



REPORT

Geotechnical Design Report (GDR)

T8 R3 NBPP Big Meadow Bridge #0064 Replacement (WIN 027270.00)

Submitted to:

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MaineDOT WIN 027270.00 / WSP US0040947.9162

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1.0 INTRODUCTION

This Geotechnical Design Report (GDR) summarizes the results of WSP's geotechnical investigation and recommendations for the replacement of Big Meadow Bridge #0064 in T8 R3 NBPP, Maine (WIN 027270.00). The location of this culvert is shown in Figure 1 (attached). The purpose of this GDR is to present soil and bedrock information for the culvert replacement site obtained from subsurface investigations and laboratory tests; present recommended geotechnical design parameters for the proposed replacement culvert; and provide geotechnical design recommendations for construction of the replacement culvert.

2.0 PROJECT BACKGROUND

Big Meadow Bridge No. 0064 carries Route (RT) 169 over Meadow Brook in T8 R3 NBPP, Maine. The existing structure consists of a corrugated steel plate pipe arch constructed in 1979. The replacement design is a buried box culvert based on the estimated 16-foot bank-full width of Meadow Brook at this location.

3.0 GEOLOGIC SETTING

3.1 Regional Surficial Geology

The site lies upstream of the Mattawamkeag River in the Potter Hill Quadrangle (formerly known as the Millinocket Quadrangle and the Wytopitlock Quadrangle), where surficial soils are characterized as Holocene-aged swamp and stream deposits and Pleistocene-aged glacial till.¹ The surficial geology of the site is mapped as swamp or marsh deposits, described as silt, clay, and sand. These sediments overlie glacial till, described as a heterogeneous, non-stratified mixture of sand, silt, clay, and stones. Regional mapping indicates the presence of both basal till and ablation till. The basal till is described as fine grained and very compact, with low permeability and poor drainage; the ablation till is described as loose, sandy, and stony, with moderate permeability and fair to good drainage. The glacial till generally is present as a blanket deposit overlying and conforming to the bedrock surface.^{1,2}

3.2 Regional Bedrock Geology

The bedrock beneath the site was previously mapped as Silurian-aged unnamed metasiltstone and slate, interbedded with thin layers of metasandstone.³ A fault or shear zone was previously mapped as separating this unnamed unit from the Silurian-aged Daggett Ridge Formation (grey slate, metasiltstone, and quartzite) and the Silurian-aged Baskahegan Stream Slate (interbedded grey slate and metasiltstone with quartzite and cross-bedded sandstone) to the west. More recent mapping⁴ identifies the bedrock at site as Silurian-aged unnamed sandstone, weakly metamorphosed. The fault boundary was re-mapped further east, and the Daggett Ridge Formation and Baskahegan Slate were re-mapped as the Ordovician-aged Belle Lake Slate. This more recent mapping interprets the fault as a high-angle normal fault, and notes the location is uncertain.

¹ Newman, William A. and Holland, William R., 1986, Reconnaissance surficial geology of the Wytopitlock [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 86-27, map scale 1:62,500. *Maine Geological Survey Maps 755*. http://digitalmaine.com/mgs_maps/755

² Hanson, Lindley S., Caldwell, Dabney W. and Lowell, Thomas V. (compilers), 1987, Surficial geology of the Millinocket 1 x 2 degree quadrangle, Maine: Maine Geological Survey, Open-File Map 87-12, map scale 1:250,000. *Maine Geological Survey Maps 729*. http://digitalmaine.com/mgs_maps/729

³ Larrabee, D.M., Spencer, C.W., and Swift, D.J.P., 1965, Bedrock geology of the Grand Lake area, Aroostook, Hancock, Penobscot, and Washington Counties, Maine, U.S. Geological Survey Bulletin 1201-E.

⁴ Osberg, P.H., Hussey, A.M. II, and Boone, G.M. (editors), 1985, Bedrock geologic map of Maine: Maine Geological Survey, map scale 1:500,000. *Maine Geological Survey Maps 23*. http://digitalmaine.com/mgs_maps/23.

4.0 SUBSURFACE INVESTIGATIONS

WSP completed two (2) borings (BB-T8R3MB-101 and BB-T8R3MB-102) and two (2) rock probes (BP-T8R3MB-103 and BP-T8R3MB-104) between March 18 and March 20, 2025 to collect subsurface geotechnical information. WSP surveyed the as-drilled boring locations following completion of the drilling program. Boring and probe location coordinates and ground surface elevations are summarized in Table 1. Boring and probe locations with respect to existing and proposed site features are illustrated on the Boring Location Plan (BLP) shown in Figure 2. A WSP engineer monitored drilling activities, selected sampling intervals, logged subsurface conditions encountered, and obtained soil samples for use in visual descriptions and subsequent laboratory testing and classification.

WSP subcontracted Seaboard Drilling, LLC (Seaboard) of Bangor, Maine to complete the borings and probes using a Diedrich D-50 Turbo drill rig and both solid stem auger (SSA) and cased wash boring methods. Borings BB-T8R3MB-101 and BB-T8R3MB-102 were advanced using 4-inch diameter solid stem augers (SSA) in 5-foot lengths to 10 feet below ground surface (bgs) and 4-inch diameter casing in 5-foot lengths installed with an automatic safety hammer. Soil in the casing was washed out using a roller bit and water to the depth where samples were subsequently collected. Borings BB-T8R3MB-101 and BB-T8R3MB-102 were drilled to refusal between 14 feet and 17 feet below ground surface (bgs), where 12.3 feet and 15.8 feet of rock core was collected, respectively. Rock probes BP-T8R3MB-103 and BP-T8R3MB-104 were advanced using solid stem augers (SSA) without sampling to refusal between 19 feet and 20 feet below ground surface (bgs). Once refusal was met, no rock core was collected in BP-T8R3MB-103 or BP-T8R3MB-104.

Seaboard performed Standard Penetration Testing (SPT) using a calibrated automatic hammer system and standard 2-inch split spoon sampler in accordance with American Society for Testing and Materials (ASTM) D1586 for all borings. Sampling of overburden materials was conducted at approximately 5-foot intervals for all borings. Seaboard drove the split spoon sampler 24 inches with a 140-pound hammer dropped 30 inches and WSP recorded the number of hammer blows required to advance the split spoon sampler through each 6-inch increment. Measured, uncorrected N-values, calculated as the sum of the hammer blows to advance the sampler during the 6-inch to 18-inch interval, are provided in the boring logs in Appendix A. WSP used the average hammer energy transfer ratio of 100% for the Diedrich D-50 Turbo Rig (provided by Seaboard⁵) to convert the measured N-values to N_{60} values for further calculations. Soil samples were collected and stored in sealed glass jars for later evaluation and laboratory testing.

Seaboard collected rock core using NQ2 size (1.98-inch diameter) diamond tipped core barrels following refusal of either the casing, split spoon sampler, or roller bit to advance within the borehole. Rock core was placed in wooden boxes and transported to the WSP office. WSP recorded the lithology, Total Core Recovery (TCR), Rock Quality Designation (RQD), and coring rates for each core run in the boring logs provided in Appendix A. A detailed summary of the rock core quality parameters for the recovered rock core is presented in Table 2 and photographs of the rock core are presented in Appendix B.

The boring logs provided in Appendix A present details of the sampling methods used, field data obtained, and soil and rock conditions encountered during the investigation. Soils were field categorized in accordance with ASTM D2488. WSP field characterized the bedrock lithology, and the descriptions were revised in the office. A

⁵ 2024PA00175 - Seaboard - SPT Energy Calibration, prepared by GRL Engineers Inc., dated 10/23/2024.

description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs in Appendix A.

5.0 LABORATORY TESTING PROGRAM

After reviewing the samples collected in the office, WSP transferred select samples to GeoTesting Express, LLC (GTX) of Acton, Massachusetts for geotechnical laboratory testing in accordance with applicable AASHTO and ASTM testing procedures. WSP selected representative SPT split spoon soil samples from each of the borings for geotechnical laboratory testing to assist in soil classification. The types and numbers of each of the laboratory tests conducted on soil samples are presented in Table 5-1. Measured index and classification test results from soil samples are summarized in Table 2 and include AASHTO and ASTM Unified Soil Classification System (USCS) classifications. Soil testing results are included on the boring logs in Appendix A. Complete laboratory testing results are provided in Appendix C.

Table 5-1: Type and Number of Laboratory Tests Performed

Soil Laboratory Test	Material	Test Standard	No. Tests Completed
Grain Size Analysis (sieve only)	Soil	ASTM D 6913	4
Grain Size Analysis (sieve and hydrometer)	Soil	ASTM D 6913 ASTM D 7929	1
Moisture Content	Soil	ASTM D 2216	5

6.0 SUBSURFACE CONDITIONS

The soils WSP encountered in the borings generally include fill materials placed during construction of the existing culvert and roadway, naturally occurring moderately plastic silt with sand and gravel interpreted to be stream deposits, and naturally occurring sand and gravel interpreted to be glacial till. The boring logs in Appendix A provide detailed descriptions of the soil conditions encountered in the borings. WSP's Interpretive Subsurface Profile (ISP) along the centerline of the roadway is presented in Figure 3 (attached). The following descriptions summarize the major stratigraphic units from the existing ground surface to depth.

Table 6-1: Summary of Encountered Subsurface Soils and Fill Materials

Material	Approximate Thickness (feet)	Generalized Description	Classification ¹	N ₆₀ ²
Asphalt Pavement	0.3 to 0.4	N/A	N/A	N/A
Fill	7.0 to 9.8	Brown, fine to coarse sand, trace to some fine to coarse gravel, and trace to some silt.	A-1-a or A-1-b SM	18 (medium dense) to 77 (very dense) (denser at top)

Material	Approximate Thickness (feet)	Generalized Description	Classification ¹	N ₆₀ ²
Stream Deposits	2.2	Light brown, medium plasticity, silt, some fine to medium gravel, little fine to coarse sand.	A-4 ML	12 (stiff)
Glacial Till	6.2	Olive to brown, fine to coarse sand, some fine to coarse gravel, some silt.	A-4 SM	38 (dense)
Weathered Bedrock	3.3	N/A	N/A	160 (very dense)

Notes: ¹Classification based on AASHTO M 145 (A-1 through A-7) or ASTM D2487 for Unified Soil Classification System (USCS) or ASTM D2488 for Visual-Manual Classification. ²SPT N-value corrected for hammer efficiency.

Bedrock: Bedrock was encountered in borings BB-T8R3MB-101 and BB-T8R3MB-102 between 13.1 feet and 15.7 feet bgs, respectively. Refusal was encountered in BP-T8R3MB-103 and BP-T8R3MB-104 at depths of 19.0 and 20.0 feet, respectively, which we assumed to be top of bedrock. The bedrock elevation trend is a downward slope from southwest to northeast, with bedrock at elevation (EL) 575.3 feet in BB-T8R3MB-101 and EL 572.7 feet in BB-T8R3MB-102. The bedrock lithology generally consists of grey, fine grained, medium strong (R3) to strong (R4), fresh (W1) to moderately weathered (W3) slate from the Belle Lake Slate Formation. The Rock Quality Designation (RQD) was very poor (RQD of 0% to 22%) and estimated Rock Mass Rating (RMR) ranged from 31 to 50. Table 2 (attached) provides detailed information about recovery, RQD, RMR, and descriptions of lithology, rock mass, and discontinuities.

Groundwater: Groundwater levels were measured in each of the borings upon their completion and prior to removal of the casing. Measured groundwater levels ranged from 3.9 feet bgs (BB-T8R3MB-102) and 9.0 feet bgs (BB-T8R3MB-101) for all borings. Measured water levels ranged from EL 584.5 feet (BB-T8R3MB-102) and EL 579.4 feet (BB-T8R3MB-101). Per the Draft PS&E Plans⁶, the river level associated with ordinary high water and rust staining in the culvert is EL 579.1 feet (Headwater Elevation Q1.1) and the river level associated with design Q₅₀ is EL 580.7 feet. The interpreted groundwater levels shown on the ISP (Figure 3, attached) are based on these water level measurements and moisture contents (e.g., dry, moist, wet) observed in the soil samples collected from the borings.

