounds.



REQUIRED
14 Steel plates 18"x1'-6"
14 Phosphor bronze plates 10"x10"
36-3/4" csk head bolts
12" long threaded full length with one nut each.
Bronze plates to be true and smooth either by rolling or by a finishing cut.

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION
GRIST MILL BRIDGE
OVER
SQUADABSCOOK STREAM
IN THE TOWN OF
HAMPDEN, PENOBSCOT, CO.
SHEET 5 of 5 AUGUSTA, MAINE Jan. 26, 1925

Note:
Break the bond in the Construction Joint by painting the concrete surface with Asphalt paint.
Cover the back side of the Construction Joint with two layers of heavy roofing 10' wide.
Coat the surface of the concrete and the back side of each layer as applied with a suitable grade of roofing cement. The area to be covered is to be recessed 1/4" by nailing thin strips to the forms before concrete is placed.



REIN. STEEL

See following sheet for Bar shapes

MARK	SIZE	NO.	A	R	LENGTH	REMARKS
F1	3/8"	2	13'-4 1/2"	3 1/2"	16'-1"	Slab-Transv.
F2	"	"	13'-12"	"	15-11"	"
F3	"	"	12'-10 1/2"	"	15-8"	"
F4	"	"	12'-8"	"	15-5"	"
F5	"	"	12'-5 1/2"	"	15-2"	"
F6	"	"	12'-2 1/2"	"	15-0"	"
F7	"	"	11'-10 1/2"	"	14-9"	"
F8	"	"	11'-8"	"	14-6"	"
F9	"	"	11'-5 1/2"	"	14-3"	"
F10	"	"	11'-3"	"	14-1"	"
F11	"	"	11'-0 1/2"	"	13-10"	"
F12	"	"	10'-10"	"	13-7"	"
F13	"	"	10'-7 1/2"	"	13-4"	"
F14	"	"	10'-5 1/2"	"	13-2"	"
F15	"	"	10'-3"	"	12-11"	"
F16	"	"	9'-11"	"	12-8"	"
F17	"	"	9'-8 1/2"	"	12-5"	"
F18	"	"	9'-6 1/2"	"	12-3"	"
F19	"	"	9'-4 1/2"	"	12-0"	"
F20	"	"	9'-2 1/2"	"	11-9"	"
F21	"	"	8'-10 1/2"	"	11-6"	"
F22	"	"	8'-8 1/2"	"	11-4"	"
F23	"	"	8'-6 1/2"	"	11-1"	"
F24	"	"	8'-4 1/2"	"	10-10"	"
F25	"	"	8'-2 1/2"	"	10-7"	"
F26	"	"	7'-10 1/2"	"	9-9"	"
F27	"	"	7'-8 1/2"	"	9-6"	"
F28	"	"	7'-6 1/2"	"	9-3"	"
F29	"	"	7'-4 1/2"	"	9-0"	"
F30	"	"	7'-2 1/2"	"	8-9"	"
F31	"	"	6'-11"	"	8-6"	"
F32	"	"	6'-9 1/2"	"	8-3"	"
F33	"	"	6'-7 1/2"	"	8-0"	"
F34	"	"	6'-5 1/2"	"	7-9"	"
F35	"	"	6'-3 1/2"	"	7-6"	"
F36	"	"	6'-1 1/2"	"	7-3"	"
F37	"	"	5'-11"	"	7-0"	"
F38	"	"	5'-9 1/2"	"	6-11"	"
F39	"	"	5'-7 1/2"	"	6-9"	"
F40	"	"	5'-5 1/2"	"	6-8"	"
F41	"	"	5'-3 1/2"	"	6-6"	"
F42	"	"	5'-1 1/2"	"	6-5"	"
F43	"	"	4'-11"	"	6-3"	"
F44	"	"	4'-9 1/2"	"	6-2"	"
F45	"	"	4'-7 1/2"	"	6-0"	"
F46	"	"	4'-5 1/2"	"	6-0"	"
F47	"	"	4'-3 1/2"	"	6-0"	"
F48	"	"	4'-1 1/2"	"	6-0"	"
G1	1 1/2"	6	58'-6"	3 1/2"	61'-6"	Beam ①
G2	"	1	57'-11"	"	60-11"	"
G3	"	1	56'-6"	"	59-6"	"
G4	"	3	55'-0"	"	58-0"	"
G5	"	1	53'-7"	"	56-7"	"
H1	3/4"	18	"	"	3-7"	Int. Posts
H2	"	6	"	"	3-5"	End
H3	1/2"	16	"	"	8-3"	Posts
S1	1/2"	51	"	"	11-5"	Beam ②
S2	"	39	"	"	12-7"	"
S3	"	12	"	"	8-8"	Curb
S4	"	49	"	"	4-10"	Rail-Curb.

Dimensions to E of bars.

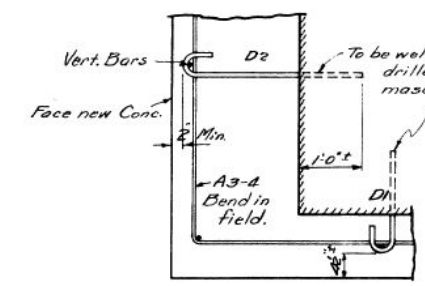
NOTE:
Place K-bars at intersection of wing with crest wall, 6' from back form @ 1'-6" c/c vertically, also @ 4' c/c horizontally, 6' below bridge seat elev.



SECT. ABUT. 1

SOUNDINGS
① Ground 92.0, Ledge 80.0
② Ground 92.0, Ledge 81.5
③ Ground 92.0, Ledge 84.0
All soundings thru fairly easy driving to refusal, ledge. Last 2 ft thru very firm material.

CONCRETE CLASSIFICATION
CLASS A- Abutment 2 & Superstructure.
CLASS B- Abutment 1.
CLASS Y- Concrete Rail.



PLAN VIEW OF SETTING DOWELS.

