TRACY BROOK BRIDGE OVER MEADOW BROOK

WIN 23657.00

Town of Durham
Androscoggin County
Maine

GEOTECHNICAL DESIGN REPORT

OCTOBER 28, 2022

PREPARED FOR

Maine Department of Transportation
16 State House Station
Augusta, Maine 04333

PREPARED BY

HNTB Corporation
9 Entin Road, Suite 202
Parsippany, NJ 07054

HNTB

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Durham, Maine

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November 14, 2022

STATE OF MAINE
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1.0 PROJECT DESCRIPTION AND SCOPE

1.1 Introduction

HNTB Corporation has been retained by The Maine Department of Transportation to provide final design engineering services for the replacement of the Tracy Brook Bridge carrying Pinkham Brook Road (State Route 125) over the Meadow Brook at the mouth to the Androscoggin River in Durham, Androscoggin County, Maine. A project location plan is included in Figure 1. The scope of work includes performing detailed design, the preparation of plans, specifications, and construction cost estimates for bridge replacement and approach modifications. This report focuses on the geotechnical evaluation and design for the proposed modifications.

1.2 Scope of Services

HNTB's Geotechnical and Foundation Services group has been selected to provide geotechnical design services for the subject project at the Tracy Brook Bridge. The overall objective of this study is to characterize the subsurface conditions within the project limits and to develop geotechnical design criteria and foundation recommendations for support of the proposed replacement structure and approach embankments.

In completing this study, HNTB has performed the following scope of services:

- Reviewed available geotechnical data and as-built plans for the project site.
- Implemented a subsurface investigation and laboratory testing program to gather additional information where needed.
- Analyzed the resulting data collected to identify subsurface conditions that impact the design and construction of the project.
- Prepared a geologic subsurface profile for the project site to summarize geotechnical data from the borings and laboratory testing.
- Established geotechnical design parameters based on the available borings and laboratory test results.
- Conducted geotechnical analysis and provided geotechnical foundation recommendations for the support of the proposed bridge.
- Conducted geotechnical analysis of the overall stability of the roadway and developed plans and special provisions for the proposed soil nail slope stabilization solution.

1.3 Existing Site Conditions

The project limits encompass approximately 875 linear feet of Pinkham Brook Road (Route 125), primarily a north/south roadway which is aligned east/west along the southern bank of Androscoggin River. The project limits extend from approximate project baseline Station 12+00 at the southern limit to 20+75 at the northern limit. The roadway crosses the Meadow Brook, a small tributary of the Androscoggin located at a vertical sag curve. Within these limits, the roadway consists of two travel lanes without shoulders, one lane in each direction. The northbound lanes at the bridge are oriented in a northwest direction, with southbound lanes heading southeast. The existing bridge is centered at approximate Station 16+10 and consists of a single 18-foot concrete span rigidly framed into two cast-in-place concrete abutment walls founded on bedrock, with skewed monolithic cast-in-place wingwalls. The existing bridge has been classified as structurally deficient and functionally obsolete.

The project site is bounded to the northeast by the Androscoggin River running parallel to the roadway, with Meadow Brook flowing from southwest to northeast under the roadway and into the river. Topographic relief of the area surrounding the project primarily slopes down from higher land to the southwest to the Androscoggin River to the northeast, and secondarily slopes towards Meadow Brook from the southeast and northwest. Near the southbound project limit around Station 12+25 the roadway approaches the Androscoggin River, transitioning from cut to fill to cross the cut channel created by Meadow Brook. The roadway profile slopes down towards Tracy Brook Bridge, with embankment sloping down steeply toward the river on the northeast side and toward Meadow Brook to the southwest. Embankment height above the river varies from around 45 feet at Station 12+25 to around 20 feet at the bridge near Station 16+10. North of the bridge, the roadway profile rises gradually to approximately 26 feet above the river at Station 20+75. The ground surface on the southwest side of roadway rises above the roadway, while the northeast side continues to slope steeply to the river. Existing side slopes along the Androscoggin River are as steep as 1 horizontal:1 vertical (1H:1V).

1.4 Proposed Improvements

Pinkham Brook Road is to be realigned for approximately 875 feet within the project limits to improve overall roadway geometry and allow for staged construction. The roadway is to be shifted southwest of the existing alignment, away from the Androscoggin River and towards Meadow Brook, and raised by approximately 2.5 feet at the bridge location. The existing bridge is to be replaced with a new single-span integral abutment bridge to accommodate the widened and realigned roadway, consisting of a precast concrete NEXT beam superstructure with a 75-foot span length. Abutments will be supported on H-piles and will be set back behind the existing abutments and proposed rip-rap slopes to minimize impacts to the waterway. The existing steepened slopes along the Androscoggin River will be stabilized against global stability failure by installation of a soil nail retention system. The realigned embankment is to be expanded towards Meadow Brook along the southbound approach to accommodate the wider approach embankment and allow for staged construction, requiring the realignment of approximately 200 linear feet of Meadow Brook.

1.5 Survey Control

All elevations presented in this report are provided in feet and refer to the North American Vertical Datum of 1988 (NAVD 88). Horizontal coordinates (northings/eastings) are provided in feet and refer to the North American Datum of 1983 (NAD), Maine West State Plane coordinate system. Stations and offsets reference the project baseline for Pinkham Brook Road.

2.0 GEOLOGY AND SITE CONDITIONS

2.1 General Site Geology

The project is located within the New England physiographic province, a region is characteristically composed of gneiss, schist, marble, quartzite, and slate, with frequent folding, faulting, and igneous intrusions. The region has been subject to glaciation, resulting in a large number of lakes and streams, and surficial geology composed primarily of glacial deposits (glacial till, glaciofluvial deposits and glacial lake deposits).

Existing geologic mapping available for the project site includes Soil Survey data provided by the USDA Natural Resource Conservation Service (NRCS) and bedrock and surficial geology mapping prepared by the Maine Geological Survey (MGS) for the Lisbon Falls South Quadrangle. Excerpts from the surficial and bedrock geology mapping are included in Figure 2 and Figure 3, respectively, and NRCS web soil survey results are included in Appendix A.

NRCS mapping indicates that the project site consists of Adams loamy sand, derived from crystalline rock and deposited from glacial meltwaters. MGS surficial geology mapping identifies soil overburden in the area of the project as artificial fill within the embankment footprint overlying/surrounded by marine nearshore deposits. Marine nearshore deposits consist of Pleistocene gravel, sand, and mud deposited by wave activity in shallow marine environments. MGS bedrock geology mapping shows the site is within the biotite gneiss of the Vassalboro Group, consisting of medium gray, medium grained, quartz-plagioclase biotite and/or muscovite and/or sillimanite gneiss and schist bedrock.

3.0 SUBSURFACE EXPLORATIONS

3.1 General

To facilitate the geotechnical assessment and design for the project, HNTB reviewed available existing geotechnical data from the project site and evaluated additional investigation needs. Working with our geotechnical subconsultant Schonewald Engineering Associates, Inc. (Schonewald), HNTB planned and executed a subsurface investigation program including a total of five soil borings and a geophysical investigation. The historic and project-specific subsurface investigations are summarized in the following sections.

3.2 Existing Subsurface Data

An initial phase of borings was drilled and logged by MaineDOT staff in March 2019. Borings were taken near the locations of the proposed north and south abutments. The boring locations are summarized in the table below and shown on the boring location plan in Figure 4. The borings were advanced using mud rotary drilling techniques with a CME 45C truck mounted drill rig.

Table 3.2.1 – Summary of Phase 1 Borings

Boring	Northing (ft)	Easting (ft)	Elevation (feet)	Depth Drilled (feet)		Location
				Soil	Rock	
DMB-BB-101	422072	1068727	84.7	29	16	South Abutment
DMB-BB-102	422111	1068694	84.7	21.2	32.3	North Abutment

3.3 Project-Specific Geotechnical Subsurface Investigation

3.3.1 Supplemental Boring Investigation

For design of the proposed bridge foundations and approach embankments, existing Phase 1 data was supplemented with a subsurface investigation consisting of five additional soil borings, with one boring behind each existing abutment, two along the south approach, and one along the north approach. This second phase of borings was advanced in May 2022 by New England Boring Contractors, with field coordination and boring inspection carried out by HNTB's subconsultant, Schonewald. Borings were advanced using mud rotary drilling techniques with a track-mounted Mobile B-53 drilling rig. Samples were generally collected at 5-foot intervals. With the exception of HB-DMB-202, borings were advanced into bedrock using an NQ2-sized core barrel. Bridge borings were advanced a minimum of 10 feet into competent bedrock to identify a suitable bearing stratum.

Samples from both series of borings were collected using Standard Penetration Test (SPT) methods in general accordance with ASTM D-1586. Soil samples were retrieved using a 24-inch long split-spoon sampler (2-inch O.D., 1.375-inch I.D.), driven by an automatic hammer, delivering energy equivalent to a 140-lb free falling 30 inches. Soil samples were visually classified in the field using the Burmister Soil Identification System as outlined in Reference 6. Other pertinent information assessed includes natural color, mottling, texture, moisture, odor, organics, and the results of field tests, as applicable.

Boring locations are summarized below in Table 3.3.1. The stations and offsets shown are in reference to the Pinkham Brook Road project baseline. Looking up-station, offsets are indicated to either the left (L) or right (R) of the baseline. A Boring Location Plan is included in Figure 4. Boring logs for the 100-series borings are included in Appendix B. Boring logs and rock core photographs for the 200-series borings are included in Appendix C.

Table 3.3.1 – Summary of Subsurface Investigation

Boring	Northing (ft)	Easting (ft)	Estimated Elevation (ft)	Depth Drilled (ft)		Location
				Soil	Rock	
HB-DMB-201	421876	1068943	103	53.6	5.8	South Approach
HB-DMB-202	421994	1068846	91	51.0	0	South Approach
BB-DMB-203	422044	1068755	85	38.5	10.5	South Abutment
BB-DMB-204	422085	1068658	84	31.0	10.0	North Abutment
HB-DMB-205	422152	1068565	87	33.7	5.4	North Approach

3.3.2 Geophysical Testing

Geophysical testing was performed by Hager-Richter Geoscience, Inc. (HRGS) of Salem, NH to investigate the variation in the bedrock surface as a means to reduce the risk of unanticipated rock excavation along the west side of the north approach and the proposed realignment of Meadow Brook. The geophysical testing performed included both Seismic Refraction and Horizontal/Vertical Spectral Ratio (H/V) testing. Geophysical testing was selected for investigation of these areas due to geographic restrictions limiting access for available drilling equipment, including steep slopes and dense vegetation. A summary report, location plan, and test results are all provided within Appendix C.

Seismic Refraction Lines 1 and 2 were performed west of the north approach while Lines 3 through 5 were performed along the proposed realignment of Meadow Brook. The testing did not indicate the presence of shallow bedrock within the proposed excavation limits at either area.

H/V tests 1 through 5 were performed at borehole locations to calibrate the methods given the known stratification at these locations, and the bedrock depth at four of the borings. Along the north approach, H/V tests 6 through 10 were performed at the western edge of roadway near toe of adjacent slope from which bedrock was interpreted to vary from approximately 31 to 37 feet below ground surface (or elevations 51 to 58 feet). Along the riverbank at south approach, southeast toe of slope, H/V tests 11 through 15 were performed from which bedrock was interpreted to vary between approximately 47 to 50 feet below ground surface (or elevations 17 to 21 feet). Examination of the interpreted bedrock depths along the north approach and the riverbank indicate that the bedrock is dipping towards the southeast.

4.0 GEOTECHNICAL LABORATORY TESTING

4.1 Summary of Testing

A geotechnical laboratory testing program was performed by Geotesting Express of Acton, Massachusetts to verify the visual-manual field classifications and to aid in determination of the engineering design parameters. Testing included index tests to aid in identification and classification of soils, corrosion testing for use in evaluating corrosivity of steel substructure elements, and rock testing for establishing strength and elasticity parameters of intact bedrock. The geotechnical laboratory test results are discussed below in Sections 4.2 through 4.4.

4.2 Soil Index Testing

Index testing was conducted on select samples from 200-series borings in accordance with ASTM D 422 for identification and classification of soil types encountered. Grain size distribution analyses were performed to determine percentages of gravel, sand, and finer fractions in the soil samples. Cohesive and organic soils were not encountered, therefore Atterberg Limits or Organics Content testing was not assigned. Gradation curves for all Grain Size testing are presented in Appendix D for 200-series borings.

4.3 Rock Testing

Unconfined compressive strength testing was conducted performed in accordance with ASTM D7012 to determine the strength and elastic properties of intact rock on four samples. The results are summarized below in Table 4.3.

Table 4.3 – Rock Unconfined Compression Test Results

Boring No.	Sample No.	Depth (feet)	Tested Depth (feet)	Unit Weight, γ (pcf)	Compressive Strength, C_0 (psi)	Peak Strain, ε_{100} (%)
BB-DMB-203	R1	39 – 44	42.22-42.60	171	11,521	0.0019
BB-DMB-203	R2	44 – 49	45.29-45.66	169	15,042	0.0048
BB-DMB-204	R1	31 – 36	31.89-32.27	168	7,979	0.0042
BB-DMB-204	R2	36 – 41	36.81-37.21	169	11,659	0.0044

4.4 Corrosion Testing

Corrosion testing of 200-series borings was completed per AASHTO and/or ASTM testing standards in accordance with the following standards:

- Sulfate Content AASHTO T290
- Chloride Content AASHTO T291 – Method B
- Soil pH AASHTO T289
- Laboratory Resistivity ASTM G57

The results of corrosion testing on 200-series borings are summarized below in Table 4.4. The results of the resistivity test for HB-DMB-202 sample is somewhat low. This is typically an indicator of salt content higher than identified and considered a concern if the sulfates and chlorides are found to be in excess of 200ppm and 100ppm, respectively. However, given the aggregate pH, chloride content, and sulfate content results were representative of a non-aggressive subsurface environment no further consideration of an aggressive corrosion environment is justified.

Table 4.4 – Corrosion Test Results, 200-Series Borings

Boring No.	Layer	Sample No.*	Depth (feet)	Resistivity (ohm-cm)	pH (dim)	Sulfate Content (ppm)	Chloride Content (ppm)
HB-DMB-202	Fill	3D, 4D	9 – 11, 14 – 16	1,240	6.41	< 10	19
BB-DMB-204	Fill	3D, 4D	9 – 11, 14 – 16	27,892	6.65	< 10	< 10
BB-DMB-204	Till	5D, 6D	19 – 21, 24 – 26	8,264	5.83	24	< 10

*Due to insufficient sample size, first sample was composited with second sample for resistivity testing.

Based on the AASHTO and ASTM test results and following the evaluation procedures indicated in FHWA GEC 7 and FHWA GEC 12, the conditions at the site represent a low possibility of uniform or macrocell corrosion and the design was advanced given this conclusion.

5.0 SUBSURFACE CONDITIONS

5.1 Generalized Subsurface Stratification

A thorough review of the boring logs and laboratory test results was conducted to establish the general stratigraphy of the site. The purpose of "stratifying" a site is to group similar soil materials that have the same geologic origin. Stratification not only aids in visualizing the subsurface conditions, but also facilitates computations since the soil material throughout a particular stratum usually exhibits similar engineering characteristics, e.g., strength and compressibility. It should be noted that the strata boundaries shown on the logs have been inferred using available subsurface data.

In general, the site stratigraphy consists of granular embankment fill over native glacial till, underlain by schist bedrock. Sampled layers with common physical properties and similar SPT-N values, corrected to 60% hammer efficiency (N60), were grouped together as a single layer when the coefficient of variation between the corrected SPT N values was low and similar soil behavior characteristics and index properties were expected. An average corrected N value was then assigned to each layer. Several references were utilized to correlate the SPT N values to quantitative soil properties and strength values, as presented later in Section 5.2. A general description of each stratum encountered is included below. The prime component is identified in all capital letters, and secondary components are quantified by percent using Burmister terminology (in quotations) as described in Reference 6 and summarized below.

- **F – Marine Sand/Fill**: The site is overlain by approximately 18.0 to 24.6 feet of granular material. This material generally consists of medium dense to dense, fine to medium grained, brown to tan-brown SAND with trace (0 to 10%) to some (20 to 35%) silt and gravel. SPT-N60 values ranged from 6 to 37 blows per foot, with an average value of 18 blows per foot. This material is similar in composition to nearby marine sand deposits and appears to be a combination of in-situ marine sand and locally-sourced borrow.

- **T – Glacial Till:** The fill layer overlies native soil composed of glacial till material. The glacial till layer generally consists of medium dense to very dense, fine to medium grained gray-brown SAND with trace to some silt and gravel, and trace clay. Cobbles were occasionally encountered. In borings where bedrock was encountered this stratum ranged from 2.2 to 27.1 feet thick. The bottom of the layer was not reached in boring HB-DMB-202, which was terminated at a depth of 51 feet. SPT-N60 values ranged from a minimum of 49 blows per foot to greater than 100 blows per foot. A sublayer of cobbles and coarse gravel of approximately 3.2 feet thick was encountered in borings BB-DMB-101 just above the top of rock.

- **R – Schist Bedrock:** The entire project area is underlain by light and dark gray schist bedrock of the Vassalboro Formation. This rock is fine to medium grained, exhibits slight to no weathering, and generally consists of close to moderate joint spacing. A total of 52.4 feet of rock cores were collected in this stratum. Recovery and RQD had weighted averages of 98% and 66%, respectively.

A subsurface profile was developed for the proposed structure location along the Pinkham Brook Road baseline. This profile is included in the Drawings and reproduced as Figure 5. The stratification at each substructure location is summarized below in Table 5.1.

Table 5.1 – Summary of Site Stratification

Layer	Top of Layer (feet)							
	Abutment 1				Abutment 2			
	BB-DMB-101		BB-DMB-203		BB-DMB-102		BB-DMB-204	
	Depth	Elev.	Depth	Elev.	Depth	Elev.	Depth	Elev.
F – Marine Sand/Fill	0.4	84.3	0.0	85.0	0.4	84.3	0.0	84.0
T – Glacial Till	19.0	65.7	22.5	62.5	19.0	65.7	18.0	66.0
R – Schist Bedrock	29.0	55.7	38.5	46.5	21.2	63.5	31.0	53.0

5.2 Subsurface Material Properties

The geotechnical design parameters for soil and rock selected for analysis were developed using laboratory test results, standard published correlations, and engineering judgment. The soil parameters were developed for each stratum at each of the substructure/boring locations. A summary of soil and rock design properties is presented below in Tables 5.2.1 and 5.2.2. Proposed abutment backfill should be evaluated based on the parameters recommended for Granular Borrow by the MaineDOT "Bridge Design Guide", as discussed below in Section 6.

Table 5.2.1 –Engineering Properties of Soils for Embankments

Layer	$\overline{N_{60}}$ (bpf)	$\overline{N1_{60}}$ (bpf)	γ (pcf)	ϕ (deg.)	E (ksi)	υ (dim)
F – Marine Sand/Fill	24	20	121	35	6.11	0.29
T – Glacial Till	49	40	125	40	10.9	0.40

Table 5.2.2 –Engineering Properties of Soil and Rock at Abutments

Layer*	$\overline{N_{60}}$ (bpf)	$\overline{N1_{60}}$ (bpf)	γ (pcf)	ϕ (deg)	k_{aw} (pci)	k_{bw} (pci)	E (ksi)	υ (dim)	k_{rm} (dim)	C_o (ksi)
F – Marine Sand/Fill	18	22	118	34	115	71	5.28	0.27	---	---
T – Glacial Till	50	45	126	40	275	155	11.11	0.40	---	---
R – Schist Bedrock	---	---	170	---	---	---	4,800	---	0.002	11.5

* Based upon borings BB-DMB-101, BB-DMB-102, BB-DMB-203, BB-DMB-204.

Where:
$\overline{N_{60}}$= Average SPT-N value of stratum, corrected for hammer efficiency.
$\overline{N1_{60}}$= Average SPT-N value of stratum, corrected for hammer efficiency and effective overburden pressure.
γ = Total unit weight of soil, based on grain size and relative density/consistency per Bowles 5th Edition, Table 3-2.
ϕ = Internal friction angle of soil, per SPT-N value correlations in separate analysis.
k_{aw}= Subgrade modulus above water table, based on Figure 6.8.7-1, pp. 70 of API RP 2A.
k_{bw}= Subgrade modulus below water table, based on Figure 6.8.7-1, pp. 70 of API RP 2A.
E= Soil modulus of elasticity, per separate analysis based on typical values and SPT-N value correlations as presented in AASHTO LRFD Table C10.4.6.3-1. Intact rock modulus of elasticity per test results.
ν = Poisson's ratio, based on typical values per grain size and relative density as presented in AASHTO LRFD Table C10.4.6.3-1.
k_{rm} = strain factor

C_o = Uniaxial compressive strength of rock, average value per test results.

5.3 Groundwater

Groundwater depths could not be measured in the borings due to depth of water, use of wash drilling, and drilling access/schedule limitations. Given the majority of the embankment is constructed of marine sand and gravel with trace to little fines, it is assumed to be well-draining. As such, the design groundwater elevation was assumed to be equal to the normal high water level of 68.45 feet as given by the Hydrology and Hydraulics Report separately prepared and submitted by HNTB.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 General

Geotechnical design recommendations for the abutment foundations and approach embankments associated with the Tracy Brook Bridge Replacement project are discussed in the following sections. Recommendations have been developed in accordance with the AASHTO LRFD Bridge Design Specifications, 9th Ed. (AASHTO LRFD), and the MaineDOT Bridge Design Guide (BDG).

6.2 Integral Abutment Design and Recommendations

6.2.1 Foundation Type Selection

Pile supported integral abutments were chosen as the preferred abutment superstructure/substructure combination due to advantages associated with fast construction, low maintenance costs, and a long service life associated with the jointless construction. Integral abutments are typically supported on driven H-piles, with the weak axis oriented perpendicular to girder centerline to increase flexibility. Due to the shallow depth of bedrock which has been identified as low as seven feet below bottom of abutment, however, driven pile foundations were not considered viable. Piles were analyzed under the short embedment but were unable to achieve adequate lateral fixity with the limited overburden, resulting in the potential for double-hinge type behavior and abutment instability. To ensure a stable foundation system, rock-socketed H-piles were also evaluated.

An analysis was performed comparing the performance of available 10-inch, 12-inch, and 14-inch H-pile sizes under the defined longitudinal thermal displacements and with axial loading for 5-pile, 6-pile and 7-pile configurations. All piles were assumed to conform to the requirements of ASTM A572, Grade 50. HP14x102 piles were chosen as the preferred foundation type to resist the relatively high bending moments from thermal displacement and required per-pile axial demands for the given 75-foot concrete span and abutment configuration. Five (5) HP14x102 piles are proposed at each abutment, to be installed in predrilled holes with a minimum 24-inch diameter, 4-foot-deep grout-filled rock socket.

6.2.2 Design Methodology

Integral abutment piles have been designed for relevant Service and Strength limit state conditions in accordance with AASHTO LRFD Articles 3.4.1 and 11.5.5. The abutment pile foundations have been designed to resist the bridge and substructure vertical dead loads, live loads, vertical component of earth pressure loading, and horizontal thermal deformations following the integral abutment pile design procedure indicated in BDG Section 5.4.2.4 and the "simplified method" of integral abutment design as presented in the "Integral Abutment Bridge Design Guidelines" by the Vermont Department of Transportation (VTrans).

Under the Service limit state condition, a resistance factor of 1.0 has been used to assess pile vertical displacement. Additionally, the overall stability of each abutment has been assessed under Service limit state conditions with a minimum resistance factor of 0.75 (which equates to a factor of safety using limit equilibrium stability analyses of approximately 1.3) in accordance with AASHTO LRFD Article 11.6.3.7.

The design of H-Pile foundations under Strength limit state conditions has been advanced for the compressive axial geotechnical resistance of individual piles and their rock sockets, the structural resistance of individual piles in axial compression, and the structural resistance of individual piles in combined axial compression and flexure.

Geotechnical axial resistance of the rock-socketed piles was assessed based on AASHTO LRFD methodology for design of drilled shaft foundations. In order to limit displacements and simplify field verification requirements, the piles were designed such that the factored side resistance of the rock socket alone would meet or exceed the maximum Strength condition factored axial pile demands. Resistance factors under the Strength limit state conditions have been determined in accordance with AASHTO LRFD Table 10.5.5.2.4-1. A geotechnical resistance factor (φ_{qs}) of 0.55 has been selected for side resistance calculations, with side resistance estimated without load testing per the Kulhawy et al. method (2005). The factored geotechnical resistance using a resistance factor of 0.55 was assessed relative to the factored structural resistance to determine the limiting resistance.

An axial structural resistance factor (φ_c) of 0.6 has been used to assess the structural resistance of the H-Pile in compression and not subjected to bending in accordance with AASHTO LRFD Article 6.5.4.2, given that the piles will be installed in a pre-drilled holes without concern of installation damage. The structural resistance of each H-Pile has also been checked against combined axial compression and flexure in accordance with AASHTO LRFD Articles 6.9.2.2 and 6.15.2. As per Article 6.5.4.2, an axial resistance factor (φ_c) of 0.7 has been used when piles are subjected to both axial compression and flexure and the corresponding flexure resistance factor (φ_f) of 1.0 has been used in the interaction equation specified in AASHTO LRFD Article 6.9.2.2 to determine structural resistance. Following the VTrans "simplified method" of analysis, a single plastic hinge has been permitted to form at the pile head, provided that the remaining length of pile below the hinge remains elastic, within factored limits, and fully fixed.

6.2.3 Assessment of Corrosion Potential

Based on the guidance provided in AASHTO LRFD Section 10.7.5, corrosion testing performed in accordance with AASHTO and ASTM standards at the proposed abutments indicated generally non-aggressive corrosion conditions. The potential for aggressive corrosion is expected to be small given the results of this testing and the necessity to remove and replace a significant portion of the existing fill above the water table in order to demolish the existing structure and construct the proposed abutments. For these reasons, a limited design corrosion loss of 1/16 of an inch along all exposed surfaces is assumed for long-term pile design. No further mitigation measures are recommended, provided that all new fill placed on site in the vicinity of the proposed abutments meets the requirements of MaineDOT Specification 703.19.

6.2.4 Pile Demands

Design pile loading information at each abutment was developed by evaluating each of the applicable limit states in accordance with BDG Section 5.4.1.2. The limit states evaluated include Strength I and Service I. Table 6.2.4 summarizes the maximum factored pile head axial demands and maximum thermal displacements that were assessed for design.

Table 6.2.4 – Summary of Maximum Pile Head Demands per Pile

Limit State Condition	Abutments 1 and 2		
	Max Factored Axial Compressive Force (kips)	Max Thermal Displacement – Expansion (inches)	Max Thermal Displacement – Contraction (inches)
Strength I	398	0.234	0.307
Service I	290	0.234	0.307

6.2.5 Axial Pile Resistance and Design Considerations

The maximum factored axial compression demand from Table 6.2.4 is 398 kips per pile at Abutments 1 and 2 corresponding to the Strength I limit state. Using a geotechnical resistance factor of 0.55 associated with side resistance of a concrete-filled drilled shaft in rock yields a required nominal geotechnical resistance of 724 kips per pile at both abutments under Strength limit state conditions. Using a geotechnical resistance factor of 1.0 under the Service I limit states yields a required nominal geotechnical resistance of 290 kips at Abutment 1 and Abutment 2. Therefore, the governing required nominal geotechnical resistance occurs under the Strength limit state and is 724 kips at Abutments 1 and 2.