7.0 GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

The proposed box culvert invert as specified in the Draft PS&E Plans⁶ will be 84 feet long including the wingwalls with a 20-foot span and the culvert invert will be between EL 573.7 feet and EL 574.3 feet. Assuming the base of the culvert is 16 inches thick, the culvert will be embedded into the bedrock and weathered bedrock layer. An additional 1 foot minimum of excavation below those elevations is required to accommodate Granular Borrow for Underwater Backfill and to facilitate fine grading and dewatering below the culvert. Installation of the proposed box culvert will require excavation of site soils at and adjacent to RT 169. The approximate maximum depth of

⁶ State of Maine Department of Transportation, T8 R3 NBPP Big Meadow Bridge over Meadow Brook, Springfield Road, Federal Aid Project No 2727000, Bridge No. 0064, Draft PS&E Plan Set, March 20, 2026.

excavation is 17 feet bgs at the location investigated. Figure 2 (attached) shows the culvert location in plan and Figure 3 (attached) shows the culvert location with respect to the interpreted stratigraphy.

7.1 Design Parameters

WSP evaluated the subsurface geotechnical data collected and developed geotechnical material design parameters to serve as a basis for calculations, analyses, and evaluations discussed herein. Table 7-1 provides a summary of design parameters for each in situ soil layer encountered (refer to Figure 3) and material properties for construction fills as identified in the MaineDOT Standard Specifications.⁷

Table 7-1: Engineering Design Parameters for In Situ Soils and Fills

Soil Layer		γ (pcf)	ϕ (°)	1K_0
In Situ Materials	Asphalt Pavement	140	N/A	N/A
	Existing Fill	125	35	0.43
	Stream Deposits	115	31	0.48
	Glacial Till	130	38	0.38
Construction Materials	Asphalt Pavement	140	N/A	N/A
	Granular Borrow (Section 703.19)	125	32	0.47
	Gravel Borrow (Section 703.20)	135	36	0.41
	Special Fill (Stream Bed) (Section 203.33)	N/A	N/A	N/A
	Streambed Rock	N/A	N/A	N/A
	Riprap (Section 703.26)	140	42	N/A

Note: ¹Lateral earth pressure coefficient at rest was determined using the equation $K_0 = 1 - \sin\phi$. The term N/A indicates these parameters are not used in design.

7.2 Frost Penetration Depth

WSP estimated an average frost penetration depth for roadway and culvert backfill using the MaineDOT Bridge Design Guide⁸. The site has a design freezing index of approximately 1,900 °F-degree days. For roadway culvert coarse-grained fills with an approximate average water content of 12.6%, we estimate a frost penetration depth of 7.4 feet at the site. Refer to calculations in Appendix D.

The proposed culvert base is below the estimated frost penetrations depth below the roadway. While the culvert is open to cold temperatures that might lead to frozen ground below the culvert, the height of the overlying embankment is expected to mitigate the effects of potential frost heave. However, the inlet and outlet portions of

⁷ State of Maine, Department of Transportation. March 2020. Standard Specifications. <https://www.maine.gov/dot/sites/maine.gov.dot/files/docs/contractors/publications/standardspec/docs/2020/2020%20Standard%20Specification.pdf>.

⁸ Maine Department of Transportation, Bridge Design Guide, 2003 updated 2018. <https://www.maine.gov/mdot/bdg/docs/BDGupdateJune2018.pdf>

the culvert are subject to little overburden and we recommend that the materials used in these areas below concrete features be non-frost susceptible.

7.3 Lateral Earth Pressure

For a non-yielding, stiff structure like the box culvert and wingwall system proposed for this project, we recommend the use of the at-rest (K_0) lateral earth pressure coefficient of 0.48 for the in situ stream deposits around the culvert. Calculations, references, and assumptions are provided in Appendix D. Per AASHTO LRFD⁹ Article 3.11.6.4, an applied live load was required in the calculations based on wall height of the culvert's inlet/outlet and distance of the wall back face to the edge of traffic. A minimum height of soil loading (H_{eq}) of 2 feet should be used when determining lateral earth pressure using the coefficients provided.

7.4 Bearing Resistance and Settlement

The profile for the culvert and embankment fills below the roadway in the Draft PS&E Plans⁶ shows the following:

- No substantial grade change at the roadway surface is planned that will increase embankment loading at the culvert location.
- The proposed culvert is taller than the existing culvert and will displace some of the in situ soils below the embankment, which will result in a net stress reduction in the underlying soils at the culvert.
- The culvert will be founded on a 12-inch thick layer of Granular Borrow for Underwater Backfill (MaineDOT Standard Specification⁷ Section 703.19) directly above the interpreted bedrock surface

By inspection, there will not be a net increase in effective stresses within the in situ soils at the proposed culvert that would exceed either a total or differential settlement tolerance limit. Thus, we did not perform a project-specific settlement evaluation.

By inspection, the bedrock and weathered bedrock will have adequate bearing resistance at the strength limit state due to its immediate proximity to the culvert, thus we did not evaluate bearing resistance. We propose to use the presumptive bearing resistance values for spread footings at the service limit state per AASHTO LRFD⁹. The bedrock is characterized as slate; however, the rock is highly fractured and moderately weathered. AASHTO LRFD⁹ Table C10.6.2.5.1-1 recommends a bearing resistance of 16 ksf to 24 ksf for weathered to broken bedrock of any kind. We propose the use of 16 ksf presumptive bearing resistance at the service limit state for the proposed culvert.

7.5 Subgrade Modulus

We recommend a modulus of subgrade reaction (K) of 109 pci (pounds per cubic inch) for use in the structural design of the proposed culvert based on a friction angle of 32 degrees for layer of Granular Borrow for Underwater Backfill based on its estimated value using an interpolated N_{60} value to relative density relationship.

7.6 Global Stability of Approach Embankments

The profile for the culvert and embankment fills below the roadway in the Draft PS&E plan set shows the following:

⁹ American Association of State Highway and Transportation Officials (2024). "AASHTO LRFD Bridge Design Specifications," 10th edition, Washington, DC

- No substantial grade change at the roadway surface that increases embankment loading within the in situ soils at and near the culvert location.
- No substantial embankment widening or slope changes along the project extents, where slopes appear to be the same maximum 2H:1V grade on the east and west embankments outside of the riprap armoring area, which has a design slope of 1.75H:1V.

Our proposed minimum factor of safety (FS) of 1.3 for global stability correlates to the AASHTO LRFD⁹ recommended earth slope global stability resistance factor of $\phi = 0.75$ for slopes with and without foundations when geotechnical parameters (e.g., stratigraphy and strength) are well defined.

The proposed 2H:1V slopes are comprised of embankment fills underlain by stream deposits, glacial till, and bedrock. We assume the strengths of the existing materials are adequate to support minor changes to the existing roadway slopes described above. We do not anticipate minimum global stability failure surfaces that extend into the in situ soils to have $FS < 1.3$. Shallow portions of the proposed 2H:1V slopes will be grubbed and infilled to the PDR design slopes with granular borrow that has a design unit weight of 125 pcf and a design friction angle of $\phi = 32^\circ$ provided in the MaineDOT Bridge Design Guide⁸. The 2H:1V embankment slope angle of 26.5° is less than the internal friction angle of the embankment fill material (32°); however, this combination of friction angle and slope angle typically results in calculated surficial $FS < 1.3$. Based on discussions with MaineDOT, we understand that slopes constructed with granular borrow using methods commensurate with MaineDOT Standard Specifications⁷ have historically performed adequately.

The proposed 1.75H:1V slopes nearest the proposed box culvert will be grubbed and armored with a 2-foot thick layer of rip rap that has a design unit weight of 140 pcf and a design friction angle of $\phi = 42^\circ$ (see Table 7-1). The 1.75H:1V embankment slope angle is 30° which is less than the internal friction angle of the riprap material (42°); however, the ratio of riprap internal friction angle to embankment slope angle is ~ 1.4 , similar to the factor of safety. We understand that 1.75H:1V slopes constructed with riprap using methods commensurate with MaineDOT Standard Specifications⁷ have historically performed adequately and we anticipate an adequate factor of safety against surficial failures can be met for this scenario.

7.7 Scour and Riprap

The inlet of the culvert will be armored with void-filled riprap in accordance with MaineDOT Standard Details¹⁰ and Standard Specifications.⁷ The inlet should include a 2-foot thick by 10-foot long Plain Riprap Apron (Detail 610(04)) comprised of Plain Riprap (Section 703.26) capped with a 2-foot-thick minimum layer of Special Fill (Item No. 203.33, which will be specified in a Special Provision later in the project). The riprap slope will be underlain by a Class 1 non-woven erosion control geotextile meeting the requirements of Section 722.03. The Draft PS&E Plans⁶ design provides the recommended extents of scour and riprap protection of Meadow Brook embankments.

Based on the approximate average elevation of the stream channel (EL 576 feet), site visits where exposed bedrock was identified near the existing culvert outlet, and the range of bedrock elevations from the borings and probes performed (EL 572.7 feet to EL 586.6 feet), the channel near the outlet of the culvert is expected to be comprised of bedrock exposed at the surface or shallow streambed sediments over bedrock. We do not recommend excavating bedrock to place void-filled rip rap at the outlet.

¹⁰ Maine Department of Transportation Standard Details, March 2020, https://www.maine.gov/dot/sites/maine.gov.dot/files/inline-files/Standard%20Detail%202020%204_2020.pdf

8.0 GEOTECHNICAL CONSTRUCTION RECOMMENDATIONS

Excavation Support and Dewatering

At the writing of this report, the sequence of the work is unknown. Construction activities will require soil excavation to below both the stream water level and water table encountered during the geotechnical site investigation. Construction will require cofferdams and earth support systems to support culvert and stream bed excavations and to control water flow. Design of all temporary earth support systems should comply with Section 511 Cofferdams of the MaineDOT Standard Specifications⁷ and OSHA requirements. Temporary dewatering of the stream and excavation may be required and should be performed to limit disturbance to the culvert bearing surfaces. Dewatering activities should comply with Section 656 Temporary Soil Erosion and Water Pollution Control of the MaineDOT Standard Specifications.⁷ The contractor should additionally control groundwater and surface water infiltration so that construction can be completed under dry conditions.

Subgrade and Embankments

All areas proposed for embankment fill placement should be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil. During the grubbing and stripping process, unsuitable materials exposed at the subgrade level, such as wood, logs, tree stumps, forest mat materials, peat, debris fill, or other materials that may decay or collapse should be removed. Prior to placing new fill materials, the existing slopes should be benched to provide a stable surface for compaction and ensure new fills are keyed into the existing embankment. Bench widths should be at least 3 feet, and each bench should be excavated as the layers of the new embankment are being constructed.

Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Specifications⁶ Section 203.09 for preparation of the embankment area which requires a firm, unyielding surface. Embankment fill should meet the requirements of MaineDOT Standard Specifications⁷ Section 703.19 for Granular Borrow. Embankment excavation and fill placement should follow MaineDOT Specifications⁷ Section 203.13 for embankment construction and should meet the requirements described in this report and compacted to at least 90 percent of the maximum dry density as determined by AASHTO T 180. Structural fill materials and placement methods for abutment construction should meet the requirements of MaineDOT Standard Specifications⁷, specifically Section 203.10 through Section 203.12, and 203.17.

Prior to placing the box culvert and wingwalls, we recommend undercutting the foundation 12 inches and placing 12 inches of Granular Borrow for Underwater Backfill (Section 534.02) to facilitate fine grading and dewatering for the installation of the culvert.

If wet subgrade conditions are encountered in embankment fill areas, wet and disturbed subgrade material should be excavated and replaced with an appropriate size stone and covered with a geotextile (both per MaineDOT Standard Specifications⁷) to allow proper compaction of overlying fills. If seepage persists over a broad subgrade area, provisions should be made to allow for positive drainage beneath and within the new embankment fills. Positive drainage could be provided by a column of crushed stone wrapped in filter fabric that daylight beyond the new toe of slope.

Excavation of both intact and weathered rock is anticipated and should follow MaineDOT Bridge Design Guide¹⁴ Section 5.3.4.2, and MaineDOT Standard Specifications⁷ Section 203 Excavation and Embankment. When structures are to be placed on solid bedrock, the rock should be excavated to a firm surface, either level, stepped or serrated. When solid or disintegrated rock or boulders are encountered, the rock should be excavated to a

designated depth below the bottom of the proposed structures, as shown in the Contract Documents or as directed by the Resident.