DESIGN - ALLEN
TRACE - 1/2"
CHECK - 1/2"

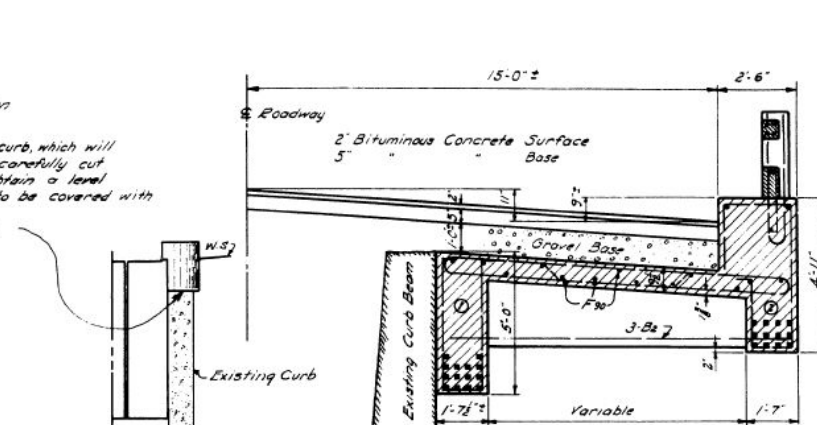
BRIDGE - 2334

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

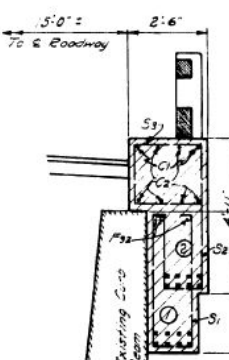
GRIST MILL BRIDGE
OVER
SOUADABCOOK STREAM
IN THE TOWN OF
HAMPDEN
PENOBSCOT COUNTY
SUBSTRUCTURE

SHEET 1 OF 3, AUGUSTA, MAINE MAY 1947

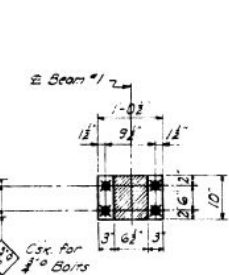
M-831



SECTION C-C



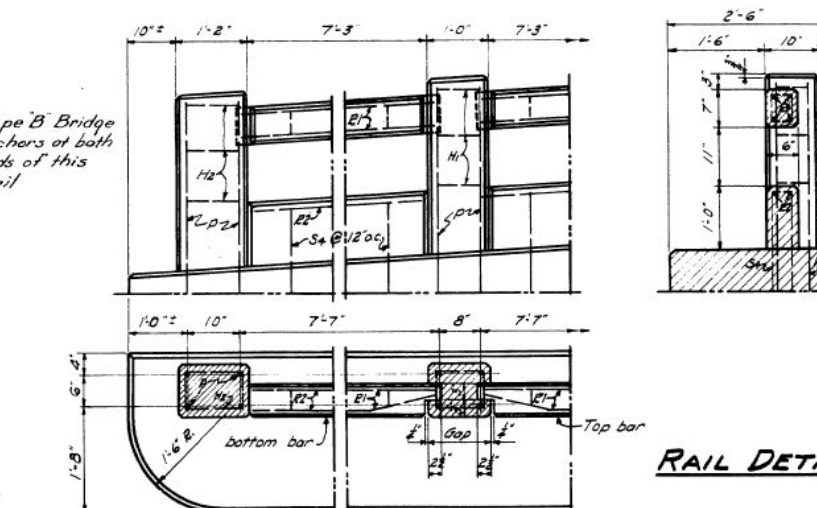
SECTION D-D



Top R. BP2
(Bottom View)
 $10'' \times \frac{15}{16}'' \times 1'-0\frac{1}{2}'' - 1 \text{ Reg'd.}$

NOTE: Machine shaved area $\frac{1}{8}$ " deep. Grooves may be cut $\frac{1}{8}$ " to $\frac{1}{4}$ " deeper to facilitate machining.

Si-Spacing
SECT. A-A - BEAM ①



NOTE:

Curb to be cast with slab. Steel for posts and lower bar to be in position before curb is placed. The lower bar is to be cast in place with steel projecting into posts. The top bar is to be precast and set in position so that the ends project into post forms $2\frac{1}{2}$ ". Wrap the tongue ends with two (2) layers of heavy roofing.

Wire stirrups for top rail bars shall be constructed in the field from a single strand of #9 annealed wire. In forming the stirrups make a complete turn around each bar.

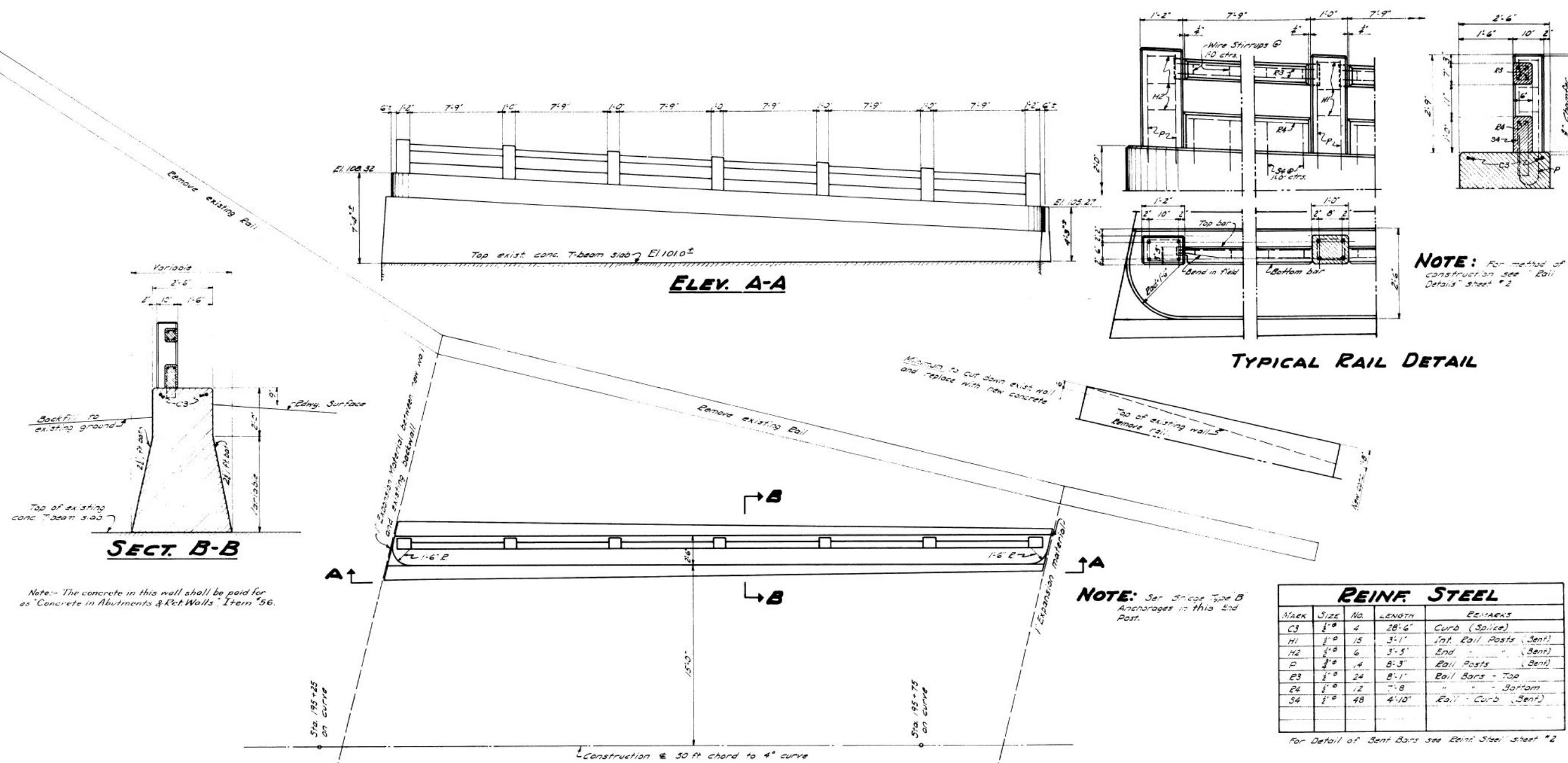
All exposed edges of concrete to be chamfered & unless otherwise indicated.

