



REPORT

Geotechnical Design Report (GDR)

*Prentiss Township T7 R3 NBPP Mattagodus Brook Bridge #5185 Replacement
(WIN 027532.00)*

Submitted to:

Rich Myers, PE

Assistant Bridge Program Manager
Maine Department of Transportation
16 State House Station
Augusta, Maine 04333-0016

Submitted by:

WSP USA, Inc.

428 Dow Highway Eliot, Maine 03903

+1 207 865 4024

MaineDOT WIN 027532.00 / WSP US0040947.9162

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1.0 INTRODUCTION AND PROJECT BACKGROUND

This Geotechnical Design Report (GDR) pertains to the replacement of Mattagodus Brook Bridge #5185 in Prentiss Township T7 R3, Maine, MaineDOT WIN 027532.00. 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 parameters for site materials; and provide geotechnical design and recommendations for construction of the replacement culvert.

Mattagodus Brook Bridge #5185 carries Route (RT) 169 (Averill Road) over Mattagodus Stream in Prentiss Township T7 R3 NBPP, Maine. The existing structure consists of three (3) corrugated steel sectional plate pipe culverts constructed in 1948. The bank-full width for this project is 22.0 feet. The replacement design provided in the Draft PS&E plan set¹ is a 70-foot-long pre-cast concrete box culvert with a 26-foot clear span and 11-foot rise, that includes an approximately 11 foot miter at each end.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The site lies in the Bowers Mountain² and Springfield³ 15-minute Quadrangles. Surficial soils are mapped as glacial stream deposits overlying glacial till deposits. The glacial-stream deposits consist of sand and gravel with moderate to high permeability that may include minor till and is commonly overlapped or entirely buried by glacial marine deposits in coastal lowlands. The glacial-stream deposits overlie glacial till deposits comprised of rarely stratified, heterogeneous mixture of sand, silt, clay, and stones. Two varieties of glacial till are mapped in the geologic unit: basal till, which is fine grained very compact, with low permeability and poor drainage, and ablation till, described as loose, sandy, and stony, with moderate permeability and fair to good drainage. The unit generally overlies bedrock, however it may overlie or include sand and gravel deposits. Significant sand and gravel aquifers are mapped in the area, consisting primarily of glacial sand and gravel that may include areas of sandy till and alluvium connected with surface-water bodies.⁴

2.2 Regional Bedrock Geology

Bedrock beneath the site is mapped as the Carys Mills Formation, consisting of interbedded dark gray micrite and lighter gray dolomicrite with subordinate calcareous and non-calcareous phyllite and sparse calcareous wacke. Bedding is thin to medium (5 cm to 20 cm) and is commonly obscured by transposed layering. The carbonates are converted in contact aureoles to dramatically banded biotite-rich purplish-gray (originally siltstone) and white

¹ State of Maine Department of Transportation, Prentiss TWP T7 R3 NBPP, Penobscot County, Mattagodus Brook Bridge over Mattagodus Stream, Averill Road, Federal Aid Project No. 2753200, Bridge No. 5185, Draft PS&E Plan Set, March 27, 2026.

² Weddle, Thomas K. (compiler), 2001, Surficial materials of the Bowers Mountain quadrangle, Maine: Maine Geological Survey, Open-File Map 01-311, map, scale 1:24,000. *Maine Geological Survey Maps*. 909.

³ Holland, W.R., 1986, Reconnaissance Surficial Geology of the Springfield 15' Quadrangle, Maine, Maine Geological Survey, Open-File 86-23, 1:62,500.

⁴ Neil, C.D. and Weddle, T.K., 2001, Significant sand and gravel aquifers in the Bowers Mountain quadrangle, Maine: Maine Geological Survey, Open-File Map 01-334, map, scale 1:24,000.

(limestone) or greenish actinolite-bearing (dolomitic) calcsilicate hornfels.^{5,6} The Carys Mills Formation is Silurian and Ordovician in aged.

3.0 SUBSURFACE INVESTIGATION

WSP completed three (3) borings: BB-PMS-101, BB-PMS-102 and BB-PMS-102A offset from BB-PMS-102 between March 17 and March 18, 2025, to collect subsurface geotechnical information. WSP surveyed the as-drilled boring locations following completion of the drilling program. Boring location coordinates and ground surface elevations are summarized in Table 1 (attached). Boring 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 using a Diedrich D-50 Turbo drill rig and both solid stem auger (SSA) and cased wash boring methods. Borings BB-PMS-101, BB-PMS-102, and BB-PMS-102A were advanced using 4-inch solid stem augers (SSA) to 10 feet, 7.5 feet (shallow refusal encountered), and 10 feet below ground surface (bgs), respectively, after which they were advanced using a 4-inch inside diameter casing in 5-foot lengths installed using an automatic safety hammer. When refusal was encountered, 3-inch inner diameter casing was telescoped through the 4-inch inner diameter casing for rock coring. Soil in the casing was washed out using a roller bit and water to the depth where samples were subsequently collected. Borings BB-PMS-101 and BB-PMS-102A were drilled to refusal at depths of 26 feet and 25.8 feet bgs, respectively, after which 10 feet of rock core was collected.

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 2-foot intervals within the upper 9 feet of the ground surface and at 5-foot intervals thereafter 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 within the 6-inch to 12-inch and 12-inch to 18-inch intervals 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 Seaboard⁷) 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 additional 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

⁵ Ludman, A. and Hopeck, J.T., 2020, Bedrock geology of the Springfield 15' quadrangle, Maine, Maine Geological Survey, Open-File Report 20-22, 1:24,000.

⁶ Ludman, A., 2024, Bedrock geology of the Lincoln 1:100,000 quadrangle, Maine, Maine Geological Survey, Geologic Map 24-11, 1:100,000.

⁷ 2024PA00175 - Seaboard - SPT Energy Calibration, prepared by GRL Engineers Inc., dated 10/23/2024.

Appendix A. A detailed summary of the rock core quality parameters for the recovered rock core is presented in Table 2 (attached) 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 description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs in Appendix A.

4.0 LABORATORY TESTING

GeoTesting Express, LLC tested the collected soil samples in accordance with applicable American Society for Testing and Materials (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 4-1. Measured index and classification test results from soil samples are summarized in Table 2 (attached) 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 4-1: Type and Number of Laboratory Tests Performed

Soil Laboratory Test	Test Standard	No. Tests Completed
Grain Size Analysis (sieve only)	ASTM D 6913	5
Grain Size Analysis (sieve and hydrometer)	ASTM D 6913 ASTM D 7929	4
Moisture Content	ASTM D 2216	7

5.0 SUBSURFACE CONDITIONS

The soils WSP encountered in the borings generally include fill materials placed during construction of the existing culvert and roadway and naturally occurring fine to coarse sand with varying amounts of fine to coarse gravel and trace to little silt interpreted as glacial till. The boring logs in Appendix A provide detailed descriptions of soil conditions encountered in the borings. WSP's Interpretive Subsurface Profile (ISP) along the centerline of the roadway is presented in Figure 3 (attached). Table 5-1 summarizes the major stratigraphic units from the existing ground surface to depth.

Table 5-1: Summary of Encountered Subsurface Soils and Fill Materials

Material	Approximate Thickness (feet)	Generalized Description	Classification ¹	N ₆₀ ²
Asphalt Pavement	0.5	N/A	N/A	N/A
Fill	5.0 to 15.0	SAND, trace to some silt, trace to little gravel	A-1-b SM or SW-SM	28 to 165 (medium dense to very dense)
Loose Fill (BB-PMS-101)	8.0	SAND, little to some silt, little gravel, trace wood	A-2-4, A-1-b SM	3 to 10 (very loose to loose)
Glacial Till	10.8 to 13.0	GRAVEL, little silt, trace sand to sandy SAND, some gravel to gravelly, trace to little silt	A-1-a or A-1-b SM, SW-SM, GP-GM, or GM	23 to 45 (medium dense to 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 the borings between 25.8 feet (Elevation (EL) 339.1 feet) and 26.0 feet bgs (EL 338.8 feet). The bedrock lithology generally consists of grey, fine grained, strong (R4) to extremely strong (R6), fresh (W1) to moderately weathered (W3) phyllite from the Carys Mills Formation. Rock Quality Designation (RQD) ranged from very poor to fair (0% to 64%) and estimated Rock Mass Rating (RMR) ranged from 40 to 64. Table 2 (attached) provides detailed information about recovery, RQD, RMR, and descriptions of lithology, rock mass, and discontinuities.