Temporary Detour

The Draft PS&E Plans⁶ recommend an on-site temporary detour bridge. We assume the selected Contractor will be responsible for temporary embankment and/or temporary culvert geotechnical stability or bearing resistance during temporary operations per the MaineDOT Standard Specifications⁷ and Bridge Design Guide⁸.

9.0 REPORT AND EXPLORATION LIMITATIONS

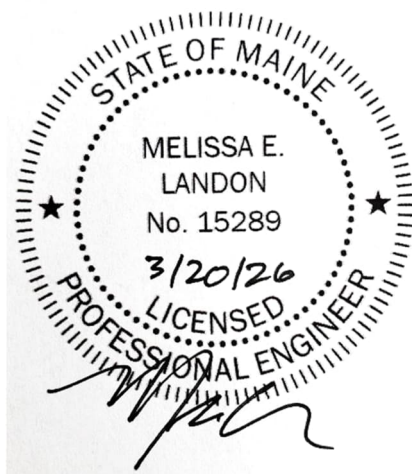
This Geotechnical Design Report (GDR) was prepared for the use of MaineDOT for the specific application to the proposed replacement of the Meadow Brook Bridge culvert that passes RT 169 over Meadow Brook in T8 R3 NBPP, Maine. We conducted our evaluations and compiled our recommendations in accordance with generally accepted soil and foundation engineering practices in this geographical area and under similar time and financial constraints. WSP makes no other warranty, either express or implied. If changes in the nature, design, or location of the proposed project are planned, WSP should be notified to review the appropriateness of our conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design.

Our analyses and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Variations in the nature and extent of subsurface conditions between explorations should be expected. WSP should be notified if conditions encountered during construction vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report.

The professional services provided by WSP for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

Signature Page

WSP USA, Inc.



Melissa E. Landon, PhD, PE
Assistant Vice President, Geotechnical Engineering

A handwritten signature in blue ink, appearing to read "J. Lloyd", written in a cursive style.

Jeffrey D. Lloyd, PE
Vice President, Geotechnical Engineering

MEL/CCB/JDL

[https://wspnlinenam.sharepoint.com/sites/us-mainedot-northern/shared documents/0064-big meadow bridge - 27270_00/technical/design/geotechnical/09 reports/02 gdr/win 027070.00 bridge 0064 gdr .docx](https://wspnlinenam.sharepoint.com/sites/us-mainedot-northern/shared%20documents/0064-big%20meadow%20bridge%20-%2027270_00/technical/design/geotechnical/09%20reports/02%20gdr/win%20027070.00%20bridge%200064%20gdr%20.docx)

Tables

Table 1: Subsurface Exploration Locations
Geotechnical Design Report
Bridge #0064 at Meadow Brook, T8 R3 NBPP, Maine
MaineDOT WIN 027270.00

Boring Designation ^{1,2,3}	As-Drilled Locations ⁴		Existing Ground Surface Elevation ⁴	Bedrock Depth	Maximum Depth of Investigation
	Northing	Easting	(feet)	(feet bgs)	(feet bgs ⁵)
BB-T8R3MB-101	623577.69	2261835.30	588.4	13.1	26.3
BB-T8R3MB-102	623608.83	2261871.51	588.4	15.7	32.8
BP-T8R3MB-103 ⁶	623624.71	2261859.96	588.6	13.5	20.0
BP-T8R3MB-104 ⁶	623561.03	2261845.92	588.3	13.5	19.0

Notes:

1. Boring and rock probe locations illustrated on the Boring Location Plan in the Preliminary Geotechnical Design Report.
2. Borings were performed by Seaboard Drilling LLC in March 2025.
3. Boring and rock probe logs are presented in Appendix A of the Preliminary Geotechnical Design Report.
4. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" dated 5/6/25.
5. bgs = below ground surface.
6. Probes (BP) were advanced without collection of soil samples or rock core.

Prepared By: RJN

Checked By: LDN

Reviewed By: MEL

Table 2: Summary of Rock Core Quality
Geotechnical Design Report
Bridge #0064 at Meadow Brook, T8 R3 NBPP, Maine
MaineDOT WIN 027270.00

Test Boring Designation	Core Size (in)	Run						TCR ¹		RQD ²		Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description ^{5,6}	
		No.	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation	Weathering ³	Estimated Field Strength ³		Rock Mass Rating [RMR] ⁴
				Start	End	Midpoint										
BB-T8R3MB-101	NQ2 (2)	R1	3.4	14.0	19.0	16.5	5.0	4.4	88%	0.00	0%	Very Poor	Moderately Weathered (W3)	Medium Strong (R3)	31	Grey, fine grained, SLATE, medium strong, moderately weathered; discontinuities smooth, highly fractured, fine sand infilling, broken rock core.
		R2	7.9	19.0	23.0	21.0	4.0	3.5	88%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	33	Grey, fine grained, SLATE, medium strong, slightly weathered; discontinuities smooth, highly fractured, fine sand infilling, broken rock core.
		R3	11.6	23.0	26.3	24.7	3.3	3.3	100%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	43	Grey, fine grained, SLATE, strong, fresh; discontinuities vertical to low angle dipping, close to very closely spaced, tight, no infilling, thin laminated vertical and angled sheets, smooth, planar; average 1.52 fractures per foot.
BB-T8R3MB-102	NQ2 (2)	R1	2.3	17.0	19.0	18.0	2.0	0.0	0%	0.00	0%	Very Poor	-	-	-	No recovery.
		R2	5.3	19.0	23.0	21.0	4.0	3.0	75%	0.00	0%	Very Poor	Slightly Weathered (W2)	Medium Strong (R3)	37	Grey, fine grained, SLATE, medium strong, slightly weathered; highly fractured, rough, fine sand infilling, broken rock core.
		R3	9.7	23.0	27.8	25.4	4.8	2.9	60%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	44	Grey, fine grained, SLATE, strong, slightly weathered; discontinuities very rough, highly fractured, fine sand infilling, broken rock core.
		R4	14.6	27.8	32.8	30.3	5.0	4.8	96%	1.10	22%	Very Poor	Fresh (W1)	Strong (R4)	50	Grey, fine grained, SLATE, strong, fresh; discontinuities vertical to horizontal dipping, close to very closely spaced, tight to open, no infilling, thin laminated vertical and angled sheets, rough to very rough, planar; average 2.6 fractures per foot.

Notes:

1. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
2. RQD = rock quality designation. RQD is the total length of intact, full diameter core pieces recovered with a length greater than or equal to 4 inches measured along the core axis. The percent RQD is the total length of RQD measured divided by the run length. Note that vertical discontinuities are not included in determination of RQD.
3. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Wyllie and Mah, 2004, Rock Slope Engineering: Civil and Mining, 4th Edition (based on ISRM, 1981).
4. Rock Mass Rating (RMR) System (Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, groundwater conditions, and orientation of discontinuities. These ratings are summed to provide the RMR value. The rating adjustment for joint orientation was assigned a value of 0; correlation of geologic field mapping data of exposed rock outcrops with the rock core samples and proposed foundation type may allow for a different rating adjustment for joint orientation, and thus a modification to the RMR value shown on this table.
5. Bedrock formation is labelled as "unnamed" sandstone, metasiltstone, or slate at site location.
6. ft = feet, in = inches
- Prepared by: LMP/LDN
Checked by: RJN
Reviewed by: MEL

Table 3: Summary of Soil Index and Classification Laboratory Testing Results
Geotechnical Design Report
Bridge #0064 at Meadow Brook, T8 R3 NBPP, Maine
MaineDOT WIN 027270.00

Test Boring Designation ¹	Ground Surface Elevation ² (feet)	Sample Number	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet) ²	Laboratory Testing ³		Soil Classification	
					Sieve Minus No. 200 ⁴ (%)	Moisture Content ⁵ (%)	AASHTO ⁶	USCS ⁷
BB-T8R3MB-101	588.4	2D	3.0 - 5.0	585.4 - 583.4	14.7	6.9	A-1-a (0)	<i>SM</i>
		3D	5.0 - 7.0	583.4 - 581.4	16.7	5.9	A-1-b (0)	<i>SM</i>
BB-T8R3MB-102	588.4	3D	5.0 - 7.0	583.4 - 581.4	19.7	5.2	A-1-b (0)	<i>SM</i>
		4D	7.0 - 9.0	581.4 - 579.4	59.9	32.5	A-4 (0)	<i>ML</i>
		5D	10.0 - 12.0	578.4 - 576.4	35.1	9.2	A-4 (0)	<i>SM</i>

Notes:

- Boring locations are illustrated in Figure 2 - Boring Location Plan of the Preliminary Geotechnical Design Report.
- As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" dated 5/6/2025.
- Laboratory testing was performed by Geotesting Express of Acton, MA.
- Particle size testing was performed in accordance with ASTM D6913, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
- Moisture content testing was performed in accordance with ASTM D2216, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.
- AASHTO classification is based on AASHTO M145-91 and includes both Group and Subgroup classification and Group Index (in parentheses) based on the percent passing the #200 sieve and Atterberg Limits. Classification provided by GeoTesting Express.
- USCS classification was performed by WSP based on interpreted laboratory test results. Items in italics use assumed fines type based on field characterization.
- Complete laboratory test results for soil testing are provided in Appendix B of the Preliminary Geotechnical Design Report.

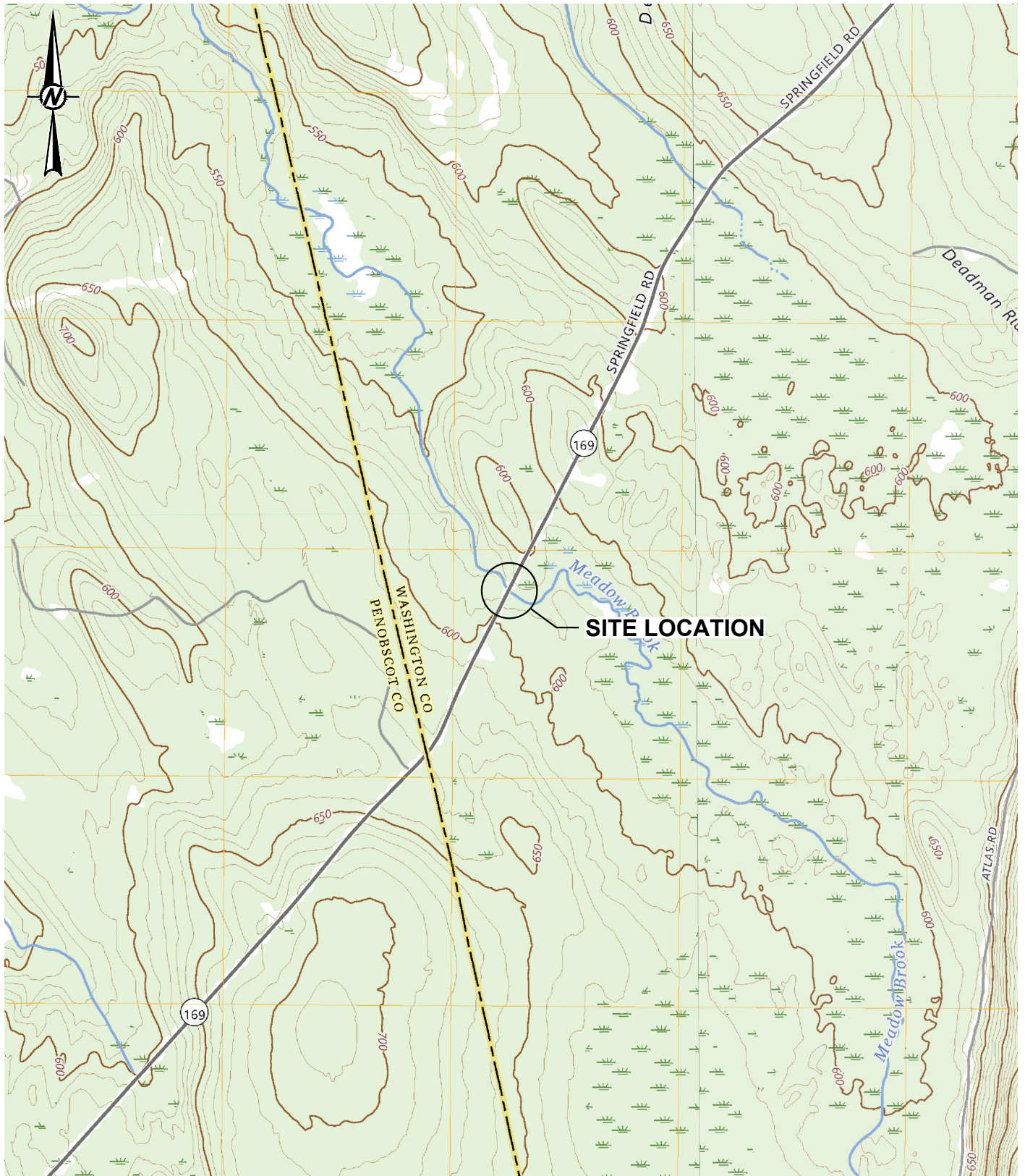
Prepared By: RJN

Checked By: LDN

Reviewed By: MEL

Figures

Last Edited By: usg700668 Date: 2025-06-17 Time: 3:13:59 PM | Printed By: USRG700668 Date: 2025-06-17 Time: 3:14:15 PM
Path: \\corp.pbowen.net\USCentralData\USMT100\CADD\Projects\MaineDOT\NBB_Bridge099_PROJECT\SU0040947.9162\0002_0002.dwg | File Name: US0040947.9162-0002-001.dwg