The nominal geotechnical resistance of the piles was estimated based on the side resistance of the grout-filled rock sockets. Geotechnical resistance was estimated following the procedures provided for computing nominal side friction for drilled shafts in rock per AASHTO LRFD Section 10, Article 10.8.3.5.4. Based on the subsurface investigation performed at the project site and inspection of recovered rock cores, it is anticipated that the rock mass is sufficiently intact such that rock sockets may be excavated without artificial support. Unit side friction therefore was estimated based on intact rock strength and proposed grout strength of 6 ksi using AASHTO Equation 10.8.3.5.4b-1, assuming a regression coefficient of 1.0. As the design rock strength exceeds grout strength, the ultimate unit side friction is found to be controlled by the strength of grout. Following this procedure, the nominal unit side friction of rock socket is estimated as 42.8 ksf, resulting in a minimum 2-foot-diameter rock socket length of 2.7 feet to satisfy the required 724 kip nominal resistance. The recommended minimum rock socket length has been rounded up to 4 feet, or two diameters, to ensure lateral fixity.

The factored axial structural resistance for piles in compression without flexure was calculated following the procedures outlined in AASHTO LRFD Section 6, Article 6.9.4.1. This procedure incorporated buckling effects for integral abutment pile free length as defined in BDG section 5.4.2.4. Compression resistance was calculated for top, middle, and bottom (fully braced) segments of pile, with the minimum controlling resistance for selected pile size presented herein.

Table 6.2.5 summarizes the maximum factored axial structural compression demand on a single HP14x102 pile with 4-foot-long, 2-foot-diameter rock socket, compared to the factored geotechnical and structural compression resistance under the governing strength limit state. For this evaluation, structural resistance is evaluated in compression only as discussed above, without bending and combined stress effects

Table 6.2.5 – Factored Axial Demand and Resistance of HP 14x102 at Strength Limit State

HP 14x102	Factored Structural Demand (kips)	Factored Structural Resistance[1] (kips)	Factored Geotechnical Resistance[2] (kips)	Governing Factored Axial Compressive Resistance (kips)
Abutment 1	398	669	591	591
Abutment 2	398	700	591	591

Note: 1. Based on axial compression loading only with piles remaining undamaged and a resistance factor of 0.6.
2. Assumes piles derive resistance by rock socket side friction only and a resistance factor of 0.55.

Controlling factored axial compressive resistance is governed by factored geotechnical resistance at both abutments, with rock sockets providing excess geotechnical resistance. The governing factored axial compressive resistance is greater than the maximum factored demand and satisfies design requirements. Axial-only loading does not control the design, and pile size selection was based on structural resistance in combined axial compression and flexure.

6.2.6 Lateral Pile Resistance

Single pile lateral analysis for integral abutment piles was conducted using the LPILE computer program from Ensoft, Inc., version 2019.11.001. The ground surface was assumed to be horizontal at the top of abutment for analysis of pile reaction due to thermal expansion, and sloped from bottom of abutment for thermal contraction. Piles were analyzed under the thermal displacements and factored axial demands indicated above in Table 6.2.4.

Lateral analyses were first performed with defined pile head deflections corresponding to the thermal expansion and contraction at each abutment, assuming a fixed-head condition (zero slope condition). The plastic moment resistance of the pile was calculated using unbraced lengths and effective length factors as a result of the first LPILE analysis. The structural capacity of the pile was checked to determine if a plastic hinge formed for the specified displacement and specified axial thrust load based on the provisions of Article 6 of AASHTO and as per the recommendations of Section 5.4.2.4 of the MaineDOT BDG.

The resulting moment from the first LPILE iteration exceeded the calculated plastic moment, therefore a second LPILE analysis was performed to model the hinged condition using a specified pile head displacement and moment. The hinge allows the pile head to rotate with a constant moment, i.e. the plastic moment. The pile head transforms from a fixed connection to a pinned connection, thereby changing the pile behavior and the effective length of the top segment for stability checks. The specified pile head displacement in the second iteration remained unchanged while the specified pile head moment was equivalent to the plastic moment calculated from the results of the first LPILE iteration. The results of the second LPILE iteration were used to recalculate the pile axial and flexural structural resistance based on revised unbraced lengths and to ensure that a second plastic hinge does not develop.

Based on the above lateral analysis, for the proposed abutments and pile configuration, an HP14x102 pile was determined to be the minimum required pile size to satisfy combined stress requirements.

6.2.7 Pile Settlement and Elastic Pile Compression

HNTB recommends that the piles be socketed into bedrock with the factored resistance satisfied based on rock socket side resistance alone. In this configuration, the pile will experience negligible settlement at the pile tip. Therefore, the majority of the settlement at the bridge abutments will be a result of the elastic compression of the H-Pile itself. This compression is estimated to be less than 1/8 inch at each abutment.

6.2.8 Pile Recommendations

Based on the results of the above analysis, the recommended foundations for the proposed integral abutments shall consist of a total of five (5) HP14x102 piles per abutment, conforming to the requirements of ASTM A572, Grade 50. Piles shall be installed in predrilled holes with 24-inch diameter, 4-foot-deep grout-filled rock sockets. Grout for rock sockets shall have a minimum 28-day compressive strength of 6,000 psi. Above the top of grout, pile holes shall be backfilled with a non-compacting backfill material, such as underdrain backfill Type C. Final rock socket tip elevation and top of grout elevation shall be verified in the field to ensure a minimum 7.5-foot length of pile above the top of grout to bottom of abutment pile cap to ensure adequate pile flexibility. Any casing used for excavation of predrilled holes and rock sockets must be removed before construction of the abutment.

Minimum pile requirements and estimated top of rock elevations are summarized below in Table 6.2.8. Top of rock elevation varies along and between abutments and is therefore estimated for each stage of abutment construction. Stage 1 represents the final Route 125 southbound travel lane, while Stage 2 represents the side of bridge closest to the Androscoggin River and the final Route 125 northbound travel lane.

Table 6.2.8 – Pile Tip Elevations and Nominal Driving Resistances

Location	Min. Factored Geotechnical Compression Resistance (kips)	Min. Nominal Geotechnical Compression Resistance (kips)	Estimated Top of Rock Elev. (ft)	Minimum Rock Socket Length (ft)	Estimated Tip of Pile Elev. (ft)
Abutment 1 Stage 1	398	724	46.5	4.0	42.5
Abutment 1 Stage 2	398	724	55.7	4.0	51.7
Abutment 2 Stage 1	398	724	53.0	4.0	49.0
Abutment 2 Stage 2	398	724	63.5	4.0	59.5

HNTB recommends ordering lengths of piles reflect a minimum of 5 additional feet per pile beyond the deepest estimated pile tip elevation per abutment to accommodate variations in pile penetration and depth to bedrock.

6.2.9 Lateral Earth Pressures

As per MaineDOT BDG Section 5.4.2.11, integral abutment and wingwall reinforcement should be designed to resist the full passive earth pressure that acts on the back face of the abutment or wingwall when the bridge expands. Passive pressure acts in a triangular pressure distribution on the full height of the abutment backwall from the bottom of the approach slab to the bottom of the abutment/pile cap. Calculation of passive earth pressure should assume a Coulomb passive earth pressure coefficient in accordance with the BDG Section 3.6.6 using the material properties defined in BDG Table 3-3 for a Granular borrow.

Developing full passive earth pressure requires significant displacement of the wall into the soil mass. Per 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 that required to develop full passive pressure, Rankine passive earth pressure coefficients may be considered in accordance with BDG Section 5.4.2.11. A load factor for passive earth pressure is not specified in AASHTO LRFD, however a load factor for passive earth pressure (γ_{EH}) equal to 1.50 should be applied in accordance with BDG Section 5.4.2.11.

Wingwalls should generally not exceed 10 feet in length and should be straight cantilevered extension wings where possible. As per BDG Section 5.4.2.11, piles should not be placed under wingwalls that are integral with the abutment stem. Because of the high bending moments due to passive pressure in wingwalls 10 feet or longer, it may be necessary to support longer wingwalls on their own independent foundations that are isolated from the abutment. If this is the case, a flexible joint must be provided between the wingwalls and the backwall.

Active earth pressure coefficients using either Coulomb or Rankine theory can be used for the design of yielding walls that include: gravity walls and abutments, semi gravity walls, prefabricated modular walls with steep backfaces and cantilever walls and abutments with short heels as defined by AASHTO LRFD Figure C3.11.5.3-1. Table 6.2.9 summarizes Coulomb and Rankine active and passive earth pressure coefficients for a granular borrow material with a friction angle (ϕ) of 32 degrees and an interface friction angle (δ) between soil and concrete of 24 degrees in accordance with BDG Table 3-3 for a Granular borrow, assuming a vertical backface, horizontal backfill, and a total unit weight of 125 pcf.

Table 6.2.9 – Recommended Earth Pressure Coefficients

Earth Pressure Theory	ϕ (deg.)	δ (deg.)	α (deg.)	β (deg.)	k_p (dim)	k_a (dim)
Coulomb Theory	32	24	90	0	8.378	0.275
Rankine Theory	32	24	90	0	3.255	0.307

Where: ϕ = Internal friction angle of soil, standard value per BDG Table 3-3.
δ = Interface friction angle (δ) between soil and concrete, standard value per BDG Table 3-3.

α = Angle of back face of wall to horizontal.
β = Angle of backfill to horizontal.
k_p = Passive earth pressure coefficient, per BDG Section 3.6.6.
k_a = Active earth pressure coefficient, per BDG Section 3.6.5.

For any retaining walls required for the support of existing embankment fill, such as temporary sheet pile retaining walls, the active and passive earth pressure coefficients may be recalculated using the equations provided in BDG Section 3.6.5 and 3.6.6 based on the properties previously provided for existing Marine Sand/Fill in Section 5.2. For this existing fill, a total unit weight (γ) of 121 pcf, a friction angle (ϕ) of 35 degrees, and an interface friction angle (δ) of 23 degrees may be assumed.

A live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height, H, behind the back of any wall. The additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by an equivalent height of soil defined in Table 3-4 of the BDG. The surcharge will result in the application of an additional uniform, constant horizontal pressure on the back of the wall. When a structural approach slab is specified, a reduction of the live load surcharge is permitted in accordance with AASHTO LRFD Article 3.11.6.5.

6.2.10 Global Stability at Bridge Abutments Longitudinal to Bridge Alignment

Global stability was evaluated for the proposed conditions at the bridge abutments following the ground surface profile specified along the baseline of Pinkham Brook Road. Subsurface conditions were modeled by laying out the stratigraphy of each boring at their respective distances from the abutment with strata boundaries assumed to follow straight lines between borings. Two-dimensional limit equilibrium analyses were performed at Abutment 1 (south abutment) using SLIDE2 V9.018 by Rocscience Inc. Analysis of the south abutment alone was considered adequate as the geometry and soil conditions of the abutments as taken through the baseline are nearly identical, with the exception being the depth to bedrock. Bedrock was encountered at the north abutment in boring BB-DMB-102 at less than 8 feet below the base of the proposed north abutment pile cap, leaving little room for deep-seated failure surfaces and proving a less critical design condition than the south abutment.

Global stability analysis was performed for long term loading conditions using the drained soil strength design parameters specified in Table 5.2.1. Short term loading conditions were not assessed due to the granular nature of the soils located within the project area. A surcharge load of 250 psf was applied to the approach embankment for all slope stability analyses to simulate the vehicular live load.

AASHTO LRFD requires a resistance factor of 0.65 (equivalent to a factor of safety of 1.5) for the global stability of slopes supporting structures. Spencer's method of analysis has been used to perform all global stability analyses and satisfies both force and moment equilibrium, meeting the recommendations of AASHTO LRFD commentary Article C11.6.2.2 for slope stability analysis. Results of the analysis were assessed using optimized failure surfaces, the results of which have been provided herein.

The minimum factor of safety for the proposed conditions with the ordinary/normal high water level (as provided by the "Preliminary Hydrologic and Hydraulic Report" by HNTB) of 68.45 feet was 2.0, satisfying AASHTO criteria. This analysis incorporates resistance provided by the integral abutment as represented by an active earth pressure applied horizontally as an external load distributed across the vertical limits of the proposed abutment. The active earth pressure is the minimum resistance the substructure would provide, given the abutments are independently stable, framed into the superstructure, and will vary between active and passive conditions, making this assumption conservative. An analysis was also performed with the water level raised to the 100-year flood elevation of 83.50 feet (as provided by the "Preliminary Hydrologic and Hydraulic Report" by HNTB), which also resulted in a satisfactory condition with a 1.9 factor of safety.

6.3 Approach Embankment Design and Recommendations

6.3.1 Basis of Analysis

The existing embankment carrying Pinkham Brook Road slopes down steeply toward the Androscoggin River, with portions of some cross-sections as steep as 1:1 (horizontal : vertical). This steepened embankment condition begins near the southern project limits and extends well beyond the project limits to the north. These slopes show some signs of historic shallow sloughing, as evidenced by local areas of surficial riprap, but are generally well vegetated and nominally stable. In the proposed condition, the roadway alignment will be raised and shifted, and these steepened slopes will be modified or subjected to additional loading. These modifications have the potential to compromise the stability of the proposed roadway footprint from approximately Station 12+50 through 17+50. This shift will also require an embankment widening with considerable fill placed towards Meadow Brook along the southwest slope from approximately Station 13+25 to 15+00.

Global stability was analyzed for the proposed roadway within the limits of widening and slope modification in order to check for compliance with AASHTO LRFD stability requirements, and to identify any areas requiring stabilization. Stabilization and reinforcement needs were limited to areas where unsatisfactory global stability failure surfaces extend into the limits of proposed roadway, and within the above-stated limits of modification. Shallow stability surfaces not imminently affecting the proposed roadway were not considered for stabilization. Modifying these slopes to meet factor of safety requirements would require significant clearing along steep vegetated slopes outside of current project limits and right-of-way, and would trigger potential impacts to federally-regulated floodplain areas. As such, it is our opinion that these areas may be left in their current condition provided that the Owner monitor these slopes and provide maintenance/repair of any observed sloughing or surficial erosion.

6.3.2 Stability Analysis – Unreinforced Slopes

In analyzing the north and south approach embankments, HNTB performed 2D limit-equilibrium slope stability analysis of critical embankment cross-sections using the SLIDE2 V9.018 by Rocscience, Inc. Proposed roadway cross-sections were provided by the project highway engineer. Sections representing maximum heights and steepest slopes were assessed, with at least one section analyzed over every hundred feet from Sta 12+00 to the south abutment, and from the north abutment to Sta 18+25.

Subsurface conditions for global stability analysis were selected based on conditions at the nearest two borings. Spencer's method of analysis has been used to perform all global stability analyses and satisfies both force and moment equilibrium, meeting the recommendations of AASHTO LRFD commentary Article C11.6.2.2. Results of the analysis were assessed using a search for non-circular failure surfaces, which permits a larger sampling of failure surfaces be included in the assessment, including more refined failure surfaces than those found in a search for circular failure surfaces.

Global stability analysis was performed for long term loading conditions using drained soil strength design parameters as specified in Table 5.2.1. Short term loading conditions were not assessed due to the granular nature of the soils located within the project area. Stability was assessed using the design river water elevation for a 1.1-year flood (ordinary high water) of 68.45 feet, which was considered for the service limit state. These conditions were analyzed with a target minimum factor of safety of 1.3, equivalent to a service limit state resistance factor of 0.75 as specified by AASHTO LRFD for the global stability of slopes not supporting structures. As an additional check, analyses were performed with water raised to the FEMA 100-year flood of 83.50 feet, with a target factor of safety of 1.1 for this extreme condition. A surcharge load of 250 psf was applied within the proposed roadway limits for all slope stability analyses to simulate vehicular live load.

Analysis results for stability of unreinforced slopes are summarized below in Table 6.3.2. Where the minimum factor of safety for global stability extending into the proposed roadway was found to be less than 1.3, stabilization of the slope is recommended to achieve adequate factors of safety. Stabilization system design recommendations are addressed in the subsequent sections of this report.

Table 6.3.2 – Summary of Global Stability Results

Station / Transverse Direction	Minimum Factor of Safety Impacting Roadway		Recommendation
	Normal High Water	100-Year Flood	
12+25 / Right	1.3	1.1	Does not impact roadway beyond the shoulder – No Modification Needed
13+00 / Right	1.0	0.9	**Reinforcement Required**
14+50 / Right	1.2	1.2	**Reinforcement Required**
14+75 / Right	1.3	1.3	No Modification Needed
15+00/ Right	1.3	1.4	No Modification Needed
16+75 / Right	1.2	1.2	**Reinforcement Required**
18+25 / Right	1.3	1.3	No Modification Needed
13+75 / Left	1.3	1.2	No Modification Needed
14+50 / Left	1.3	1.2	No Modification Needed
15+25 / Left	1.8	2.0	No Modification Needed

6.3.3 Stability Analysis - Soil Nail Reinforced Slopes

The slopes along the Androscoggin River extend steeply from the top of embankment to the water below, creating a challenging condition for stabilization where needed. Several methods of stabilization were considered to balance cost and performance. Shallow surficial stabilization systems such as riprap and erosion control measures were not deemed adequate since the controlling conditions impacting the roadway extended deep and beyond the practical limits of such systems. Launched soil nails were evaluated due to their ease of installation and limited disturbance to existing ground surface, but these lighter elements lack pullout strength and operate primarily in shear against shallow failures. These systems were not found to be effective in resisting the deeper failure surfaces controlling the factors of safety for this project. Traditional drilled and grouted soil nails were considered, and while viable, require a multi-staged installation process along the steepened slopes likely requiring staging of drill rods and casings with limited space or access. To limit the steps of installation and simplify construction, hollow-bar soil nails were selected as the preferred method of stabilization. Hollow-bar soil nails will be installed in a single process, grouted through the center of hollow bar, without need for installation and removal of casing, use of separate drill rods, or installation of separate reinforcement.

As with the unreinforced slopes, the hollow-bar reinforced slopes were analyzed using the limit-equilibrium computer program SLIDE2 by Rocscience, Inc. The "active method" for reinforcement resistance was utilized, where an independent factor of safety (or resistance factor) is applied to the resistance provided by the reinforcing elements. This is the preferred methodology as it allows the appropriate factor of safety specific to the strength of the reinforcing element to be applied. The global stability factor of safety is not applied to the reinforcing elements. Soil nails were designed using Allowable Stress Design procedures in general accordance with FHWA GEC 7, "Soil Nail Walls Reference Manual." Soil nail layout, spacing and length were adjusted to yield a model with acceptable global stability factor of safety, without exceeding the factored structural and geotechnical resistance of individual soil nails. The software accounts for nail spacing across the slope by dividing the resistance offered by a single element by the user-input longitudinal spacing. The resistance provided by the reinforcement is provided parallel to the reinforcement length. A minimum of 3 rows of nails was considered to provide appropriate coverage and redundancy. Nail lengths and inclination were selected to intercept potential failure surfaces shown to provide inadequate factors of safety in the proposed conditions as assessed without reinforcement.

Design of individual soil nails assumed an ultimate geotechnical grout-to-ground bond strength of 2.9 ksf based upon values given for rotary drilled nails in sand/gravel per Table 4.4a of FHWA GEC 7. For structural sizing of the steel hollow bars, a minimum yield strength of 75 ksi was assumed. A sacrificial steel thickness of 1/8" was assumed around the outer perimeter of the hollow bars, based on the hollow bar design recommendations from GEC 7 Section 10.4.2, Equation 7.1 and assuming a conservative design life of 100 years. Nail design assumed a geotechnical pullout factor of safety of 2.0 and a tendon structural tensile resistance factor of safety of 1.8 per GEC 7 Table 5.1.

6.3.4 Hollow-Bar Soil Nail Recommendations

Based on the analysis and assumptions presented above, the recommended hollow bar soil nail geometry and design loads for reinforcement of the right-size (northeast) slopes are summarized below in Table 6.3.4.

Table 6.3.4 – Summary of Hollow Bar Soil Nail Design

Station Limits	Inclination from Horizontal	Soil Nail Arrangement	Bottom Nail Elev. (feet)	Hollow Bar Dimensions		Grout Diameter (inches)	Max. Service Design Load (kips)
				Min. Nominal Outer Diameter (inch)	Min. Net Area Through Threads (sq. inch)		
12+50 to 12+90	30°	4 rows, 20ft length	92.0	1.50	1.067	4.0	23
12+91 to 13+30	30°	4 rows, 20 ft length	81.0	1.50	1.067	4.0	23
13+31 to 13+60	30°	4 rows, 20 ft length	79.0	1.50	1.067	4.0	23
13+61 to 14+00	30°	3 rows, 20 ft length	79.0	1.50	1.067	4.0	23
14+01 to 14+50	30°	3 rows, 20 ft length	77.0	1.50	1.067	4.0	23
16+65 to 17+50	30°	3 rows, 15ft length	73.0	1.50	1.067	4.0	23

Based on iterative analysis, a design spacing of 7 feet between soil nail rows (vertical spacing) and a 7-foot spacing between columns (horizontal spacing) is recommended, as measured along the slope face. This spacing provides good coverage of the steeper upper limits of the slope while generally within reach of equipment operating from top of embankment, and results in reasonable soil nail demands. A stagger is recommended between soil nail rows, equal to ½ of the column spacing. While this was not incorporated into the global stability analysis, it is common practice to incorporate staggering to reduce the risk of failure between columns of nails. The lower/bottom-most row of soil nails should be installed at the bottom nail elevation indicated in Table 6.3.4, with additional rows located up-slope in 7 foot increments. To avoid conflicts with the proposed bridge wingwalls, the upper-most row of nails should be located as to ensure a minimum 2-foot vertical clearance between soil nails and bottom of concrete wingwall. A minimum 4-inch drilled/grouted diameter is recommended for all soil nails to satisfy geotechnical requirements at this spacing. A steel hollow bar of minimum 1.5-inch outer diameter is recommended to satisfy structural demands and allow for use of drill bits necessary to achieve the recommended hole diameter.

To satisfy design assumptions, soil nails should be constructed to the size, length, spacing, and inclination recommended in the above table, using hollow bar tendons conforming to the requirements of ASTM A615 with a minimum yield strength of 75 ksi. Grout used for final grouting of soil nails should have a minimum 28-day compressive strength of 4 ksi.

6.3.5 Surface Detail Recommendations

After completion of soil nail installation, a slope stabilization mat consisting of an erosion control blanket and galvanized double-twist wire mesh should be installed over the soil nails, from one foot above the top row of nails to at least one foot below the bottom-most row of nails and at least 1 foot beyond the nail limits at end columns. The erosion control blanket should extend above the wire mesh to the crest of the slope. This mat will serve to temporarily stabilize the surface conditions from erosion and sloughing until vegetation of the surface is restored, and will provide catchment and retainage in the event of a local failure between or around nails, transferring load to the soil nails to resist further soil movement.

A square steel bearing plate, washer, and threaded bar nut should be threaded over the slope stabilization mat and hand-tightened down to the finished ground surface to retain the mat. For corrosion protection, the plate, washer, and nut should be fusion-bonded epoxy coated in accordance with ASTM A775. All bar stickup length above top of grout should be epoxy coated in the field with compatible patching epoxy supplied by the manufacturer of the Fusion-Bonded Epoxy used on the other components. The epoxy coating of the plate, washer, and nut typically perform for a service life of 15 to 20 years and are not intended to provide a 100-year service life without maintenance. It is recommended that the epoxy coating of the nail heads be inspected periodically, coating be repaired as necessary with compatible patching epoxy, and nail head hardware be replaced if coating repair is not achievable.

6.3.6 Hollow-Bar Soil Nail Considerations

Installation of traditional, solid-bar soil nails is typically a multi-staged process. In the first stage, the nail is excavated by advancing a drill string consisting of a reusable drill bit and segmental threaded drill rods, with spoils returned to the surface typically through reverse circulation using compressed air or drilling fluid pumped through the drill rods and out the bit. In collapsible soils such as sands and gravels, temporary casing is often advanced with the drill string to maintain support of the drill hole. Once the drill hole has been advanced to the required length and cleaned out, the drill string must be removed. Permanent reinforcement with centralizers and grout tube are placed into the hole. Finally, grout is pumped through the grout tube to the tip of nail until a clean grout return is observed at the surface. Any temporary casing required for excavation must be removed as the grouting advances. This process requires frequent access to the nail head location and an accessible staging area for storage of the drill string and casing. Given this limited access at the project site and the lack of a feasible staging area at the toe of slope, this process is especially challenging for this project.

An alternative to solid bar soil nails is the use of hollow-bar soil nails. A hollow-bar soil nail can be installed in a single stage utilizing externally threaded hollow bar as both the drill string and permanent reinforcement, with a sacrificial drill bit installed at the tip of the bar. Bars are added as needed during advancement of the soil nails using threaded bar couplers, and drill hole stability in potentially collapsible soils is achieved by pumping of grout down the hollow bar and through the drill bit as the excavation advances. The grout serves in the place of a drilling mud to resist hole collapse, returns spoils to the surface, and cools the drill bit. This process eliminates the need for installation, removal, and staging of temporary casing or reusable drill string.

A common concern with hollow-bar soil nails in long-term applications is corrosion of the tendon. Epoxy coating, galvanization, and encapsulation commonly used in corrosion protection of standard soil nails would be damaged during hollow-bar drilling, and damaged coatings could unintentionally result in locally accelerated corrosion. A 2010 study published by the FHWA (Reference 11) identified a lack of standardized guidance for hollow-bar soil nail corrosion mitigation leading to a general hesitancy in their use, and recommended several avenues to standardize protection using grout cover and/or sacrificial steel. Grout cover offers some protection, but grout can be cracked during testing and loading, potentially exposing the tendon. Detailed guidance was provided in Chapter 10 of the 2015 publishing of FHWA GEC 7 (Reference 12) incorporating sacrificial steel as the primary means of corrosion protection. For non-aggressive environments, a method is provided for calculation of sacrificial steel thickness based on design service life, and the effective outer diameter of the hollow-bar is reduced by twice this thickness when calculating effective cross-sectional area of the bar. This approach has been employed on this project to mitigate corrosion risk, with sacrificial steel thickness conservatively estimated based on a design service life of 100 years.

7.0 SEISMIC CONDITIONS

7.1 Seismic Activity

Unlike the seismically active regions in the western United States where the North American and the Pacific plates meet, earthquakes in the northeast occur less frequently, are typically smaller in magnitude, more complex and less understood than plate boundary activity. The project site is situated nearly halfway between the plate boundaries to the west and the Mid-Atlantic Ridge to the east where the North American Plate diverges from the Eurasian Plate. Seismic events occurring deep within these boundaries are known as interplate earthquakes. In the last few decades a common explanation for the cause of these earthquakes is that ancient zones of weakness are being reactivated in the present day stress field. Due to these uncertainties the level of seismic hazard in the northeast, as presented by USGS mapping, is based primarily on the past records of seismic activity rather than the location of geologically mapped faults.

According to USGS earthquake history, several significant earthquake epicenters have been recorded in the Northeast region. Table 7.1 summarizes some of the notable earthquakes of magnitude 4.0 or greater centered within 200 miles of the project site over the last 100 years, as reported by USGS earthquake archives.