Groundwater: Groundwater levels were measured in BB-PMS-101 and BB-PMS-102A upon completion and prior to removal of the casing. Measured groundwater levels ranged from 6.2 feet bgs (EL 358.6 feet) and 7.0 feet bgs (EL 357.9 feet). Per the PS&E Plan Set¹, the river level associated with ordinary high water (Q_{1.1}) at the existing bridge is EL 358. feet and the river level associated with design Q₅₀ is EL 360.9 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. In general, groundwater measurements made during drilling are influenced by the addition of water used during the drilling process and the measurements may not be representative of a stabilized water level. Groundwater levels also fluctuate due to other natural variations, such as seasonal/climatic influences, precipitation, barometric pressure, and temperature, and due to other variations, such as construction and groundwater pumping.

6.0 GEOTECHNICAL ANALYSIS AND RECOMMENDATIONS

The proposed box culvert as specified in the PS&E Plan Set¹ will be 70 feet long including the wingwalls with a 26 foot span and the culvert invert between EL 350.5 feet and EL 350.8 feet. Assuming the base of the culvert is 20 inches thick, the culvert will be embedded into the glacial till 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. The approximate maximum depth of excavation is 17.5 feet bgs. Installation of the proposed box culvert will require removal of all three existing sectional plate pipe culverts which will leave a large void to the north of the proposed culvert location that is planned to be infilled with compacted

Granular Borrow. Figure 2 (attached) shows the culvert location in plan and Figure 3 (attached) shows the culvert location with respect to the interpreted stratigraphy.

The record survey plans⁸ dated December 29, 1947, indicate a previous pony truss bridge with large dry-laid field stone abutments was replaced by three ten-foot tall sectional plate pipe culverts. The proposed box culvert centerline is near the southern-most existing sectional plate pipe culvert centerline and the proposed box culvert north wall will extend toward the centerline of the middle existing sectional plate pipe culvert. The southern extent of the proposed box culvert may require excavation of the original dry laid stone southern abutment if the abutment was not previously removed. The installation of the three sectional plate pipe culverts likely resulted in the full removal of the original dry laid stone northern abutment.

6.1 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 6-1 provides a summary of design parameters for each in situ soil layer encountered (refer to Figure 3, attached) and material properties for construction fills as identified in the MaineDOT Standard Specifications.⁹ These design parameters served as a basis for calculations, analyses, and evaluations discussed herein.

Table 6-1: Engineering Design Parameters for In Situ Soils and Construction Fills

Soil Layer		γ (pcf)	ϕ (°)	1K_0
In Situ Materials	Asphalt	140	N/A	N/A
	Existing Fill	125	35	0.43
	Existing Loose Fill	115	28	0.53
	Glacial Till	130	36	0.41
Construction Materials	Asphalt	140	N/A	N/A
	Granular Borrow (Section 703.19)	125	32	0.47
	Gravel Borrow (Section 703.20)	135	36	0.41

Note: ¹Lateral earth pressure coefficient at rest was determined using the equation $K_0 = 1 - \sin\phi$.

6.2 Frost Penetration Depth

WSP estimated an average frost penetration depth for roadway and culvert backfill using the MaineDOT Bridge Design Guide¹⁰. The site has a design freezing index of approximately 1,900 °F-degree days. For roadway and

⁸ Maine Highway Commission Bridge Division, Mattagodus Bridge, Survey Plan, dated Dec. 29, 1947, and Details of Pipes plan, dated July 1948.

⁹ 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>.

¹⁰ Maine Department of Transportation, Bridge Design Guide, 2003 updated 2018. <https://www.maine.gov/mdot/bdg/docs/BDGupdateJune2018.pdf>

bridge abutment coarse-grained fills with an approximate average water content of 16.3%, we estimate a frost penetration depth of 6.9 feet at the site. Refer to assumptions, references, and 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 the culvert are subject to little overburden and we recommend that the materials used in these areas below concrete features be non-frost susceptible.

6.3 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 0.47 for the Granular Borrow backfill that will be acting on the culvert. Per AASHTO LRFD¹¹ Article 3.11.6.4, and applied 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. Calculations, references, and assumptions are provided in Appendix D.

6.4 Subgrade Modulus

We recommend a modulus of subgrade reaction (K) of 225 pci (pounds per cubic inch) for the use in structural design of the proposed culvert was determined based on the corrected average N_{60} -values encountered for the glacial till deposit layer and estimating values using interpolation using relative density correlated from N_{60} values. Refer to assumptions, references, and calculations in Appendix D.

6.5 Global Stability Analysis

WSP evaluated global stability at one cross-section transverse to RT 169 with the greatest change in embankment geometry, for proposed conditions at Sta. 14+10, north of the proposed box culvert. Both north and south slopes were analyzed in the transverse cross-section. We performed the analyses using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience¹² for the post-construction static conditions. Embankment geometry was created using the proposed grades from the PDR plan set. We interpreted the subsurface stratigraphy from Figure 3 (attached).

We analyzed the models using the Spencer method with auto refined search for circular failure surfaces and the Spencer method with a Cuckoo search plus surface altering optimization for non-circular failure surfaces. The search algorithm that produced a lower factor of safety (FS) was selected, and the analyses show that non-circular surfaces govern potential slope failures. Our proposed minimum factor of safety (FS) of 1.3 for global stability correlates to the AASHTO LRFD¹¹ 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.

WSP analyzed the lowest factor of safety for two scenarios: Case 1: the lowest FS for a potential failure surfaces within embankment fills only and Case 2: the lowest FS for a potential failure surface that extend into the in situ soils below the embankment. Results are summarized in Table 6-2 for Case 1, and Table 6-3 for Case 2. The

¹¹ American Association of State Highway Transportation Officials (2024). "AASHTO LRFD Bridge Design Specification," 10th edition, Washington, DC

¹² Rocscience Inc. Slide2 software package, Version 9.038, build date February 28, 2025.

proposed embankment geometry at the north and south embankment slopes produced potential failure surfaces that exceed the minimum allowable design FS values for both failure surfaces within proposed embankment fills and that extend through underlying in situ soils. Refer to assumptions, references, and calculations in Appendix D.

Table 6-2: Minimum Global Stability Factors of Safety for Potential Failure Surfaces in Embankment Fills

Surface Type	Location	Static Factor of Safety
Circular	South Slope ¹	1.46
Non-Circular		1.42
Circular	North Slope ¹	1.41
Non-Circular		1.42

Note: ¹Minimum FS of 1.3 recommended for potential failure surfaces in fill soils only under static loading.

Table 6-3: Minimum Global Stability Factors of Safety for Potential Failure Surfaces Through In Situ Soils

Surface Type	Location	Static Factor of Safety
Circular	South Slope ¹	1.73
Non-Circular		1.69
Circular	North Slope ¹	1.48
Non-Circular		1.47

Note: ¹Minimum FS of 1.3 recommended for potential failure surfaces in fill soils only under static loading.

6.6 Bearing Resistance and Settlement

The profile for the culvert and embankment fills below the roadway in the PS&E Plan Set¹ shows the following:

- No substantial grade change at the roadway surface that will increase embankment loading at the culvert location.
- The proposed culvert is taller than the existing culverts and will displace some of the in situ soils below the embankment. This will result in a net effective stress reduction in the underlying soils at the culvert.
- The removal of the remaining two existing 10-foot tall culverts located to the north of the proposed box culvert will require additional Granular Borrow to fill the void previously occupied by the culverts. The record survey plans⁸ dated December 29, 1947, indicate the area where the two existing steel culverts are located was previously occupied by embankment fill and large field stone abutments. Based on the materials and/or structures previously occupying the space of the culverts to be removed, we anticipate there will be no new stress increase on the underlying soils below the existing culverts.
- The proposed box culvert will be founded on a 12-inch thick layer of Granular Borrow for Underwater Backfill (MaineDOT Standard Specification⁹ Section 703.19) above the in situ glacial till.

By inspection, there will not be a net increase in effective stress within the glacial till that would lead to total or differential settlement beyond the acceptable limits at the proposed culvert. Thus, we did not perform a settlement evaluation for preliminary design.

WSP evaluated the strength and service limit state bearing resistance and compared that to the bearing pressure for the centerline of the culvert at the centerline of the roadway. This represents the greatest loading expected along the length of the culvert. We recommend a strength limit state bearing resistance factor of 0.45 and a strength limit state factored bearing resistance of 37 ksf (kips [kilo-pounds] per square foot) be used for design. We recommend a service limit state bearing resistance factor of 1.0, and a factored presumptive bearing resistance of 5 ksf for the service limit state that limits settlement to 1 inch. Furthermore, WSP calculated the factored bearing pressure for the new culvert and compared the factored pressure to the bearing resistances at the strength I and service limit states and determined that demand to capacity ratios were 0.1 and 0.4, respectively. Refer to assumptions, references, and calculations in Appendix D.