REFERENCE(S)

1. BASE MAP TAKEN FROM U.S.G.S. 7.5-MINUTE STETSON MOUNTAIN AND POTTER HILL QUADRANGLES OF MAINE.

CLIENT

MAINE DEPARTMENT OF TRANSPORTATION

PROJECT

NORTH BRIDGE BUNDLE (NBB), BRIDGE #0064

CONSULTANT



YYYY-MM-DD 2025-06-17

DESIGNED LDN

PREPARED RG

REVIEWED MEL

APPROVED

TITLE

SITE LOCATION MAP

T8 R3 NBPP BIG MEADOW BRIDGE #0064 REPLACEMENT

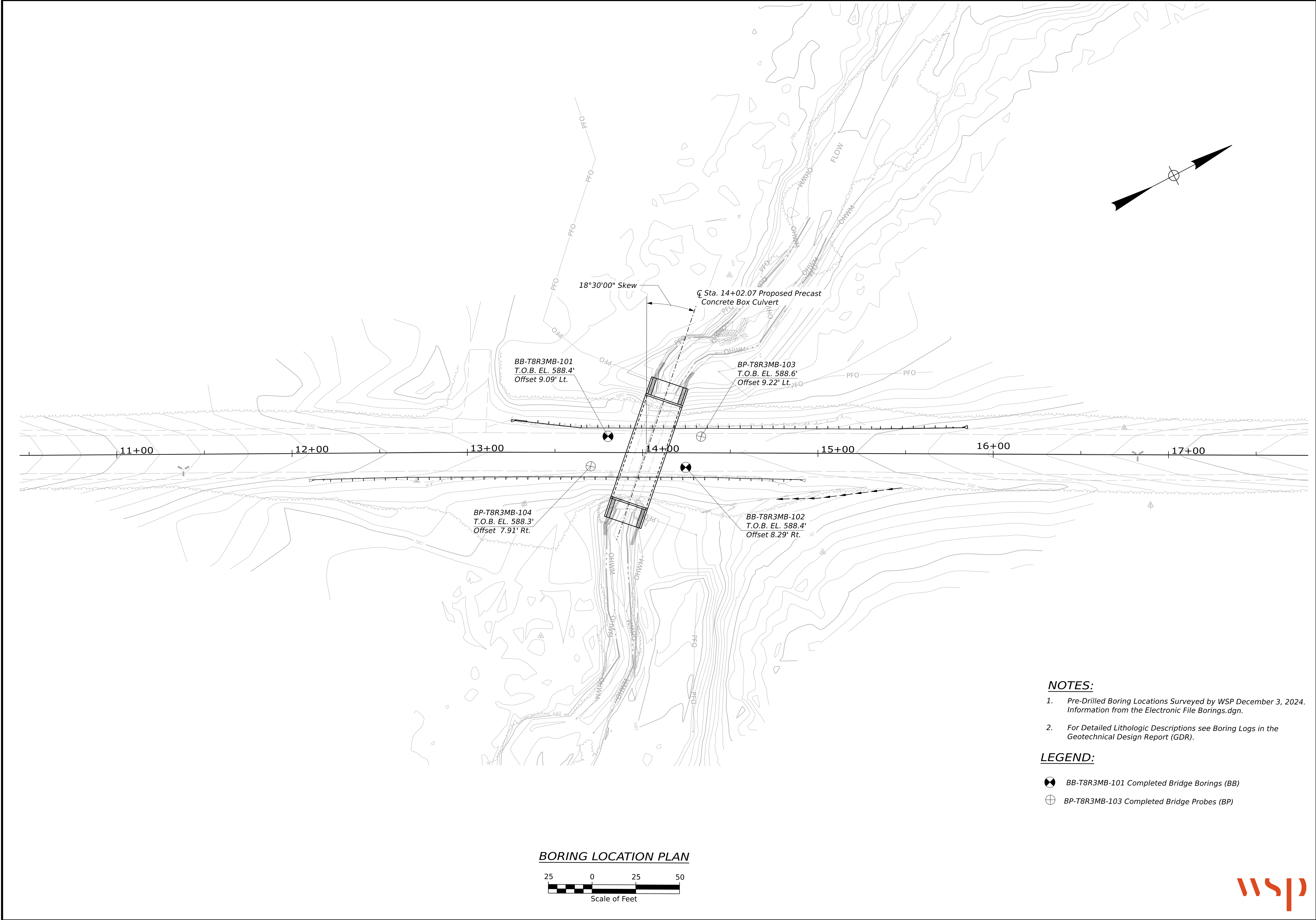
MAINEDOT WIN: 027270.00

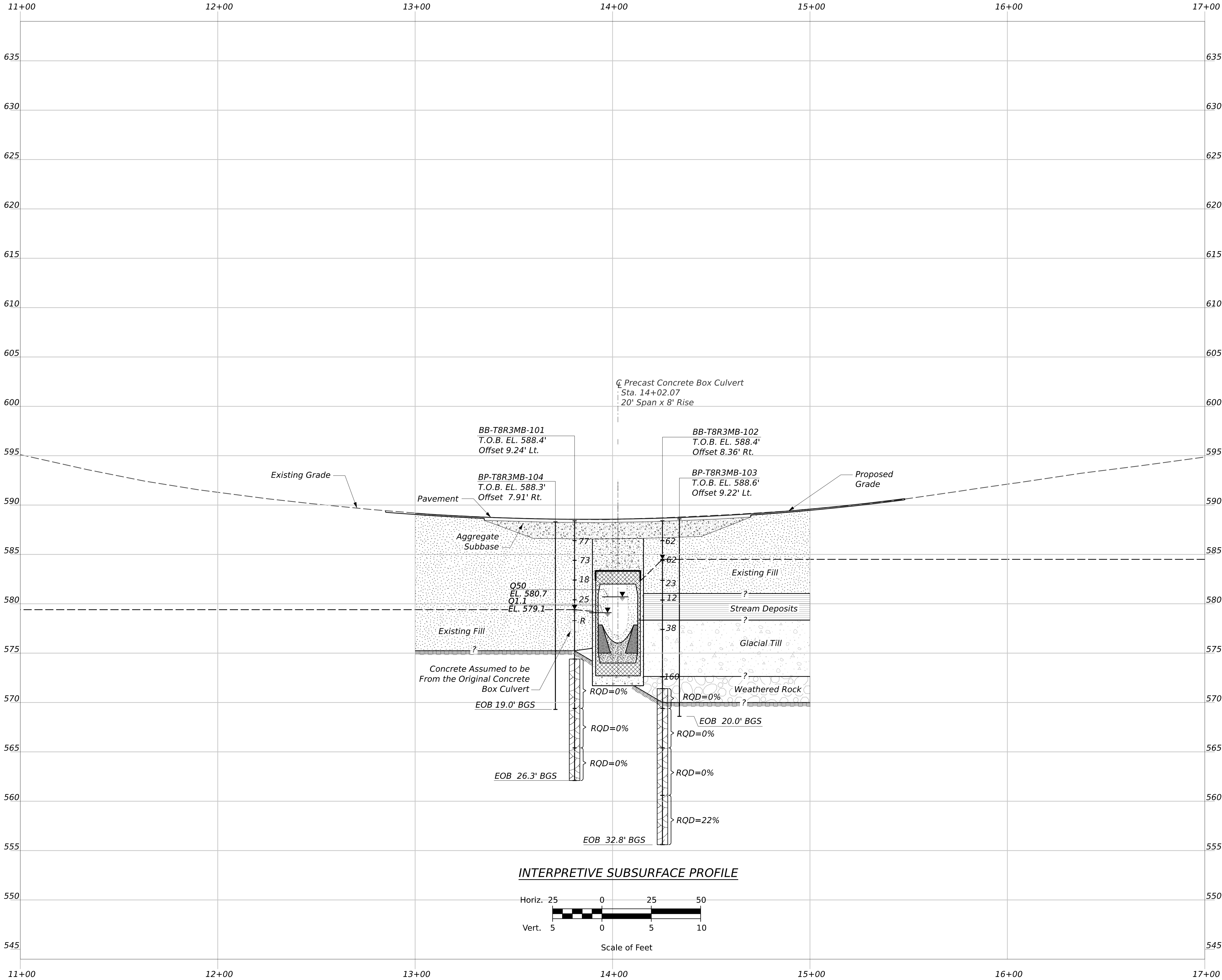
PROJECT NO. CONTROL
US0040947.9162 0002-001

REV.
0

FIGURE
1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A





LEGEND

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring Probe
SPT: N60 - Value
(Corrected for Hammer Efficiency)
R Refusal
Strata Interface
Run 1 = 92%
Core Run Number and Rock Quality Designation (RQD)
EOB End of Boring Depth Below Ground Surface (BGS)

LEGEND

Brown, Medium Dense to Dense, Fine to Coarse Sand, Fine to Coarse Gravel, Trace to Some Silt (Fill)

Dark Yellow, Soft Moderately Plastic Silt with Some Fine to Medium Gravel and Little Coarse to Fine Sand (Stream Deposits)

Olive Brown, Medium Dense Coarse to Fine Sand, Some Coarse to Fine Gravel, Some Silt (Glacial Till)

Weathered Bedrock

Grey, Fine Grained Slate (Bedrock)

Existing Ground Surface

Existing Ground Water Surface

Design Water Level

Existing Bridge Structure

Proposed Bridge Substructure and Superstructure

- NOTES:**
- Pre-drilled boring locations surveyed by WSP December 3, 2024. Information from the electronic file Borings.dgn.
 - For detailed lithologic descriptions see boring logs in the Geotechnical Design Report (GDR).
 - For complete laboratory data, see laboratory results in the Geotechnical Design Report (GDR).
 - Groundwater surface is interpreted from localized water levels and measurements taken during the subsurface exploration program, see the Geotechnical Design Report (GDR) and Boring Logs.
 - This generalized subsurface profile is intended to convey trends in the subsurface conditions. The boundaries between strata are approximate and idealized and have been developed based on interpretations of widely spaced explorations. Actual soil and rock transitions may vary and may be erratic.