Table 7.1 – Historic Nearby Earthquakes

Approximate Epicenter Location	Date (month/day/year)	Magnitude	Approximate Distance from Project Site (miles)
Waterboro, Maine	10/16/2012	4.7	41
Black Brook, New York	4/20/2002	5.3	184
Peru, Maine	5/29/1983	4.2	40
Sanbornton, New Hampshire	1/19/1982	4.5	84
Altona, New York	6/9/1975	4.2	185
Saint-Augustin-de-Woburn, Quebec	6/15/1973	4.8	101
Scarborough, Maine	4/26/1957	4.4	33
Sangerville, Maine	12/28/1947	4.5	91
Tamworth, New Hampshire	12/24/1940	5.6	61
Dannemora, New Hampshire	12/20/1940	5.3	66
Warrensburg, New Hampshire	4/15/1934	4.5	191
Ossipee, New Hampshire	4/20/1931	4.7	190
South Paris, Maine	10/9/1925	4.0	56

7.2 Seismic Evaluation

Seismic design of all proposed structures shall be in accordance with the AASHTO LRFD, as supplemented and modified by the MaineDOT BDG. For determining seismic loads and liquefaction criteria, values for the peak ground acceleration coefficient (PGA) and the short- and long-period spectral acceleration coefficients (S_S and S_1, respectively) were obtained from the USGS "U.S. Seismic Design Maps" online system. Values obtained are based on a 7% probability of exceedance over a 75-year period (1033-year return period) per 2002 USGS seismic data and 2009 AASHTO design criteria, with the project latitude and longitude in Durham, ME used as the location of interest. These values were compared and found to be consistent with the corresponding values provided in the 1000-year return period seismic response maps in AASHTO LRFD Figures 3.10.2.1-1 through 3.10.2.1-3.

The given base acceleration coefficients correspond to the peak ground acceleration at the top of rock (Site Class B). Given the shallow bedrock and primarily medium dense to dense cohesionless soil (granular embankment on stable deposits of glacial till), the project site meets the criteria given for Site Class D. The recommended design base acceleration coefficients, site factors, and maximum ground acceleration coefficients are presented below in Table 7.2.

Table 7.2 – Seismic Response Spectrum

Acceleration Coefficient	At Top of Rock	Site Factor	At Ground Surface
Peak Ground Acceleration	PGA= 0.083g	F_{pga}= 1.6	A_S= 0.132g
0.2 Second Period	S_S= 0.168g	F_a= 1.6	S_{DA}= 0.269g
1.0 Second Period	S_1= 0.045g	F_v= 2.4	S_{D1}= 0.108g

7.3 Liquefaction Assessment

As part of the seismic assessment, a determination of the liquefaction hazards present at the project site was conducted. Liquefaction is a phenomenon whereby a soil substantially loses strength in response to an applied cyclic stress, typically associated with earthquake loading. This temporary loss of soil strength causes the soil to behave like a liquid, impacting bearing capacity and lateral stiffness. Liquefaction induced ground movement can cause serious damage to structures. Damage may occur during the earthquake itself, or continue to occur or be initiated subsequent to the earthquake in situations where the static factor of safety against lateral movement is reduced to less than unity. Two types of post-liquefaction deformations are possible:

1. Horizontal shear deformation arising from the large shearing strains occurring in zones where the earthquake has induced initial liquefaction.
2. Settlements arising from volume changes that occur on reconsolidation accompanying dissipation of the large excess pore pressures in liquefied zones.

In general, strata meeting the following criteria are typically not susceptible to liquefaction and can be eliminated from the screening:

- Soil with fines content (percent passing through No. 200 sieve) more than 35 percent
- Soils classified as Marine and Lacustrine Silt and Clay
- Layers with SPT-N values greater than 30 blows per foot
- Unsaturated soils above the groundwater table

Conditions at the project site generally meet the above criterion for all layers. Embankment fill with SPT-N values of less than 30 blows per foot were typically encountered above the water table, with dense to very dense glacial till encountered below groundwater. As a verification, liquefaction potential was evaluated at all boring locations in accordance with the procedures outlined in Reference 2, EERI Monograph MNO 12 "Soil Liquefaction during Earthquakes" by Idriss and Boulanger. Based on history of past earthquakes in the vicinity of the project site, a conservative earthquake magnitude of 6.0 was used for the liquefaction analysis. Base acceleration coefficient values for the 1000-year seismic events were used for calculating earthquake-induced shear stresses, as given above in Table 7.2.

For use in the Idriss and Boulanger method, the base acceleration coefficients, also referred to as "Peak Ground Acceleration" values, are multiplied by a correction factor F_{PGA} based on the site conditions in order to estimate the maximum ground acceleration coefficient at the ground surface, A_s. This approach is outlined in Section 3.3.8 of Reference 8, FHWA GEC No. 3. By this method, the site can be classified as Site Class D, with average SPT-N values between 15 and 50 blows per foot. For Site Class D, Reference 8 Table 3-6 assigns an F_{PGA} value of 1.6 for the 1000-year event.

All layers sampled by SPT at the proposed abutment locations were analyzed for liquefaction. The minimum factor of safety calculated for the 1000-year seismic event was 1.58, with no layer exhibiting a risk of liquefaction. As such, the liquefaction hazards at the project site are considered low, and liquefaction was not considered further in the analysis and design of the foundation elements.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Abutment Construction and Pile Installation

The proposed bridge abutment foundations are located behind and partially offset from the existing abutments and will be constructed in phases with the first phase west of the existing bridge. The existing bridge footings will remain in place at the toe of slope in the proposed condition, and do not pose any obstruction to the installation of new foundations. The ground surface along the first phase of construction is sloped in its current condition and may require regrading or temporary supports to facilitate access of foundation drilling equipment at Abutment 1. Overhead utilities pass over the proposed southbound side of bridge and are to be relocated by others. The construction schedule should be coordinated with the utility relocations to ensure that overhead utilities are removed/relocated prior to start of the first phase of bridge foundation construction.

The piles are to be installed in a drilled hole with concrete-filled rock sockets. Drilled foundation risks includes loss of ground during excavation. Soils encountered include sand and gravel components and may be subject to collapse if not properly supported. Excavation by open hole may not be achievable, and temporary casing may be required to maintain hole stability. While drilling through soil below water table, positive head of water and/or slurry should be maintained to prevent running sand and blow in of material. There is also risk of encountering cobbles in the glacial till near top of rock. The Contractor should be prepared for cobbles if and where encountered, and these issues should be detailed in the Contractor's pile installation plan.

The rock should be expected to stand for extended periods in excavations and is suitable for rock socket construction. There is a potential of an uneven rock surface across the footprint of the drilled holes. It may be difficult to properly seat casing prior to advancing the rock socket. The estimated top of rock socket elevations provided in this report are generally based on the depth at which sound rock is expected based on the nearest boring but are approximate only. Actual top of sound rock must be verified in the field during pile installation. Further, depending on the Contractor's construction methods, it may be necessary to seat the temporary casing below the top of sound rock to ensure stability of the excavation and casing. The as-built rock socket length shall be measured from the top of sound rock or bottom of casing, whichever is deeper.

Grout for rock sockets should be installed after placement of the pile in-the-dry if feasible by pumping, or using tremie methods. After the grout has achieved its initial set, the hole should be backfilled with non-compacting backfill material, such as underdrain backfill Type C, and temporary casing should be removed.

8.2 Soil Nail System

8.2.1 Soil Nail Installation

As previously discussed, soil nail installation will be required along steepened slopes over water, with access and staging limited to the top of slope. It is anticipated soil nails will be installed using drilling equipment mounted to the head of a long reach excavator. It is also anticipated workers will access nail locations with a cherry picker or bucket truck. Alternatively, the Contractor may elect to utilize ropes for personnel access and/or a Spider-type slope/walking excavator anchored at top of slope. Trees and shrubs which obstruct access along the slope may require removal prior to the soil nail work. The Contractor shall take care to minimize disturbance to the slope. As discussed in Section 6.3.1, slopes are over-steepened upon which trees and shrubs may be providing resistance to raveling. The Contractor should limit clearing to within proposed stabilization limits, should leave existing root systems in place where possible, and should coordinate to minimize the time between the clearing of trees and shrubs and the installation of the soil nail retention system (including the erosion control mat).

Soil nails from Sta 12+50 to 14+50 and Sta 17+25 to 17+50 are to be positioned with the head of the top nail a minimum of one foot below the edge of proposed top of slope. The Contractor may opt to perform limited excavation/benching at the top of embankment above the top crest of slope and outside of the active roadway to improve access for drilling equipment.

Soil nails from Sta 16+65 to 17+25 are to be installed after excavation and grading is performed but prior to laying riprap. To avoid conflicts with the guardrail, the heads of the top row of nails are positioned as far as 5'-6" below the top of the riprap cut as measured along the slope face. The Contractor may opt to excavate/bench above the top nail head to provide access for the drill mounted excavator to reach the bottom nail. Care should be taken when placing riprap over soil nails to prevent damage to the soil nails, anchorages, and stabilization mat.

Hollow bar soil nails are drilled with a sacrificial drill bit attached to the front of the steel hollow bar. The Contractor shall select the drill bit appropriate for the in-situ soils to provide the design grout diameter. A low strength grout may be required to circulate through the hollow bar and drill bit during drilling to drill and maintain the hole. During drilling the drilling grout continuously overflows from the hole. The Contractor shall control the excess grout to prevent any grout from flowing into the river or fouling the slope. Upon reaching the proposed soil nail length, the structural grout will be pumped until the structural grout is observed to overflow from the hole. All excess grout should be removed upon completion of soil nail installation, prior to placing stabilization mat.

8.2.2 Soil Nail Testing Program

Prior to installation of production nails, verification load testing is to be performed on sacrificial soil nails at two locations along the south approach and two locations along the north approach. The verification load tests are taken to a maximum test load of twice (200%) the service design load to verify the nominal geotechnical bond strength. Following successful verification load testing and during installation of production nails, proof load testing is to be performed on a minimum of five percent of the production soil nails per row. The proof load tests are taken to a maximum test load of one and a half (150%) the service design load to verify consistency in construction methods and capacity of production nails.

Testing shall be performed a minimum of 3 days after grouting to allow the grout to set. Test nails are to be constructed in the same manner as the production nails, with the exceptions of having an extended bar or temporary debonded length to provide a minimum stressing length of 3 feet between the top of grout and point of load application. After verification testing of sacrificial soil nails is complete, the hollow bar should be cut at the ground surface prior to abandoning the sacrificial soil nail.

If pullout failure occurs during the verification testing, the results will be assessed by HNTB to determine if any adjustment to the soil nail lengths is required prior to start of production soil nail installation. If failure occurs during proof testing, the failed soil nail shall be abandoned, and additional nails shall be installed as determined by HNTB supplement the soil nail system. If pullout failure causes local divots in the slope face the contractor should perform local regrading to reestablish the existing grading. Regardless of whether pullout failure occurs, the maximum test load is not to be exceeded during load testing as to not intentionally load the verification test nail to failure. It is preferred to not unnecessarily induce pullout failures during these tests.

Verification and proof tests are to be accepted or rejected based upon the following criteria:

1) Total creep movement does not exceed 0.04 inch between the 1- and 10-minute readings or total creep movement does not exceed 0.08 inch between the 6- and 60-minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

2) The total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length.

3) A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

9.0 LIMITATIONS

The conclusions and recommendations contained in this report are based upon the subsurface data obtained during this investigation and on details stated in this report. The validity of the conclusions and recommendations contained in this report are necessarily limited by, among other things, the scope of field investigation and by the number of borings. Therefore, given the nature of this subsurface study, there is a possibility that actual conditions encountered will differ from those discussed in this report. Should conditions arise which differ from those described in this report, HNTB should be notified immediately and provided with all information when available regarding subsurface conditions.

As part of the geotechnical recommendations presented in this report, HNTB makes no warranty as to the absence or presence of any environmental hazard or waste present on any property evaluated hereunder and all reports generated hereto are qualified as being based upon existing data reasonably available to HNTB and not subject to independent verification. HNTB is not responsible for any latent defects that could not be reasonably discovered during the performance of its services and makes no legal representations whatsoever concerning any matter, including but not limited to, the ownership of any property or the interpretation of any law. These limitations form a material part of this report and are considered incorporated by reference therein. No warranty for the contents of this report, neither expressed nor implied, is made except that professional services were performed in accordance with generally accepted principles and practices.

10.0 REFERENCES

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FIGURES

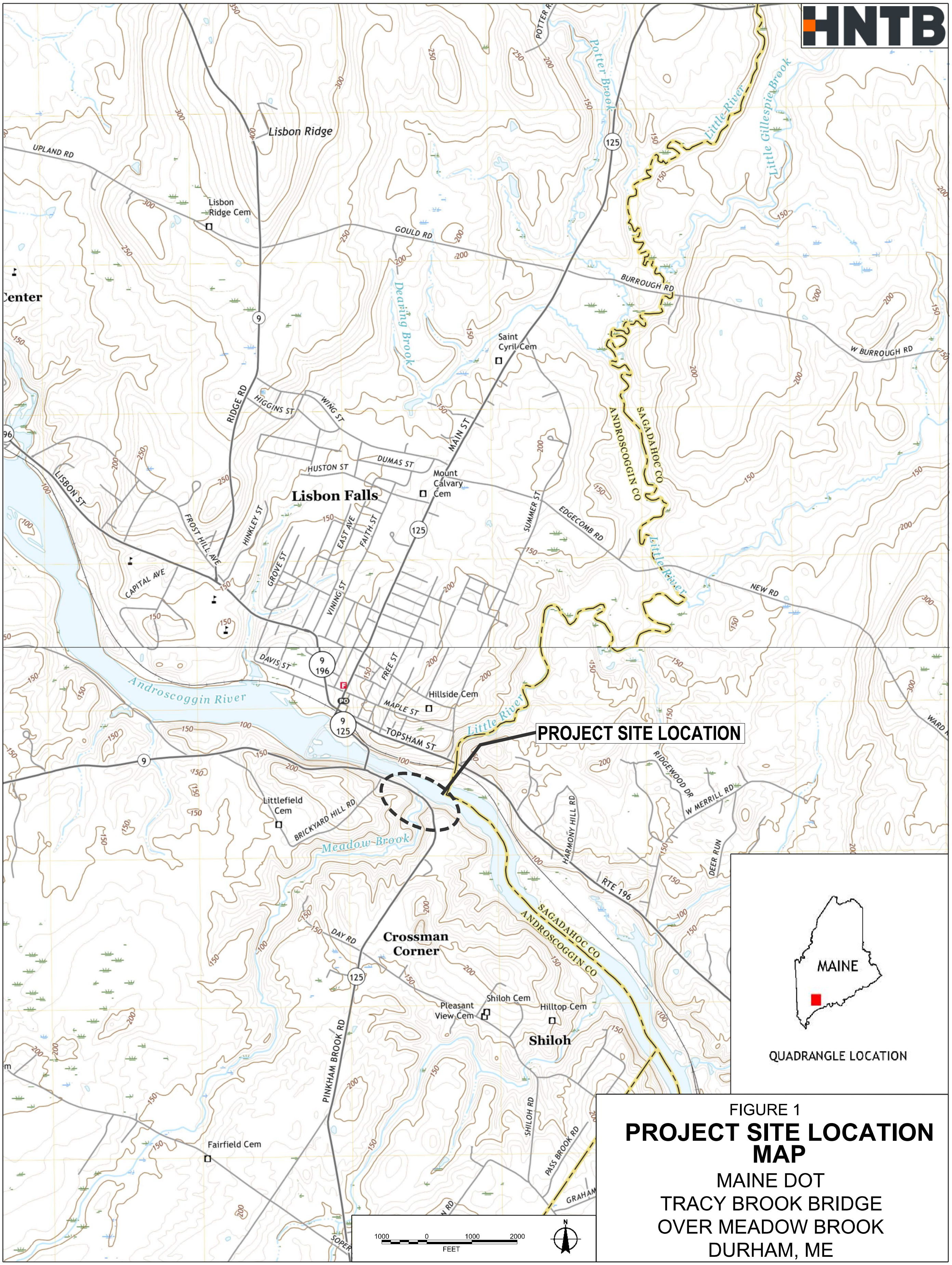
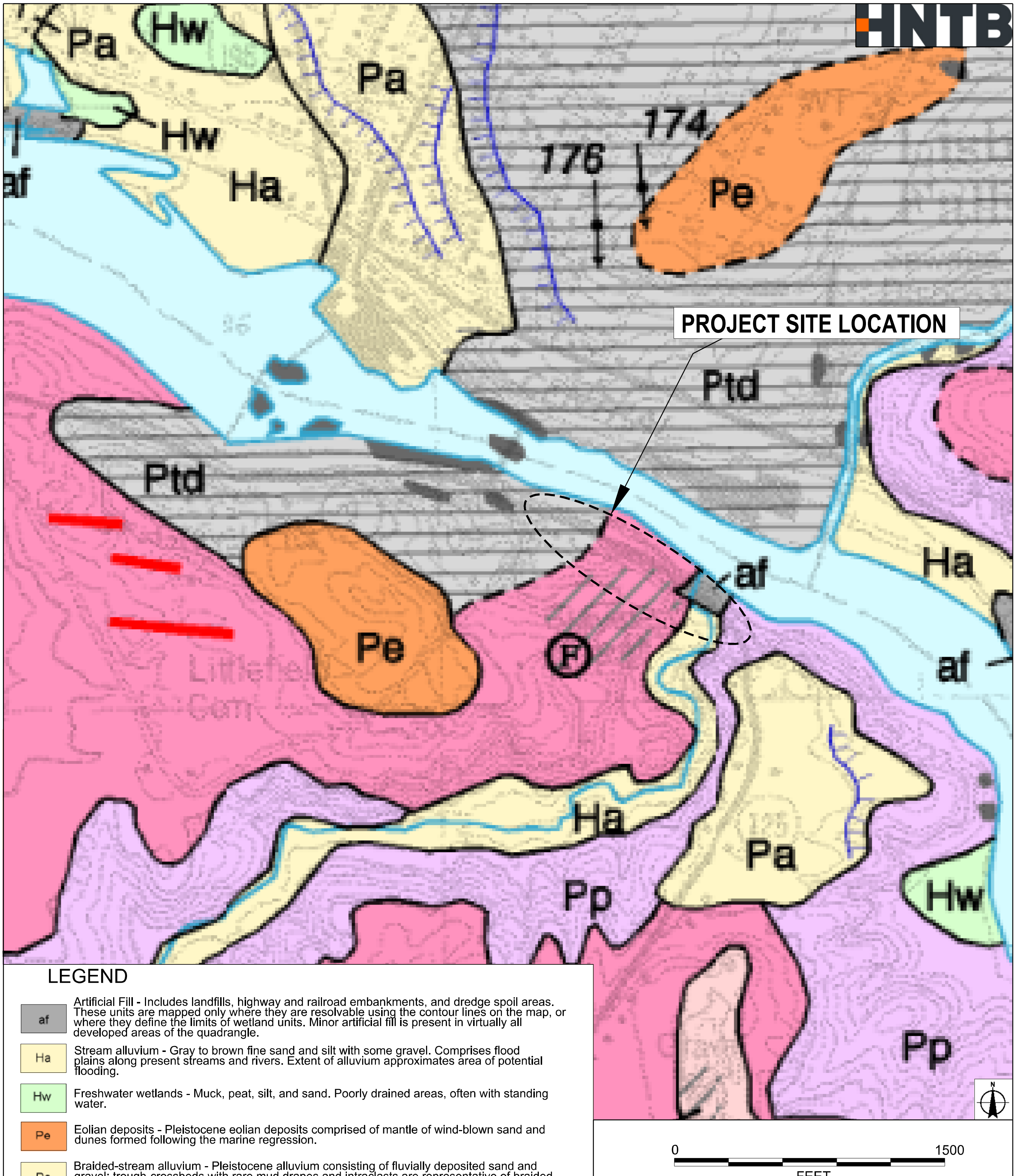


FIGURE 1
PROJECT SITE LOCATION MAP
MAINE DOT
TRACY BROOK BRIDGE
OVER MEADOW BROOK
DURHAM, ME

Basemaps: U.S.G.S. Lisbon Falls North and South, ME Quadrangle US Topo, 2021



LEGEND

af	**Artificial Fill** - Includes landfills, highway and railroad embankments, and dredge spoil areas. These units are mapped only where they are resolvable using the contour lines on the map, or where they define the limits of wetland units. Minor artificial fill is present in virtually all developed areas of the quadrangle.
Ha	Stream alluvium - Gray to brown fine sand and silt with some gravel. Comprises flood plains along present streams and rivers. Extent of alluvium approximates area of potential flooding.
Hw	Freshwater wetlands - Muck, peat, silt, and sand. Poorly drained areas, often with standing water.
Pe	Eolian deposits - Pleistocene eolian deposits comprised of mantle of wind-blown sand and dunes formed following the marine regression.
Pa	Braided-stream alluvium - Pleistocene alluvium consisting of fluvially deposited sand and gravel; trough-crossbeds with rare mud drapes and intraclasts are representative of braided streams and coastal braid-delta environment formed during the marine regression.
Pmn	Marine nearshore deposits - Pleistocene gravel, sand, and mud deposited as a result of wave activity in nearshore or shallow-marine environments; not associated with beach morphology.
Pp	Presumpscot Formation - Massive to laminated silty clays with rare dropstones and occasional shelly horizons, which overlie rock and till, and are interbedded with and overlie end moraines and marine fan deposits; includes sand deposited as a distal unit of submarine fans.
Ptd	Thin-drift areas - Area with generally less than ten feet of drift covering bedrock. Till overlies bedrock on hillslopes and ridge crests; Presumpscot Formation silty clays are present in depressions; and nearshore deposits overlie till. Presumpscot Formation, and bedrock on hillslopes and at the base of these slopes. Small rock outcrops, and areas of numerous small outcrops are shown as solid gray areas.

FIGURE 2

SURFICIAL GEOLOGY MAP

MAINE DOT
TRACY BROOK BRIDGE OVER
MEADOW BROOK
DURHAM, ME

Basemap: Surficial Geology - Lisbon Falls South Quadrangle, Maine - Maine Geological Survey - 1997

LEGEND

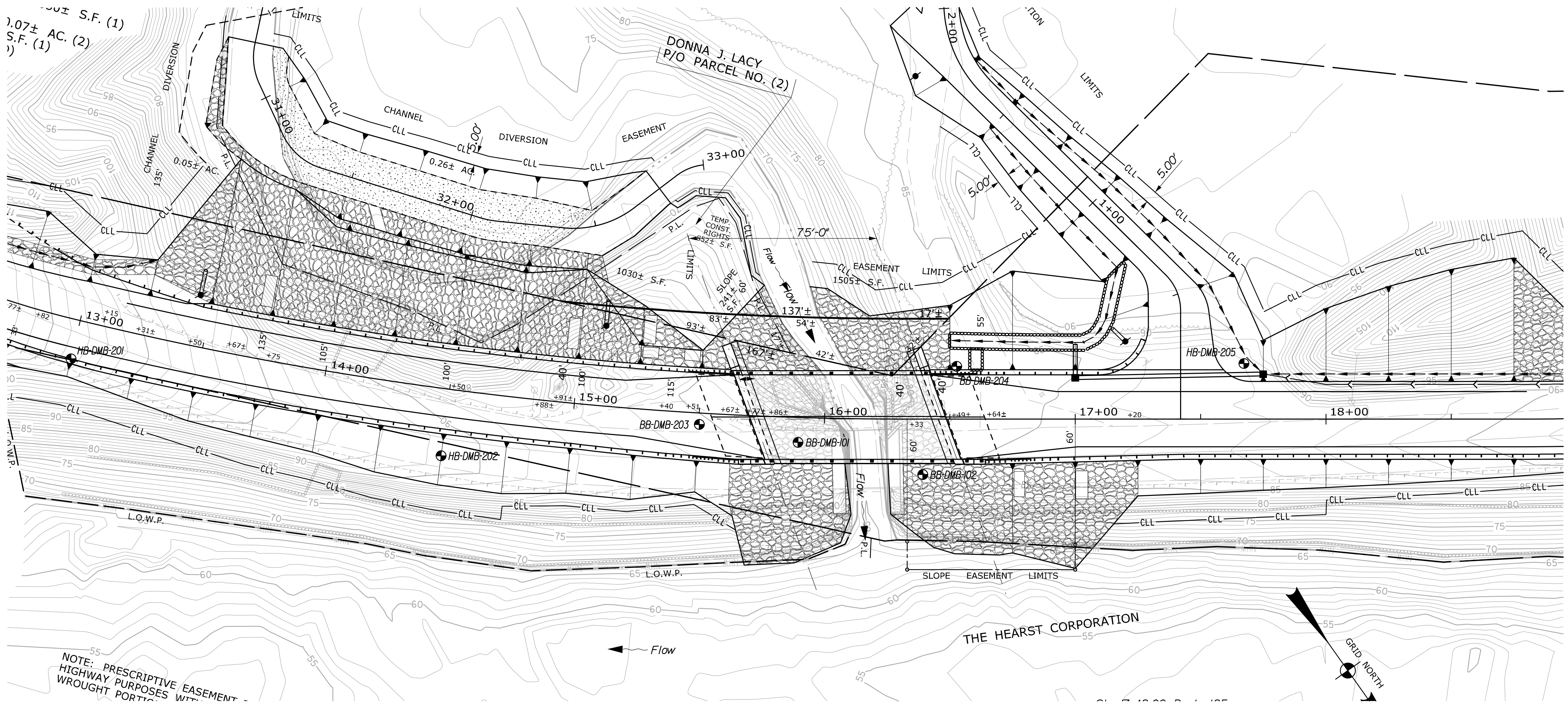
SOvu — Vassalboro Group, undifferentiated (Marvinney and others, 2010). Light to medium gray, fine-grained to medium-grained, plagioclase-quartz-biotite granofels and gneiss interlayered with subordinate amounts of greenish gray, fine-grained, calc-silicate granofels and/or medium gray, medium-grained, biotite schist. Layers range in thickness from 3 to 12 centimeters (cm). Lenses, boudins, and sills of pegmatite are common.

SOvbg — Biotite gneiss.Medium gray, medium-grained, quartz-plagioclase-biotite ± muscovite ± sillimanite gneiss and schist. Cross-cutting granitic dikes and sills up to 25 meters in width are abundant, as are small-scale textures suggestive of in situ partial melting. Compositional layering within the gneiss and schist is generally absent. Subordinate lithologies in the large outcrops at Lisbon Falls include rusty-weathering schist and impure marble.