The ends of the culvert where there is limited overburden will apply less load to the underlying soils than calculated for the portion of the culvert under the embankment. The ends of the culvert, however, bear on the same soils that have the service limit state bearing resistance of 5 ksf. By inspection, there is adequate bearing resistance for the inlet and outlet of the culvert. While scour of the underlying is mostly likely to occur at the inlet and/or outlet of the culvert, the rip rap outside the inlet and outlet of the culvert as well as the bedding material within the culvert have been designed to be scour resistant. The rip rap sizing was developed based on the results of the project Hydrology and Hydraulics Reports that are contained with the Preliminary Design Report¹³ considering native stream bed material, anticipated scour velocities, and water levels under a variety of storm conditions.

6.7 Scour and Riprap

The inlet and outlet of the culvert will be armored with void-filled riprap in accordance with MaineDOT Standard Details¹⁴ and Standard Specifications.⁹ 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 PS&E¹ design provides the recommended extents of scour and riprap protection of Mattagodus Stream embankments, which are proposed to have 3-foot thick void filled riprap at the surface.

7.0 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 Specifications⁹ and OSHA requirements. Temporary dewatering of

¹³ Preliminary Design Report, Mattagodus Brook Bridge #5185 over Mattagodus Stream, Prentiss TWP T7 R3 NBPP, Maine, WIN 027532.00, 16 pp.

¹⁴ 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

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.⁹ 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.

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 Specifications⁹) 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 daylights beyond the new toe of slope.

Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Specifications⁹ Section 203.09 for preparation of the embankment area which requires a firm, unyielding surface. Embankment fill should meet the requirements of MaineDOT Standard Specifications⁹ Section 703.19 for Granular Borrow. Embankment excavation and fill placement should follow MaineDOT Specifications⁹ 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 Specifications⁹, 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.

8.0 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 Mattagodus Brook Bridge culvert that passes RT 169 over Mattagodus Stream in Prentiss Township T7 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.

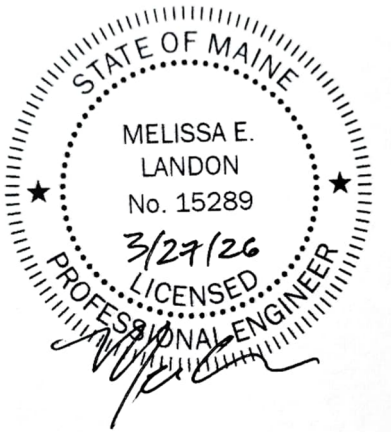
Our analyses and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Variations in the nature and extent of

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".

Jeffrey D. Lloyd, PE
Vice President, Geotechnical Engineering

LDN/MEL/CCB/JDL

[https://wspnlinenam.sharepoint.com/sites/us-mainedot-northern/shared documents/5185-mattagodus brook bridge - 27532_00/technical/design/geotechnical/09 reports/02 gdr/win 027532.00 bridge 5185 gdr.docx](https://wspnlinenam.sharepoint.com/sites/us-mainedot-northern/shared%20documents/5185-mattagodus%20brook%20bridge%20-%2027532_00/technical/design/geotechnical/09%20reports/02%20gdr/wins/027532.00%20bridge%205185%20gdr.docx)

Tables

Table 1: Subsurface Exploration Locations
Geotechnical Design Report
Mattagodus Brook Bridge #5185, Prentiss Township T7 R3, Maine
MaineDOT WIN 027532.00

Boring Designation ^{1,2,3}	As-Drilled Locations ⁴		Existing Ground Surface Elevation ⁴ (feet)	Bedrock Depth (feet bgs)	Maximum Depth of Investigation (feet bgs ⁵)
	Northing	Easting			
BB-PMS-101	588180.82	2235283.03	364.8	26.0	36.0
BB-PMS-102	588207.92	2235322.15	364.9	-	7.5
BB-PMS-102A	588203.91	2235325.15	364.9	25.8	36.0

Notes:

1. Boring and probe locations are shown in Figure 2 - 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.

Prepared By: RJN

Checked By: LDN

Reviewed By: MEL

Table 2: Summary of Rock Core Quality
Geotechnical Design Report
Bridge #5185 at Mattagodus Brook, Prentiss Township T7 R3, Maine
MaineDOT WIN 027532.00

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-PMS-101	NQ2 (2)	R1	1.5	26.0	29.0	27.5	3.0	2.9	97%	0.00	0%	Very Poor	Moderately Weathered (W3)	Strong (R4)	40	Grey, fine grained, PHYLLITE, strong, moderately weathered; discontinuities horizontal to vertically dipping, very closely spaced, tight to open, quartz infilling 26.3-26.8 ft bgs, rough, planar; average 3.33 fractures per foot. [CARYS MILLS FORMATION].
		R2	4.0	29.0	31.0	30.0	2.0	1.7	83%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	46	Grey, fine grained, PHYLLITE, strong, fresh; discontinuities horizontal to moderately dipping, close to very closely spaced, tight to open, no infilling, rough, planar; average 3.03 fractures per foot. [CARYS MILLS FORMATION].
		R3	7.5	31.0	36.0	33.5	5.0	4.7	94%	2.75	55%	Fair	Fresh (W1)	Extremely Strong (R6)	64	Grey, fine grained, PHYLLITE, extremely strong, fresh; discontinuities horizontal to vertically dipping, close to moderately closely spaced, tight, no infilling, laminated vertical sheeting, fracture faces have shiny sheen, slightly rough, planar; average 1.2 fractures per foot. [CARYS MILLS FORMATION].
BB-PMS-102A	NQ2 (2)	R1	2.7	26.0	31.0	28.5	5.0	3.9	78%	2.20	44%	Poor	Slightly Weathered (W2)	Strong (R4)	51	Grey, fine grained, PHYLLITE, strong, slightly weathered; discontinuities horizontal to steeply dipping, very close to moderately closely spaced, tight to open, quartz infilling in bottom 0.4 ft, slightly rough to rough, planar; average 2.3 fractures per foot. [CARYS MILLS FORMATION].
		R2	7.7	31.0	36.0	33.5	5.0	4.3	86%	3.20	64%	Fair	Fresh (W1)	Strong (R4)	60	Grey, fine grained, PHYLLITE, strong, fresh; discontinuities horizontal to low angle dipping, very close to widely spaced, open, no infilling, rough, planar to stepped; average 0.7 fractures per foot. [CARYS MILLS FORMATION].

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 name from: Ludman, Allan, and Hopeck, John T., 2020, Bedrock Geology of the Springfield 15' quadrangle, Maine: Maine Geological Survey, Open-File Report 20-22, 20 p. report and 4 color maps, scale 1:24,000. Maine Geological Survey Publications. 602. https://digitalmaine.com/mgs_publications/602
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 #5185 at Mattagodus Brook, Prentiss Township T7 R3, Maine
MaineDOT WIN 027532.00

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-PMS-101	364.84	3D	5.0 - 7.0	359.8 - 357.8	13.1	9.0	A-1-b (0)	SM
		4D	7.0 - 9.0	357.8 - 355.8	33.7	30.2	A-2-4 (0)	SM
		7D	20.0 - 22.0	344.8 - 342.8	8.1	14.9	A-1-b (1)	SW-SM
		8D	25.0 - 25.8	339.8 - 339.0	9.2	8.5	A-1-a (1)	GP-GM
BB-PMS-102	364.91	3D	5.0 - 6.6	359.9 - 358.3	24.1	15.6	A-1-b (0)	SM
BB-PMS-102A	364.89	1D	10.0 - 12.0	354.9 - 352.9	8.6	17.2	A-1-b (1)	SW-SM
		3D	15.0 - 17.0	349.9 - 347.9	12.6	-	A-1-a (0)	GM
		4D	20.0 - 22.0	344.9 - 342.9	15.8	10.9	A-1-b (0)	SM
		5D	25.0 - 25.8	339.9 - 339.1	18.6	-	A-1-b (0)	SM