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION		Federal Project No. 2727000		WIN 27270.00	
BIG MEADOW BRIDGE NO. 0064		CROSSING MEADOW BROOK		T8 R3 NBPP		INTERPRETIVE SUBSURFACE PROFILE	
SHEET NUMBER		3		OF 3			

PROJ. MANAGER	DATE	BY	SIGNATURE	P.E. NUMBER	DATE
DESIGN-DETAILED	7/2025	D. HENDERSON			
CHECKED-REVIEWED	7/2025	M. LADDON			
DESIGN-DETAILED2					
DESIGN-DETAILED3					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER SYSTEM																																																						
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES																																																							
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	<u>Descriptive Term</u>		<u>Portion of Total (%)</u>																																																				
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace		0 - 10																																																				
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	little		11 - 20																																																				
			GC	Clayey gravels, gravel-sand-clay mixtures.	some		21 - 35																																																				
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, Gravelly sands, little or no fines	adjective (e.g. Sandy, Clayey)		36 - 50																																																				
		(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.																																																							
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures	TERMS DESCRIBING DENSITY/CONSISTENCY <u>Coarse-grained soils</u> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value). <table><tr><th><u>Density of Cohesionless Soils</u></th><th><u>Standard Penetration Resistance</u> N₆₀-Value (blows per foot)</th></tr><tr><td>Very loose</td><td>0 - 4</td></tr><tr><td>Loose</td><td>5 - 10</td></tr><tr><td>Medium Dense</td><td>11 - 30</td></tr><tr><td>Dense</td><td>31 - 50</td></tr><tr><td>Very Dense</td><td>> 50</td></tr></table> <u>Fine-grained soils</u> (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated. <table><tr><th><u>Consistency of Cohesive soils</u></th><th><u>SPT N₆₀-Value (blows per foot)</u></th><th><u>Approximate Undrained Shear Strength (psf)</u></th><th><u>Field Guidelines</u></th></tr><tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr><tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr><tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr><tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr><tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr><tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr></table> <u>Rock Quality Designation (RQD):</u> RQD (%) = <u>sum of the lengths of intact pieces of core* > 4 inches</u> length of core advance *Minimum NQ rock core (1.88 in. OD of core) Rock Quality Based on RQD <table><tr><th><u>Rock Quality</u></th><th><u>RQD (%)</u></th></tr><tr><td>Very Poor</td><td>≤25</td></tr><tr><td>Poor</td><td>26 - 50</td></tr><tr><td>Fair</td><td>51 - 75</td></tr><tr><td>Good</td><td>76 - 90</td></tr><tr><td>Excellent</td><td>91 - 100</td></tr></table> <u>Desired Rock Observations (in this order, if applicable):</u> Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))				<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance</u> N ₆₀ -Value (blows per foot)	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50	<u>Consistency of Cohesive soils</u>	<u>SPT N₆₀-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	Hard	>30	over 4000	Indented by thumbnail with difficulty	<u>Rock Quality</u>	<u>RQD (%)</u>	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100
		<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance</u> N ₆₀ -Value (blows per foot)																																																								
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Fair	51 - 75																																																										
Good	76 - 90																																																										
Excellent	91 - 100																																																										
SILTS AND CLAYS (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.																																																									
	CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.																																																									
	OL	Organic silts and organic Silty clays of low plasticity.																																																									
SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.																																																									
	CH	Inorganic clays of high plasticity, fat clays.																																																									
	OH	Organic clays of medium to high plasticity, organic silts.																																																									
	HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.																																																								
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level					Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth																																																						
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information																																																											

Table A-2**Classification of Rock Material Strengths¹**

Grade	Description	Field Identification	Approx. Range of Uniaxial Compressive Strength	
			MPa	psi
S1	Very soft clay	Easily penetrated several inches by fist	<0.025	<4
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05	4-7
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.10	7-15
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10-0.25	15-35
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.50	35-70
S6	Hard clay	Indented with difficulty by thumbnail	>0.50	>70
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0	35-150
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife	1-5	150-725
R2	Weak rock	Can be peeled by a pocket knife with difficulty; shallow indentations made by firm blow with point of geological hammer	5-25	725-3,500
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer	25-50	3,500-7,000
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100	7,000-15,000
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250	15,000-36,000
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250	>36,000

Note: Grades S1 to S6 apply to cohesive soils, for example clays, silty clays, and combinations of silts and clays with sand, generally slow draining. Discontinuity wall strength will generally be characterized by grades R0-R6 (rock) while S1-S6 (clay) will generally apply to filled discontinuities. Rock material strength descriptions are included in the rock core descriptions in the boring logs. Rock material strength grades (R0-R6) are not included in the rock core descriptions to avoid confusion with the numbering of the rock core runs.

¹ International Society for Rock Mechanics (ISRM), Commission on standardization of laboratory and field tests (1978): Suggested methods for the quantitative description of discontinuities in rock masses. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., Vol. 15, No. 6, pp. 319-368.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: North Bridge Bundle (NBB), Bridge #0064 Location: T8-R3, ME				Boring No.: BB-T8R3MB-101 WIN: 027270.00																																																																																																																																																																																																															
Driller: Seaboard Drilling, LLC				Elevation (ft.): 588.39				Auger ID/OD: 4 in																																																																																																																																																																																																															
Operator: Eric Baron				Datum: NAVD88				Sampler: 2 in Standard Split Spoon																																																																																																																																																																																																															
Logged By: Ryan Nagle				Rig Type: Diedrich D-50 (SN:367)				Hammer Wt./Fall: 140 lbs / 30 in																																																																																																																																																																																																															
Date Start/Finish: 8:43 - 11:40 3/20/25				Drilling Method: SSA / Cased Washed				Core Barrel: NQ2																																																																																																																																																																																																															
Boring Location: N:623577.69 E:2261835.3				Casing ID/OD: 4.0/4.5 in and 3.0/3.5 in				Water Level*: 9.0 ft at 9:00 on 3/17/25																																																																																																																																																																																																															
Hammer Efficiency Factor: 1.0				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																			
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_u(lab) = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																																																							
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Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: T8-R3, ME

WIN: 027270.00

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

Remarks: 1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024. 2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25. 3. Water level reading taken on 3/18/25 at 14:18 was before the start of rock coring with bottom of casing at 16.3 ft bgs.
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Page 1 of 2

Boring No.: BB-T8R3MB-102

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: North Bridge Bundle (NBB), Bridge #0064 Location: T8-R3, ME				Boring No.: BB-T8R3MB-102 WIN: 027270.00							
Driller: Seaboard Drilling, LLC				Elevation (ft.): 588.42				Auger ID/OD: 4 in							
Operator: Eric Baron				Datum: NAVD88				Sampler: 2 in Standard Split Spoon							
Logged By: Ryan Nagle				Rig Type: Diedrich D-50 (SN:367)				Hammer Wt./Fall: 140 lbs / 30 in							
Date Start/Finish: 12:52 - 4:00 3/18/25				Drilling Method: SSA / Cased Washed				Core Barrel: NQ2							
Boring Location: N:623607.83 E:2261871.51				Casing ID/OD: 4.0/4.5 in and 3.0/3.5 in				Water Level*: 3.9 ft at 2:18 on 3/18/25							
Hammer Efficiency Factor: 1.0				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected				T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)							
25										R3 (23.0' - 27.8'): Grey, fine grained, SLATE, strong, slightly weathered; discontinuities very rough, highly fractured, fine sand infilling, broken rock core. Rock Mass Quality = very poor 60% Recovery Rock Core Rate (min:sec) 23.0-24.0 ft (2:04) 24.0-25.0 ft (2:14) 25.0-26.0 ft (5:42) 26.0-27.0 ft (4:07) 27.0-27.8 ft (5:20) R4 (27.8' - 32.8'): Grey, fine grained, SLATE, strong, fresh; discontinuities vertical to horizontal dipping, close to very closely spaced, tight to open, no infilling, rough to very rough, planar, 2.6 fractures per foot. Rock Mass Quality = very poor 96% Recovery Rock Core Rate (min:sec) 27.8-28.8 ft (2:30) 28.8-29.8 ft (2:21) 29.8-30.8 ft (2:25) 30.8-31.8 ft (2:31) 31.8-32.8 ft (2:51) Bottom of Exploration at 32.8 feet below ground surface. Boring backfilled with bentonite chips in the rock core socket, gravel to bottom of pavement, and patched with cold patch asphalt.					
	R4	60/57.6	27.80 - 32.80	RQD = 22%											
50															
Remarks: 1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024. 2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25. 3. Water level reading taken on 3/18/25 at 14:18 was before the start of rock coring with bottom of casing at 16.3 ft bgs.															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 2					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-T8R3MB-102					

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: North Bridge Bundle (NBB), Bridge #0064 Location: T8-R3, ME				Boring No.: BP-T8R3MB-103 WIN: 027270.00					
Driller: Seaboard Drilling, LLC				Elevation (ft.): 588.60				Auger ID/OD: 4 in					
Operator: Eric Baron				Datum: NAVD88				Sampler: N/A					
Logged By: Ryan Nagle				Rig Type: Diedrich D-50 (SN:367)				Hammer Wt./Fall: N/A					
Date Start/Finish: 12:10 - 12:23 3/18/25				Drilling Method: SSA				Core Barrel: N/A					
Boring Location: N:623624.71 E:2261859.96				Casing ID/OD: N/A				Water Level*: N/A					
Hammer Efficiency Factor: 1.0				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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test													
Depth (ft.)	Sample Information									Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows						
0								SSA			Augered without sampling to 20 ft bgs.		
5											Driller Notes: It is gravelly to approximately 4 ft bgs due to rig behavior.		
10											Driller Notes: A significant reduction in drilling resistance between 7.0 ft and 13.5 ft bgs.		
15											Driller notes: Drilling becomes stiffer starting at 13.5 ft bgs.		
20									568.6		Driller notes harder rock encountered at 14.8 ft and continued to 20 ft bgs.		
25											Bottom of Exploration at 20.0 feet below ground surface. Boring backfilled with gravel to bottom of pavement, and patched with cold patch asphalt.		
Remarks: 1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024. 2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25.													
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.											Page 1 of 1		
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											Boring No.: BP-T8R3MB-103		

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Location: T8-R3, ME

WIN: 027270.00

Auger ID/OD:	4 in
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Sampler:	N/A
-----------------	-----

Hammer Wt./Fall: N/A

Core Barrel:	N/A
---------------------	-----

Water Level*:	N/A
----------------------	-----

Hammer Type: Automatic ☐ Hydraulic ☐ Rope & Cathead ☐

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

[illegible]

Remarks:

1. Hammer Efficiency factor provided by Seaboard and taken from "2024PA00175-Seaboard-SPT Energy Calibration" by GRL Engineers Inc., dated 10/23/2024.
2. As-drilled boring locations and ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" on 5/6/25.

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

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

Page 1 of 1

Boring No.: BP-T8R3MB-104

APPENDIX B

Rock Core Photographs

APPENDIX B
ROCK CORE PHOTOGRAPHS
BRIDGE #0064 AT MEADOW BROOK, T8 R3 NBPP, MAINE
MAINEDOT WIN 027270.00

Boring	Run	Depth Below Surface		Recovery			RQD			Rock Type	Box Row	Date Cored
		Feet		Feet		%	Feet		%			
BB-T8R3MB-101	R1	14.0	- 19.0	4.4	- 5.0	88.0	0.0	- 5.0	0	SLATE	Row 1	3/20/2025
	R2	19.0	- 23.0	3.5	- 4.0	87.5	0.0	- 4.0	0	SLATE	Row 2	3/20/2025
	R3	23.0	- 26.3	3.3	- 3.3	100.0	0.00	- 3.3	0	SLATE	Row 3	3/20/2025



Notes:

1. "Box row" indicates the section of the box where the core is contained: 1 = top, 4 = bottom.
2. Top of each core run is on the left and increases with depth to the right.
3. Data for Rows 4, and part of 3 is not provided as they are samples from Bridge #3189.

APPENDIX B
ROCK CORE PHOTOGRAPHS
BRIDGE #0064 AT MEADOW BROOK, T8 R3 NBPP, MAINE
MAINEDOT WIN 027270.00

Boring	Run	Depth Below Surface		Recovery		RQD		Rock Type	Box Row	Date Cored
		Feet		Feet		%	Feet	%		
BB-T8R3MB-102	R1	17.0	- 19.0	0.0	- 2.0	0.0	0.0 - 2.0	0	-	3/18/2025
	R2	19.0	- 23.0	3.0	- 4.0	75.0	0.0 - 4.0	0	Row 1	3/18/2025
	R3	23.0	- 27.8	2.9	- 4.8	60.4	0.0 - 4.8	0	Row 2	3/18/2025
	R4	27.8	- 32.8	4.8	- 5.0	96.0	1.1 - 5.0	22	Row 3	3/18/2025



- Notes:
1. "Box row" indicates the section of the box where the core is contained: 1 = top, 4 = bottom.
 2. Top of each core run is on the left and increases with depth to the right.