RAIL DETAILS

P									
* Bend in field 3 or 4 bars on necessity at about 2 end									
MARK	SIZE	NO.	A	B	C	D	E	LENGTH	REMARKS
F ₁	1/2"	1	4'-10"	6'-1"	3'-7"	9'-1"	3'-1"	16'-5"	S/100
F ₂	"	1	4'-10"	"	3'-6 1/2"	"	"	16'-3"	"
F ₃	"	1	4'-8 1/2"	"	3'-3 1/2"	"	"	16'-0"	"
F ₄	"	1	4'-7 1/2"	"	3'-6 1/2"	"	"	15'-9"	"
F ₅	"	1	4'-6 1/2"	"	3'-4"	"	"	15'-6"	"
F ₆	"	1	4'-5"	"	3'-3 1/2"	"	"	15'-4"	"
F ₇	"	1	4'-3 1/2"	"	3'-2 1/2"	"	"	15'-1"	"
F ₈	"	1	4'-2 1/2"	"	3'-1 1/2"	"	"	14'-10"	"
F ₉	"	1	4'-1 1/2"	"	3'-1"	"	"	14'-7"	"
F ₁₀	"	1	4'-0"	"	3'-0 1/2"	"	"	14'-5"	"
F ₁₁	"	1	3'-10 1/2"	"	2'-11 1/2"	"	"	14'-2"	"
F ₁₂	"	1	3'-9 1/2"	"	2'-10 1/2"	"	"	13'-11"	"
F ₁₃	"	1	3'-8 1/2"	"	2'-9"	"	"	13'-8"	"
F ₁₄	"	1	3'-7 1/2"	"	2'-8 1/2"	"	"	13'-6"	"
F ₁₅	"	1	3'-6 1/2"	"	2'-7 1/2"	"	"	13'-5"	"
F ₁₆	"	1	3'-6 1/2"	"	2'-7 1/2"	"	"	13'-5"	"
F ₁₇	"	1	3'-5 1/2"	"	2'-6 1/2"	"	"	13'-3"	"
F ₁₈	"	1	3'-5 1/2"	"	2'-6 1/2"	"	"	13'-3"	"
F ₁₉	"	1	3'-4 1/2"	"	2'-5 1/2"	"	"	13'-1"	"
F ₂₀	"	1	3'-4 1/2"	"	2'-5 1/2"	"	"	13'-1"	"
F ₂₁	"	1	3'-3 1/2"	"	2'-4 1/2"	"	"	12'-10"	"
F ₂₂	"	1	3'-3 1/2"	"	2'-4 1/2"	"	"	12'-10"	"
F ₂₃	"	1	3'-2 1/2"	"	2'-3 1/2"	"	"	12'-7"	"
F ₂₄	"	1	3'-2 1/2"	"	2'-3 1/2"	"	"	12'-7"	"
F ₂₅	"	1	3'-1 1/2"	"	2'-2 1/2"	"	"	12'-4"	"
F ₂₆	"	1	3'-1 1/2"	"	2'-2 1/2"	"	"	12'-4"	"
F ₂₇	"	1	3'-0 1/2"	"	2'-1 1/2"	"	"	12'-1"	"
F ₂₈	"	1	3'-0 1/2"	"	2'-1 1/2"	"	"	12'-1"	"
F ₂₉	"	1	2'-11 1/2"	"	2'-0 1/2"	"	"	11'-10"	"
F ₃₀	"	1	2'-10 1/2"	"	2'-0 1/2"	"	"	11'-10"	"
F ₃₁	"	1	2'-9 1/2"	"	2'-0 1/2"	"	"	11'-8"	"
F ₃₂	"	1	2'-9 1/2"	"	2'-0 1/2"	"	"	11'-8"	"
F ₃₃	"	1	2'-8 1/2"	"	2'-0 1/2"	"	"	11'-5"	"
F ₃₄	"	1	2'-8 1/2"	"	2'-0 1/2"	"	"	11'-5"	"
F ₃₅	"	1	2'-7 1/2"	"	2'-0 1/2"	"	"	11'-2"	"
F ₃₆	"	1	2'-6 1/2"	"	2'-0 1/2"	"	"	11'-2"	"
F ₃₇	"	1	2'-5 1/2"	"	2'-0 1/2"	"	"	10'-11"	"
F ₃₈	"	1	2'-5 1/2"	"	2'-0 1/2"	"	"	10'-11"	"
F ₃₉	"	1	2'-4 1/2"	"	2'-0 1/2"	"	"	9'-10"	"
F ₄₀	"	1	2'-4 1/2"	"	2'-0 1/2"	"	"	9'-10"	"
F ₄₁	"	1	2'-3 1/2"	"	2'-0 1/2"	"	"	9'-7"	"
F ₄₂	"	1	2'-3 1/2"	"	2'-0 1/2"	"	"	9'-7"	"
F ₄₃	"	1	2'-2 1/2"	"	2'-0 1/2"	"	"	9'-4"	"
F ₄₄	"	1	2'-2 1/2"	"	2'-0 1/2"	"	"	9'-4"	"
F ₄₅	"	1	2'-1 1/2"	"	2'-0 1/2"	"	"	9'-1"	"
F ₄₆	"	1	2'-1 1/2"	"	2'-0 1/2"	"	"	9'-1"	"
F ₄₇	"	1	2'-0 1/2"	"	2'-0 1/2"	"	"	8'-10"	"
F ₄₈	"	1	2'-0 1/2"	"	2'-0 1/2"	"	"	8'-10"	"
F ₄₉	"	1	1'-8"	"	2'-2"	"	"	8'-7"	"
F ₅₀	"	1	1'-6"	"	2'-4"	"	"	8'-4"	"
F ₅₁	"	1	1'-5"	"	2'-0 1/2"	"	"	8'-1"	"
F ₅₂	"	1	1'-3 1/2"	"	1'-11 1/2"	"	"	7'-10"	"
F ₅₃	"	1	1'-2"	"	1'-11"	"	"	7'-9"	"
G ₁	1 1/2"	2	4'-9'-2"	4'-3 1/2"	3'-0"	5'-0"	3'	33'-7"	Beam ①
G ₂	"	2	4'-6'-6"	3'-11"	3'-5 1/2"	5'-2 1/2"	"	34'-9"	"
G ₃	"	2	33'-6"	3'-11"	3'-5 1/2"	5'-2 1/2"	"	46'-11"	"
G ₄	"	2	33'-6"	4'-0"	3'-5 1/2"	5'-2 1/2"	"	49'-6"	"
G ₅	"	2	43'-0"	3'-11"	3'-5 1/2"	5'-2 1/2"	"	50'-8"	"
G ₆	"	2	37'-2 1/2"	3'-11"	3'-5 1/2"	5'-2 1/2"	"	48'-10"	"
G ₇	"	2	29'-4 1/2"	3'-11"	3'-5 1/2"	5'-2 1/2"	"	42'-10"	"
G ₈	1 1/2"	2	36'-4"	3'-6 1/2"	3'-1 1/2"	4'-8 1/2"	"	38'-10"	"
Dimms/ans to E of caps.									