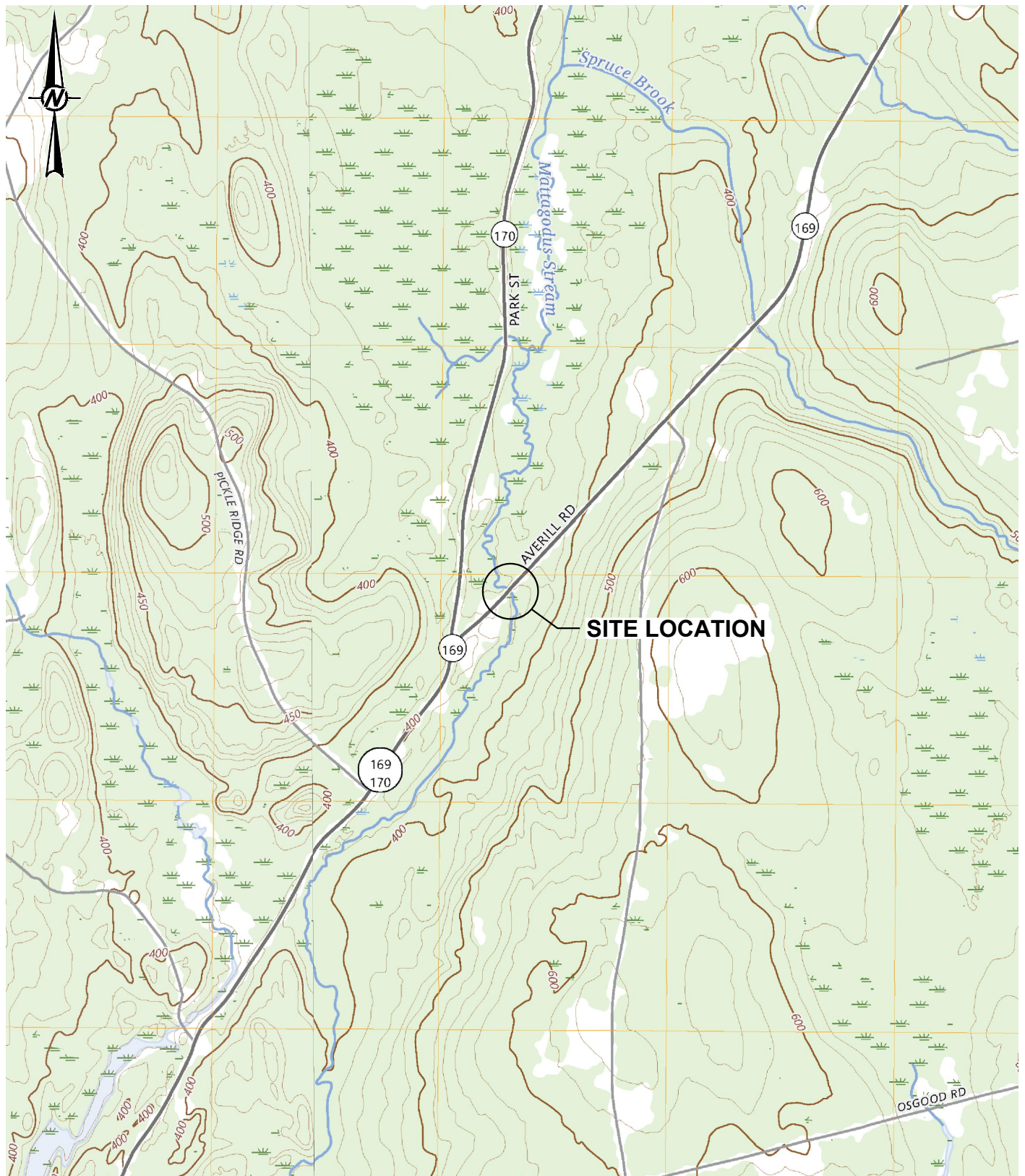
Notes:

1. Boring locations are illustrated in Figure 2 - Boring Location Plan of the Preliminary Geotechnical Design Report.
2. As-drilled boring ground surface elevations were obtained by WSP in the electronic file "Borings.dgn" dated 5/6/25.
3. Laboratory testing was performed by Geotesting Express of Acton, MA.
4. Particle size testing was performed in accordance with ASTM D6913, Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis.
5. 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.
6. 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.
7. USCS classification was performed by WSP based on interpreted laboratory test results. Items in italics use assumed fines type based on field characterization.
8. Complete laboratory test results for soil testing are provided in Appendix B of the Preliminary Geotechnical Design Report. Laboratory test results provided by Geotesting Express incorrectly labeled boring BB-PMS-102 sample S-3 ending depth below ground surface, incorrectly labeled boring BB-PMS-102 as "P8-PMS-102", and incorrectly label boring BB-PMS-102A as "PB-PMS-102A".

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

Figures

Last Edited By: usrg700666 Date: 2025-06-17 Time: 10:25:04 AM | Printed By: USRG700666 Date: 2025-06-17 Time: 3:21:41 PM
Path: \\corp.pben.net\USCentralData\USMT100\CADD\Projects\MaineDOT\NBB_Bridge09_PROJECT\SU0040947.9162\0003.dwg | File Name: US0040947.9162-0003-001.dwg



REFERENCE(S)

1. BASE MAP TAKEN FROM U.S.G.S. 7.5-MINUTE BOWERS MOUNTAIN AND SPRINGFIELD QUADRANGLES OF MAINE.



CLIENT
MAINE DEPARTMENT OF TRANSPORTATION

PROJECT
NORTH BRIDGE BUNDLE (NBB), BRIDGE #5185

CONSULTANT



YYYY-MM-DD 2025-06-17

DESIGNED LDN

PREPARED RG

REVIEWED MEL

APPROVED

TITLE

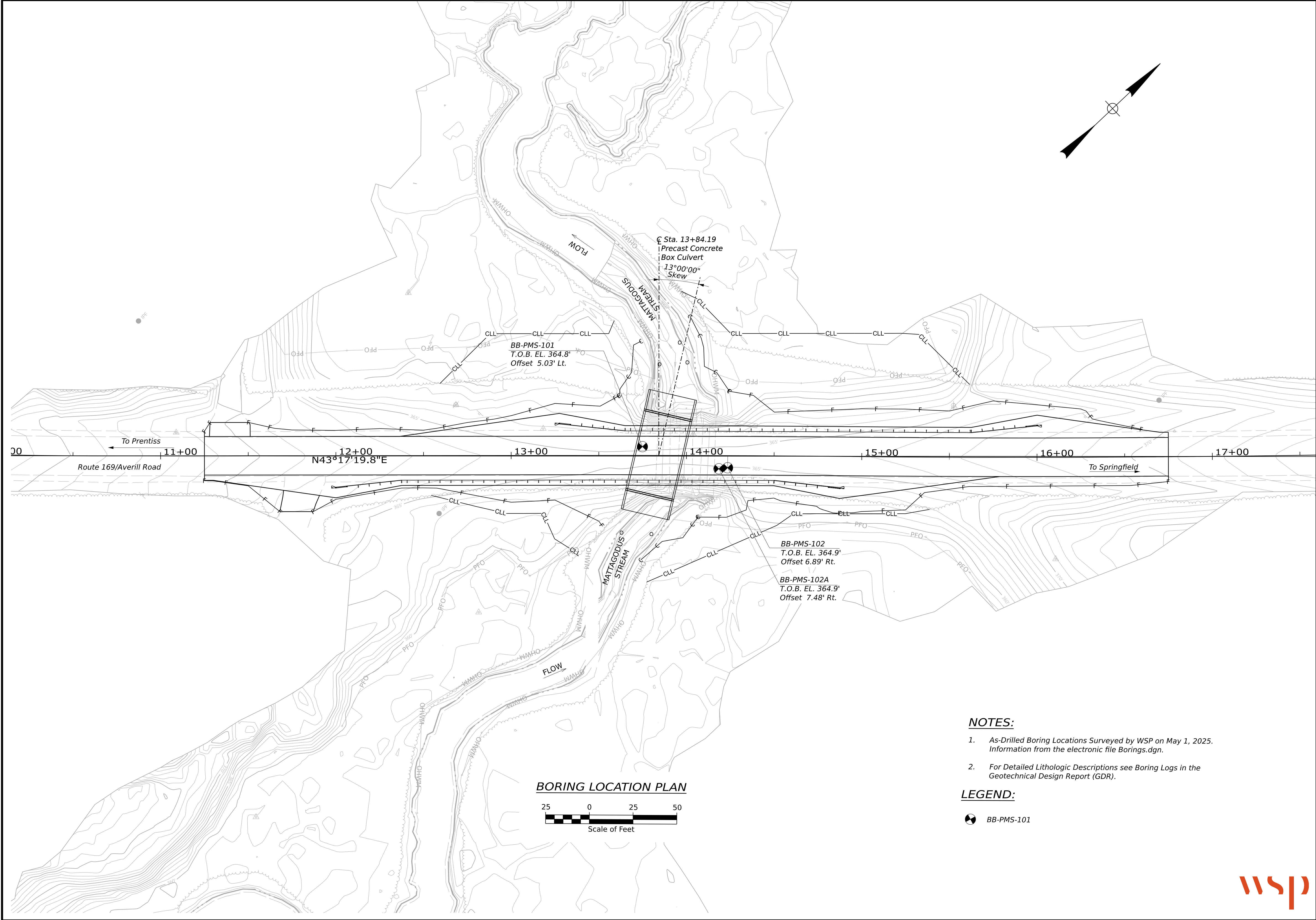
SITE LOCATION MAP
PRENTISS TOWNSHIP T7 R3 NBPP MATTAGODUS BRIDGE #5185
REPLACEMENT, MAINEDOT WIN: 027532.00