Prepared By: RJN/LDN
 Checked By: LMP
 Reviewed By: MEL

APPENDIX C

Laboratory Test Results

Client:	WSP USA, Inc.	Project No:	GTX-320898
Project:	MEDOT N. Bridge Bundle		
Location:	ME		
Boring ID:	---	Sample Type:	---
Sample ID:	---	Test Date:	04/23/25
Depth :	---	Test Id:	812353
		Tested By:	ajl
		Checked By:	ank

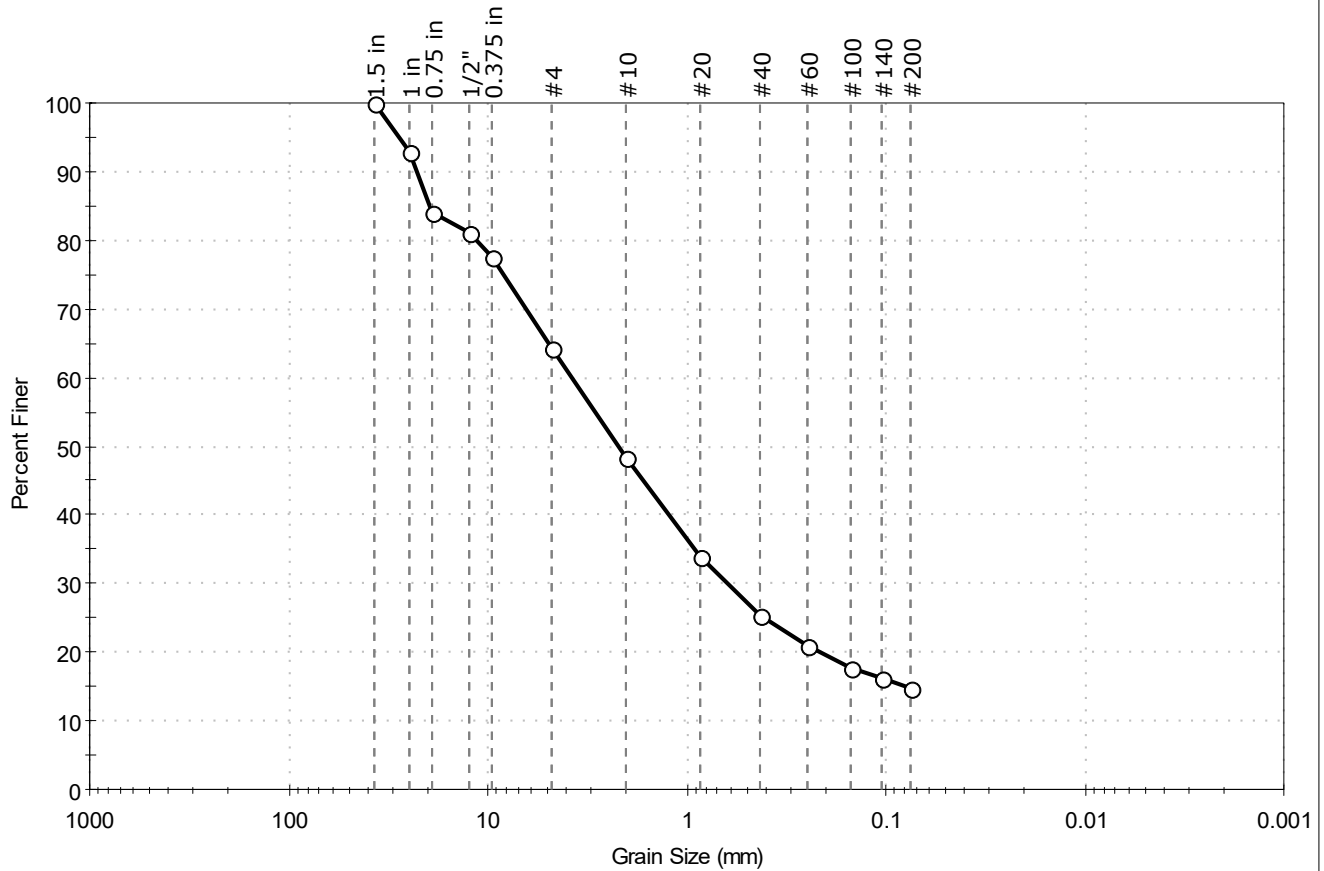
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-T8R3MB-101	S- 3(3D)	5.0-7.0 ft	Moist, brown silty sand with gravel	5.9
BB-T8R3MB-102	S- 3(3D)	5.0-7.0 ft	Moist, brown silty sand with gravel	5.2
BB-T8R3MB-102	S- 4(4D)	7.0-9.0 ft	Moist, dark yellow silt with sand and gravel	32.5
BB-T8R3MB-102	S- 5(5D)	10.0-12.0 ft	Moist, olive brown silty sand with gravel	9.2
BB-T8R3MB-101	S- 2(2D)	3.0-5.0 ft	Moist, olive brown silty sand with gravel	6.9

Notes: Temperature of Drying : 110° Celsius

Client: WSP USA, Inc.	Project No: GTX-320898
Project: MEDOT N. Bridge Bundle	
Location: ME	
Boring ID: BB-T8R3MB-101	Sample Type: Jar
Sample ID: S-2(2D)	Test Date: 04/26/25
Depth: 3.0-5.0 ft	Test Id: 812307
Test Comment: ---	Tested By: ajl
Visual Description: Moist, olive brown silty sand with gravel	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	35.8	49.5	14.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	93		
0.75 in	19.00	84		
1/2"	12.50	81		
0.375 in	9.50	77		
#4	4.75	64		
#10	2.00	48		
#20	0.85	34		
#40	0.42	26		
#60	0.25	21		
#100	0.15	18		
#140	0.11	16		
#200	0.075	15		

Coefficients

$D_{85} = 19.6076 \text{ mm}$ $D_{30} = 0.6148 \text{ mm}$
 $D_{60} = 3.7611 \text{ mm}$ $D_{15} = 0.0798 \text{ mm}$
 $D_{50} = 2.1749 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

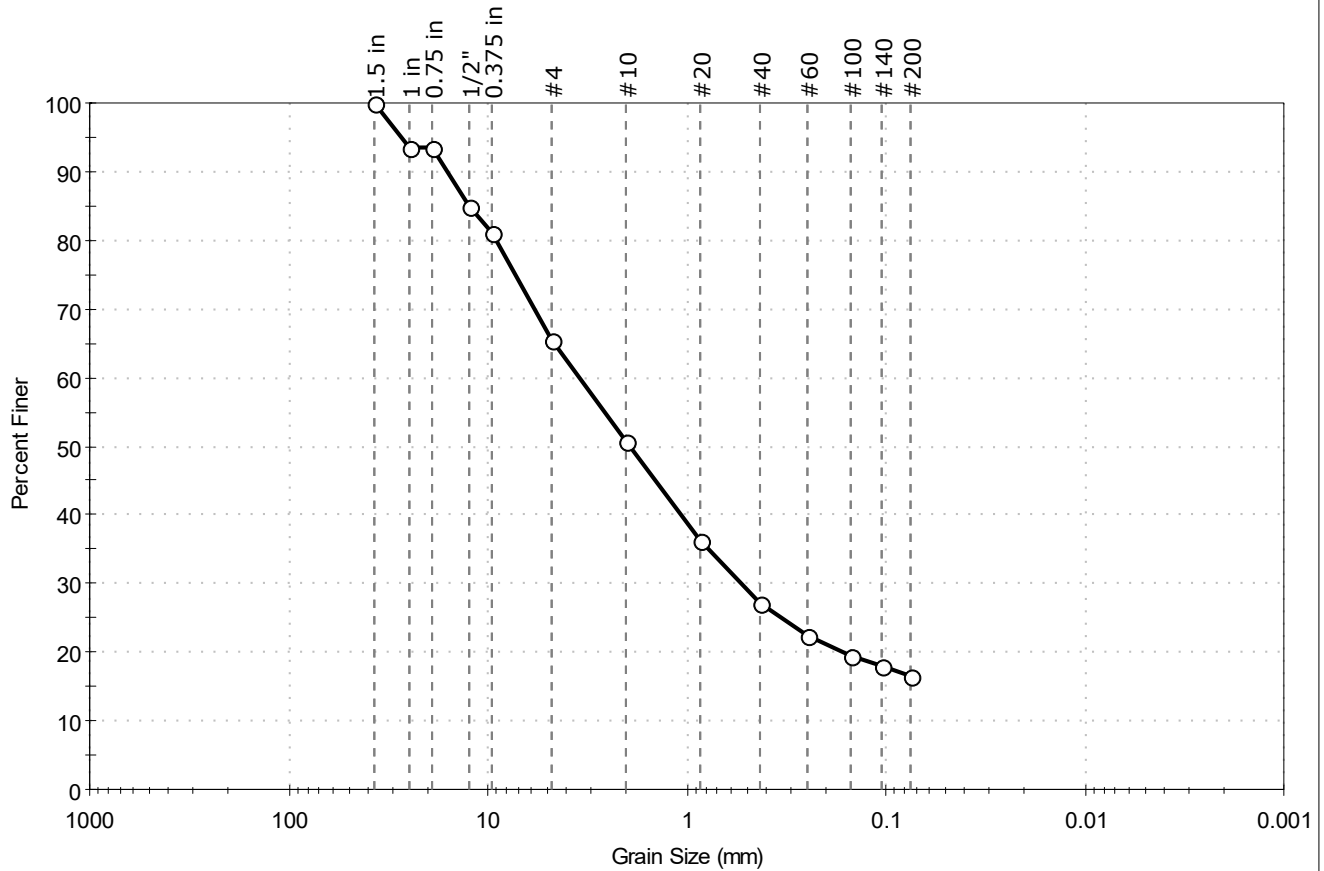
AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD

Client: WSP USA, Inc.	Project No: GTX-320898	
Project: MEDOT N. Bridge Bundle		
Location: ME		
Boring ID: BB-T8R3MB-101	Sample Type: Jar	Tested By: ajl
Sample ID: S-3(3D)	Test Date: 04/25/25	Checked By: ank
Depth: 5.0-7.0 ft	Test Id: 812308	
Test Comment: ---		
Visual Description: Moist, brown silty sand with gravel		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.5	48.8	16.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	93		
0.75 in	19.00	93		
1/2"	12.50	85		
0.375 in	9.50	81		
#4	4.75	65		
#10	2.00	51		
#20	0.85	36		
#40	0.42	27		
#60	0.25	22		
#100	0.15	20		
#140	0.11	18		
#200	0.075	17		

Coefficients

$D_{85} = 12.5944 \text{ mm}$ $D_{30} = 0.5257 \text{ mm}$
 $D_{60} = 3.4459 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 1.9099 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

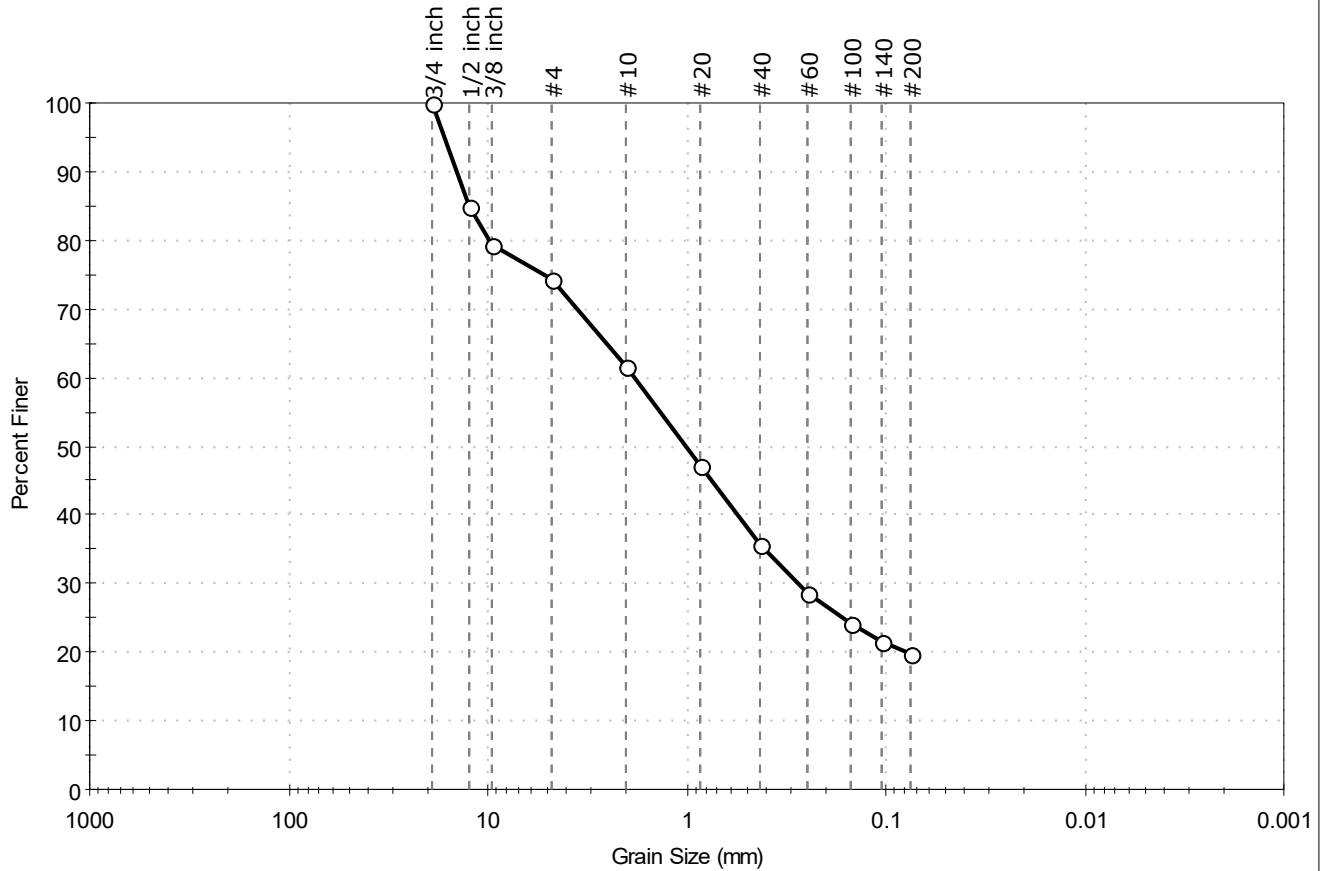
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD

Client: WSP USA, Inc.	Project No: GTX-320898	
Project: MEDOT N. Bridge Bundle		
Location: ME	Boring ID: BB-T8R3MB-102	Sample Type: Jar
	Sample ID: S-3(3D)	Test Date: 04/24/25
	Depth: 5.0-7.0 ft	Test Id: 812310
Test Comment: ---		
Visual Description: Moist, brown silty sand with gravel		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.6	54.7	19.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	85		
3/8 inch	9.50	79		
#4	4.75	74		
#10	2.00	62		
#20	0.85	47		
#40	0.42	36		
#60	0.25	29		
#100	0.15	24		
#140	0.11	22		
#200	0.075	20		

Coefficients

$D_{85} = 12.4784 \text{ mm}$ $D_{30} = 0.2763 \text{ mm}$
 $D_{60} = 1.8200 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 1.0071 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

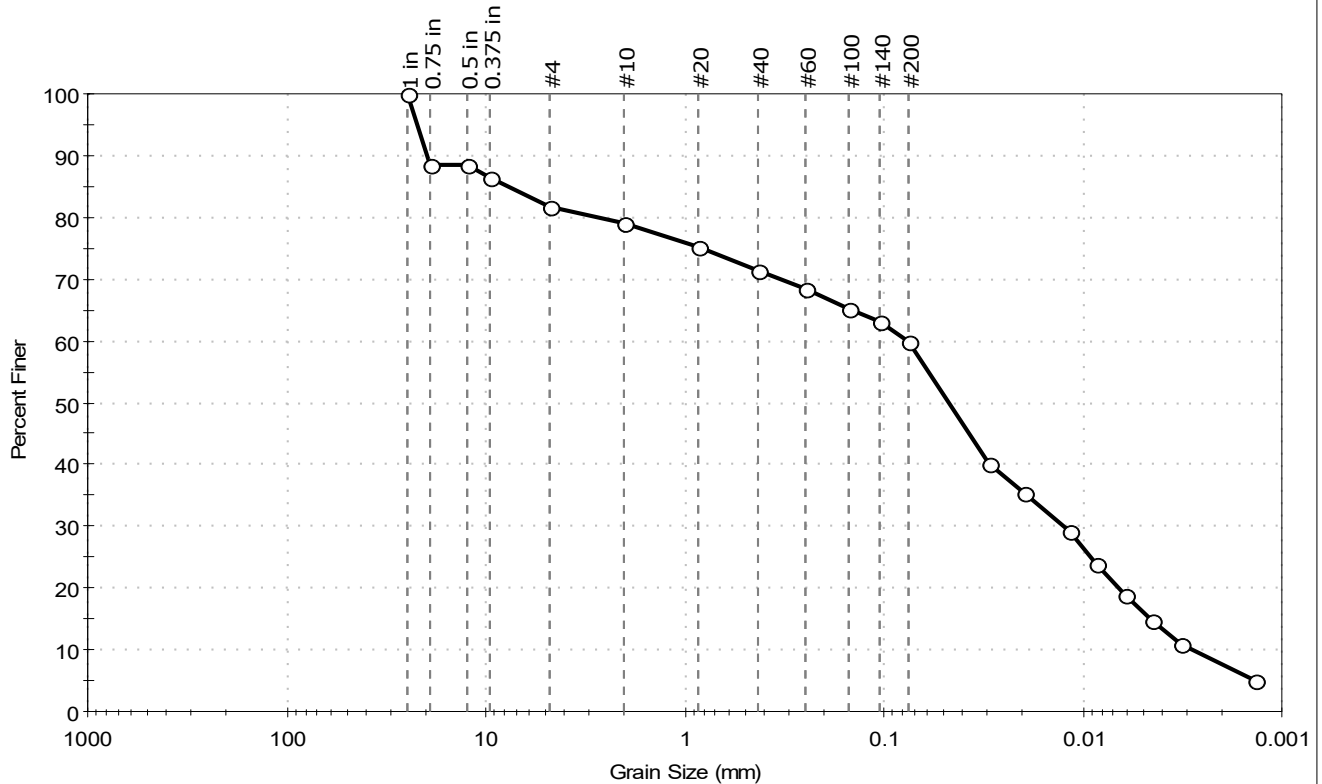
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD

Client: WSP USA, Inc.	Project No: GTX-320898
Project: MEDOT N. Bridge Bundle	
Location: ME	
Boring ID: BB-T8R3MB-102	Sample Type: Jar
Sample ID: S-4(4D)	Test Date: 04/25/25
Depth: 7.0-9.0 ft	Test Id: 812316
Test Comment: ---	
Visual Description: Moist, dark yellow silt with sand and gravel	
Sample Comment: ---	

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	18.3	21.8	59.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	89		
0.5 in	12.50	89		
0.375 in	9.50	87		
#4	4.75	82		
#10	2.00	79		
#20	0.85	75		
#40	0.42	71		
#60	0.25	68		
#100	0.15	65		
#140	0.11	63		
#200	0.075	60		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0294	40		
---	0.0199	35		
---	0.0117	29		
---	0.0085	24		
---	0.0062	19		
---	0.0045	15		
---	0.0032	11		
---	0.0014	5		

Coefficients

D₈₅ = 7.6047 mm D₃₀ = 0.0125 mm
 D₆₀ = 0.0758 mm D₁₅ = 0.0046 mm
 D₅₀ = 0.0468 mm D₁₀ = 0.0028 mm
 C_u = 27.071 C_c = 0.736

Classification

ASTM N/A

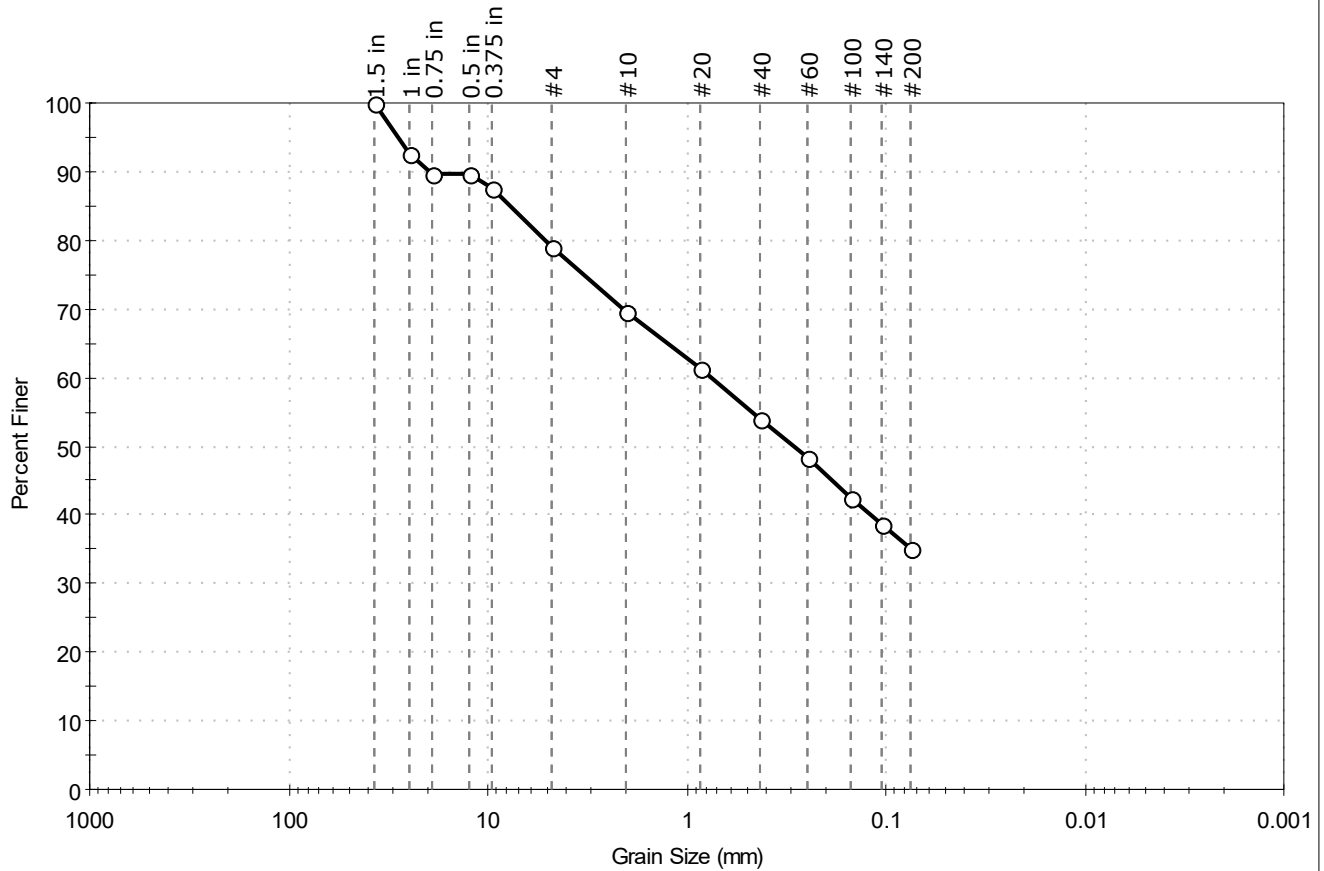
AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD
 Dispersion Device : Apparatus A - Mech Mixer
 Dispersion Period : 1 minute
 Est. Specific Gravity : 2.65
 Separation of Sample: #200 Sieve

Client: WSP USA, Inc.	Project No: GTX-320898	
Project: MEDOT N. Bridge Bundle		
Location: ME		
Boring ID: BB-T8R3MB-102	Sample Type: Jar	Tested By: ajl
Sample ID: S-5(5D)	Test Date: 04/24/25	Checked By: ank
Depth: 10.0-12.0 ft	Test Id: 812309	
Test Comment: ---		
Visual Description: Moist, olive brown silty sand with gravel		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.9	44.0	35.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	92		
0.75 in	19.00	90		
0.5 in	12.50	90		
0.375 in	9.50	88		
#4	4.75	79		
#10	2.00	70		
#20	0.85	61		
#40	0.42	54		
#60	0.25	48		
#100	0.15	43		
#140	0.11	39		
#200	0.075	35		

Coefficients

$D_{85} = 7.6028 \text{ mm}$ $D_{30} = \text{N/A}$
 $D_{60} = 0.7504 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.2919 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD

APPENDIX D

Calculations



CALCULATIONS

Date:	6/23/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027270.00	Checked by:	LDN
Subject:	Frost Penetration Depth - Bridge Structures in Fill	Reviewed by:	CCB
Project Short Title:	Springfield Road Bridge No. 0064, Crossing Meadow Brook, T8 R3 NBPP, ME		

OBJECTIVE

Determine the depth of frost penetration at the proposed bridge site.