Loading H2O-44
 $f_g = 18,000\%$
 $f_c = 1,000\%$
 $n = 10$

DESIGN - ALLEN	BRIDGE - 2334
TRACE - CLARK	
CHECK - <i>AK</i>	
STATE HIGHWAY COMMISSION BRIDGE DIVISION	
GRIST MILL BRIDGE OVER SOUADABSCOOK STREAM IN THE TOWN OF HAMPDEN PENOBSCOT COUNTY SUPERSTRUCTURE	
SHEET 2 OF 3 AUGUSTA, MAINE MAY 1941	



REIN. STEEL				
MARK	SIZE	NO.	LENGTH	REMARKS
C3	1"Ø	4	28'-6"	Curb (Splice)
H1	1"Ø	15	3'-1"	Int. Rail Posts (Sent)
H2	1"Ø	6	3'-5"	End " (Sent)
P	1"Ø	4	8'-3"	Rail Posts (Sent)
E3	1"Ø	24	8'-1"	Rail Bars - Top
E4	1"Ø	12	7'-8"	" - Bottom
S4	1"Ø	48	4'-10"	Rail - Curb (Sent)

For Detail of Bent Bars see Reinf. Steel Sheet #2

DESIGN - ALLEN	
TRACE - CLARK	
CHECK - <i>fc</i>	BRIDGE - 2334

CHECK - ☒	BRIDGE LOG
STATE HIGHWAY COMMISSION BRIDGE DIVISION	

GRIST MILL BRIDGE

OVER
SQUADABSCOOK STREAM

SQUADABSCOOK STREAM
IN THE TOWN OF

HAMPDEN

PENOBSCOT COUNTY

DOWN STREAM RETAINING WALL AND RA
SHEET 2 OF 2 AUGUSTA MAINE NOV 1

SHEET 3 OF 3 AUGUSTA, MAINE NOV. 19

14-00000

APPENDIX D

Calculations

File No.	130732-003
Sheet	1 of 8
Date	20-Mar-18
Computed by	MMB
Checked by	BCS

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Bearing Resistance of Bedrock for Abutment Footings

PROBLEM STATEMENT & OBJECTIVE

Calculate the strength, service, and extreme limit state bearing resistance.

EXECUTIVE SUMMARY

A factored bearing resistance of	32	ksf for the strength limit state is recommended.
A factored bearing resistance of	20	ksf for the service limit state for 1 in. settlement is recommended.
A factored bearing resistance of	57	ksf for the extreme event limit state is recommended.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. NCHRP Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, 2010.

AVAILABLE INFORMATION

1. Sheet 6 titled "Boring Location Plan & Interpretive Subsurface Profile" from plan set dated November 2016.
2. Draft boring logs BB-HSS-101 and BB-HSS-102
3. Results of laboratory testing on bedrock specimens.

ASSUMPTIONS

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

File No.	130732-003
Sheet	2 of 8
Date	20-Mar-18
Computed by	MMB
Checked by	BCS

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Bearing Resistance of Bedrock for Abutment Footings

PROCEDURE FOR STRENGTH LIMIT STATE

Excerpt from AASHTO LRFD 2014:

10.6.3.2 - Bearing Resistance of Rock

10.6.3.2.1 - General

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

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

10.6.3.2.2 Semiempirical Procedures

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

C. 10.6.3.2.2

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

Nominal bearing resistance equation based on Carter and Kulhawy (1988) from NCHRP Report 651:

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

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

RMR using Tables 15 and 16

m and s using Tables 17 and 19

Resistance factor ϕ from Table 10.5.5.2.2-1 in AASHTO LRFD 2014 for bearing resistance of footings on rock

File No.	130732-003
Sheet	3 of 8
Date	20-Mar-18
Computed by	MMB
Checked by	BCS

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Bearing Resistance of Bedrock for Abutment Footings

PROCEDURE FOR SERVICE LIMIT STATE

Excerpt from AASHTO LRFD 2014:

10.6.2.6 - Bearing Resistance at the Service Limit State

10.6.2.6.1 - Presumptive Values for Bearing Resistance

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

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

Use AASHTO LRFD 2014 presumptive bearing resistance for service limit state for settlement stated.

PROCEDURE FOR EXTREME EVENT LIMIT STATE

Excerpt from AASHTO LRFD 2014:

11.5.8 - Resistance Factors for Extreme Event Limit state

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state. For overall stability of the retaining wall when earthquake loading is included, a resistance factor ϕ shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSEWalls.

Use nominal resistance calculated for the Strength Limit State and apply a resistance factor of 0.8 from AASHTO LRFD 2014 Section 11.5.8 to obtain the factored resistance.

CALCULATIONS

File No. 130732-003
 Sheet 4 of 8
 Date 20-Mar-18
 Computed by MMB
 Checked by BCS

Client Maine Department of Transportation
 Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
 Subject Bearing Resistance of Bedrock for Abutment Footings

LABORATORY TEST DATA

Boring No.	Core No.	Top of Bedrock Elevation	Elevation of Core Run	Core Depth Below Top of Bedrock (ft)	Rock Type	RQD (%)	Joints	Joint Condition
BB-HSS-101	R1	3.4	3.4 to 0.2	0 to 3.2	Schist	11	Very close to close, vertical joints moderately close	Discolored to moderately weathered, vertical joints discolored to slightly weathered
BB-HSS-101	R2	3.4	0.2 to -4.5	3.2 to 7.9	Schist	91	Moderately close	Discolored to moderately weathered
BB-HSS-101	R3	3.4	-4.5 to -7.5	7.9 to 10.9	Schist	94	Close	Discolored to fresh
BB-HSS-102	R1*							
BB-HSS-102	R2	7.4	5.4 to 2.9	2 to 4.5	Schist	0	Close, vertical joints moderately close	Slightly weathered to silt-clay infilling, vertical joints discolored
BB-HSS-102	R3	7.4	2.9 to -2.1	4.5 to 9.5	Schist	78	Close to moderately close	Discolored to slightly weathered

*R1 encountered concrete in upper 2 ft of core (El. 10.4 to 7.4), concrete portion was not included in RQD.