PROJECT NO. CONTROL
US0040947.9162 0003-001

REV.
0

FIGURE
1

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



BORING LOCATION PLAN



NOTES:

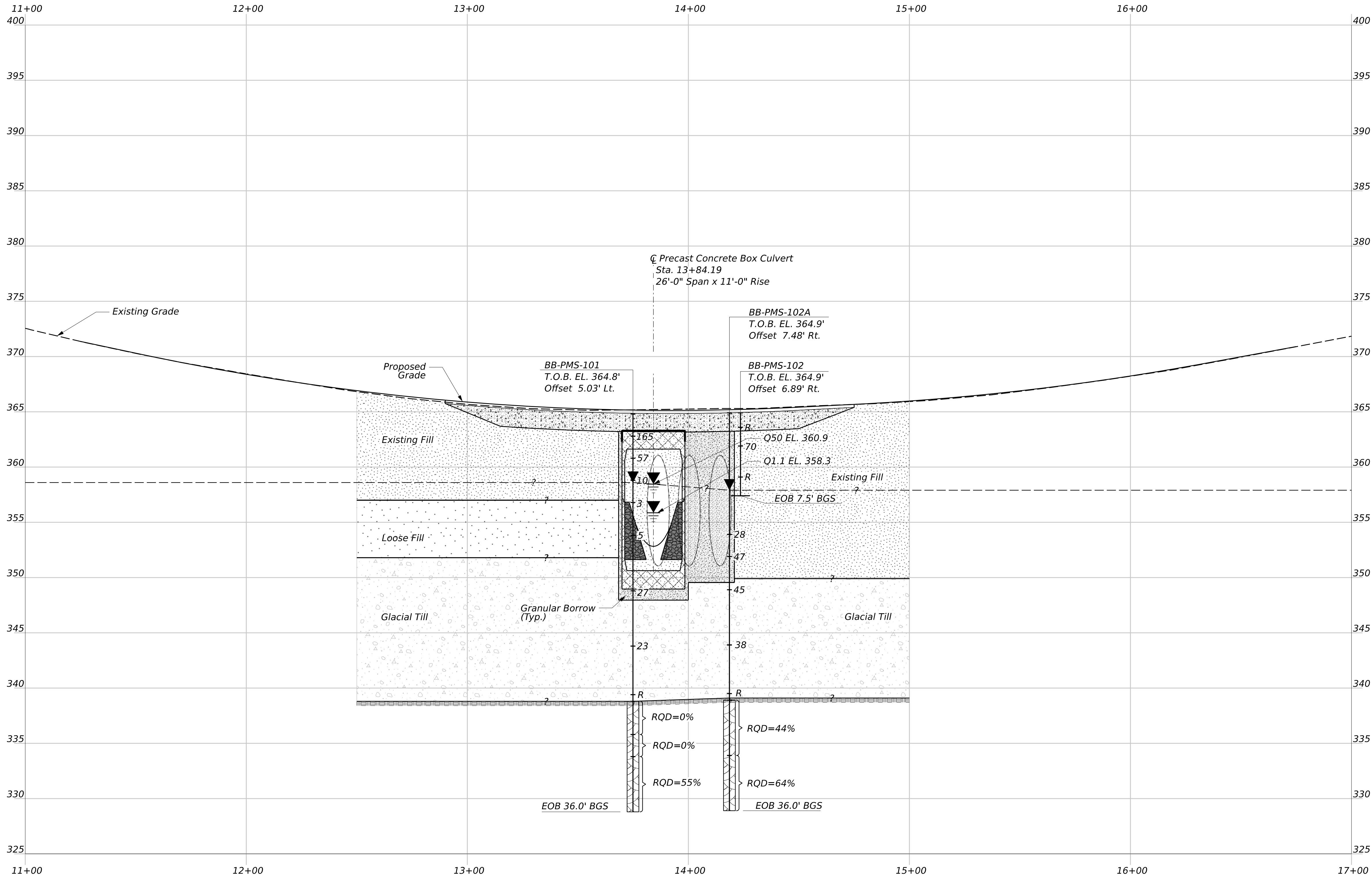
- 1. As-Drilled Boring Locations Surveyed by WSP on May 1, 2025. Information from the electronic file Borings.dgn.
- 2. For Detailed Lithologic Descriptions see Boring Logs in the Geotechnical Design Report (GDR).

LEGEND:

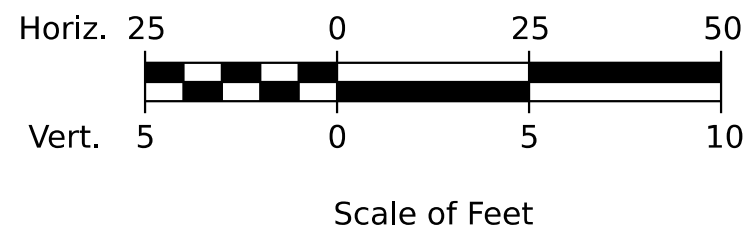
BB-PMS-101



OF 3		SHEET NUMBER		BORING LOCATION PLAN			
2							
MATTAGODUS BROOK BRIDGE NO. 5185		PROJ. MANAGER		BY		DATE	
CROSSING MATTAGODUS STREAM		DESIGN-DETAILED		D. HENDERSON		03/2026	
PRENTISS TWP T7 R3 NBPP		CHECKED-REVIEWED		M. LANDON		03/2026	
		DESIGN-DETAILED2					
		DESIGN-DETAILED3					
		REVISIONS 1				P.E. NUMBER	
		REVISIONS 2					
		REVISIONS 3					
		REVISIONS 4				DATE	
		FIELD CHANGES					



INTERPRETIVE SUBSURFACE PROFILE



Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring Probe
SPT: N60 - Value
(Corrected for
Hammer Efficiency)
Boulder
R Refusal
- ? Strata Interface
Rock Quality
Designation (RQD)
of Bedrock Core
Sample
EOB End of Boring Depth
Below Ground
Surface (BGS)

LEGEND

Brown, Very Loose to Very
Dense, Coarse to Fine Sand,
Little to Some Silt, Little Fine
to Coarse Gravel (Fill)

Grey, Very Loose to Loose,
Fine to Medium Sand, Little to
Some Silt, Trace Wood
Fragments (Loose Fill)

Grey to Dark Grey, Medium
Dense, Coarse to Fine Gravel,
Trace Sand to Sandy, Trace to
Little Silt (Glacial Till)

Grey, Fine Grained, Phyllite
(Bedrock)

Existing Ground Surface
Existing Ground
Water Elevations
Existing Ground
Water Surface
Design Water Level
Existing Bridge Structure
Proposed Bridge Substructure
and Superstructure

- NOTES:
- As-Drilled Boring Locations Surveyed by WSP on May 1, 2025. 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. 2753200		WIN 27532.00	
MATTAGODUS BROOK BRIDGE NO. 5185		CROSSING MATTAGODUS STREAM		PRENTISS TWP T7 R3 NBPP		INTERPRETIVE SUBSURFACE PROFILE	
SHEET NUMBER		3		OF		3	

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> trace little some adjective (e.g. Sandy, Clayey)	<u>Portion of Total (%)</u> 0 - 10 11 - 20 21 - 35 36 - 50																																								
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.																																										
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.																																										
	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		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 N₆₀-Value (blows per foot)</u></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>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N₆₀-Value (blows per foot)</u>	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>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N₆₀-Value (blows per foot)</u>																																											
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(little or no fines)	SP	Poorly-graded sands, Gravelly sand, little or no fines.																																												
SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures																																												
	SC	Clayey sands, sand-clay mixtures.																																												
	FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.																																										
CL			Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.																																											
OL			Organic silts and organic Silty clays of low plasticity.																																											
SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.																																											
		CH	Inorganic clays of high plasticity, fat clays.																																											
		OH	Organic clays of medium to high plasticity, organic silts.																																											
	HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.																																											
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level					Rock Quality Designation (RQD): 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> Desired Rock Observations (in this order, if applicable): Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock quality (very poor, poor, etc.) ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12 Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))				<u>Rock Quality</u>	<u>RQD (%)</u>	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100																										
<u>Rock Quality</u>	<u>RQD (%)</u>																																													
Very Poor	≤25																																													
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Good	76 - 90																																													
Excellent	91 - 100																																													
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth																																									