0 1500
FEET

FIGURE 3
BEDROCK GEOLOGY MAP
MAINE DOT
TRACY BROOK BRIDGE OVER
MEADOW BROOK
DURHAM, ME

Basemap: Bedrock Geology - Lisbon Falls South Quadrangle, Maine - Maine Geological Survey - 2019



PLAN

25 0 25 50

1" = 25'

98% Plans
September 14, 2022

FIGURE 4

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

WIN 023657.00

BRIDGE NO. 2852

BRIDGE PLANS

PROJ. MANAGER	M. KERSBERGEN	BY	DATE
DESIGN-DETAILED	H. Walton	E. Beausoleil	9/2022
CHECKED-REVIEWED	B. Greiner	A. Stephens	9/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

TRACY BROOK BRIDGE
MEADOW BROOK
DURHAM ANDROSCOGGIN

BORING LOCATION PLAN

SHEET NUMBER
10
OF 79

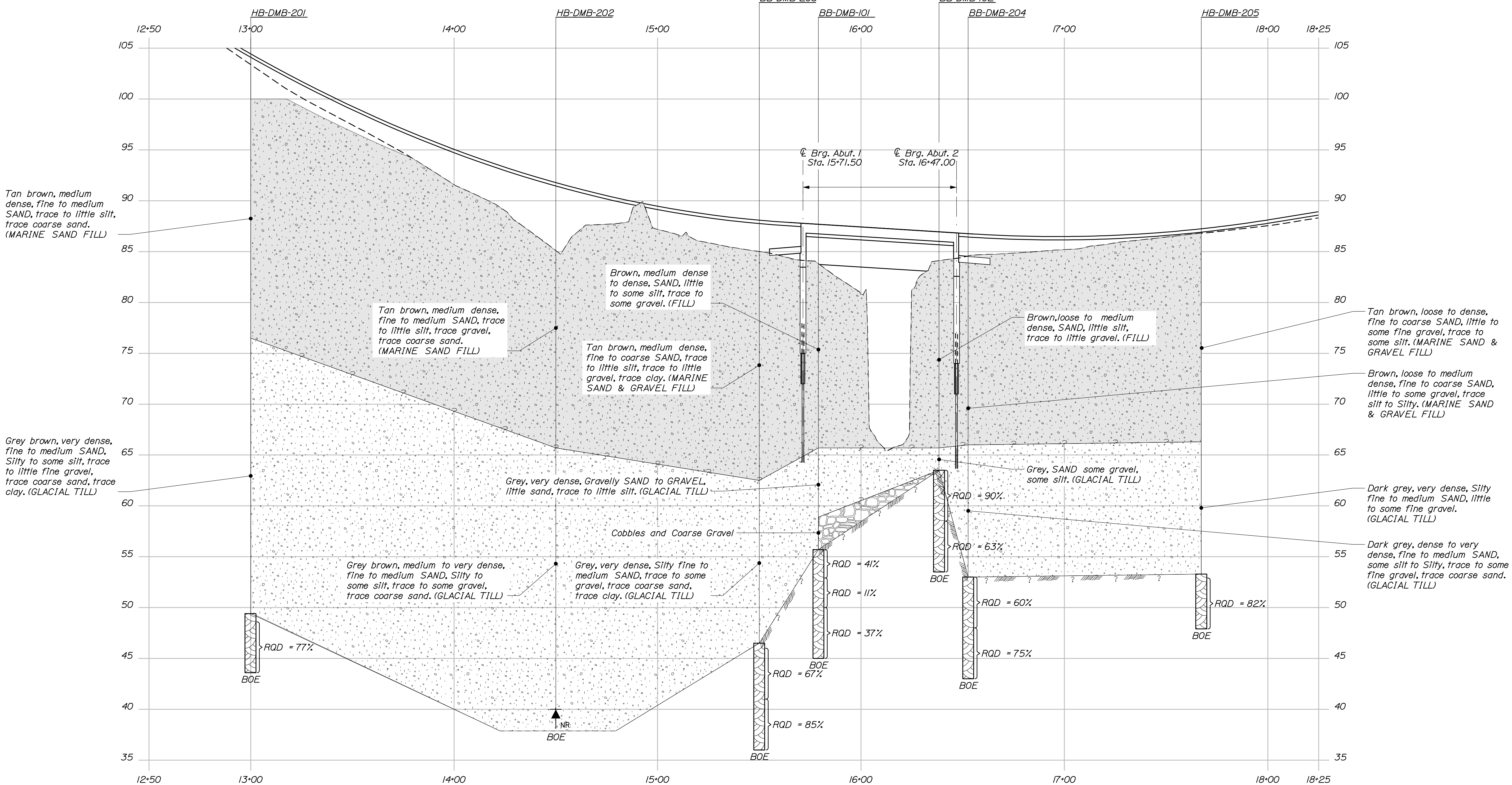
Filename: 010_Boring Location Plan.dgn
Division:
Username:
Date:9/13/2022

Date:9/13/2022

Username:

Division:

Filename: 011_Interpretative Subsurface Profile.dgn



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 explorations and samples. Actual soil and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

LEGEND

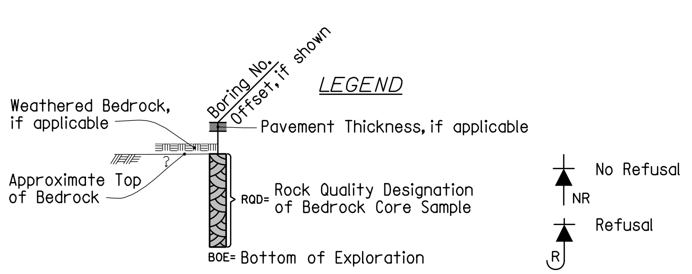


FIGURE 5

98% Plans
September 14, 2022

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

WIN 023657.00

BRIDGE PLANS

BRIDGE NO. 2852

	BY	DATE
PROJ. MANAGER	M. KERSBERGEN	
DESIGN-DETAILED	H. Walton	E. Beausoleil 9\2022
CHECKED-REVIEWED	B. Grenier	A. Stephens 9\2022
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

TRACY BROOK BRIDGE
MEADOW BROOK
DURHAM ANDROSCOGGIN

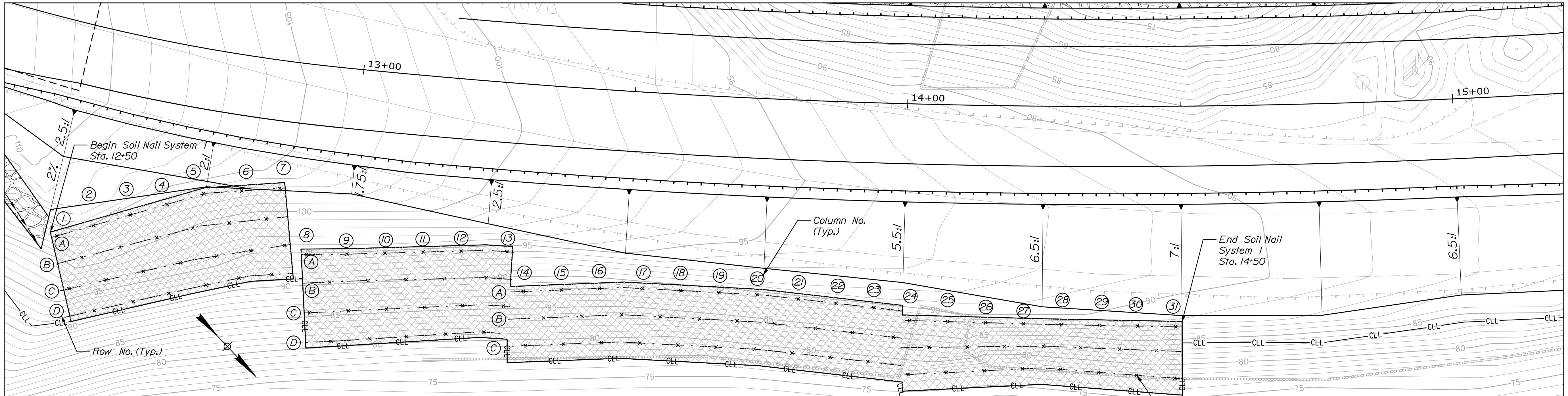
INTERPRETATIVE SUBSURFACE PROFILE

SHEET NUMBER

11

OF 79

Date:10/28/2022
Username:
Division:
Filename: 015_Soil Nail Plan.dgn



PLAN

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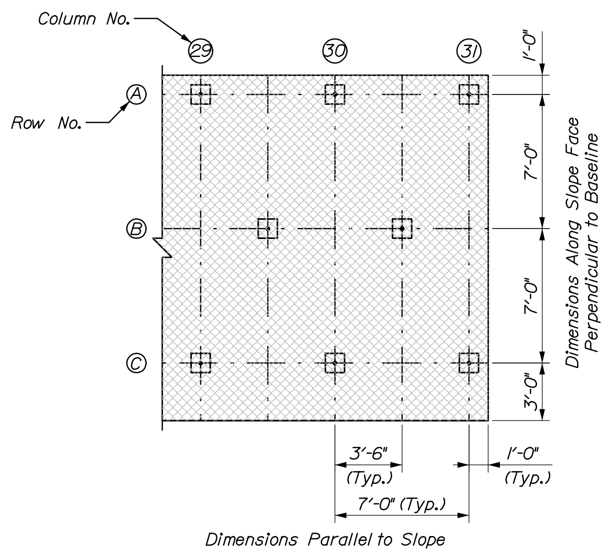
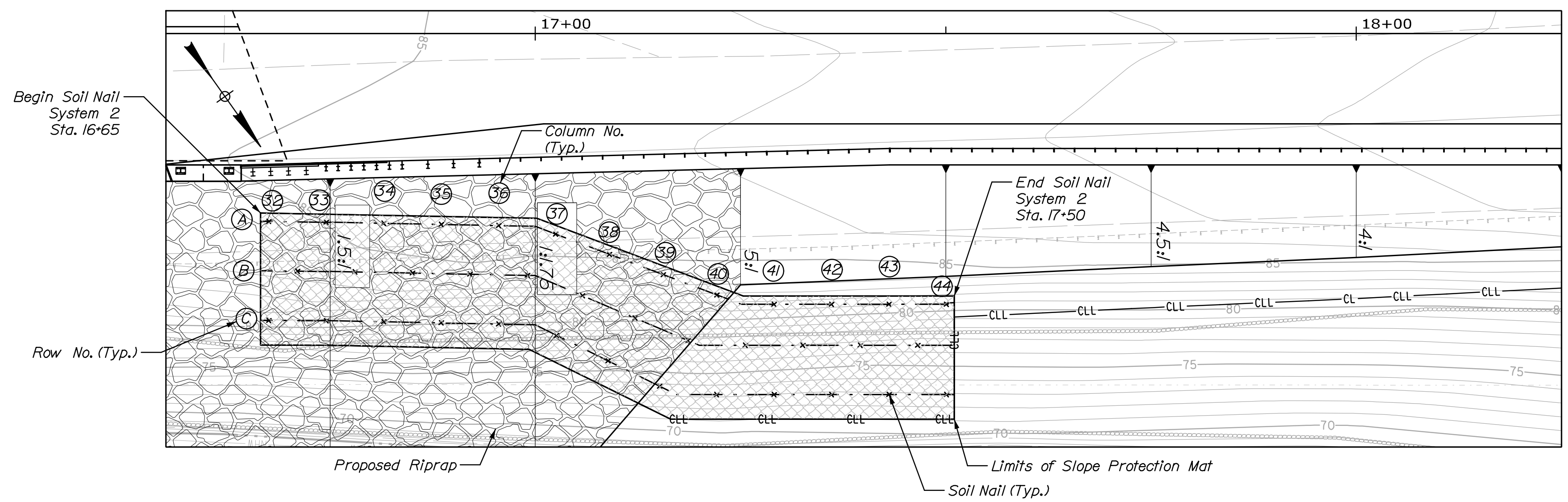
Scale of Feet





PLAN

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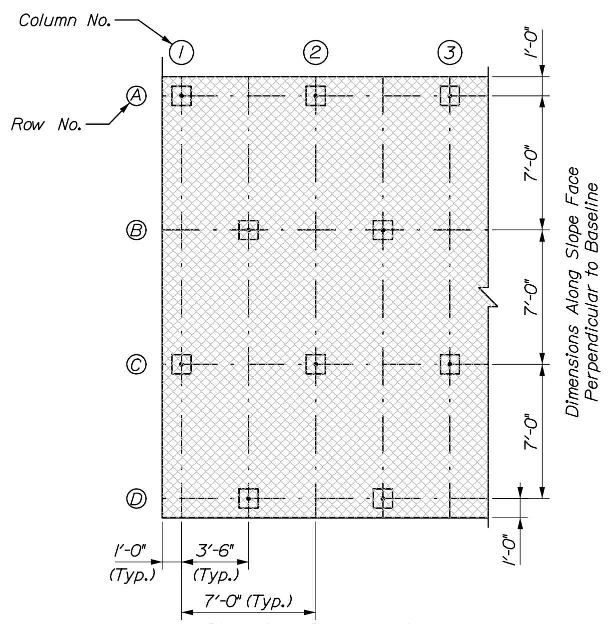
Scale of Feet



TYPICAL SOIL NAIL SPACING
NOT TO SCALE

Note:
1. Soil nail plan layout as shown is schematic only. Final layout shall be determined per Contractor's shop drawing as reviewed and approved by the Department. Soil nails shall be located in accordance with the Typical Soil Nail Spacing detail on this sheet and per the Soil Nail Details.

FIGURE 6

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

WIN
023657.00

BRIDGE PLANS

BRIDGE NO. 2852

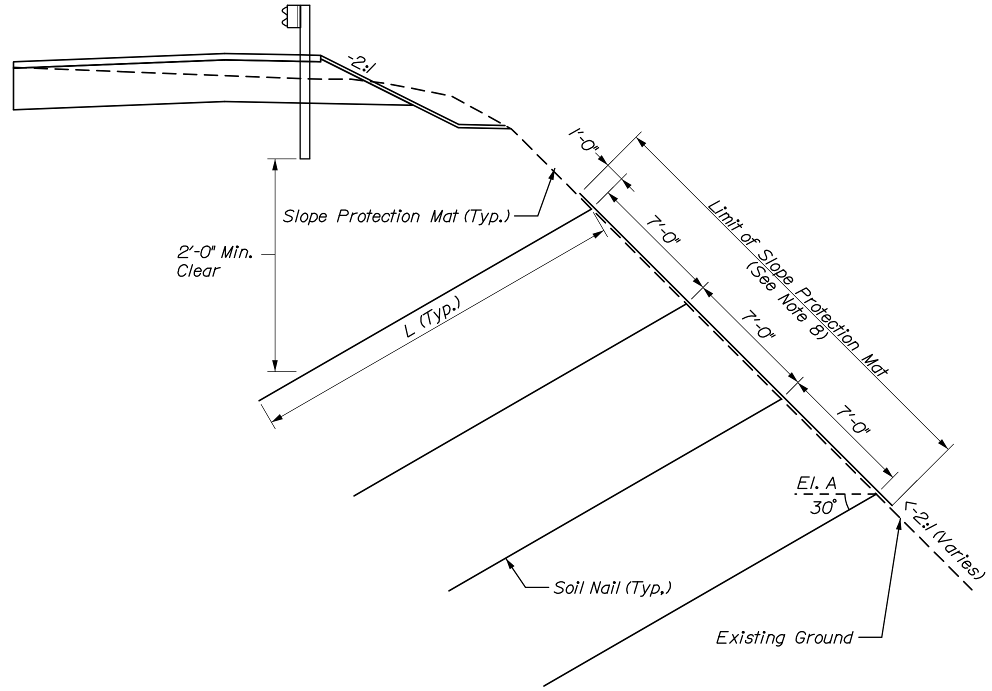
		BY	DATE
PROJ. MANAGER	M. KERSBERGEN		
DESIGN-DETAILED	J. Zentilkeaboum	P. Mckechnie	10/2022
CHECKED-REVIEWED	B. Materek	J. Juzwin	10/2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

TRACY BROOK BRIDGE
MEADOW BROOK
DURHAM ANDROSCOGGIN

SOIL NAIL PLAN

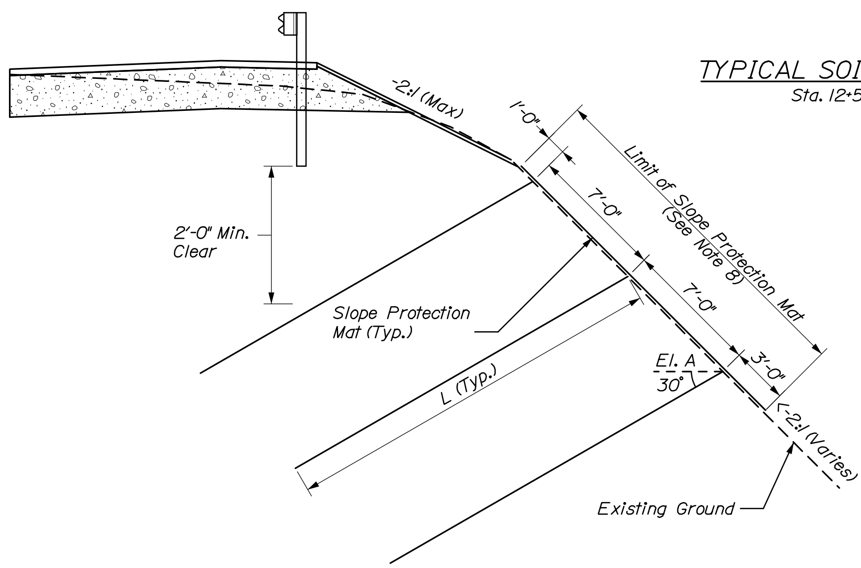
SHEET NUMBER
15
OF 79



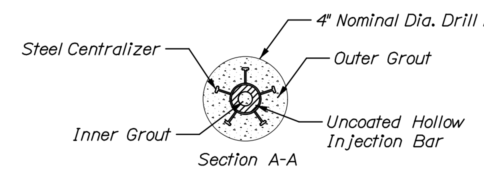
TYPICAL SOIL NAIL SECTION
Sta. 12+50 to Sta. 13+60

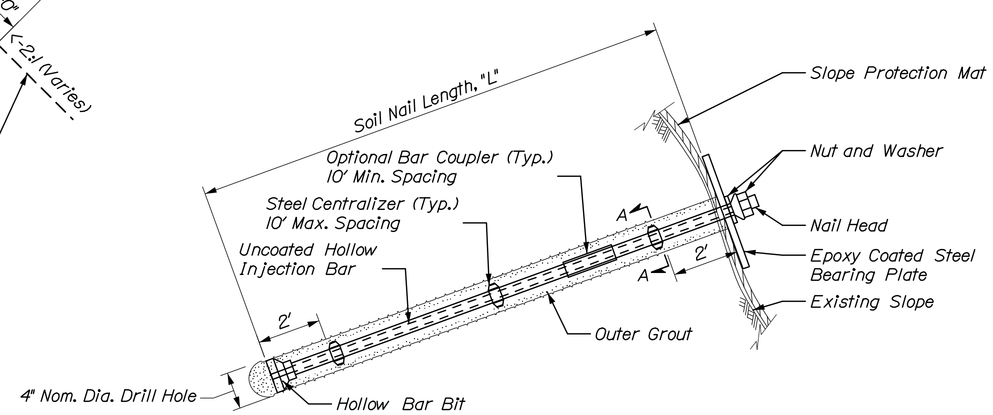
Verification Load Test Table						
Test Nail Designation	Station	Offset (Ft.)	Elevation (Ft.)	Bond Length (Ft.)	Inclination (Degrees)	Service Design Load (KIPS)
T1	13+00	+37.5	92.5	20	30	23
T2	13+90	+40.0	86.5	20	30	23
T3	17+00	+27.0	78.5	15	30	23
T4	17+25	+41.0	75.0	15	30	23

Production Soil Nail Table					
Station Limits	Number of Rows	Soil Nail Length, "L" (Ft.)	Inclination (Degrees)	Service Design Load (KIPS)	EL. A (Ft.)
12+50 TO 12+90	4	20	30	23	92.00
12+91 TO 13+30	4	20	30	23	81.00
13+31 TO 13+60	4	20	30	23	79.00
13+60 TO 14+00	3	20	30	23	79.00
14+01 TO 14+50	3	20	30	23	77.00
16+65 TO 17+50	3	15	30	23	73.00



TYPICAL SOIL NAIL SECTION
Sta. 13+61 to Sta. 14+50
Sta. 17+25 to Sta. 17+50





TYPICAL SOIL NAIL DETAIL
Not to Scale

Soil Nail Notes:

1. On the Androscoggin River side (Stage 2), temporary excavation and grading at the top of slope to accommodate drilling equipment is permitted. Soil nails must be installed before placing fill to finished grade and paving.

2. Construct and test Hollow Bar Soil Nails in accordance with Special Provision Section 675, Soil Nail Wall (Hollow Bar Soil Nails).

3. Uncoated hollow injection bar shall be continuously threaded high-strength steel tubing meeting the requirements of ASTM A519, with a minimum yield strength of 75 ksi, a minimum nominal outer diameter of 1.5 inches, and a minimum net area through threads of 1.067 square inches. Soil nail hollow injection bars have been sized accounting for corrosion for a design life of 100 years.

4. Final grout shall have a minimum 28 day compressive strength of 4,000 psi.

5. Epoxy-coated steel bearing plate with dimensions no less than 10" x 10" x 2" shall meet requirements of ASTM A709 Grade 50 and shall be fastened to each nail head using a threaded bar nut and washer.

6. The Contractor shall be responsible for submitting the final nail lengths, locations, diameters and details as part of the requirements for working drawings as per the Special Provisions. Soil nail locations shall be approved in the field by the Department prior to the start of construction. Locations as provided may be adjusted in the field by the Department.

7. Efforts shall be made to minimize disturbance to the existing slope face prior to and during construction. No benching or excavation of the existing slope shall be permitted to facilitate construction, except where indicated in the Plans. Trees and vegetation obstructing construction shall be cut at the ground line such that the root system is left in place.

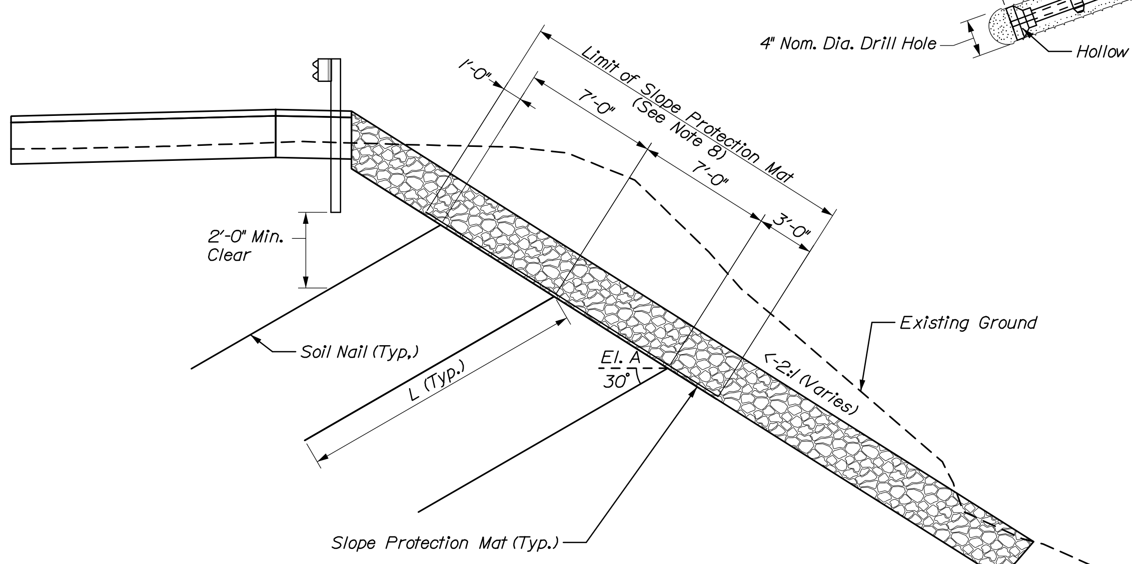
8. The Contractor shall seed the finished ground and install a Slope Protection Mat over the ground surface and under soil nail bearing plates within the indicated limits. Slope Protection Mat shall consist of an erosion control blanket in accordance with Section 613 of the Standard Specifications, overlain by double-twisted galvanized steel mesh in accordance with Special Provision Section 513, Slope Protection (Wire Mesh). Extend erosion control blanket above top of wire mesh to the top of slope.

9. Where soil nails overlap proposed riprap, the soil nails and steel mesh shall be installed before the riprap is placed. The riprap shall be placed over the steel mesh, bearing plate, and nail head. Seeding and erosion control blanket shall be omitted within riprap limits.

10. Should the Contractor notice any displacement of the existing slope during construction, or if any raveling or local instability of the slope occurs, the Department shall be notified immediately.

11. Pre-production verification tests shall be performed by the Contractor prior to the installation of production nails to verify the Contractor's installation methods and nail pullout resistance. The Contractor shall design and perform verification testing of designated sacrificial test nails at the locations indicated on the plan and in accordance with the Special Provisions. Verification test nails shall be loaded to double the Service Design Load indicated in the Verification Load Test Table.

12. No less than 5 percent of the production soil nails in each row, a minimum of 1 nail per row, shall be proof tested in accordance with the Special Provisions. The proposed locations of the proof tests will be specified by the Department following review of Contractor soil nail layout submittals.