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Bearing Resistance of Bedrock for Abutment Footings

CALCULATIONS - STRENGTH LIMIT STATE
Table 15 from NCHRP Report 651:

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

Table 16 from NCHRP Report 651:

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

Total RMR Rating Calculation:

Parameter	Value	Relative Rating
Intact Rock Strength	1,080–2,160 ksf	7
RQD	50% to 75%	13
Joint Spacing	2 in–1 ft	12
Joint Condition	Slightly rough surfaces Separation <0.05 in Hard joint wall rock	12
Groundwater Conditions	Moist only (interstitial water)	–2
Joint Strike and Dip	Favorable	7
Total Rating =		49

from laboratory test results

based on average RQD of cored bedrock of 55%

based on range of very close to close to moderately close joint spacing, <5 cm to 100 cm (<2 in. to 40 in.) - use close (2 in. to 12 in.) range

assumed

assumed

Client Maine Department of Transportation

Date 20-Mar-18

Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine

Computed by MMB

Subject Bearing Resistance of Bedrock for Abutment Footings

Checked by BCS

Calculations - Strength Limit State

Table 19 from NCHRP Report 651:

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

Table 17 from NCHRP Report 651:

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

s and m Calculation:

Rock Quality	Rock Type	Rock Type Description	m	s
Fair	B	Lithified argillaceous rocks - mudstone, siltstone, shale, and slate (normal to cleavage).	0.183	0.00009

Semi-empirical method by Carter and Kulhawy 1988:

q_u = 9,460 psi based on laboratory test results
 q_n = 71 ksf Equation 82b
 ϕ = 0.45 from Table 10.5.5.2.2-1
 q_R = 32 ksf Equation 82b

Client Maine Department of Transportation

Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine

Subject Bearing Resistance of Bedrock for Abutment Footings

CALCULATIONS - SERVICE LIMIT STATE

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

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

Based on Table C10.6.2.6.1-1 the service limit state for bearing resistance on schist bedrock is recommended at 20 ksf for settlements of 1 in., which considers that the upper portion of the rock is fractured

CALCULATIONS - EXTREME LIMIT STATE

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

$$Q_{ult} = 71 \text{ ksf}$$

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

$$q_R = 57 \text{ ksf}$$



CALCULATIONS

File No. 130732-003

Sheet 8 of 8

Date 20-Mar-18

Computed by MMB

Checked by BCS

Client Maine Department of Transportation

Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine

Subject Bearing Resistance of Bedrock for Abutment Footings

CONCLUSIONS AND RECOMMENDATIONS

Strength Limit State

The recommended factored bearing resistance for the strength limit state is 32 ksf

Service Limit State

The recommended presumptive value for schist bedrock is 20 ksf for the service limit state for a settlement up to 1 in.

Extreme Event Limit State

The recommended factored bearing resistance for the extreme event limit state is 57 ksf

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Sliding Resistance for Footing on Bedrock

PROBLEM STATEMENT AND OBJECTIVE

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

EXECUTIVE SUMMARY

	In-the-dry	In-the-wet
The coefficient of friction between the footing and bedrock is =	0.7	0.6
The resistance factor for sliding at the <u>Strength Limit State</u> is =		0.8
The resistance factor for sliding at the <u>Extreme Event Limit State</u> is =		1.0

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
2. MaineDOT BDG 2003 with 2014 interims

AVAILABLE INFORMATION

1. Haley & Aldrich test borings BB-HSS-101 and BB-HSS-102.

ASSUMPTIONS

1. Abutment footing will bear on intact PHYLLITE bedrock.

CALCULATIONS

Coefficient of Friction Between Concrete and Bedrock

Nominal sliding resistance between the cast-in-place concrete footing/seal and bedrock is dependent on the coefficient of friction ($\tan\delta$) at the interface between the footing/seal and bedrock. Construction means and methods may significantly impact the condition of the footing/seal-bedrock interface and sliding (i.e. prepared in-the-dry or in-the-wet).

We understand that the footing/seal will likely be constructed in the wet and that inspection of the bedrock surface may not be performed prior to concrete pouring.

Estimated footing/seal-rock interface friction angle (δ):

In-the-dry: 35 deg., friction angle for mass concrete on clean, sound bedrock (AASHTO LRFD Table 3.11.5.3-1)