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 #5185 Location: Prentiss, ME				Boring No.: BB-PMS-101 WIN: 027532.00						
Driller: Seaboard Drilling, LLC				Elevation (ft.) 364.84				Auger ID/OD: 4 in						
Operator: Eric Baron				Datum: NAVD88				Sampler: 2 in Standard Split Spoon						
Logged By: Ryan Nagle				Rig Type: Dierich D-50 (SN:367)				Hammer Wt./Fall: 140 lbs /30 in						
Date Start/Finish: 14:37 3/17-10:45 3/18/25				Drilling Method: SSA / Cased Washed				Core Barrel: NQ2						
Boring Location: N:588180.82 E:2235283.03				Casing ID/OD: 4.0/4.5 in and 3.0/3.5 in				Water Level*: 6.2 ft at 08:30 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 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected						
								Ty = 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						
Sample Information														
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.		
0							SSA	364.3		6 inches of Asphalt Pavement	0.5			
	1D	24/19.2	1.00 - 3.00	20-46-53-40	99	165				Brown, dry, very dense, coarse to fine SAND, little coarse gravel, little silt, poorly graded (FILL).				
	2D	24/21.6	3.00 - 5.00	24-18-16-10	34	57				Brown, moist, dense, coarse to fine SAND, little medium to fine gravel, little silt, well graded (FILL).				
5	3D	24/14.4	5.00 - 7.00	6-4-2-3	6	10				Brown, moist, loose, trace to some coarse to fine SAND, little fine gravel, little silt (FILL).	WC = 9.0% Fines = 13.1% A-1-b (0), SM			
	4D	24/21.6	7.00 - 9.00	2-1-1-1	2	3				Light brown, wet, very loose, trace to some medium to fine SAND, little fine gravel, some silt (FILL). Grey, wet, very loose, fine SAND, little to some silt (FILL).	WC = 30.2% Fines = 33.7% A-2-4 (0), SM			
							PUSH			Grey, wet, very loose, fine to medium SAND, little to some silt, trace wood fragments (FILL).				
10	5D	24/13.2	10.00 - 12.00	2-1-2-1	3	5				Driller Notes: Change in strata inferred due to refusal of hydraulic push at 13 ft bgs.	13.0			
								76		Grey, wet, medium dense, coarse to fine GRAVEL, little silt, trace coarse sand, well graded (GLACIAL TILL).				
								90						
15	6D	24/10.8	15.00 - 17.00	8-8-8-9	16	27	OPEN			Grey, moist, medium dense, trace to some fine to coarse SAND, some fine gravel, trace silt, well graded (GLACIAL TILL).	WC = 14.9% Fines = 8.1% A-1-b (1), SW-SM			
20	7D	24/6	20.00 - 22.00	5-5-9-9	14	23								
25														
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 8:30 was taken after settling overnight with the bottom of casing at 15 ft bgs.														
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2				
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-PMS-101				

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: North Bridge Bundle (NBB), Bridge #5185 Location: Prentiss, ME				Boring No.: BB-PMS-102A WIN: 027532.00																						
Driller: Seaboard Drilling, LLC				Elevation (ft.) 364.89				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: 9:47 3/17/25-1:39 3/17/25				Drilling Method: SSA / Cased Washed				Core Barrel: NQ2																						
Boring Location: N:588207.92 E:2235322.15				Casing ID/OD: 4.0/4.5 in and 3.0/3.5 in				Water Level*: 7.0 ft at 12:25 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 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																		
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th></th></tr></table>										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	
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	364.4		6.5 inches of asphalt pavement. Offset of BB-PMS-102. Auger without sampling to 10 ft.	WC = 17.2% Fines = 8.6% A-1-b (1), SW-SM																		
											See BB-PMS-102 for descriptions.																			
5											Driller Note: Cobbly from 5 to 9 ft bgs.																			
10	1D	24/7.2	10.00 - 12.00	21-9-8-12	17	28	20		354.9		Brown, wet, medium dense, little to some fine to coarse SAND, some fine gravel, trace silt, well-graded (FILL).																			
							30																							
	2D	24/9.6	12.00 - 14.00	10-9-19-12	28	47	105				Brown, wet, dense, fine to coarse GRAVEL, little fine to coarse sand, little silt, wood fragments (FILL).																			
							OPEN				Driller Notes: Woody debris drilled out from 12.6 to 14.0 ft bgs																			
15	3D	24/13.2	15.00 - 17.00	40-15-12-9	27	45			349.9		Grey, wet, dense, trace to some coarse to fine Sandy GRAVEL, trace to little fine to coarse sand, little silt (GLACIAL TILL).	Fines = 12.6% A-1-a (0), GM																		
20	4D	24/8.4	20.00 - 22.00	11-11-12-7	23	38					Grey to brown, wet, dense, little fine to coarse Gravelly SAND, trace to some coarse to fine gravel, little silt (GLACIAL TILL).																			
25																														
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 3/17/25 at 12:25 was made before the start of rock coring with bottom of casing at 25.8 ft bgs.																														
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.											Page 1 of 2																			
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.											Boring No.: BB-PMS-102A																			

APPENDIX B

Rock Core Photograph

APPENDIX B
ROCK CORE PHOTOGRAPHS
BRIDGE #5185 AT MATTAGODUS BROOK, PRENTISS TOWNSHIP T7 R3 NBPP, MAINE
MAINEDOT WIN 027532.00

Boring	Run	Depth Below Surface	Recovery		RQD		Rock Type	Box Row	Date Cored
		Feet	Feet	%	Feet	%			
BB-PMS-102A	R1	26.0 - 31.0	3.9 - 5.0	78	2.2 - 5.0	44	PHYLLITE	Row 1	3/17/2025
	R2	31.0 - 36.0	4.3 - 5.0	86	3.2 - 5.0	64	PHYLLITE	Row 2	3/17/2025
BB-PMS-101	R1	26.0 - 29.0	2.9 - 3.0	97	0.0 - 3.0	0	PHYLLITE	Row 3	3/18/2025
	R2	29.0 - 31.0	1.7 - 2.0	83	0.0 - 2.0	0	PHYLLITE	Row 3	3/18/2025
	R3	31.0 - 36.0	4.7 - 5.0	94	2.8 - 5.0	55	PHYLLITE	Row 4	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 Testing Results

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

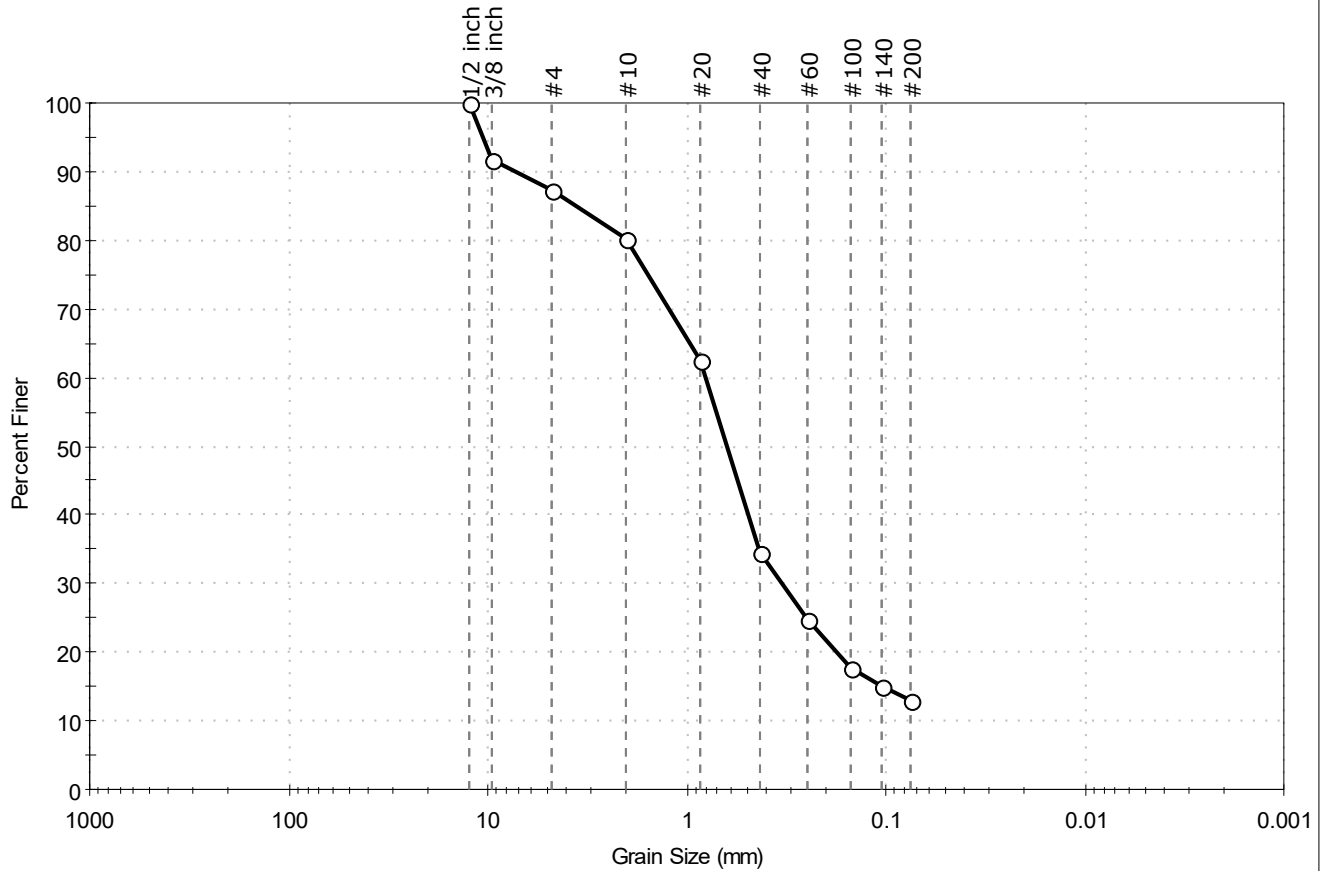
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-PMS-101	S- 3(3D)	5.0-7.0 ft	Moist, brown silty sand	9.0
BB-PMS-101	S- 4(4D)	7.0-9.0 ft	Moist, light brown silty sand	30.2
BB-PMS-101	S- 7(7D)	20.0-22.0 ft	Moist, dark gray sand with silt and gravel	14.9
BB-PMS-101	S- 8(8D)	25.0-25.8 ft	Moist, gray gravel with silt and sand	8.5
BB-PMS-102	S- 3(3D)	5.0-6.0 ft	Moist, dark yellowish brown silty sand with gravel	15.6
BB-PMS-102A	S- 1(1D)	10.0-12.0 ft	Moist, brown sand with silt and gravel	17.2
BB-PMS-102A	S- 4(4D)	20.0-22.0 ft	Moist, grayish brown silty sand with gravel	10.9