TYPICAL SOIL NAIL SECTION
Sta. 16+65 to Sta. 17+25

FIGURE 7

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

BRIDGE NO. 2852

WIN 023657.00

BRIDGE PLANS

PROJ. MANAGER	M. KERSBERGEN	
DESIGN-DETAILED	J. Zentilkeneaum	P. Mckechnie	10\2022
CHECKED-REVIEWED	B. Molerek	J. Juzwin	10\2022
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

BY | DATE | SIGNATURE | P.E. NUMBER | DATE

TRACY BROOK BRIDGE
MEADOW BROOK

DURHAM | ANDROSCOGGIN

SOIL NAIL DETAILS

SHEET NUMBER
16
OF 79

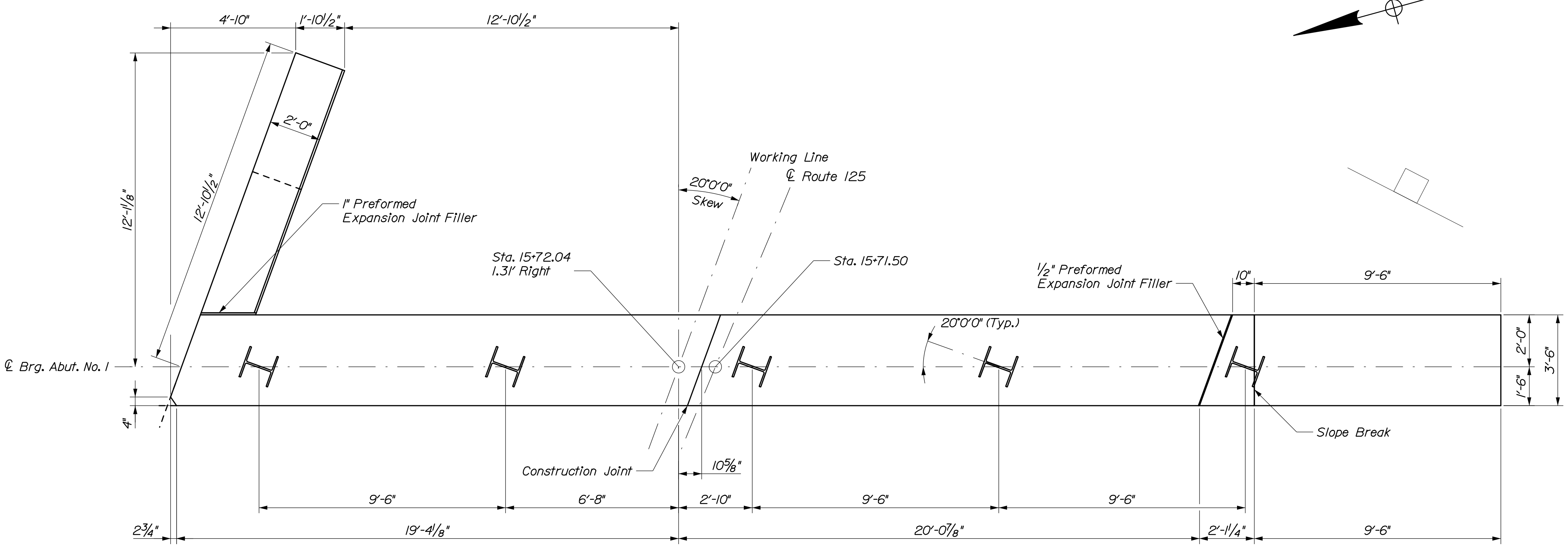
Filename: 016_Soil Nail Details.dgn
Division:
Username:
Date:10/28/2022

Date:9/13/2022

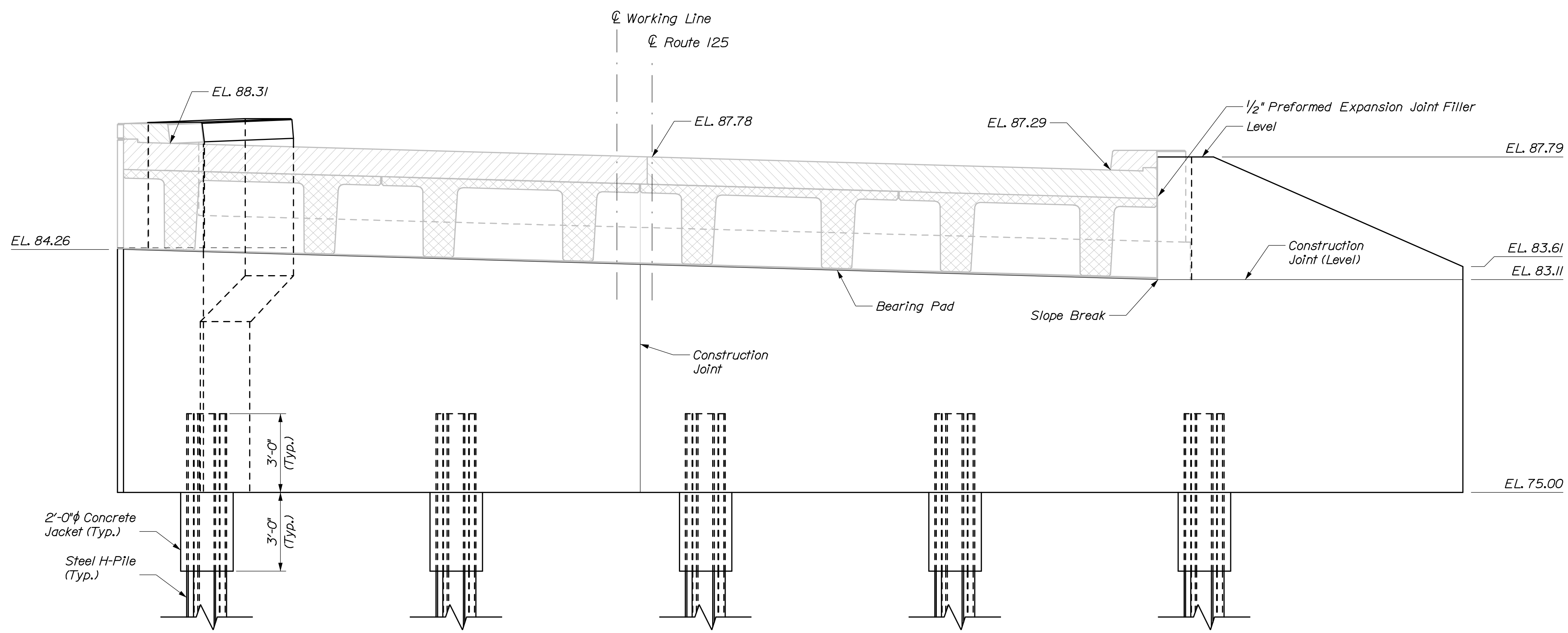
Username:

Division:

Filename: 064_Abutment 1.dgn



ABUTMENT NO. 1 PLAN



ABUTMENT NO. 1 ELEVATION
(Superstructure shown screened for clarity)

ABUTMENT NOTES

1. Reinforcing steel shall have a minimum concrete cover of 2 inches unless otherwise noted.

2. Cover joints where waterstops are not required in accordance with Standard Detail 502(01).

3. Place 4 inch diameter drains in abutment and wingwalls at 10 feet maximum spacing. The exact location will be determined by the Resident.

4. Construct Drainage Geocomposite behind the abutments and wingwalls in accordance with Special Provision Section 620, Geotextiles - Drainage Geocomposite.

5. Abutments and wingwalls shall be backfilled with Granular Borrow for Underwater Backfill. Pay limits will be the structural excavation limits as shown on the "Abutment Details" sheet.

6. All elevations are provided at centerline of bearing unless otherwise noted.

7. Payment for concrete jacket around the tops of the H-piles will not be paid for directly. Payment shall be incidental to Item 502.219, Structural Concrete, Abutments and Retaining Walls. Fill concrete may be used for the concrete jackets.

9. Anchor dowels (bars 900c) shall be installed plumb and may either be cast-in or drilled and anchored in accordance with Subsection 503.06. See Abutment Reinforcing sheets and End Diaphragm Reinforcing sheet for additional information.

PILE NOTES

1. The maximum factored pile load is 398 kips (Strength I Load Combination).

2. Estimate of Piles required (5'-0" contingency included):

   Abutment No. 1: 5 - HP 14x102 @ 29 ft
   Abutment No. 2: 5 - HP 14x102 @ 20 ft

3. H-pile material shall be ASTM A572, Grade 50.

4. Piles shall be installed in accordance with Special Provision Section 501, Rock-Socketed H-Pile Foundation.

5. Piles shall not be out of position shown by more than 2 inches in any direction.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

BRIDGE NO. 2852 | WIN 023657.00 | BRIDGE PLANS

PROJ. MANAGER	M. KERSBERGEN	BY	DATE
DESIGN-DETAILED	H. Walton	E. Beausoleil	9\2022
CHECKED-REVIEWED	B. Grenier	A. Stephens	9\2022
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

TRACY BROOK BRIDGE
MEADOW BROOK
DURHAM ANDROSCOGGIN

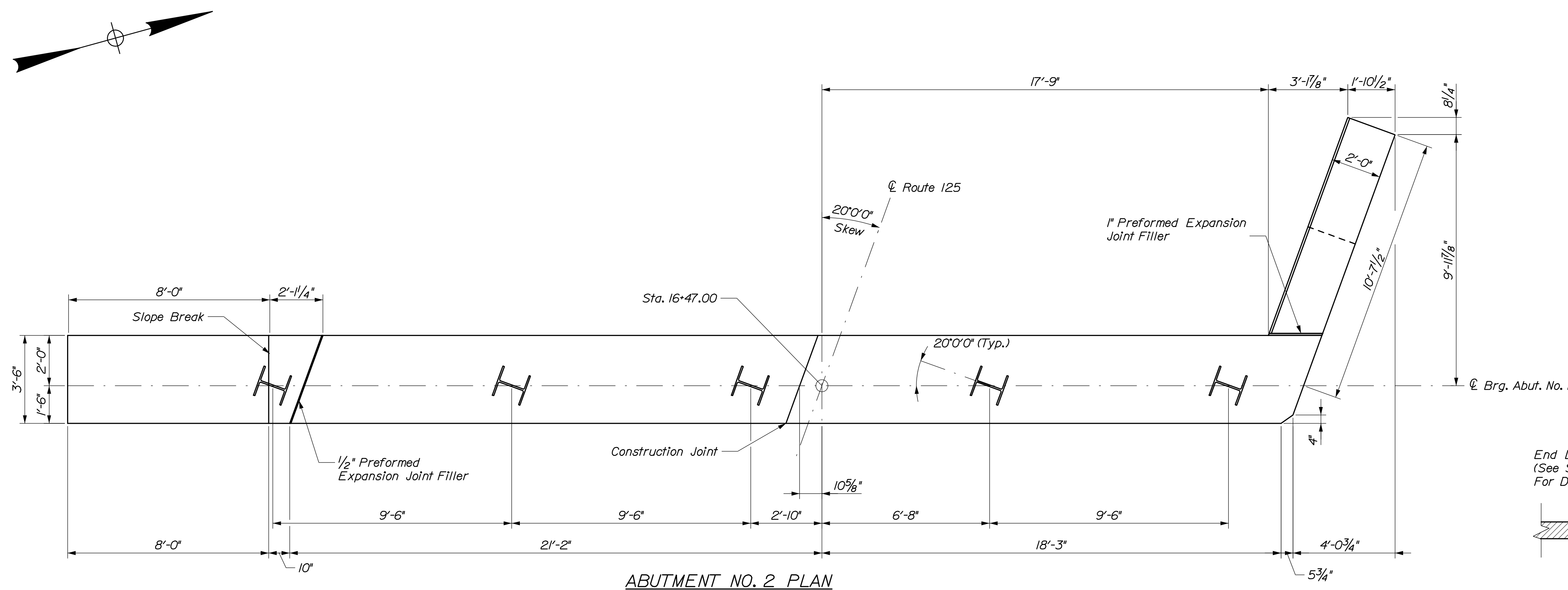
ABUTMENT NO. 1

98% Plans
September 14, 2022

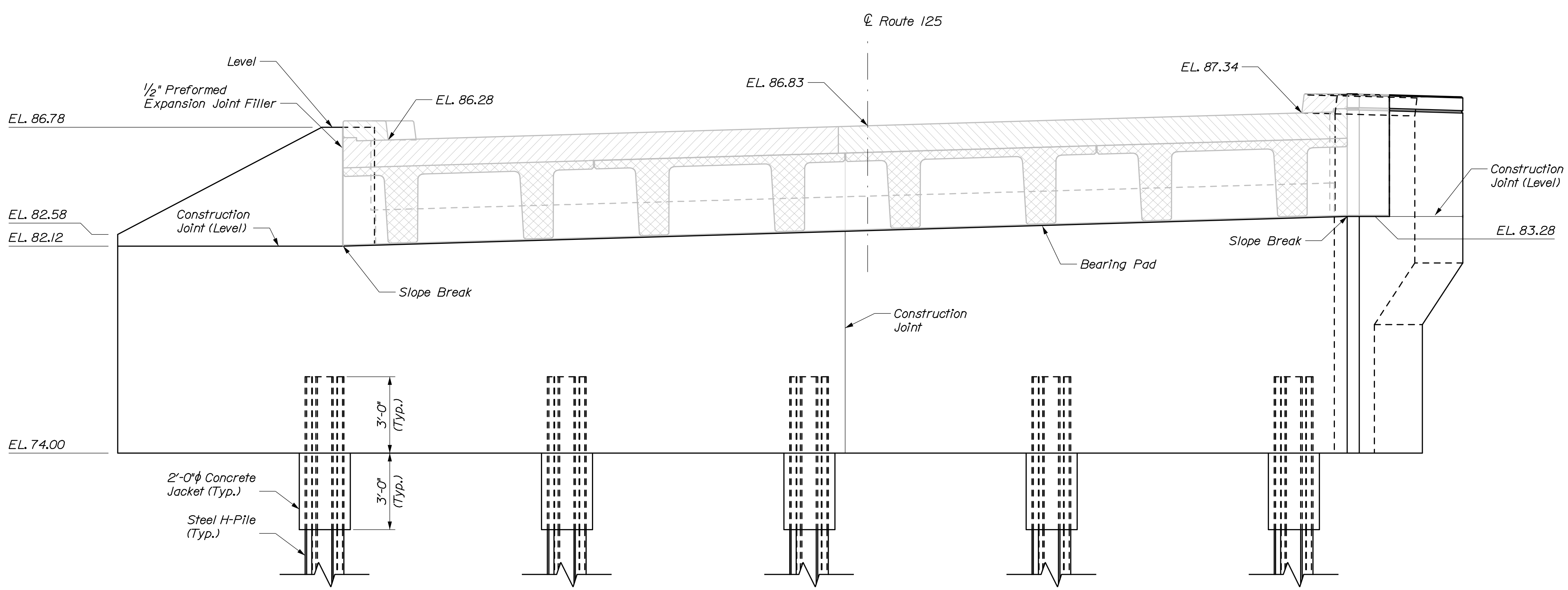
FIGURE 8

HNTB

SHEET NUMBER
64
OF 79

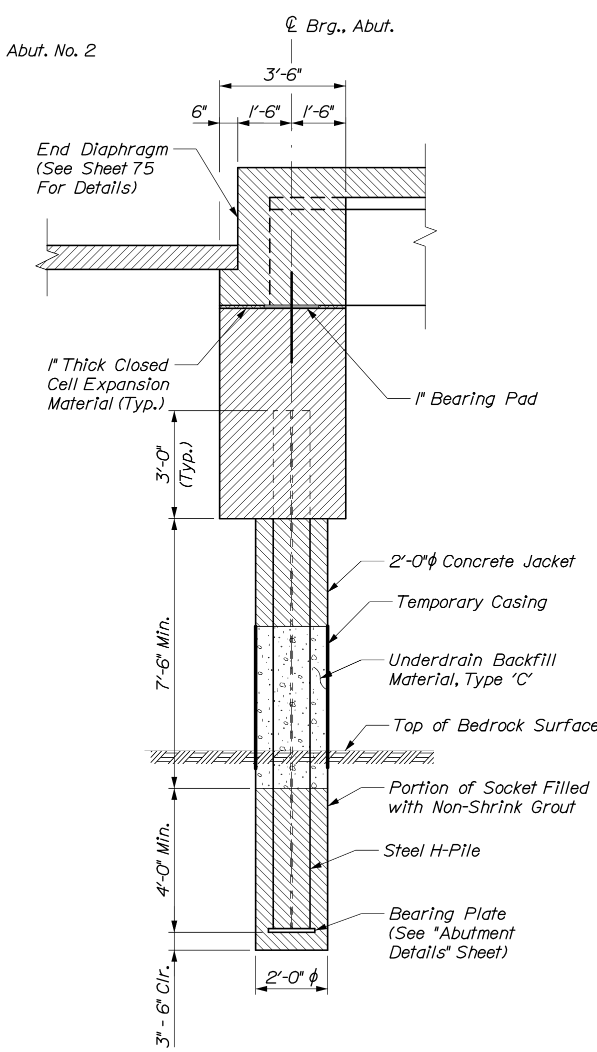


ABUTMENT NO. 2 PLAN



ABUTMENT NO. 2 ELEVATION

FIGURE 9



TYPICAL SECTION

98% Plans
September 14, 2022

HNTB

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2365700

WIN
023657.00

BRIDGE NO. 2852

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. KERSBERGEN	
DESIGN-DETAILED	H. Walton	E. Beausoleil	9\2022
CHECKED-REVIEWED	B. Grenier	A. Stephens	9\2022
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

TRACY BROOK BRIDGE
MEADOW BROOK
DURHAM ANDROSCOGGIN

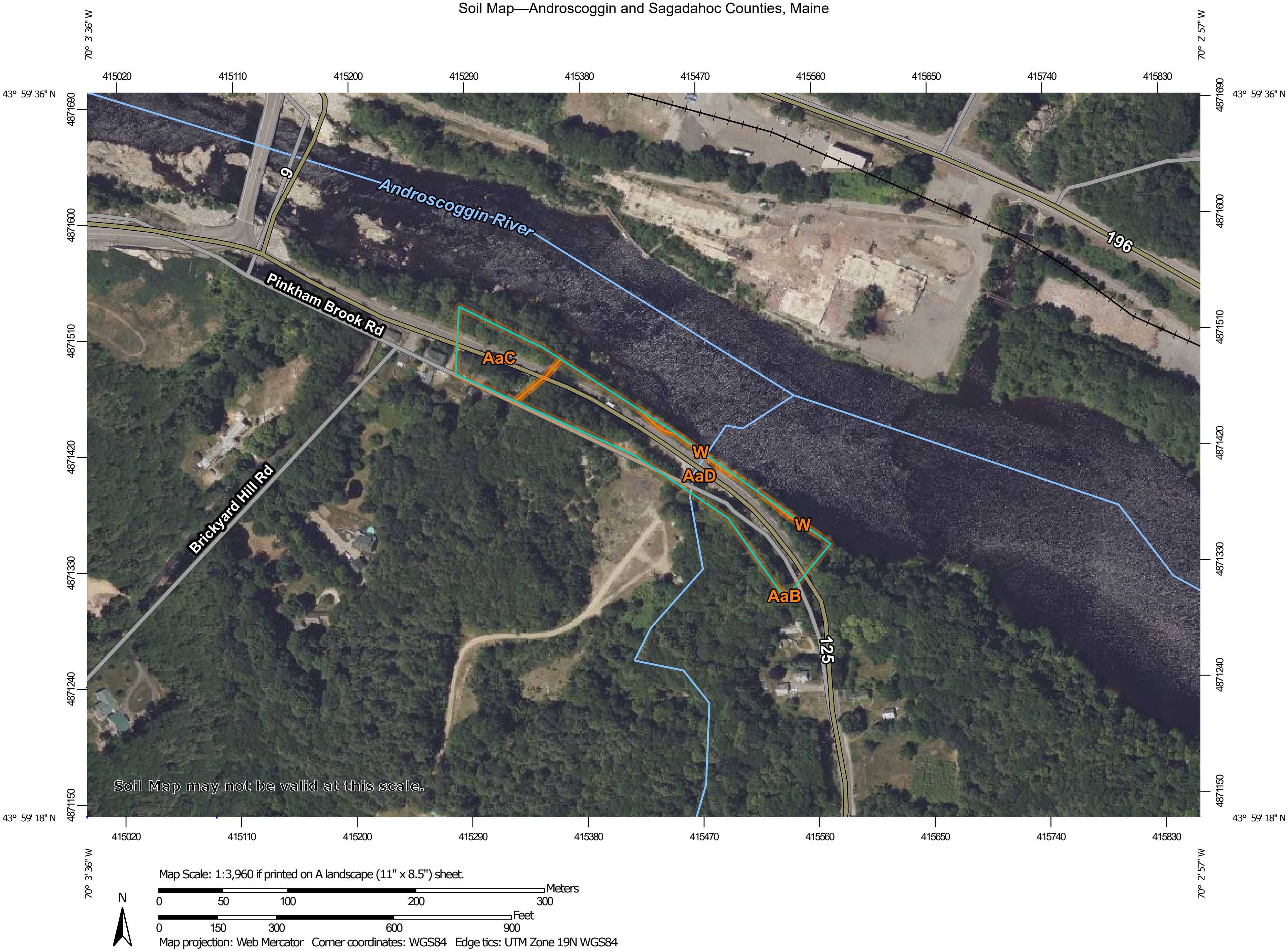
ABUTMENT NO. 2

SHEET NUMBER
65
OF 79

Date:9/13/2022
Username:
Division:
Filename: 065_Abutment 2.dgn

APPENDIX A

NRCS WEB SOIL SURVEY RESULTS



Map Scale: 1:3,960 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 50, 100, 200, 300

Feet: 0, 150, 300, 600, 900

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 19N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Androscoggin and Sagadahoc Counties, Maine
Survey Area Data: Version 22, Aug 30, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jun 19, 2020—Sep 20, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
AaB	Adams loamy sand, 0 to 8 percent slopes	0.0	0.1%
AaC	Adams loamy sand, 8 to 15 percent slopes	0.8	25.9%
AaD	Adams loamy sand, 15 to 30 percent slopes	2.3	72.5%
W	Water	0.0	1.5%
Totals for Area of Interest		**3.2**	**100.0%**

APPENDIX B

100-SERIES BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Tracy Brook Bridge #2852 carries Route 125 over Meadow Brook
Location: Durham, Maine

Boring No.: BB-DMB-101

WIN: 23657.00

Driller:	MaineDOT	**Elevation (ft.)**	84.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20/2019; 09:00-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+89.4, 9.9 ft Rt.	**Casing ID/OD:**	NW-3"/3.5"	**Water Level*:**	None observed. See note.
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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	84.3		5" HMA. 0.4	
5	1D	24/10	5.00 - 7.00	7/14/14/9	28	43				Brown, damp, dense, SAND, little silt, little gravel, (Fill).	G#337212 A-2-4, SM WC=4.7%
10	2D	24/15	10.00 - 12.00	2/2/2/5	4	6	5			Brown, moist, loose, SAND, some silt, trace gravel, (Fill).	G#337213 A-2-4, SM WC=13.8%
							7				
							41				
							61				
							69				
15	3D	24/10	15.00 - 17.00	15/4/6/5	10	15	18			Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
							22				
							24				
							41	65.7		19.0	
							68				
20	4D	9.6/5	20.50 - 21.30	60/50(3.6")	---		RC			Cobble from 20.0-20.5 ft bgs. Grey, moist, very dense, Gravelly SAND, little silt, (Glacial Till).	G#337214 A-1-b, SM WC=6.9%
25											

Remarks:

After romoval of casing, bore hole caved at 14.0 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tracy Brook Bridge #2852 carries Route 125 over Meadow Brook
Location: Durham, Maine

Boring No.: BB-DMB-101
WIN: 23657.00

Driller:	MaineDOT	**Elevation (ft.)**	84.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/20/2019; 09:00-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+89.4, 9.9 ft Rt.	**Casing ID/OD:**	NW-3"/3.5"	**Water Level*:**	None observed. See note.
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	4.8/3	25.00 - 25.40	46(4.8")	---		NQ-2			Grey, moist, GRAVEL, little sand, trace silt, (Glacial Till).	
	R1	38.4/10	25.80 - 29.00					58.9		25.8 R1: Cobbles and Coarse Gravel (25.8 - 29.0 bgs).	
	R2	34/23	29.00 - 31.83	RQD = 41%				55.7		29.0 Top of Bedrock at Elev. 55.7 ft. R2: Black and white banded, fine grained quartz, feldspar, biotite, and garnet SCHIST, moderately hard, moderately weathered, 0 to 10 degree breaks along biotite foliation, several 70 to 90 degree joints associated with core fragmentation, moderately close spacing, open with till infilling, mud infilling at 30.1 ft (29.0-31.8 ft bgs). Rock Mass Quality = Very Poor R2: Core Times (min:sec) 29.0-30.0 ft (0:41) 30.0-31.0 ft (1:13) 31.0-31.8 ft (1:57)	
30	R3	34.8/34.8	31.80 - 34.70	RQD = 11%						R3: Bedrock: Similar to R2, except moderately severe weathering with notable pyrite, and shattered from 31.8-32.6 ft bgs. Near vertical joints in the top two-thirds of the core. Rock Mass Quality = Very Poor R3: Core Times (min:sec) 31.8-32.1 ft (3:05) Core Blocked 32.1-32.8 ft (1:04) 32.8-33.8 ft (1:52) 33.8-34.7 ft (2:04) Core Blocked 100% Recovery	
35	R4	60/60	34.70 - 39.70	RQD = 37%				45.0		R4: Bedrock: Similar to R2, except with pyrite on joint faces. Joints horizontal to steep dipping. Rock Mass Quality = Poor R4: Core Times (min:sec) 34.7-35.7 ft (1:28) 35.7-36.7 ft (1:49) 36.7-37.7 ft (1:43) 37.7-38.7 ft (1:37) 38.7-39.7 ft (2:18) 100% Recovery 39.7	
40										**Bottom of Exploration at 39.7 feet below ground surface.**	
45											
50											

Remarks:

After romoval of casing, bore hole caved at 14.0 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tracy Brook Bridge #2852 carries Route 125 over Meadow Brook
Location: Durham, Maine

Boring No.: BB-DMB-102
WIN: 23657.00

Driller:	MaineDOT	**Elevation (ft.)**	84.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/26/2019; 09:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+39.2, 22.1 ft Rt.	**Casing ID/OD:**	NW-3"/3.5"	**Water Level*:**	None observed. See note.
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 No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	84.3		5" HMA. 0.4	
5	1D	24/10	5.00 - 7.00	3/2/2/3	4	6				Brown, damp, loose, SAND, little silt, trace gravel, (Fill).	G#337215 A-2-4, SM WC=4.4%
10	2D	24/10	10.00 - 12.00	6/7/10/8	17	26	29			Brown, damp, medium dense, SAND, little silt, little gravel, (Fill).	
							41				
							26				
							23			Wood in wash water.	
							16				
15	3D	24/4	15.00 - 17.00	1/2/3/3	5	8	1			Brown, wet, loose, SAND, trace silt, trace gravel.	G#337216 A-1-b, SP WC=23.5%
							7				
							19				
							42				
							160	65.7		19.0	
20	4D	7.2/4	20.50 - 21.10	17/25 (1.2")	---		60			Cobble from 20.0-20.5 ft bgs. Grey, moist, SAND, some gravel, some silt, (Glacial Till). a32 blows for 0.2 ft.	G#337217 A-2-4, SM WC=10.0%
	R1	60/60	21.20 - 26.20	RQD = 90%			a32 NQ-2	63.5		21.2 Top of Bedrock at Elev. 63.5 ft. R1: Bedrock: White with black blotches, coarse grained, granite-like MIGMATITE, hard, fresh, massive, no joint set evident, spacing close, tight to open, changing back to SCHIST in bottom 1-foot of core run, no infilling. Rock Mass Quality = Good R1: Core Times (min:sec)	
25											

Remarks:

The bore hole caved at 8.0 ft bgs after the casing was withdrawn.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tracy Brook Bridge #2852 carries Route 125 over Meadow Brook
Location: Durham, Maine

Boring No.: BB-DMB-102
WIN: 23657.00

Driller:	MaineDOT	**Elevation (ft.)**	84.7	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/26/2019; 09:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+39.2, 22.1 ft Rt.	**Casing ID/OD:**	NW-3"/3.5"	**Water Level*:**	None observed. See note.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
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SSA = Solid Stem Auger
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q_p = Unconfined Compressive Strength (ksf)
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										21.2-22.2 ft (3:53) 22.2-23.2 ft (3:02) 23.2-24.2 ft (2:23) 24.2-25.2 ft (3:01) 25.2-26.2 ft (2:04) 100% Recovery	
	R2	60/60	26.20 - 31.20	RQD = 63%						R2: Bedrock: Grey, fine grained, SCHIST (with less banding), blocky feldspar crystals interspersed and several Migmatite zones, hard, fresh, joints sub-horizontal along the foliation and one approximately 70 degrees, no infilling. Rock Mass Quality = Fair R2: Core Times (min:sec) 26.2-27.2 ft (1:41) 27.2-28.2 ft (1:44) 28.2-29.2 ft (1:59) 29.2-30.2 ft (2:17) 30.2-31.2 ft (2:59) 100% Recovery	
30								53.5		31.2	
										Bottom of Exploration at 31.2 feet below ground surface.	
35											
40											
45											
50											

Remarks:

The bore hole caved at 8.0 ft bgs after the casing was withdrawn.

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.