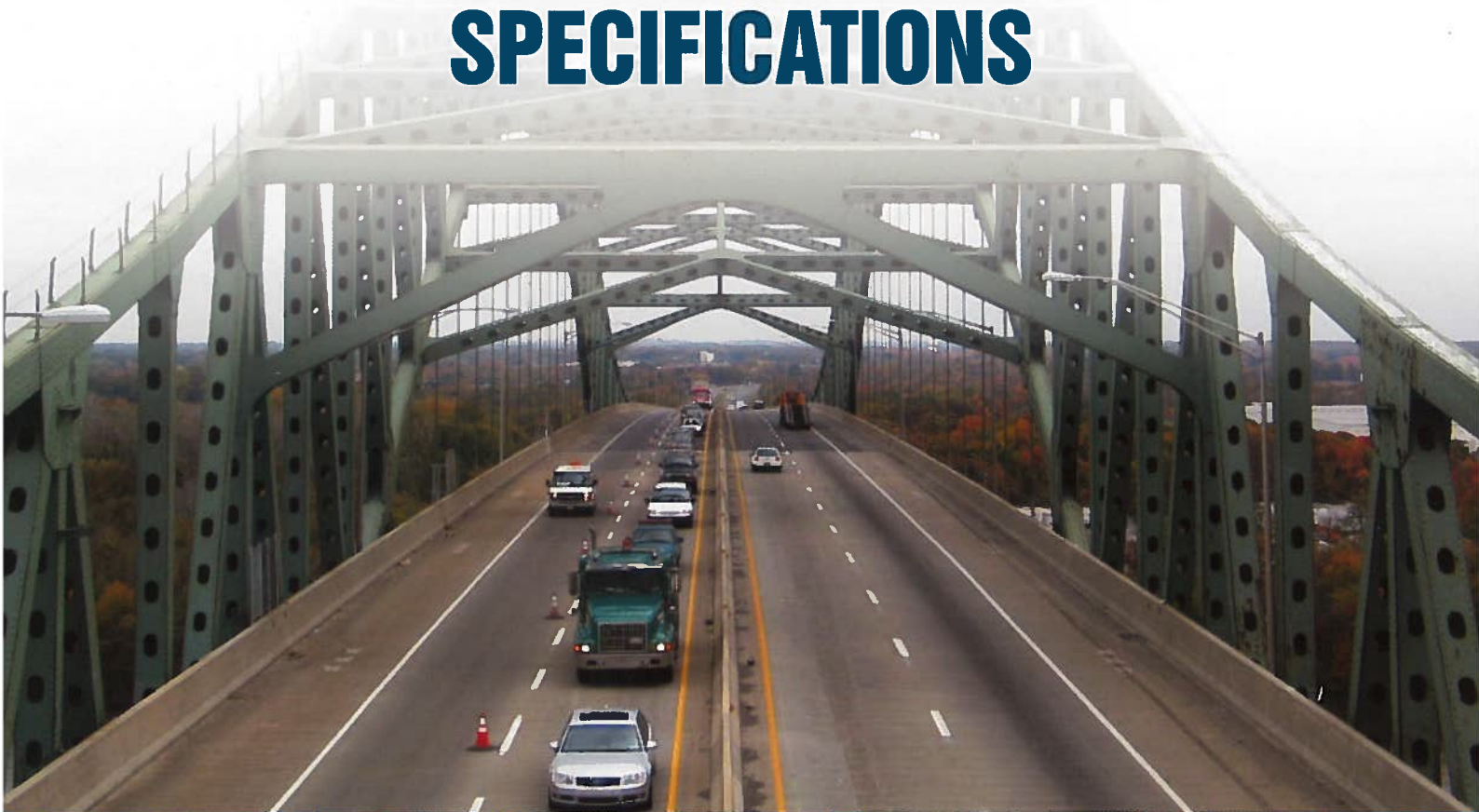
In-the-wet: 29 deg., friction angle for clean gravel
(assume lower friction angle for submerged bedrock, AASHTO LRFD Table 3.11.5.3-1)

In-the-dry: Recommended δ = 35 deg., friction angle between footing/seal and bedrock
Recommended $\tan\delta$ = 0.7 coefficient of friction

In-the-wet: Recommended δ = 29 deg., friction angle between footing/seal and bedrock
Recommended $\tan\delta$ = 0.6 coefficient of friction

HALEY ALDRICH CALCULATIONS		File No.	130732-003
		Sheet	2 of 2
Client	Maine Department of Transportation	Date	13-Jul-18
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine	Computed by	NAS
Subject	Sliding Resistance for Footing on Bedrock	Checked by	BCS
Resistance Factors Service Limit State AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on bedrock. For retaining walls, AASHTO LRFD prescribes a sliding resistance factor of = 1.0 (Section 11.5.7). Strength Limit State AASHTO LRFD does not prescribe a sliding resistance factor for shallow foundations on bedrock. For cast-in-place concrete on sand, the sliding resistance factor is = 0.8 (Table 10.5.5.2.2-1) Extreme Event Limit State Section 10.5.5.3.2 of AASHTO LRFD prescribes a resistance factor of 1.0 for the design of foundations in the Extreme Event Limit State.			

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS



AMERICAN ASSOCIATION OF
STATE HIGHWAY AND
TRANSPORTATION OFFICIALS

AASHTO
THE VOICE OF TRANSPORTATION

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**Seventh Edition, 2014
U.S. Customary Units**

Part II: Sections 7—Index

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

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

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

C3.11.5.4

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

For cohesive soils, passive pressures may be estimated by:

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

		CALCULATIONS		File No.	130732-003
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Client	Maine Department of Transportation			Date	17-Jul-18
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine			Computed by	NAS
Subject	Lateral Earth Pressure Coefficients for Abutment No. 2 and Wingwalls			Checked by	BCS

Objective

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

Assumptions

- Due to sloping backfill conditions at Abutment No. 2, calculate different lateral earth pressure coefficients for each proposed abutment and wingwalls
- Abutments and wingwalls and their footings are backfilled with Granular Borrow based on Abutment Notes on plan set.
- Free draining retaining wall, no hydrostatic pressure.

References

1. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 with interim revisions through 2016
2. MaineDOT Bridge Design Guide, August 2003, with interim revisions through March 2014

EARTH PRESSURE COEFFICIENTS FOR PROPOSED ABUTMENT NO. 2

Soil Properties and Geometry

designates input cell

Total Unit Weight, γ (pcf) =	125	pcf	Soil Type 4, BDG Table 3-3
Effective Friction Angle, ϕ' =	32	degrees	Soil Type 4, BDG Table 3-3
Backslope Angle, β =	3.86	degrees	Grade = 6.74%, per Sheet 8 from "Grist Mill Bridge over Souadabscook Stream" by T.Y. Lin International dated 16 July 2018.
Backface of Wall Angle to Horizontal, Θ =	90	degrees	
Soil and Wall Friction Angle, δ =	24	degrees	Soil Type 4, BDG Table 3-3

Static Active Lateral Earth Pressure Coefficient, K_a

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

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

$$K_a = 0.29$$

At-Rest Lateral Earth Pressure Coefficient, K_o

$$K_o = 1 - \sin\phi \quad \text{AASHTO LRFD Eq. 3.11.5.2-1}$$

$$K_o = 0.47$$

Passive Lateral Earth Pressure Coefficient, K_p

Rankine Theory

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

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

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

Coulomb Theory

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

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

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

$$K_{p, \text{Coulomb}} = 10.79$$

The derivation is similar to that for Rankine's active state.

Figure 13.8c shows the variation of passive pressure with depth. For cohesionless soils ($c' = 0$),

$$\sigma'_p = \sigma'_o \tan^2 \left(45 + \frac{\phi'}{2} \right)$$

or

$$\frac{\sigma'_p}{\sigma'_o} = K_p = \tan^2 \left(45 + \frac{\phi'}{2} \right) \quad (13.22)$$

K_p (the ratio of effective stresses) in the preceding equation is referred to as the *coefficient of Rankine's passive earth pressure*.