Notes: Temperature of Drying : 110° Celsius

Client: WSP USA, Inc.	Project No: GTX-320898
Project: MEDOT N. Bridge Bundle	
Location: ME	
Boring ID: BB-PMS-101	Sample Type: Jar
Sample ID: S-3(3D)	Test Date: 04/24/25
Depth: 5.0-7.0 ft	Test Id: 812342
Test Comment: ---	Tested By: ajl
Visual Description: Moist, brown silty sand	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.5	74.4	13.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.50	100		
3/8 inch	9.50	92		
#4	4.75	87		
#10	2.00	80		
#20	0.85	63		
#40	0.42	35		
#60	0.25	25		
#100	0.15	18		
#140	0.11	15		
#200	0.075	13		

Coefficients

$D_{85} = 3.5698$ mm $D_{30} = 0.3316$ mm
 $D_{60} = 0.7974$ mm $D_{15} = 0.1069$ mm
 $D_{50} = 0.6224$ mm $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

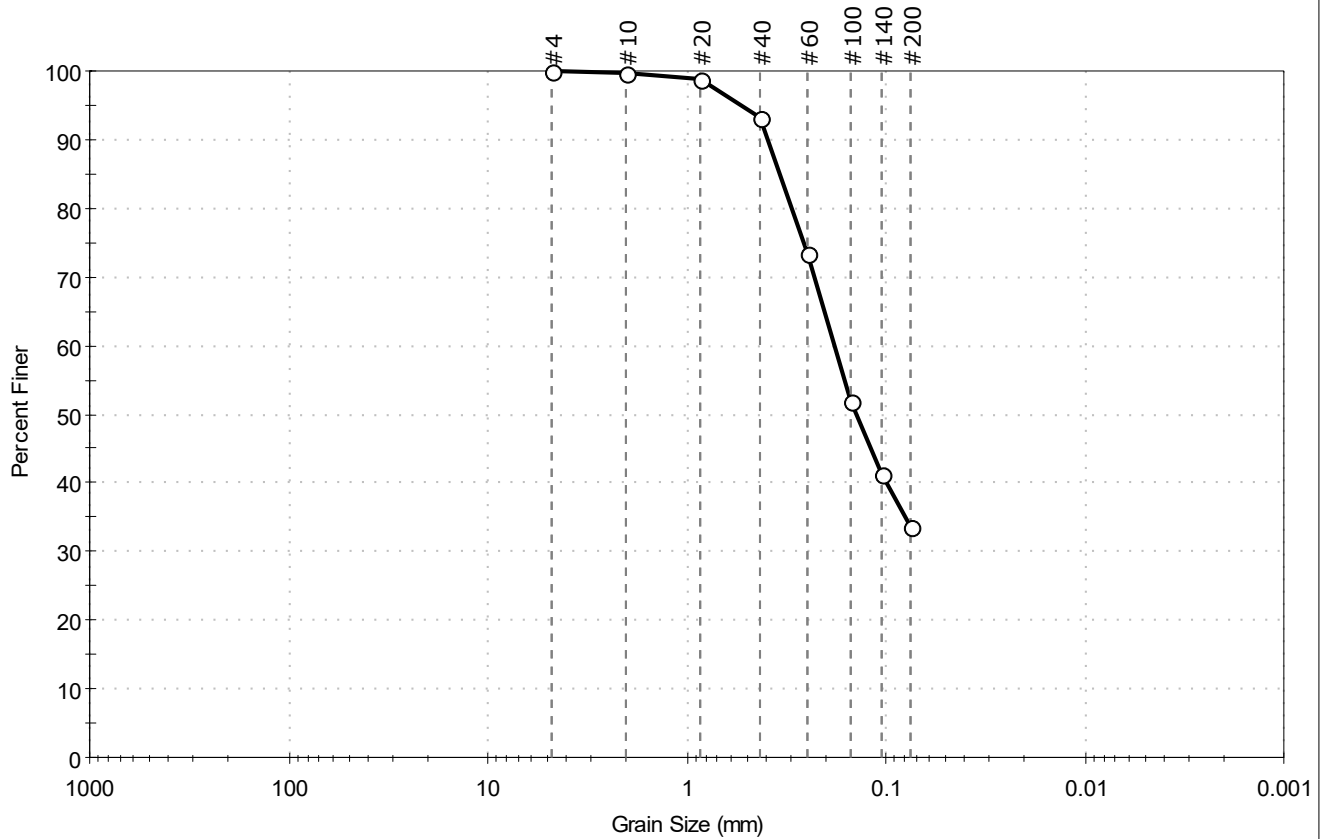
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

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-PMS-101	Sample Type: Jar
Sample ID: S-4(4D)	Test Date: 04/24/25
Depth: 7.0-9.0 ft	Test Id: 812343
Test Comment: ---	Tested By: ajl
Visual Description: Moist, light brown silty sand	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	66.3	33.7

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	93		
#60	0.25	73		
#100	0.15	52		
#140	0.11	41		
#200	0.075	34		

Coefficients

$D_{85} = 0.3406$ mm $D_{30} = \text{N/A}$
 $D_{60} = 0.1820$ mm $D_{15} = \text{N/A}$
 $D_{50} = 0.1413$ mm $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