APPENDIX C

200-SERIES SUBSURFACE INVESTIGATION DATA REPORT

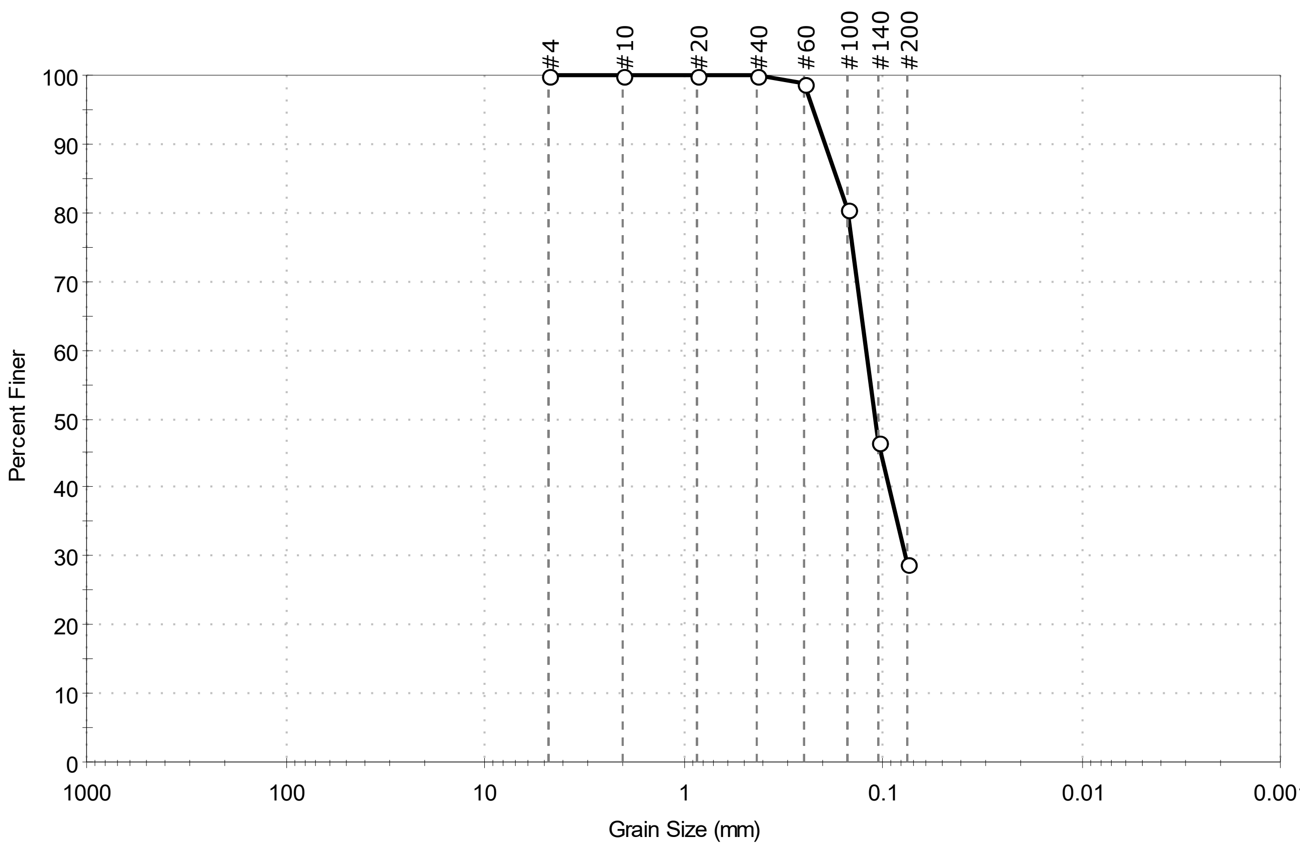
(See Separate Attachment)

APPENDIX D

LABORATORY TESTING RESULTS, 200-SERIES BORINGS

Client:	HNTB Corporation
Project:	MEDOT Tracy Brook Bridge
Location:	Durham, ME
Project No:	GTX-316027
Boring ID:	HB-DMB-201
Sample Type:	jar
Tested By:	ckg
Sample ID:	2D
Test Date:	09/07/22
Checked By:	ank
Depth :	4-6
Test Id:	684754
Test Comment:	---
Visual Description:	Moist, yellowish brown silty sand
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	71.2	28.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	81		
#140	0.11	47		
#200	0.075	29		

Coefficients

D_{85} =0.1700 mm
D_{30} =0.0767 mm
D_{60} =0.1216 mm
D_{15} =N/A
D_{50} =0.1098 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

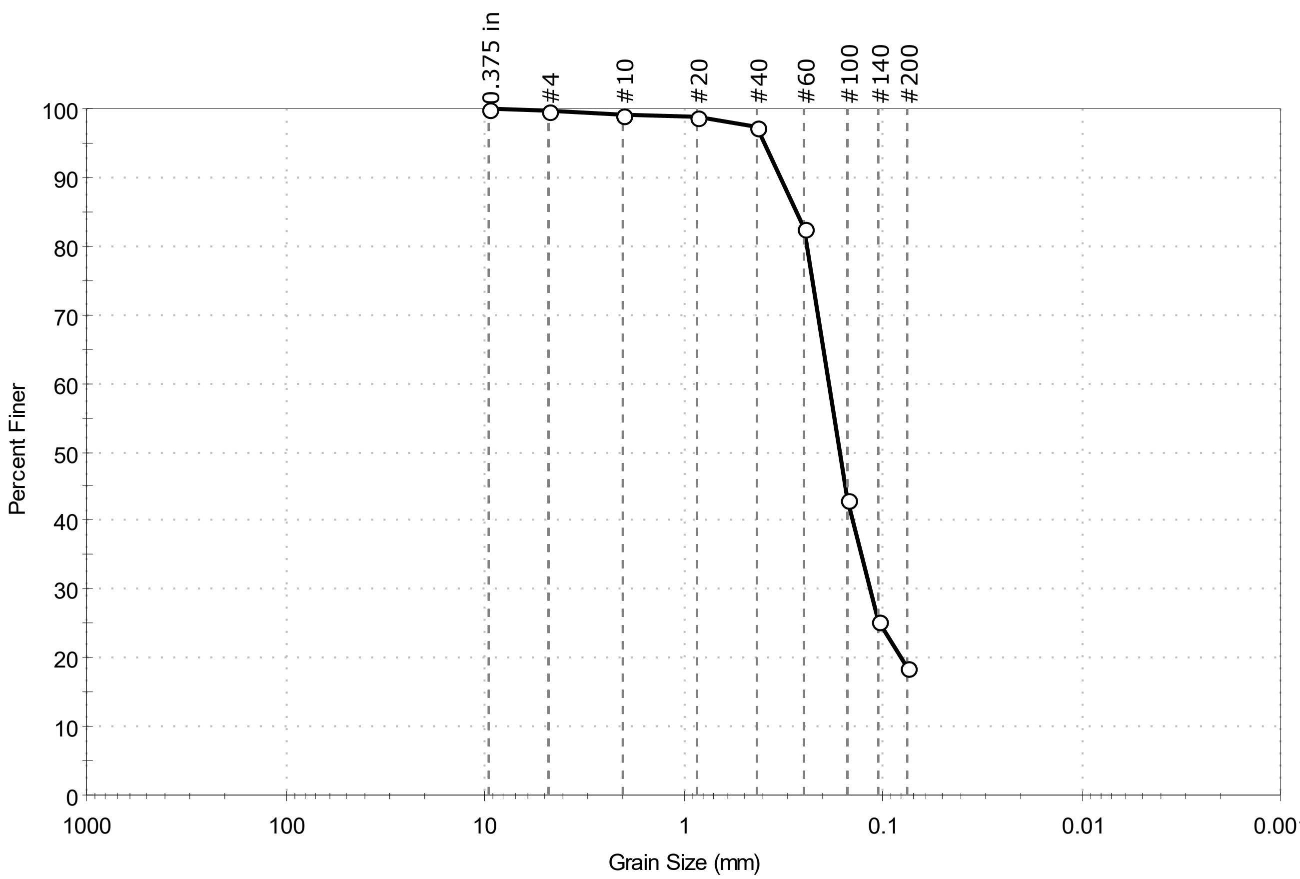
Client:	HNTB Corporation			
Project:	MEDOT Tracy Brook Bridge			
Location:	Durham, ME		Project No:	GTX-316027
Boring ID:	HB-DMB-201	Sample Type: jar	Tested By:	ckg
Sample ID:	4D	Test Date: 09/07/22	Checked By:	ank
Depth :	14-16	Test Id: 684755		

Test Comment: ---

Visual Description: Moist, yellowish brown silty sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.2	81.1	18.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	99		
#20	0.85	99		
#40	0.42	97		
#60	0.25	83		
#100	0.15	43		
#140	0.11	25		
#200	0.075	19		

Coefficients

D_{85} =0.2733 mm D_{30} =0.1161 mm

D_{60} =0.1866 mm D_{15} =N/A

D_{50} =0.1639 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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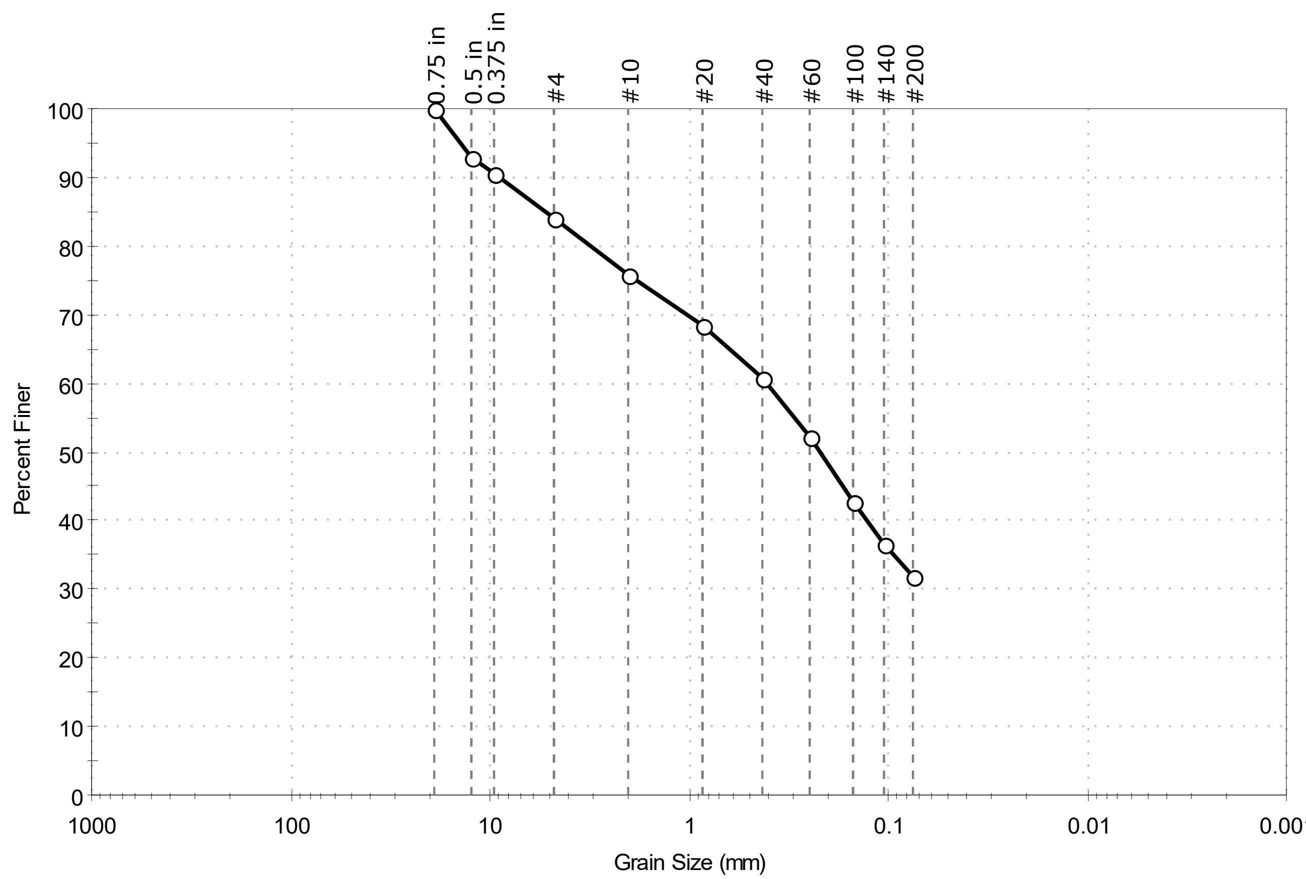
Client: HNTB Corporation			
Project: MEDOT Tracy Brook Bridge			
Location: Durham, ME		Project No:	GTX-316027
Boring ID: HB-DMB-201	Sample Type: jar	Tested By:	ckg
Sample ID: 7D	Test Date: 09/07/22	Checked By:	ank
Depth : 29-31	Test Id: 684756		
Test Comment: ---			
Visual Description: Moist, gray silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	15.9	52.3	31.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	93		
0.375 in	9.50	90		
#4	4.75	84		
#10	2.00	76		
#20	0.85	69		
#40	0.42	61		
#60	0.25	52		
#100	0.15	43		
#140	0.11	37		
#200	0.075	32		

Coefficients

D_{85} =5.2499 mm | D_{30} =N/A

D_{60} =0.4052 mm | D_{15} =N/A

D_{50} =0.2208 mm | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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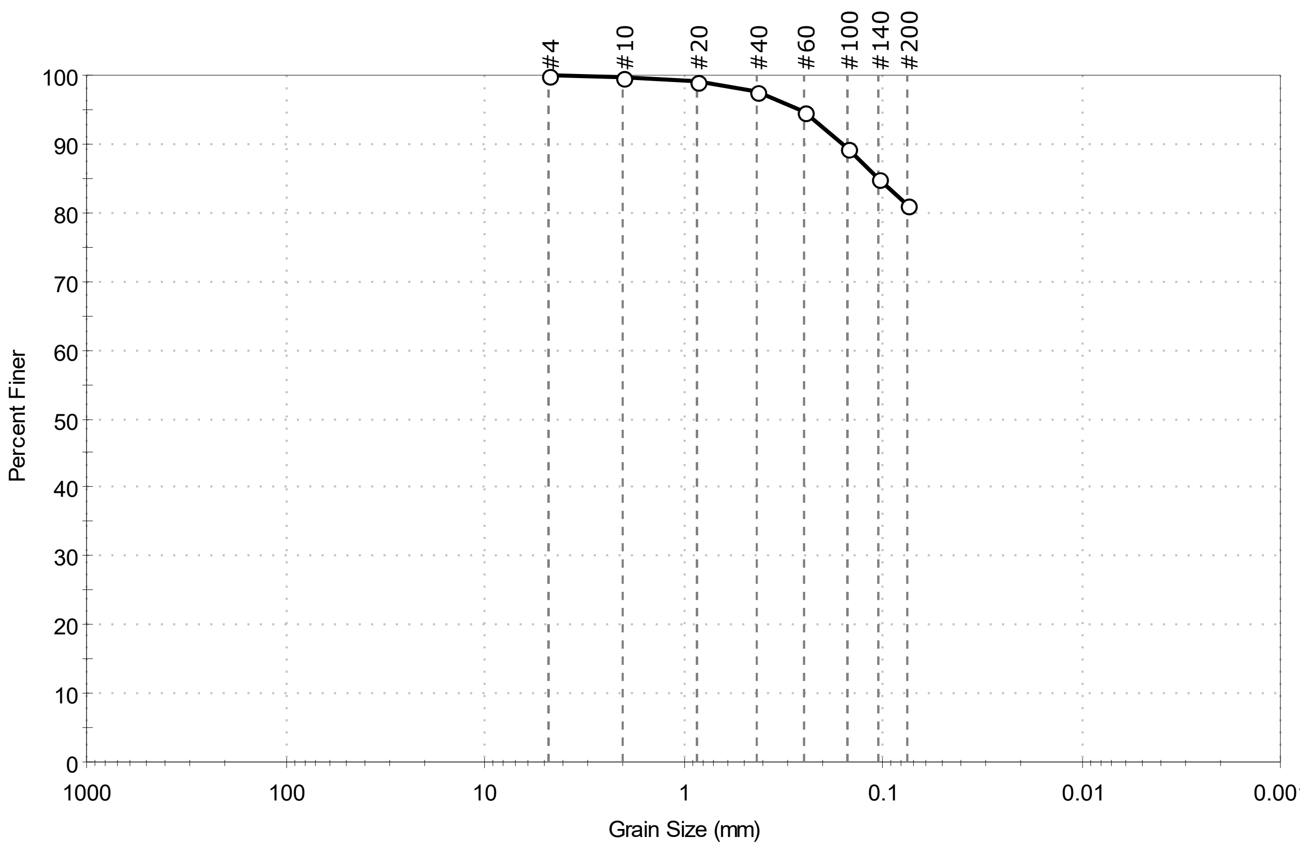
GeoTesting EXPRESS

Client:	HNTB Corporation
Project:	MEDOT Tracy Brook Bridge
Location:	Durham, ME
Project No:	GTX-316027
Boring ID:	HB-DMB-201
Sample Type:	jar
Tested By:	ckg
Sample ID:	9D
Test Date:	09/07/22
Checked By:	ank
Depth :	39-40.8
Test Id:	684757
Test Comment:	---
Visual Description:	Moist, gray silt with sand
Sample Comment:	---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	18.8	81.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	98		
#60	0.25	95		
#100	0.15	90		
#140	0.11	85		
#200	0.075	81		

Coefficients

D_{85} =0.1073 mm | D_{30} =N/A
D_{60} =N/A | D_{15} =N/A
D_{50} =N/A | D_{10} =N/A
C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

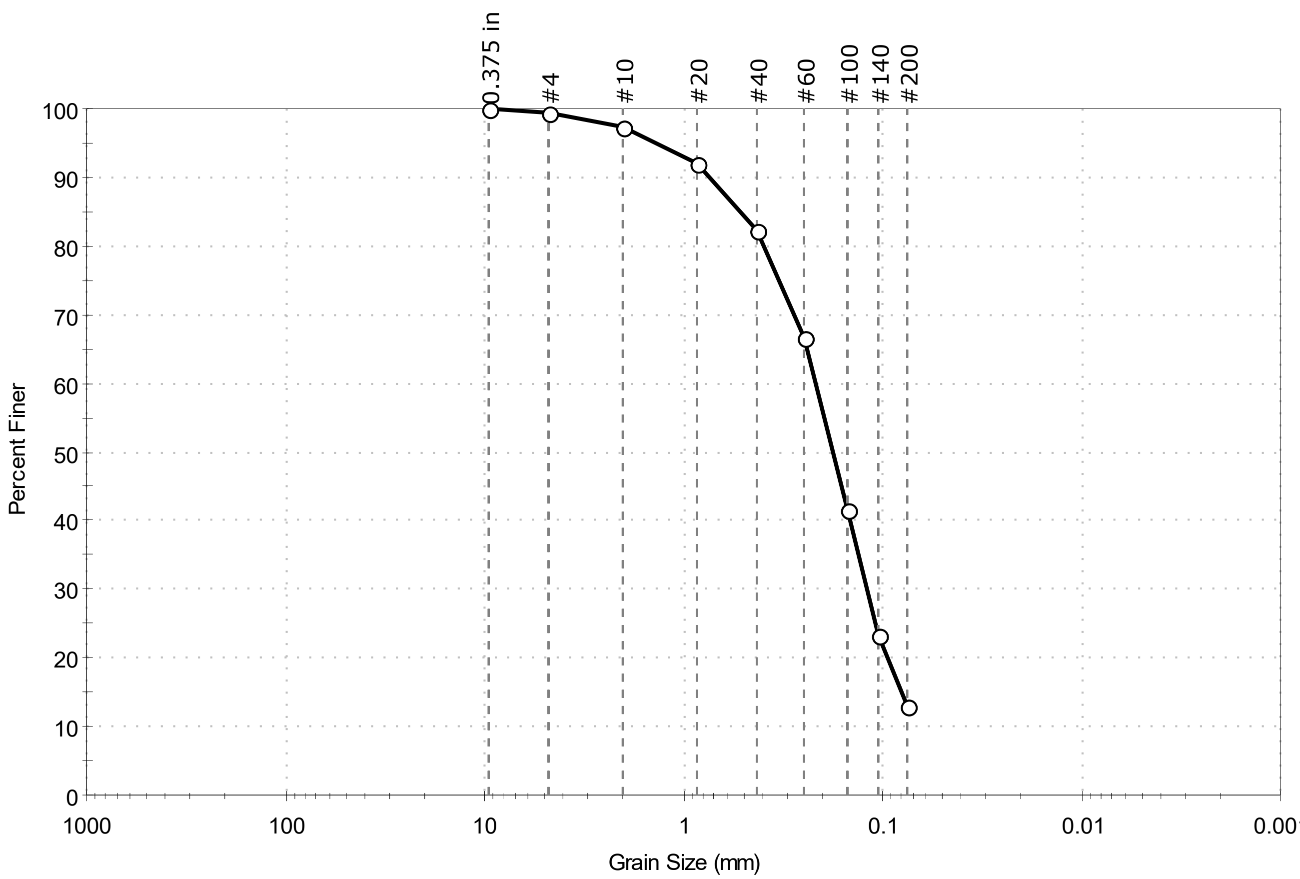
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client: HNTB Corporation			
Project: MEDOT Tracy Brook Bridge			
Location: Durham, ME		Project No:	GTX-316027
Boring ID: HB-DMB-202	Sample Type: jar	Tested By:	ckg
Sample ID: 2D	Test Date: 09/07/22	Checked By:	ank
Depth : 5-7	Test Id: 684758		
Test Comment: ---			
Visual Description: Moist, brown silty sand			
Sample Comment: ----			

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.4	86.5	13.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	97		
#20	0.85	92		
#40	0.42	82		
#60	0.25	67		
#100	0.15	42		
#140	0.11	23		
#200	0.075	13		

Coefficients

D_{85} =0.5157 mm | D_{30} =0.1204 mm

D_{60} =0.2179 mm | D_{15} =0.0800 mm

D_{50} =0.1778 mm | D_{10} =N/A

C_u =N/A | C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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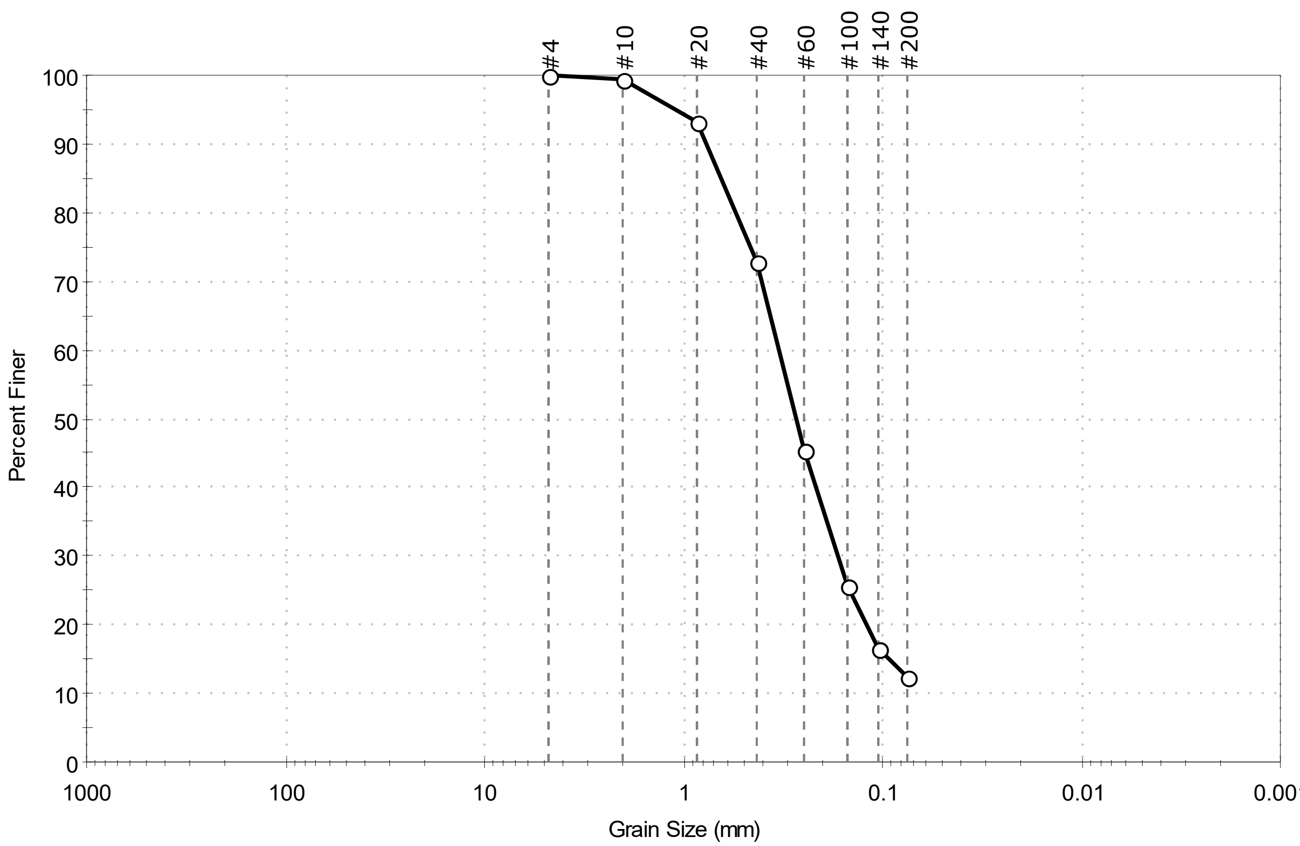
GeoTesting EXPRESS

Client:	HNTB Corporation		
Project:	MEDOT Tracy Brook Bridge		
Location:	Durham, ME	Project No:	GTX-316027
Boring ID: HB-DMB-202	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 09/07/22	Checked By:	ank
Depth : 19-21	Test Id: 684759		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	87.7	12.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	93		
#40	0.42	73		
#60	0.25	45		
#100	0.15	26		
#140	0.11	17		
#200	0.075	12		

Coefficients

D_{85} =0.6422 mm D_{30} =0.1675 mm
D_{60} =0.3319 mm D_{15} =0.0936 mm
D_{50} =0.2735 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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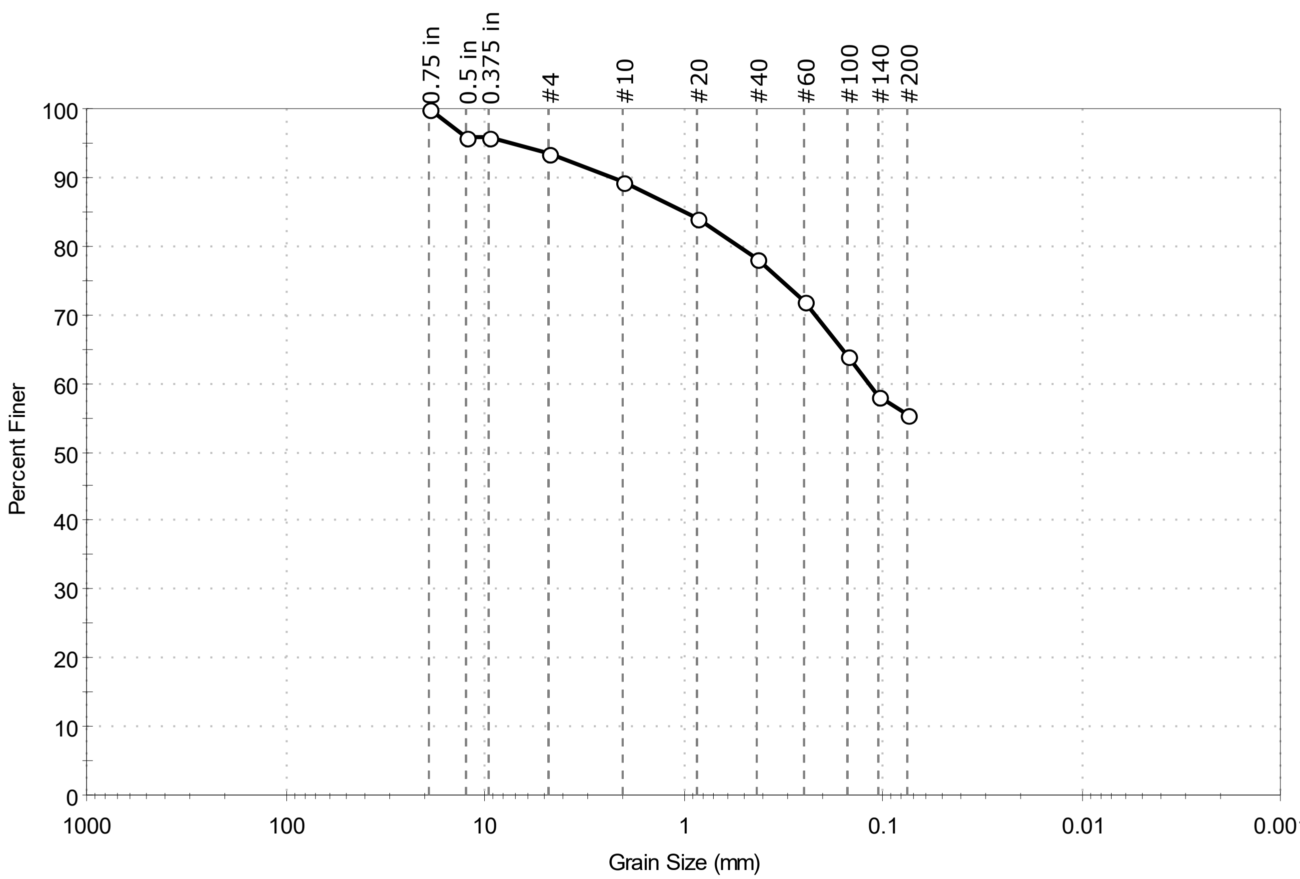
Client:	HNTB Corporation		
Project:	MEDOT Tracy Brook Bridge		
Location:	Durham, ME	Project No:	GTX-316027
Boring ID:	HB-DMB-203	Sample Type: jar	Tested By: ckg
Sample ID:	7D	Test Date: 09/07/22	Checked By: ank
Depth :	29-31	Test Id: 684760	
Test Comment:	---		
Visual Description:	Moist, gray sandy silt		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.6	38.0	55.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	96		
#4	4.75	93		
#10	2.00	89		
#20	0.85	84		
#40	0.42	78		
#60	0.25	72		
#100	0.15	64		
#140	0.11	58		
#200	0.075	55		