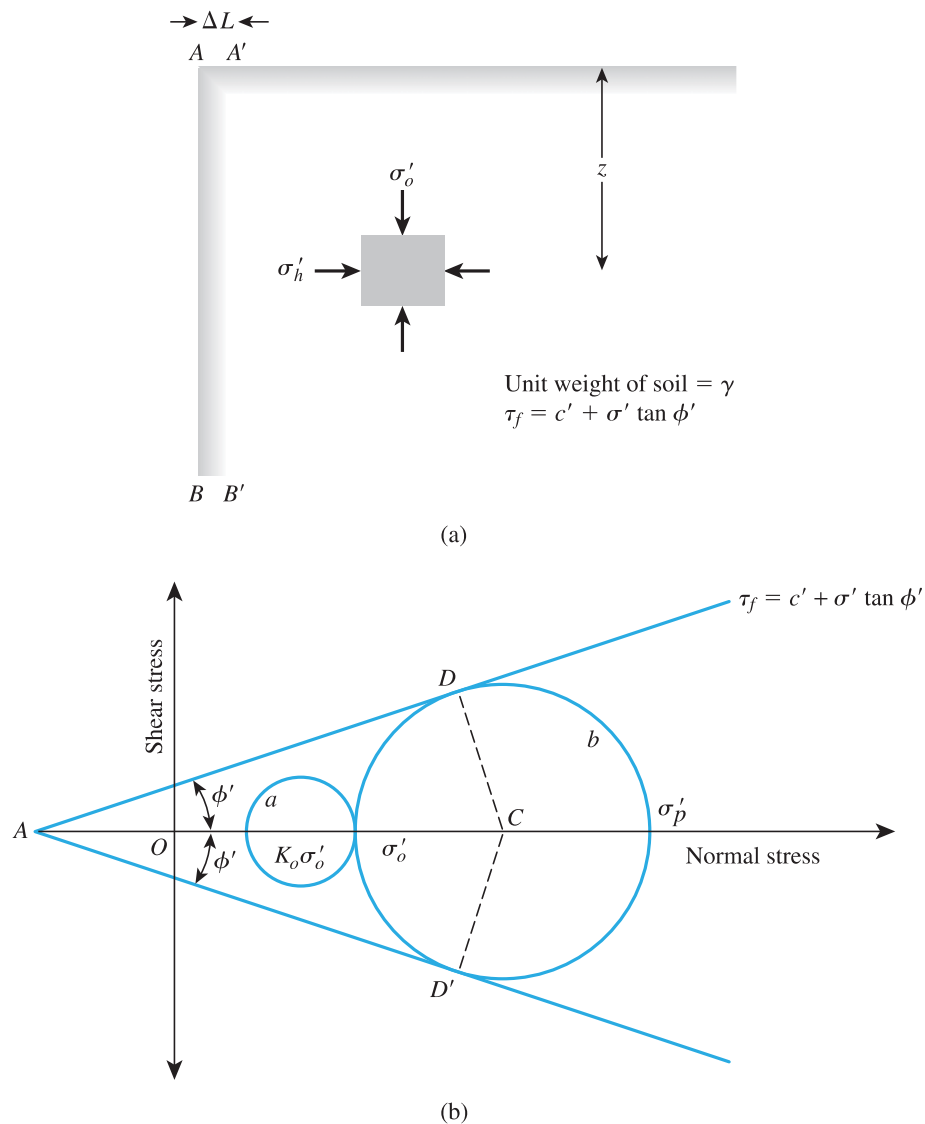


Figure 13.8 Rankine's passive earth pressure

File No.	130732-003
Sheet	1 of 7
Date	1-Oct-18
Computed by	NAS
Checked by	BCS

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Seismic Site Class Evaluation

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values from test borings.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the test borings (BB-HSS-101 and BB-HSS-102 and BB-HSS-201 through BB-HSS-205), **recommend a Seismic Site Class D.**

REFERENCES

1. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (2012 Interim Revisions).
2. AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014.
3. International Building Code 2009.
4. ASCE/SEI 7-05 Minimum Design Loads For Buildings and Other Structures
5. International Building Code 2012.
6. ASCE/SEI 7-10 Minimum Design Loads For Buildings and Other Structures

AVAILABLE INFORMATION

1. Test borings from the preliminary explorations (July 2009) and supplemental explorations (July 2018).
2. Other: Conceptual Geotechnical Design Memorandum, dated 29 September 2017, by Haley & Aldrich, Inc.
3. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. Evaluations assume the 12-inch-thick asphalt layer encountered at the test borings is soil with a SPT N-value equal to the soil beneath it.
3. WOH/WOR = SPT N-value of 1.
4. Assume the existing concrete footing encountered from El. 10.4 to El. 7.4 at BB-HSS-102 has a SPT N-value equal to the soil above it (5 bl/ft)

PROCEDURE

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

Method A

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

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

where

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

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

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

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

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Seismic Site Class Evaluation

PROCEDURE

Method B

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

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

where

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

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

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

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

Method C

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

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

where

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

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

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

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

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

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

where

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

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

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

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

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

Client Maine Department of Transportation

Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine

Subject Seismic Site Class Evaluation

SITE CLASS DEFINITIONS

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

Table 3.4.2.1-1—Site Class Definitions

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

Exceptions:

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

where:

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

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

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

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

Client Maine Department of Transportation

Project Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine

Subject Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-HSS-101
Ground Surface El.: 30.5

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	30.5	SAND (Fill)	5.0	20	0.250
2D	5	25.5	SAND (Fill)	5.0	6	0.833
3D	10	20.5	SAND (Fill)	3.2	3	1.067
4D	13.2	17.3	SAND (Fill)	5.8	21	0.276
5D	19	11.5	GRAVEL (Fill)	6.0	12	0.500
6D	25	5.5	SAND (Fill)	2.1	25	0.084
R1-R3	27.1	3.4	BEDROCK	72.9	100	0.729
Totals =				100.0		3.739

N-bar (blows/ft) = 27
Site Class = D

Exploration ID: BB-HSS-102
Ground Surface El.: 33.8

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	33.8	SAND (Fill)	5.0	36	0.139
2D	5	28.8	SAND (Fill)	5.0	11	0.455
3D	10	23.8	SAND (Fill)	5.0	6	0.833
4D	15	18.8	CLAY (Fill)	4.0	5	0.800
5D	19	14.8	No Recovery	4.4	5	0.880
R1	23.4	10.4	Concrete Ftg.	3.0	5	0.600
N/A	26.4	7.4	BEDROCK	73.6	100	0.736
Totals =				100.0		4.443

N-bar (blows/ft) = 23
Site Class = D

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: **BB-HSS-201**
 Ground Surface El.: **26.9**

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	26.9	SAND (Fill)	3.0	41	0.073
2D	3	23.9	SAND (Fill)	2.0	26	0.077
3D	5	21.9	SAND (Fill)	8.5	7	1.214
5D/6D	13.5	13.4	SAND (Fill)	10.5	6	1.750
7D	24	2.9	GRAVEL (Fill)	3.5	12	0.292
N/A	27.5	-0.6	BEDROCK	72.5	100	0.725