REFERENCES

1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. WSP interpreted subsurface section A-A' (Figure 3, Preliminary Geotechnical Design Report).
3. WSP geotechnical test boring logs (Appendix A, Preliminary Geotechnical Design Report).
4. GeoTesting Express laboratory testing results (Appendix C, Preliminary Geotechnical Design Report).

ASSUMPTIONS

1. Based on the available moisture contents in existing fill and stream deposit materials around the existing culvert, it is assumed that the new backfill and existing soils will have a similar behavior for the purpose of frost penetration depth calculations. Thus, average moisture content in existing granular fills and stream deposit materials will be used for this evaluation.
2. New fill is assumed to consist of Granular Borrow.

CALCULATION

Follow the procedure in the MaineDOT Bridge Design Guide Section 5.2.1 to determine the depth of frost penetration.

The site location is T8 R3 NBPP, Maine. Using the Maine Design Freezing Index Map (Reference 1, Figure 5-1), the maximum design freezing index at the site location is 1,900 degree-days.

Based on Reference 2, the proposed box culvert will be founded on bedrock and weathered bedrock.

Based on References 3 and 4, the values of water contents in the upper coarse grained fill are: 6.9%, 5.9%, and 5.2%, and 32.5% for fine grained stream deposits. Average water content is 12.6% for these samples.

From Reference 1, Table 5-1, with design freezing index = 1,900, coarse-grained soil, and $w = 12.6\%$.

Frost Penetration = 88.4 inches 7.4 feet

CONCLUSIONS

The depth of frost penetration at the proposed bridge site is estimated to be 7.4 feet in areas where new fill associated with bridge foundations will be placed. The foundations should be embedded below this depth. If the wingwall footings are anticipated not to be founded on bedrock, they should be embedded a minimum of 7.4 feet below the ground surface.



CALCULATIONS

Date:	6/23/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027270.00	Checked by:	LDN
Subject:	Lateral Earth Pressure	Reviewed by:	CCB
Project Short Title: Springfield Road Bridge No. 0064, Crossing Meadow Brook, T8 R3 NBPP, ME			

OBJECTIVE

Determine lateral earth pressure acting on the proposed culvert.

REFERENCES

1. Das, Braja M. 2011. Principles of Foundation Engineering, 7th ed. Cengage Learning, Stamford, CT.
2. WSP geotechnical boring logs (Appendix A, Geotechnical Design Report)
3. WSP. 2025. NBB Bridge 0064 Geotechnical Material Properties.xlsx
4. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
5. AASHTO LRFD Bridge Design Specifications, 10th Ed. 2024.

ASSUMPTIONS

1. The backfill surface behind the wingwalls is assumed to be horizontal.
2. The design friction angles for the in-situ soil layers are based on correlation (Reference 1, Equation 2.26) to the average N60-values encountered in all borings for each layer (References 2 and 3). The design friction angles are:

$\phi_{\text{Existing Fill}} =$	35	degrees
$\phi_{\text{Stream Deposits}} =$	31	degrees
$\phi_{\text{Glacial Till}} =$	38	degrees
$\phi_{\text{Granular Borrow}} =$	32	degrees
$\phi_{\text{Gravel Borrow}} =$	36	degrees
3. The design friction angles for the fill construction materials (Reference 4), based on MaineDOT standard practice:
- 4) In accordance with AASHTO LRFD Section 3.11.6.4 (Reference 5), an applied live load subcharge will be required in calculation based on wall heights of the culvert's inlet/outlet and distance of the wall backface to the edge of traffic. A minimum equivalent height of soil loading (Heq) of 2 feet will be used.

CALCULATION

1. Calculate the at-rest lateral earth pressure coefficient.

$$K_0 = 1 - \sin \phi \quad (\text{Ref. 1, Eqn. 7.3}) \quad \text{where:} \quad \phi = \text{internal friction angle of soil}$$

2. Calculate the full active and full passive earth pressure coefficients using Rankine theory.

$$K_a = \tan^2 \left(45^\circ - \frac{\phi}{2} \right) \quad (\text{Ref. 1, Eqn. 7.8}) \quad K_p = \tan^2 \left(45^\circ + \frac{\phi}{2} \right) \quad (\text{Ref. 1, Eqn. 7.61})$$

where:

ϕ = internal friction angle of soil

K_0	K_a	K_p	
0.43	0.27	3.69	Existing Fill -In Situ
0.48	0.32	3.12	Stream Deposits - In Situ
0.38	0.24	4.20	Glacial Till - In Situ
0.47	0.31	3.25	Granular Borrow - Fill
0.41	0.26	3.85	Gravel Borrow - Fill

CONCLUSIONS

For a non-yielding, stiff structure like the box culvert proposed for this project, an at-rest lateral earth pressure coefficient of 0.48 is recommended for design.



CALCULATIONS

Date:	6/24/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027270.00	Checked by:	LDN
Subject:	Modulus of Subgrade Reaction	Reviewed by:	CCB
Project Short Title:	Springfield Road Bridge No. 0064, Crossing Meadow Brook, T8 R3 NBPP, ME		

OBJECTIVE

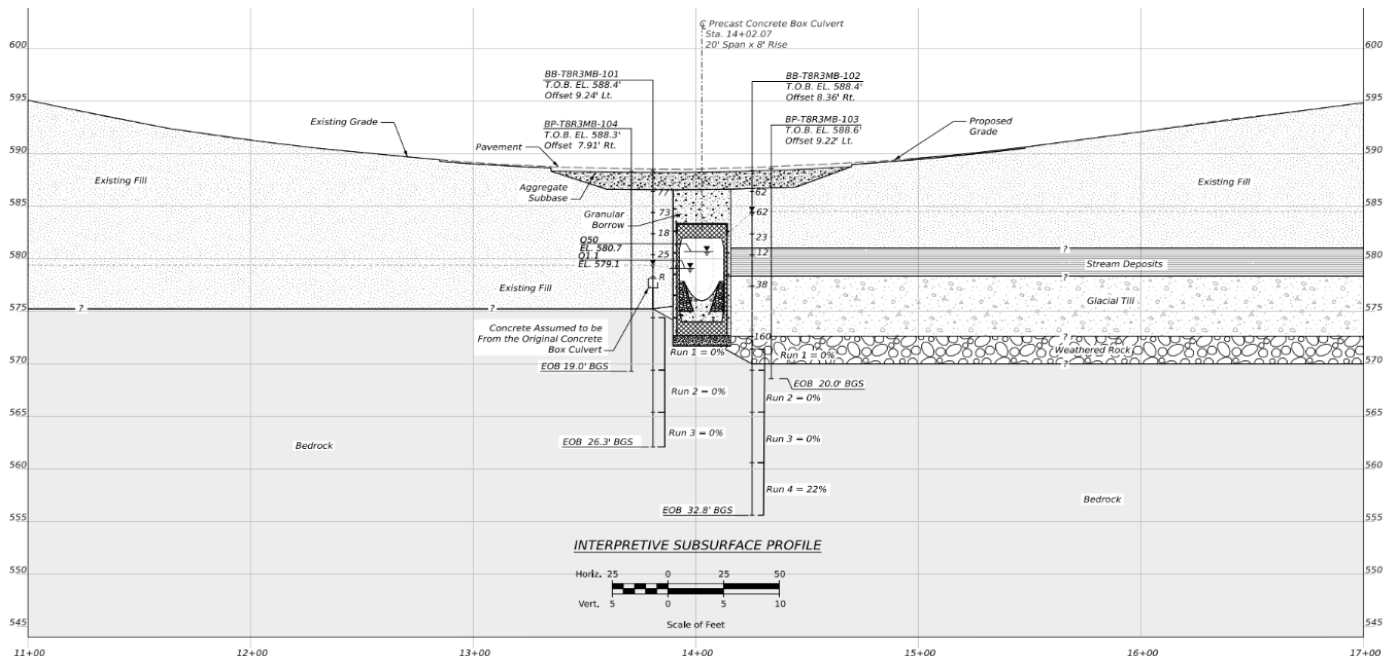
Determine the modulus of subgrade reaction in the granular borrow.

REFERENCES

1. Das, Braja M. 2010. Principles of Foundation Engineering, 7th Ed. Cengage Learning, Stamford, CT.
2. Federal Highway Administration, 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.
3. Maine DOT. 2003. Bridge Design Guide. Updated 2018.

ASSUMPTIONS

1. The proposed box culvert will be founded on a 12-inch layer of granular borrow for underwater backfill on the bedrock surface.



Interpretive Subsurface Profile

CALCULATION

1. Determine Modulus of Subgrade Reaction (Method 1).

Determine Modulus of Subgrade Reaction (K_{VI}) using Figure 8-3 below (Ref. 2). Based on the granular borrow internal angle of friction (Ref. 3), a $(N_1)_{60}$ value was calculated and the relative density was calculated using Table 18.5 of Reference 1 (below). The calculated relative density 40.3%. The calculated relative density shows the granular borrow to be in the medium dense range of the chart and the coarse grained soil curve was used to determine (K_{VI}).



CALCULATIONS

Date: 6/24/2025 **Made by:** RJN
Project No.: 0040947.9162 / WIN 027270.00 **Checked by:** LDN
Subject: Modulus of Subgrade Reaction **Reviewed by:** CCB
Project Short Title: Springfield Road Bridge No. 0064, Crossing Meadow Brook, T8 R3 NBPP, ME

$$\phi_{\text{Granular Borrow}} = 32 \text{ degrees} \quad (\text{Ref 3, Table 3-3})$$

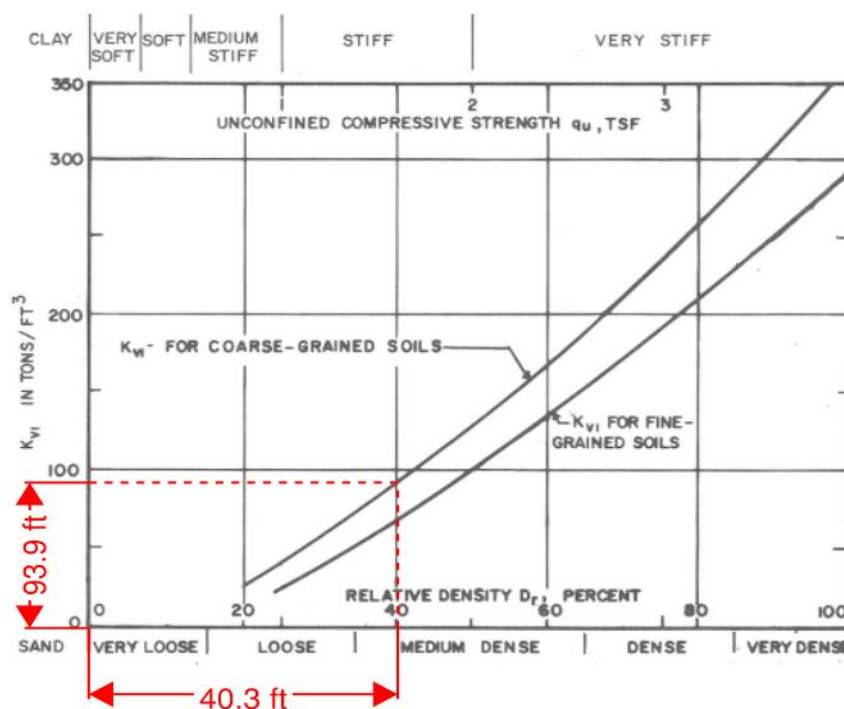
$$\phi'(\text{deg}) = 27.1 + 0.3 (N_1)_{60} - 0.00054(N_1)_{60}^2 \quad (\text{Ref 1, Eqn. 18.15})$$

$(N_1)_{60} = 17$ **Table 18.5** Approximate Relationship Between Corrected Standard Penetration Number and Relative Density of Sand

Corrected standard penetration number, $(N_1)_{60}$	Relative density, D_r (%)
0-5	0-5
5-10	5-30
10-30	30-60
30-50	60-95

$$D_r (\%) = 40.3$$

Relationship between Modulus of Subgrade Reaction and Material Stiffness (Ref 2, Table 8-3)



$$K_{vl} = 94 \text{ Tons/ft}^3$$

$$K_{vl} = 109 \text{ pci}$$

CONCLUSIONS

The recommended modulus of subgrade reaction is $K = 109$ pci as determined using FHWA GEC-6.