Coefficients

D_{85} =0.9901 mm　　D_{30} =N/A

D_{60} =0.1178 mm　　D_{15} =N/A

D_{50} =N/A　　D_{10} =N/A

C_u =N/A　　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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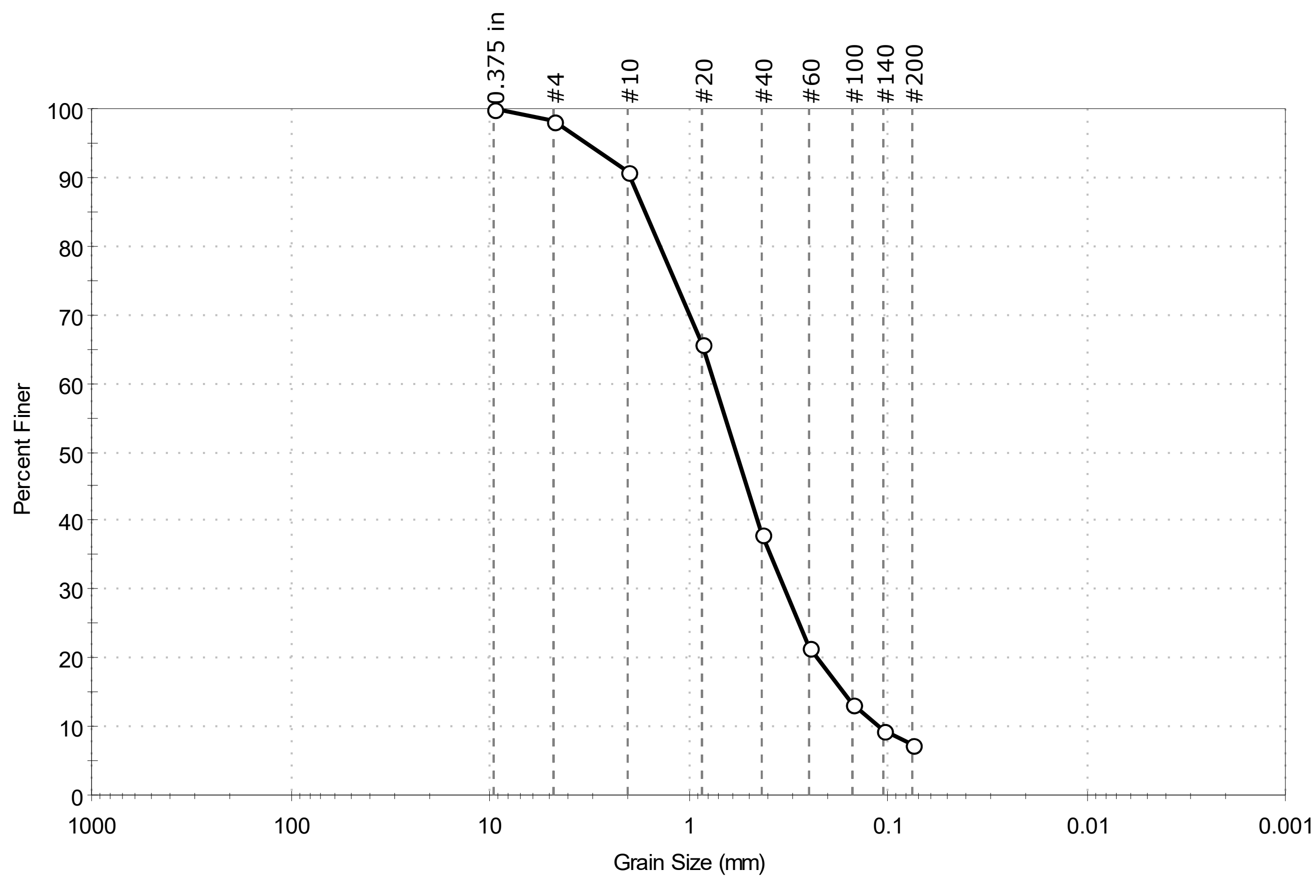
Client:	HNTB Corporation		
Project:	MEDOT Tracy Brook Bridge		
Location:	Durham, ME	Project No:	GTX-316027
Boring ID: HB-DMB-203	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 09/07/22	Checked By:	ank
Depth : 14-16	Test Id: 684783		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.7	90.9	7.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	98		
#10	2.00	91		
#20	0.85	66		
#40	0.42	38		
#60	0.25	22		
#100	0.15	13		
#140	0.11	10		
#200	0.075	7.4		

Coefficients

D_{85} =1.6359 mm
D_{30} =0.3285 mm
D_{60} =0.7359 mm
D_{15} =0.1678 mm
D_{50} =0.5740 mm
D_{10} =0.1111 mm
C_u =6.624
C_c =1.320

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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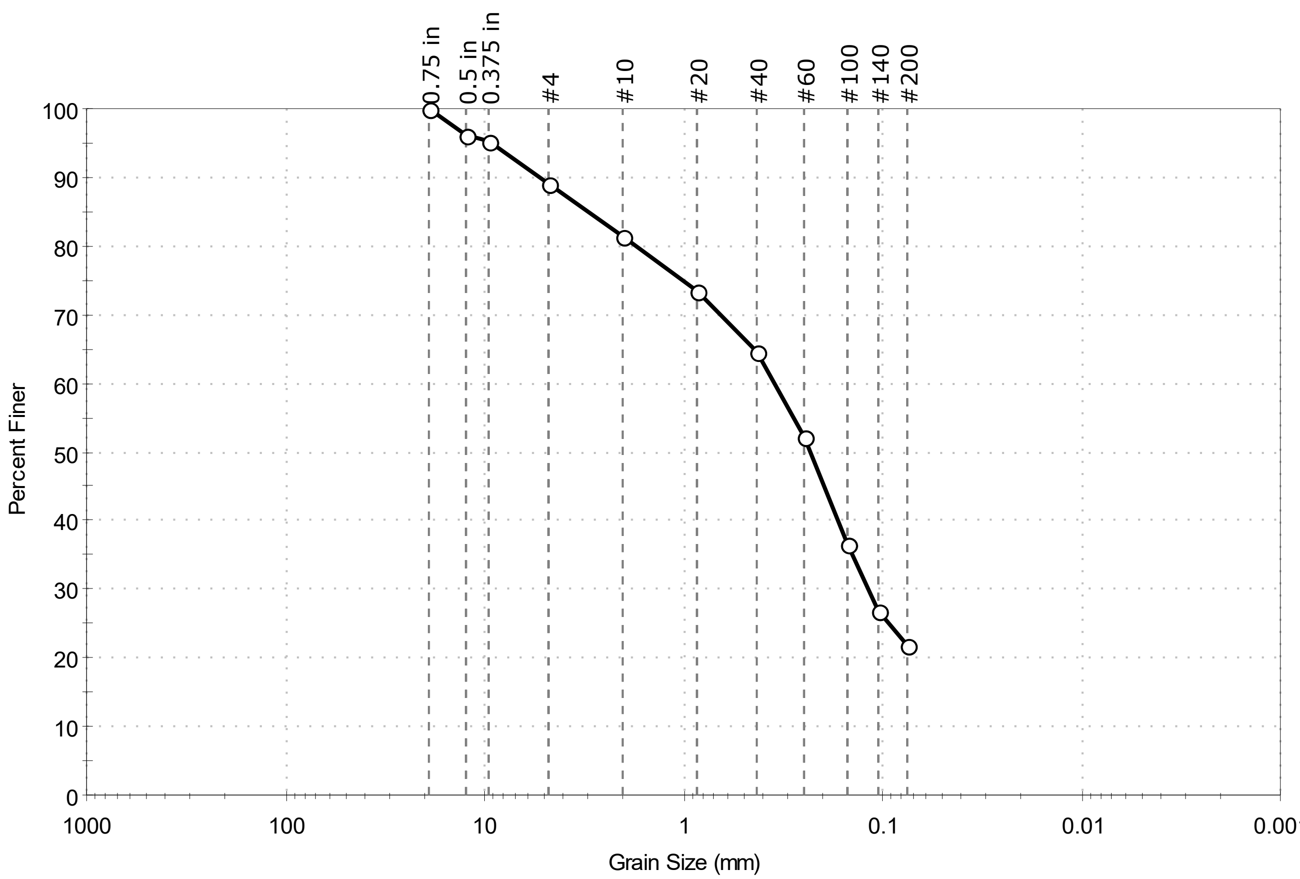
GeoTesting EXPRESS

Client: HNTB Corporation			
Project: MEDOT Tracy Brook Bridge			
Location: Durham, ME		Project No:	GTX-316027
Boring ID: HB-DMB-204	Sample Type: jar	Tested By:	ckg
Sample ID: 2D	Test Date: 09/07/22	Checked By:	ank
Depth : 4-6	Test Id: 684761		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.8	67.3	21.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	95		
#4	4.75	89		
#10	2.00	82		
#20	0.85	73		
#40	0.42	65		
#60	0.25	52		
#100	0.15	37		
#140	0.11	27		
#200	0.075	22		

Coefficients

D_{85} =2.9669 mm D_{30} =0.1189 mm
D_{60} =0.3479 mm D_{15} =N/A
D_{50} =0.2321 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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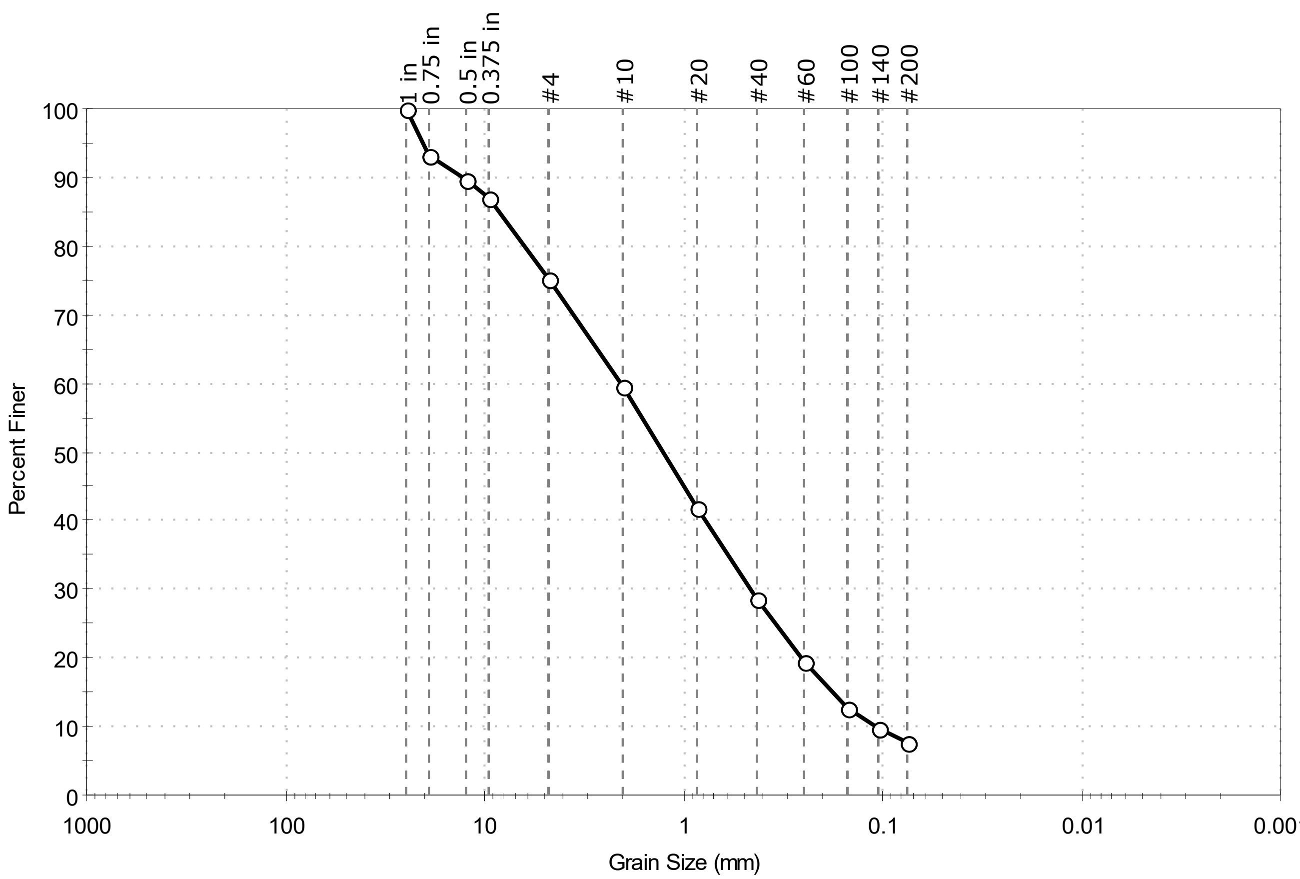
GeoTesting EXPRESS

Client:	HNTB Corporation				
Project:	MEDOT Tracy Brook Bridge				
Location:	Durham, ME			Project No:	GTX-316027
Boring ID:	HB-DMB-205	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	09/07/22	Checked By:	ank
Depth :	4-6	Test Id:	684762		
Test Comment:	---				
Visual Description:	Moist, brown sand with silt and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	24.9	67.3	7.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	93		
0.5 in	12.50	90		
0.375 in	9.50	87		
#4	4.75	75		
#10	2.00	60		
#20	0.85	42		
#40	0.42	29		
#60	0.25	19		
#100	0.15	13		
#140	0.11	10		
#200	0.075	7.8		

Coefficients

D_{85} =8.4553 mm D_{30} =0.4541 mm

D_{60} =2.0432 mm D_{15} =0.1778 mm

D_{50} =1.2530 mm D_{10} =0.1100 mm

C_u =18.575 C_c =0.917

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

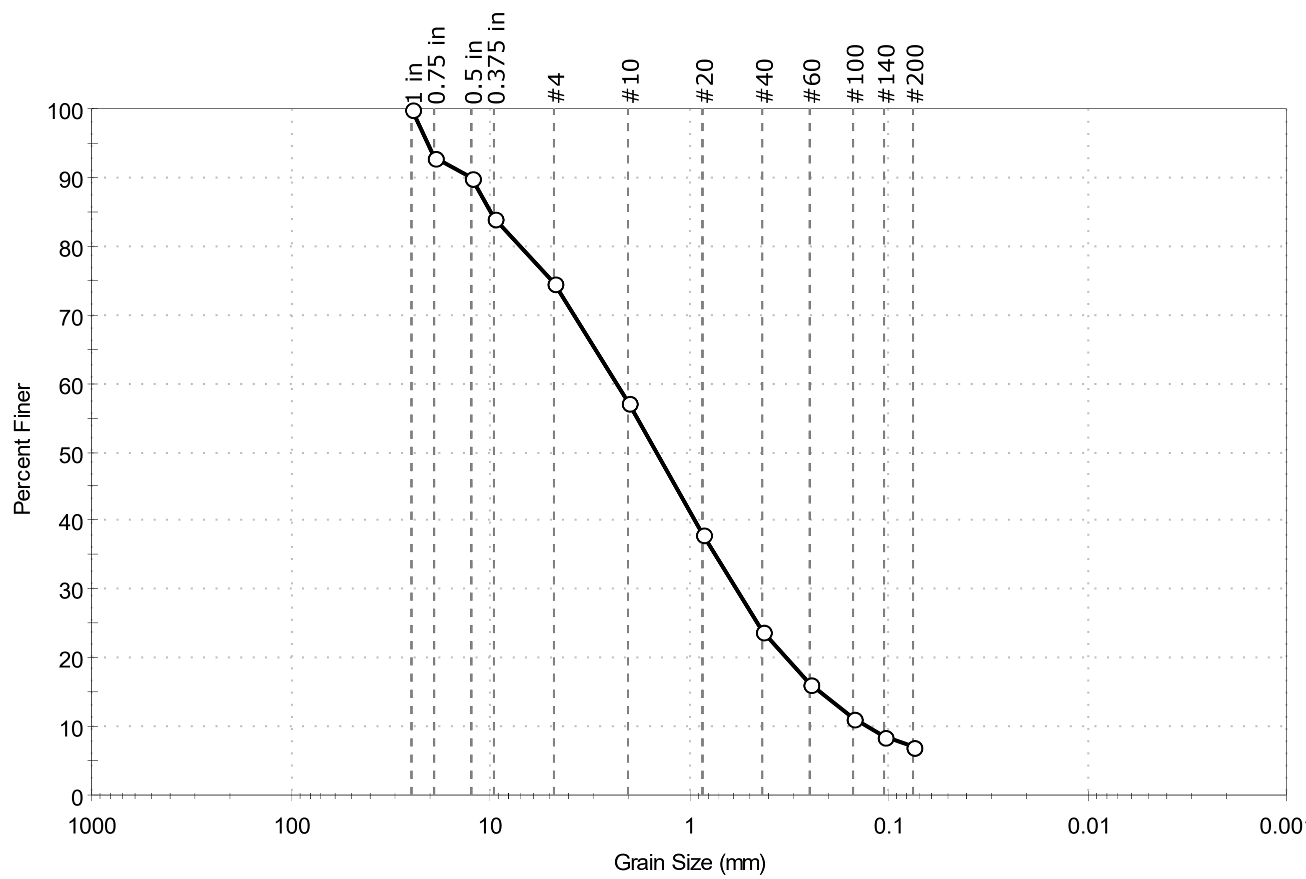
Sand/Gravel Hardness : HARD

printed 9/13/2022 2:20:16 PM

GeoTesting EXPRESS

Client:	HNTB Corporation		
Project:	MEDOT Tracy Brook Bridge		
Location:	Durham, ME	Project No:	GTX-316027
Boring ID: HB-DMB-205	Sample Type: jar	Tested By:	ckg
Sample ID: 4D	Test Date: 09/07/22	Checked By:	ank
Depth : 14-16	Test Id: 684784		
Test Comment:	---		
Visual Description:	Moist, brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.5	67.4	7.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	93		
0.5 in	12.50	90		
0.375 in	9.50	84		
#4	4.75	75		
#10	2.00	57		
#20	0.85	38		
#40	0.42	24		
#60	0.25	16		
#100	0.15	11		
#140	0.11	8		
#200	0.075	7.1		

Coefficients

D_{85} =9.8745 mm D_{30} =0.5713 mm

D_{60} =2.3001 mm D_{15} =0.2202 mm

D_{50} =1.4504 mm D_{10} =0.1293 mm

C_u =17.789 C_c =1.097

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 9/13/2022 2:20:16 PM

Client:	HNTB Corporation
Project:	MEDOT Tracy Brook Bridge
Location:	Durham, ME
GTX#:	316027
Test Date:	09/13/22
Tested By:	JB
Checked By:	ank

Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(\text{ohm-cm})^{-1}$
HB-DMB-202	3D, 4D	9-11, 14-16	Moist, brown silty sand	1,240	8.07E-04
HB-DMB-204	3D, 4D	9-11, 14-16	Moist, brown sand	27,892	3.59E-05
HB-DMB-204	5D, 6D	19-21, 24-26	Moist, gray silty sand with gravel	8,264	1.21E-04

Notes:
Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
Water added to sample to create a thick slurry prior to testing (saturated condition).
Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
Test conducted in standard laboratory atmosphere: 68-73 F

GeoTesting EXPRESS

Client:	HNTB Corportation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	09/08/22
Tested By:	JB
Checked By:	ank

pH of Soil by AASHTO T 289

Boring ID	Sample ID	Depth ft	Tare ID	Description	pH Reading
HB-DMB-202	3D,4D	9-11, 14-16		Moist, brown silty sand	6.41
HB-DMB-204	3D,4D	9-11, 14-16		Moist, brown sand	6.65
HB-DMB-204	5D, 6D	19-21, 24-26		Moist, gray silty sand with gravel	5.83

Notes: Split soil on #4 sieve pulverize larger particles, split on #4 again. Put smaller material through #10 sieve and use 30 g of air-dried minus No. 10 sieve soil into 30 mL of Distilled Water for pH test.
Let stand for 1 hour, stirring every 10-15 minutes.

Testing Services[sm]

PO Box 572455 / Salt Lake City UT 84157-2455 / USA
TEL +1 801 262 2448 · FAX +1 801 262 9870 · www.TEi-TS.com

GEOTESTING EPXRESS INCORPORATED
125 NAGOG PARK
ACTON MA 01720-3451
USA

Analysis No.	TS-A2210555
Report Date	12 September 2022
Date Sampled	02 September 2022
Date Received	09 September 2022
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Soil: Project: IMEDOT Tracy Brook Bridge; Site Location: Durham, ME; Job Number: GTX-316027

When examined to the applicable requirements of:

AASHTO T-291-18	"Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil" Method B
AASHTO T 290-20	"Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil"

Results:

AASHTO T 291 - Chloride Method B

<table>
<tr><th colspan="2" rowspan="2">Sample</th><th colspan="2">Results</th><th rowspan="2">Detection Limit</th></tr>
<tr><th>ppm (mg/kg)</th><th>%[1]</th></tr>
<tr><td colspan="2">HB-DMB-202</td><td rowspan="2">19.</td><td rowspan="2">0.0019</td><td rowspan="6">10.</td></tr>
<tr><td>3D, 4D</td><td>9-11' – 14-16'</td></tr>
<tr><td colspan="2">HB-DMB-204</td><td rowspan="2">< 10.</td><td rowspan="2">< 0.0010</td></tr>
<tr><td>3D, 4D</td><td>9-11' – 14-16'</td></tr>
<tr><td colspan="2">HB-DMB-204</td><td rowspan="2">< 10.</td><td rowspan="2">< 0.0010</td></tr>
<tr><td>5D, 6D</td><td>19-21' – 24-26'</td></tr>
</table>

NOTE: [1]Percent by weight after drying and prepared as per the Standard.

3455 South 500 West
Salt Lake City, UT 84115-4234 USA
TEL: +1 801 262 2448
FAX: +1 801 262 9870

AASHTO T 290 – Sulfates (Soluble)

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
HB-DMB-202		< 10.	< 0.0010	10.
3D, 4D	9-11’ – 14-16’			
HB-DMB-204		< 10.	< 0.0010	
3D, 4D	9-11’ – 14-16’			
HB-DMB-204		24.	0.0024	
5D, 6D	19-21’ – 24-26’			

NOTE: [1]Percent by weight after drying and prepared as per the Standard.