Totals = 100.0 4.131

N-bar (blows/ft) = 24
 Site Class = D

Exploration ID: **BB-HSS-202**
 Ground Surface El.: **27.6**

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	27.6	SAND (Fill)	4.5	23	0.196
2D	4.5	23.1	SILT (Fill)	4.0	10	0.400
3D	8.5	19.1	SAND (Fill)	6.0	5	1.200
4D	14.5	13.1	SAND (Fill)	5.5	38	0.145
5D	20	7.6	SILT (Fill)	5.5	18	0.306
6D	25.5	2.1	GRAVEL (Till)	3.5	100	0.035
7D	29	-1.4	SAND (Till)	6.0	92	0.065
8D	35	-7.4	SAND (Till)	5.1	100	0.051
N/A	40.1	-12.5	BEDROCK	59.9	100	0.599

Totals = 100.0 2.996

N-bar (blows/ft) = 33
 Site Class = D

Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: **BB-HSS-203**
 Ground Surface El.: **35.5**

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	35.5	SAND (Fill)	3.0	8	0.375
2D	3	32.5	SAND (Fill)	2.0	5	0.400
3D	5	30.5	SAND (Fill)	3.5	7	0.500
4D	8.5	27	SAND (Fill)	4.5	23	0.196
5D	13	22.5	SAND (Fill)	5.5	12	0.458
6D	18.5	17	CLAY (Fill)	3.0	19	0.158
7D	21.5	14	SAND (Fill)	4.2	100	0.042
N/A	25.7	9.8	BEDROCK	74.3	100	0.743
Totals =				100.0		2.872

N-bar (blows/ft) = 35
 Site Class = **D**

Exploration ID: **BB-HSS-204**
 Ground Surface El.: **37.9**

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	37.9	SAND (Fill)	3.0	10	0.300
2D	3	34.9	SAND (Fill)	2.0	4	0.500
3D	5	32.9	SAND (Fill)	3.5	4	0.875
4D	8.5	29.4	SAND (Fill)	6.5	14	0.464
5D/A	15	22.9	SILT (Fill)	5.0	14	0.357
6D/7D	20	17.9	GRAVEL (Till)	5.5	100	0.055
N/A	25.5	12.4	BEDROCK	74.5	100	0.745
Totals =				100.0		3.296

N-bar (blows/ft) = 30
 Site Class = **D**

CALCULATIONS

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Date	1-Oct-18
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Client	Maine Department of Transportation
Project	Grist Mill Bridge No. 2334 over Souadabscook Stream, Hampden, Maine
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD B

Exploration ID: BB-HSS-205
Ground Surface El.: 36.3

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	SPT N (blows/ft)	d/N
1D	0	36.3	SAND (Fill)	4.6	19	0.242
2D	4.6	31.7	SAND (Fill)	3.9	6	0.650
3D	8.5	27.8	SAND (Fill)	6.5	5	1.300
4D	15	21.3	SILT (Fill)	4.6	3	1.533
N/A	19.6	16.7	BEDROCK	80.4	100	0.804

Totals = 100.0 4.529

N-bar (blows/ft) = 22
Site Class = D

Design Maps Summary Report

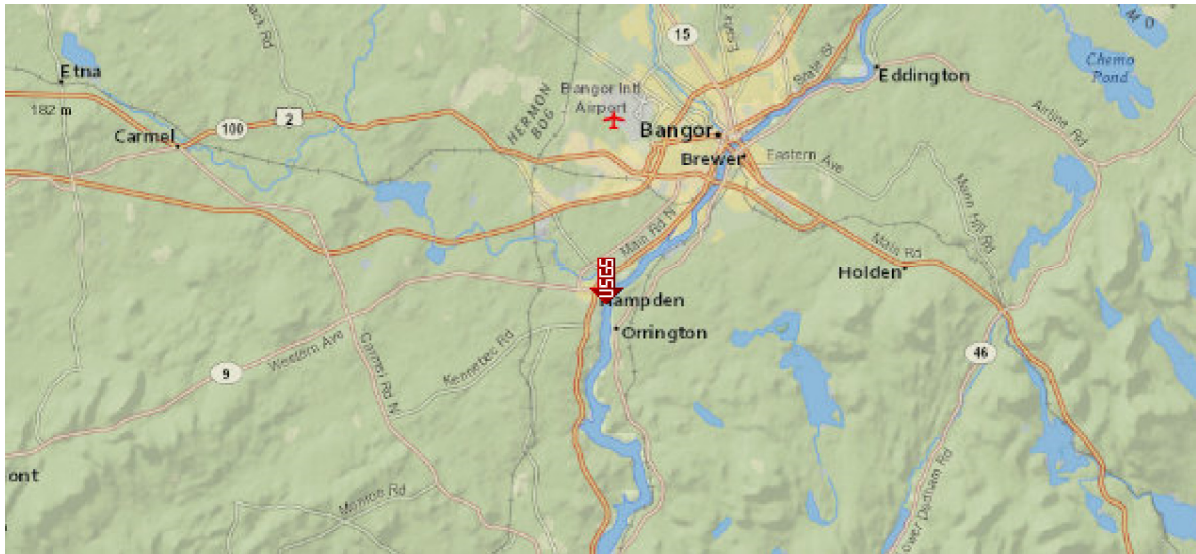
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Report Title Grist Mill Bridge
Fri July 13, 2018 14:02:10 UTC

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

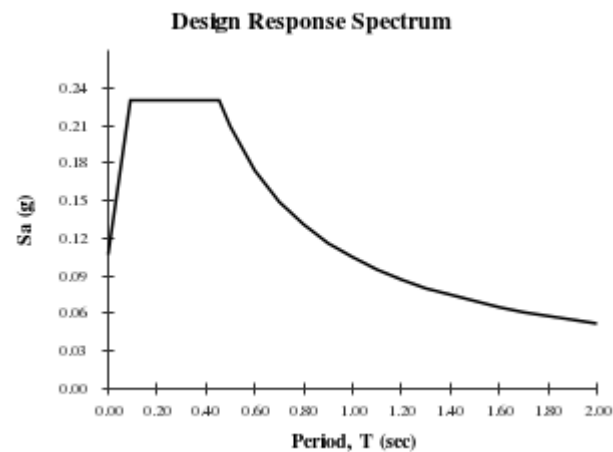
Site Coordinates 44.7488°N, 68.8324°W

Site Soil Classification Site Class D – “Stiff Soil”



USGS-Provided Output

PGA = 0.067 g	A_s = 0.107 g
S_s = 0.144 g	S_{DS} = 0.230 g
S₁ = 0.044 g	S_{D1} = 0.105 g



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