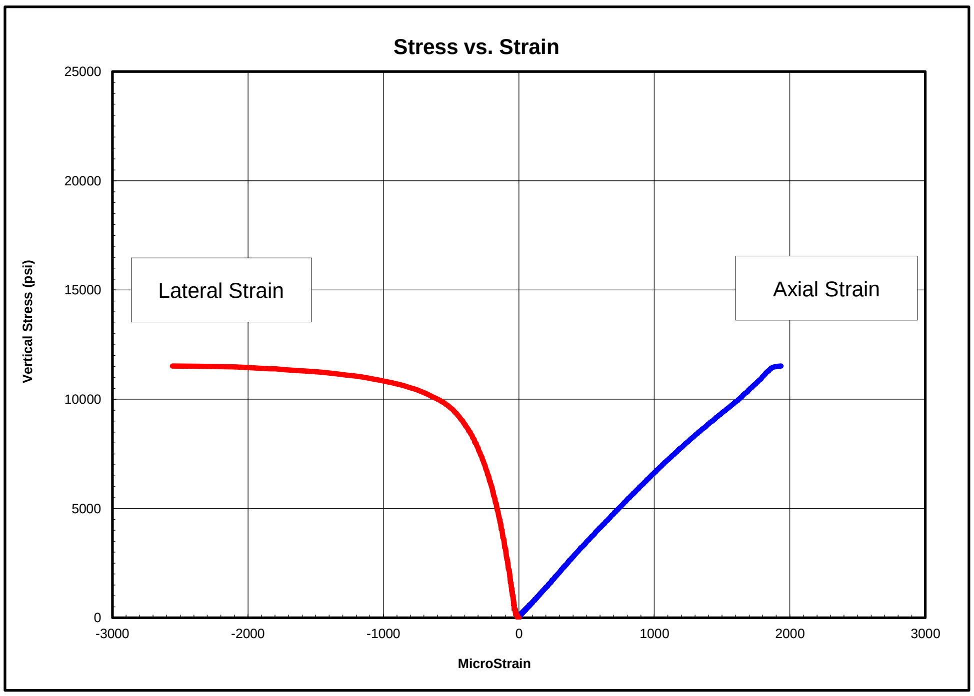
END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	HB-DMB-203
Sample ID:	R1
Depth, ft:	42.22-42.60
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 11,521 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1200-4200	6,890,000	0.19
4200-7300	6,220,000	0.28
7300-10400	5,230,000	---

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

GeoTesting EXPRESS

Client:	HNTB Corporation	Test Date:	9/15/2022
Project Name:	MEDOT Tracy Brook Bridge	Tested By:	jab/kdp
Project Location:	Durham, ME	Checked By:	smd
GTX #:	316027		
Boring ID:	HB-DMB-203		
Sample ID:	R1		
Depth:	42.22-42.60 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.33	4.35	4.34	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	599.85			
Bulk Density, lb/ft^3	171	**Minimum Diameter Tolerence Met?**	**YES**	
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**	

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

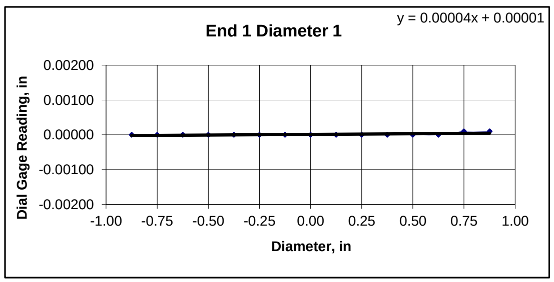
END FLATNESS AND PARALLELISM (Procedure FP1)

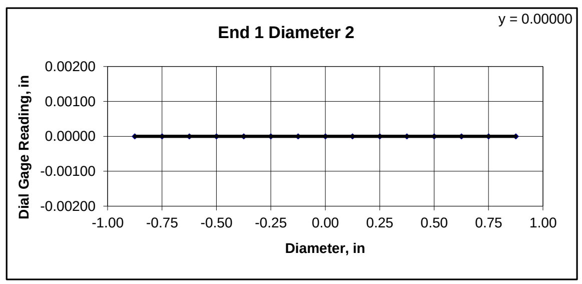
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000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

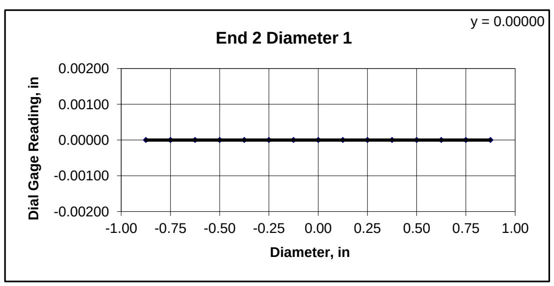
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00000

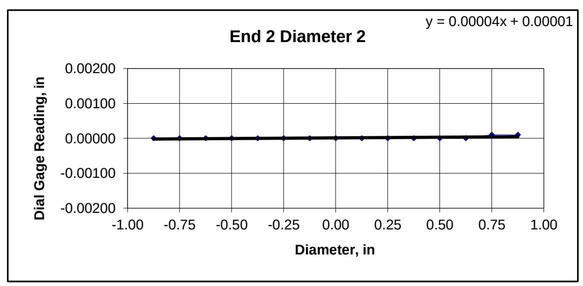
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0 90° = 0.0001
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00005
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00213
End 2:	
Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00000
Maximum Angular Difference:	0.00213

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00000
Angle of Best Fit Line:	0.00000
End 2:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00213
Maximum Angular Difference:	0.00213

Parallelism Tolerance Met? **YES**
Spherically Seated

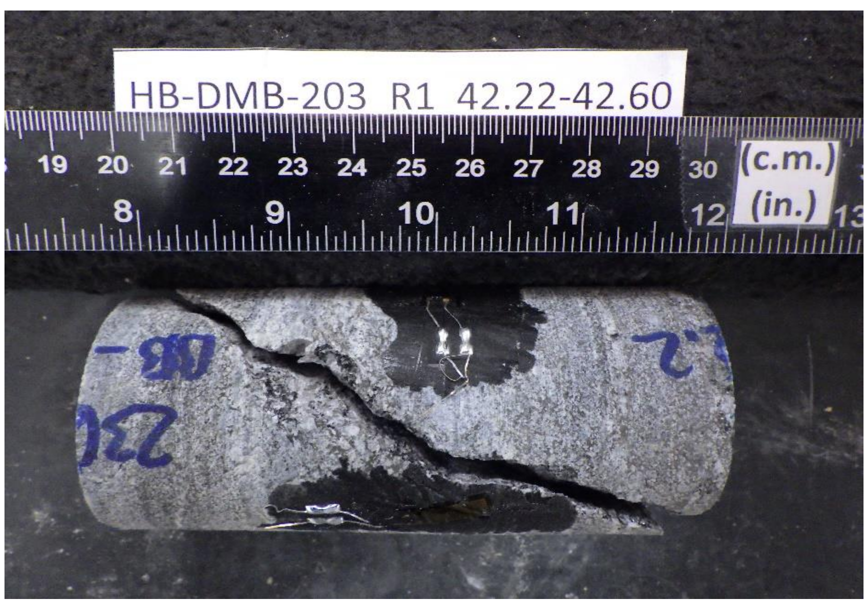
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00000	1.980	0.00000	0.000	YES
END 2					
Diameter 1, in	0.00000	1.980	0.00000	0.000	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES

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

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	smd
Boring ID:	HB-DMB-203
Sample ID:	R1
Depth, ft:	42.22-42.60

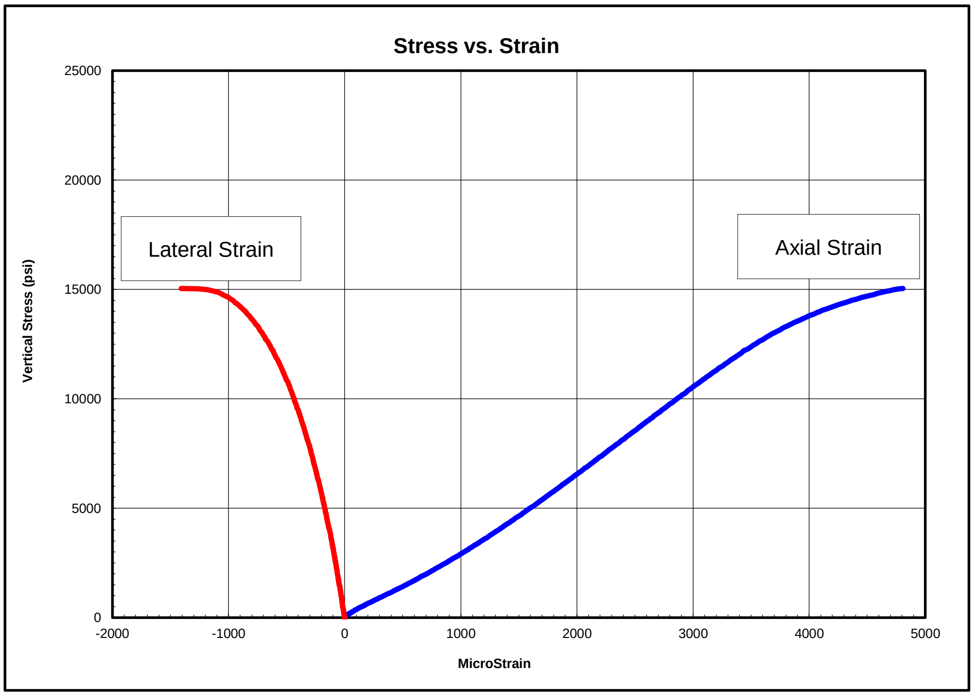
After cutting and grinding



After break

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	HB-DMB-203
Sample ID:	R2
Depth, ft:	45.29-45.66
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 15,042 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1500-5500	3,330,000	0.12
5500-9500	3,980,000	0.21
9500-13500	3,580,000	0.33

Notes:

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

GeoTesting EXPRESS

Client:	HNTB Corporation	Test Date:	9/16/2022
Project Name:	MEDOT Tracy Brook Bridge	Tested By:	jab/kdp
Project Location:	Durham, ME	Checked By:	smd
GTX #:	316027		
Boring ID:	HB-DMB-203		
Sample ID:	R2		
Depth:	45.29-45.66 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.34	4.34	4.34	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	595.63			
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**		YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**		YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

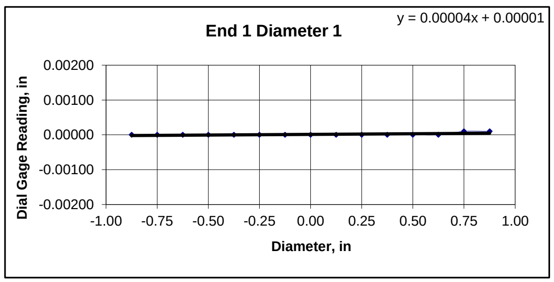
END FLATNESS AND PARALLELISM (Procedure FP1)

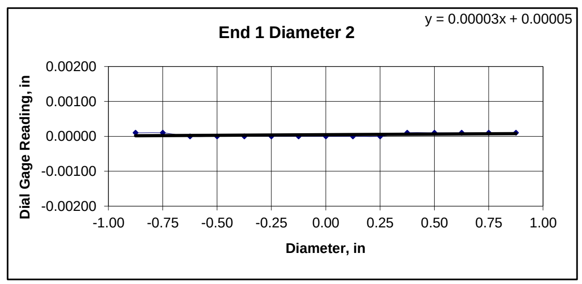
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00000	0.00000	0.00000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

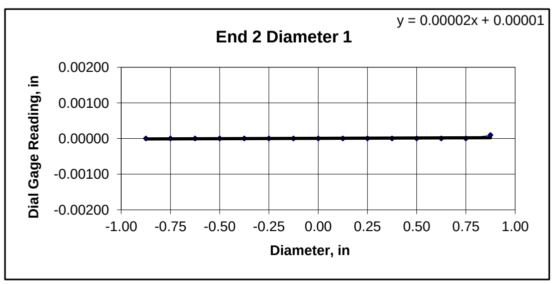
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00010

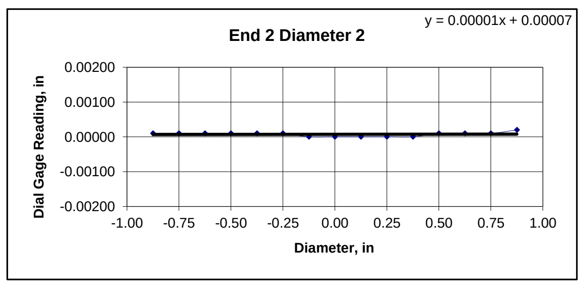
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010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

Difference between max and min readings, in:
0° = 0.0001 90° = 0.0002
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00010
Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00004
Angle of Best Fit Line:	0.00213
End 2:	
Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115
Maximum Angular Difference:	0.00098

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00003
Angle of Best Fit Line:	0.00196
End 2:	
Slope of Best Fit Line	0.00001
Angle of Best Fit Line:	0.00033
Maximum Angular Difference:	0.00164

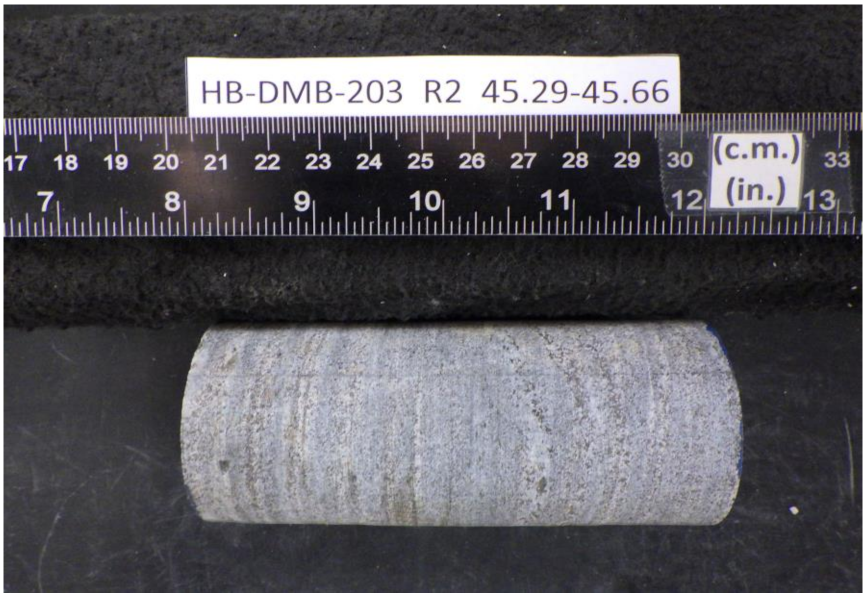
Parallelism Tolerance Met? YES
Spherically Seated

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

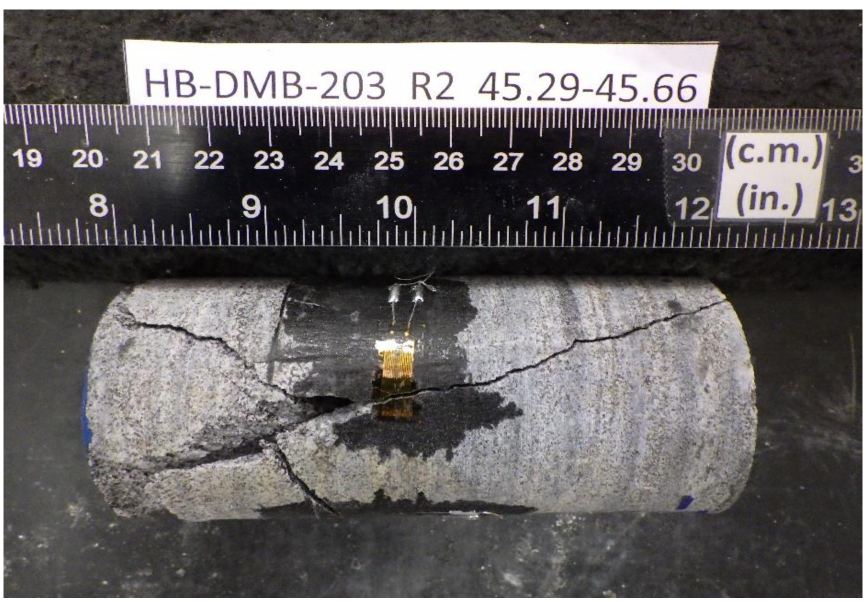
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00010	1.980	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

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

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	smd
Boring ID:	HB-DMB-203
Sample ID:	R2
Depth, ft:	45.29-45.66



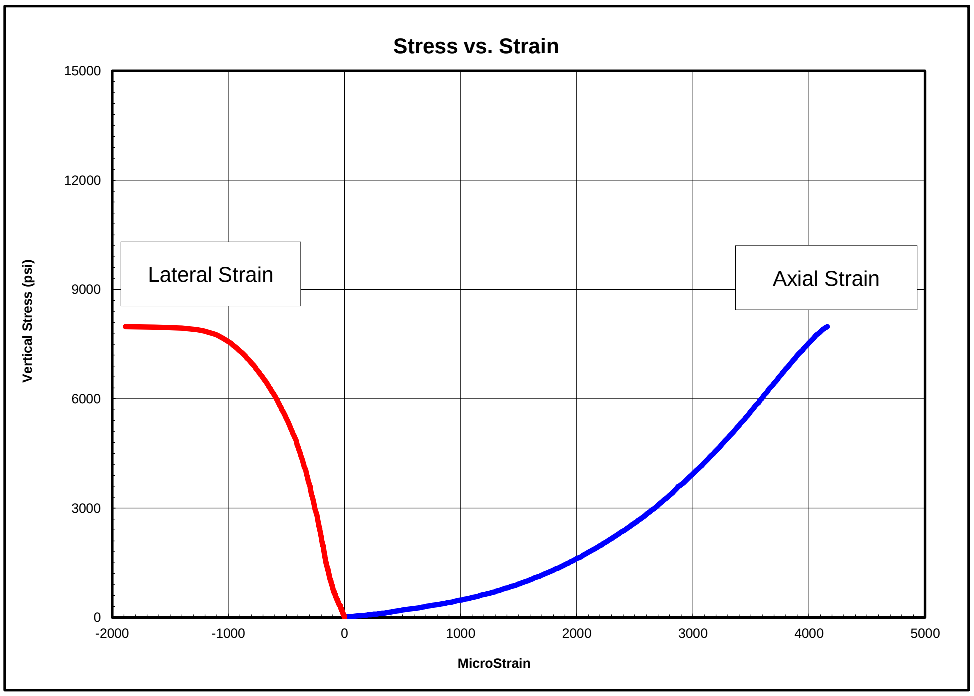
After cutting and grinding



After break

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	HB-DMB-204
Sample ID:	R1
Depth, ft:	31.89-32.27
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: 7,979 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
800-2900	1,690,000	0.11
2900-5100	3,060,000	0.28
5100-7200	3,840,000	---

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

GeoTesting EXPRESS

Client:	HNTB Corporation	Test Date:	9/19/2022
Project Name:	MEDOT Tracy Brook Bridge	Tested By:	jab/kdp
Project Location:	Durham, ME	Checked By:	smd
GTX #:	316027		
Boring ID:	HB-DMB-204		
Sample ID:	R1		
Depth:	31.89-32.27 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.35	4.35	4.35	
Specimen Diameter, in:	1.99	2.00	2.00	
Specimen Mass, g:	599.54			
Bulk Density, lb/ft^3	168	**Minimum Diameter Tolerence Met?**		YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**		YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

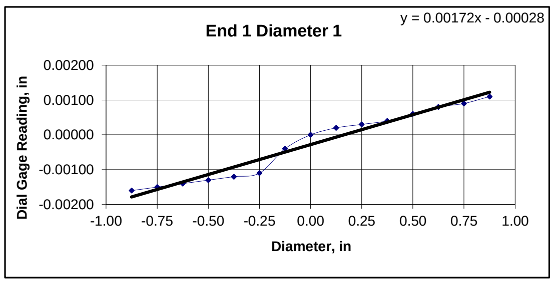
END FLATNESS AND PARALLELISM (Procedure FP1)

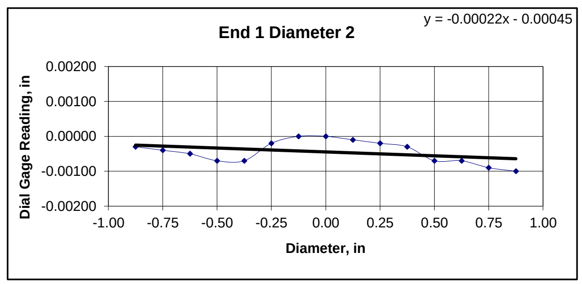
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00160	-0.00150	-0.00140	-0.00130	-0.00120	-0.00110	-0.00040	0.00000	0.00020	0.00030	0.00040	0.00060	0.00080	0.00090	0.00110
Diameter 2, in (rotated 90°)	-0.00030	-0.00040	-0.00050	-0.00070	-0.00070	-0.00020	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00070	-0.00070	-0.00090	-0.00100

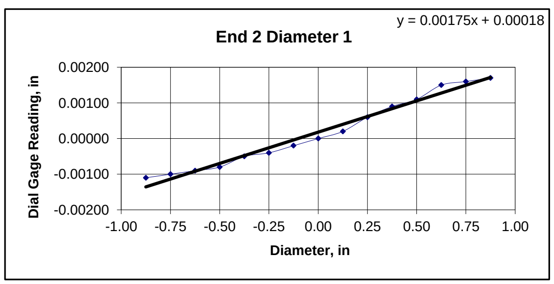
Difference between max and min readings, in:
0° = 0.00270 90° = 0.00100

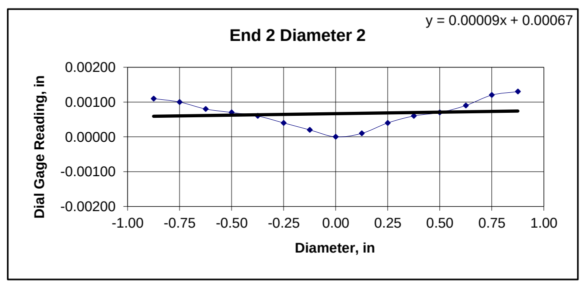
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00110	-0.00100	-0.00090	-0.00080	-0.00050	-0.00040	-0.00020	0.00000	0.00020	0.00060	0.00090	0.00110	0.00150	0.00160	0.00170
Diameter 2, in (rotated 90°)	0.00110	0.00100	0.00080	0.00070	0.00060	0.00040	0.00020	0.00000	0.00010	0.00040	0.00060	0.00070	0.00090	0.00120	0.00130

Difference between max and min readings, in:
0° = 0.0028 90° = 0.0013
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00140
Flatness Tolerance Met? NO









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00172
Angle of Best Fit Line: 0.09838

End 2:
Slope of Best Fit Line 0.00175
Angle of Best Fit Line: 0.10051

Maximum Angular Difference: 0.00213

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00022
Angle of Best Fit Line: 0.01277

End 2:
Slope of Best Fit Line 0.00009
Angle of Best Fit Line: 0.00491

Maximum Angular Difference: 0.00786

Parallelism Tolerance Met? NO
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00270	1.995	0.00135	0.078	YES
Diameter 2, in (rotated 90°)	0.00100	1.995	0.00050	0.029	YES
END 2					
Diameter 1, in	0.00280	1.995	0.00140	0.080	YES
Diameter 2, in (rotated 90°)	0.00130	1.995	0.00065	0.037	YES

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

Client:	HNTB Corporation	Test Date:	9/19/2022
Project Name:	MEDOT Tracy Brook Bridge	Tested By:	jab/kdp
Project Location:	Durham, ME	Checked By:	smd
GTX #:	316027		
Boring ID:	HB-DMB-204	Reliable dial gauge measurements could not be performed on this rock type. Tolerance measurements were performed using a machinist straightedge and feeler gauges to ASTM specifications.	
Sample ID:	R1		
Depth (ft):	31.89-32.27		
Visual Description:	See photographs		

BEST EFFORT END FLATNESS TOLERANCES OF ROCK CORE SPECIMENS TO ASTM D4543

END FLATNESS

END 1		
Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90^{o})	Is the maximum gap $\leq \pm 0.001$ in.?	YES
END 2		
Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90^{o})	Is the maximum gap $\leq \pm 0.001$ in.?	YES

End Flatness Tolerance Met? YES

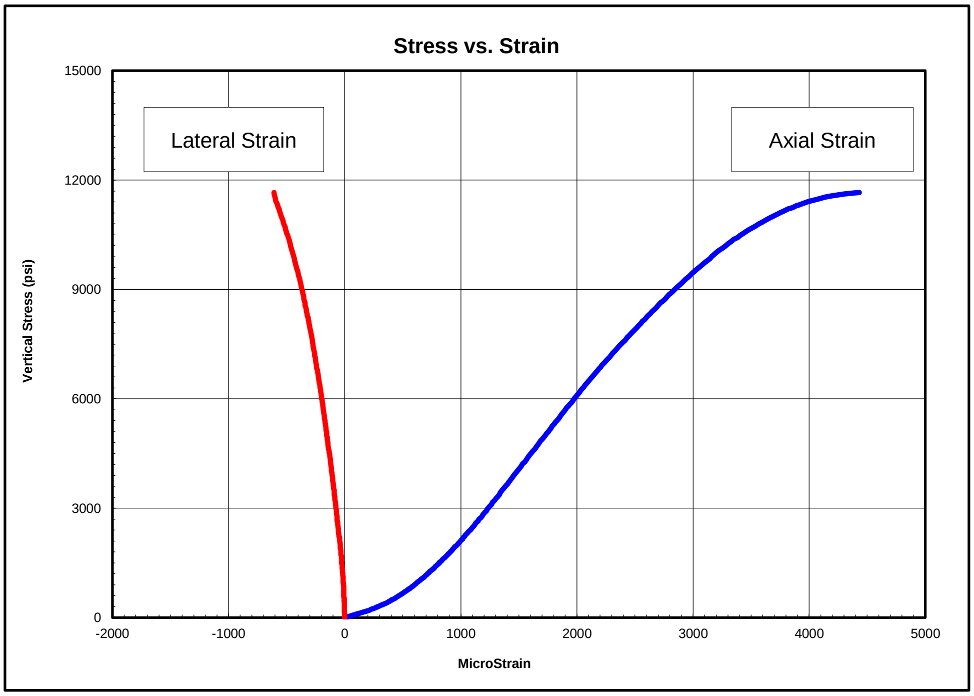
Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	smd
Boring ID:	HB-DMB-204
Sample ID:	R1
Depth, ft:	31.89-32.27

After cutting and grinding

After break

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	jsc
Boring ID:	HB-DMB-204
Sample ID:	R2
Depth, ft:	36.81-37.21
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: 11,659 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1200-4300	3,710,000	0.13
4300-7400	3,940,000	0.18
7400-10500	2,970,000	0.21

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:	HNTB Corporation	Test Date:	9/16/2022
Project Name:	MEDOT Tracy Brook Bridge	Tested By:	jab/kdp
Project Location:	Durham, ME	Checked By:	smd
GTX #:	316027		
Boring ID:	HB-DMB-204		
Sample ID:	R2		
Depth:	36.81-37.21 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average		
Specimen Length, in:	4.21	4.22	4.22		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	584.2				
Bulk Density, lb/ft^3	169			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

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

Difference between max and min readings, in:

0° = 0.00000 90° = 0.00010

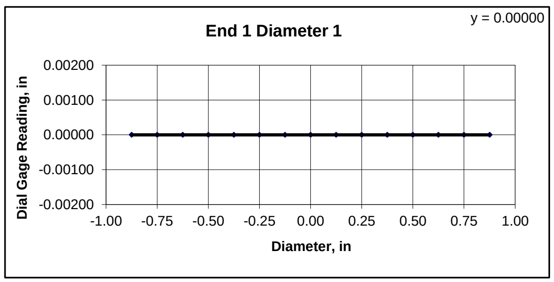
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000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

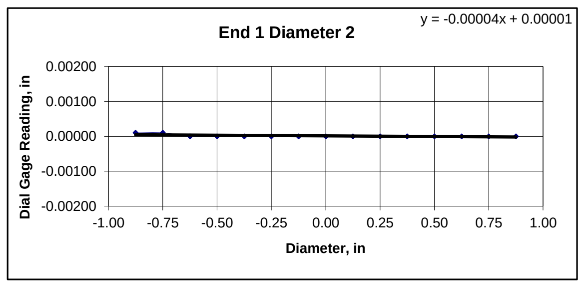
Difference between max and min readings, in:

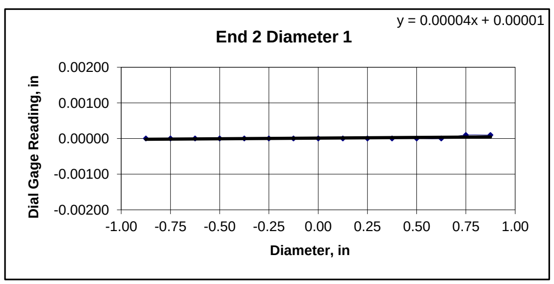
0° = 0.0001 90° = 0

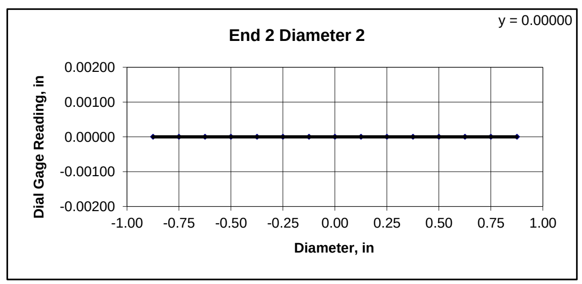
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00005

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:

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

End 2:

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

Maximum Angular Difference: 0.00213

Parallelism Tolerance Met? **YES**

Spherically Seated

DIAMETER 2

End 1:

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

End 2:

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

Maximum Angular Difference: 0.00213

Parallelism Tolerance Met? **YES**

Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00000	1.990	0.00000	0.000	YES
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00010	1.990	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00000	1.990	0.00000	0.000	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	HNTB Corporation
Project Name:	MEDOT Tracy Brook Bridge
Project Location:	Durham, ME
GTX #:	316027
Test Date:	9/27/2022
Tested By:	bp
Checked By:	smd
Boring ID:	HB-DMB-204
Sample ID:	R2
Depth, ft:	36.81-37.21

After cutting and grinding

After break