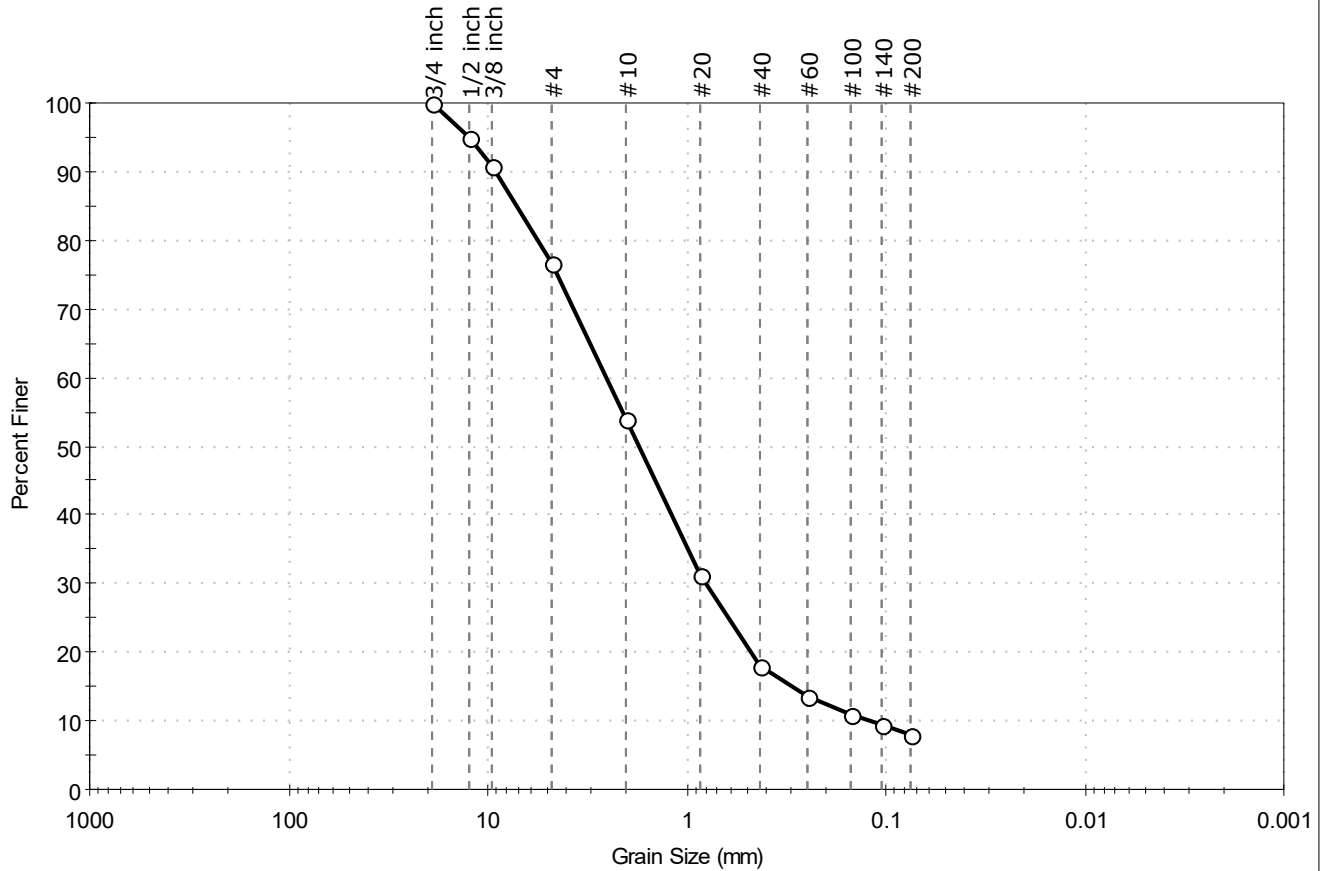
AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
 Sand/Gravel Hardness : ---

Client: WSP USA, Inc.	Project No: GTX-320898	
Project: MEDOT N. Bridge Bundle		
Location: ME		
Boring ID: BB-PMS-101	Sample Type: Jar	Tested By: ajl
Sample ID: S-7(7D)	Test Date: 04/24/25	Checked By: ank
Depth: 20.0-22.0 ft	Test Id: 812344	
Test Comment: ---		
Visual Description: Moist, dark gray sand with silt and gravel		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	23.2	68.7	8.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	95		
3/8 inch	9.50	91		
#4	4.75	77		
#10	2.00	54		
#20	0.85	31		
#40	0.42	18		
#60	0.25	14		
#100	0.15	11		
#140	0.11	9		
#200	0.075	8.1		

Coefficients

$D_{85} = 7.1223 \text{ mm}$ $D_{30} = 0.8001 \text{ mm}$
 $D_{60} = 2.5097 \text{ mm}$ $D_{15} = 0.2950 \text{ mm}$
 $D_{50} = 1.7200 \text{ mm}$ $D_{10} = 0.1206 \text{ mm}$
 $C_u = 20.810$ $C_c = 2.115$

Classification

ASTM N/A

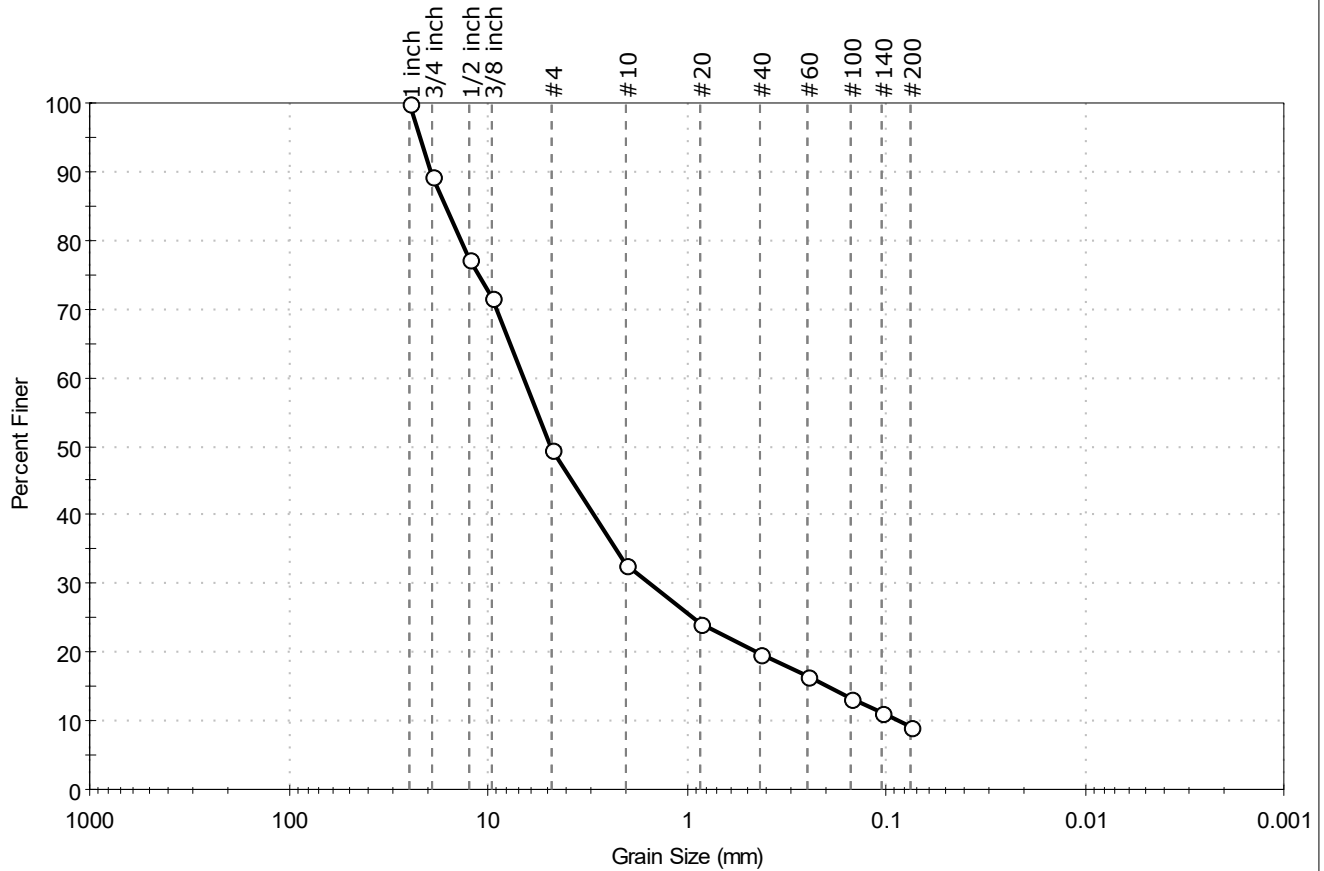
AASHTO Stone Fragments, Gravel and Sand (A-1-b (1))

Sample/Test Description

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

Client: WSP USA, Inc.	Project No: GTX-320898	
Project: MEDOT N. Bridge Bundle		
Location: ME		
Boring ID: PB-PMS-101	Sample Type: Jar	Tested By: ajl
Sample ID: S-8(8D)	Test Date: 04/25/25	Checked By: ank
Depth: 25.0-25.8 ft	Test Id: 812345	
Test Comment: ---		
Visual Description: Moist, gray gravel with silt and sand		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	50.5	40.3	9.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	89		
1/2 inch	12.50	77		
3/8 inch	9.50	72		
#4	4.75	49		
#10	2.00	33		
#20	0.85	24		
#40	0.42	20		
#60	0.25	17		
#100	0.15	13		
#140	0.11	11		
#200	0.075	9.2		

Coefficients

$D_{85} = 16.2720 \text{ mm}$ $D_{30} = 1.5069 \text{ mm}$
 $D_{60} = 6.6071 \text{ mm}$ $D_{15} = 0.1965 \text{ mm}$
 $D_{50} = 4.8275 \text{ mm}$ $D_{10} = 0.0864 \text{ mm}$
 $C_u = 76.471$ $C_c = 3.978$

Classification

ASTM N/A

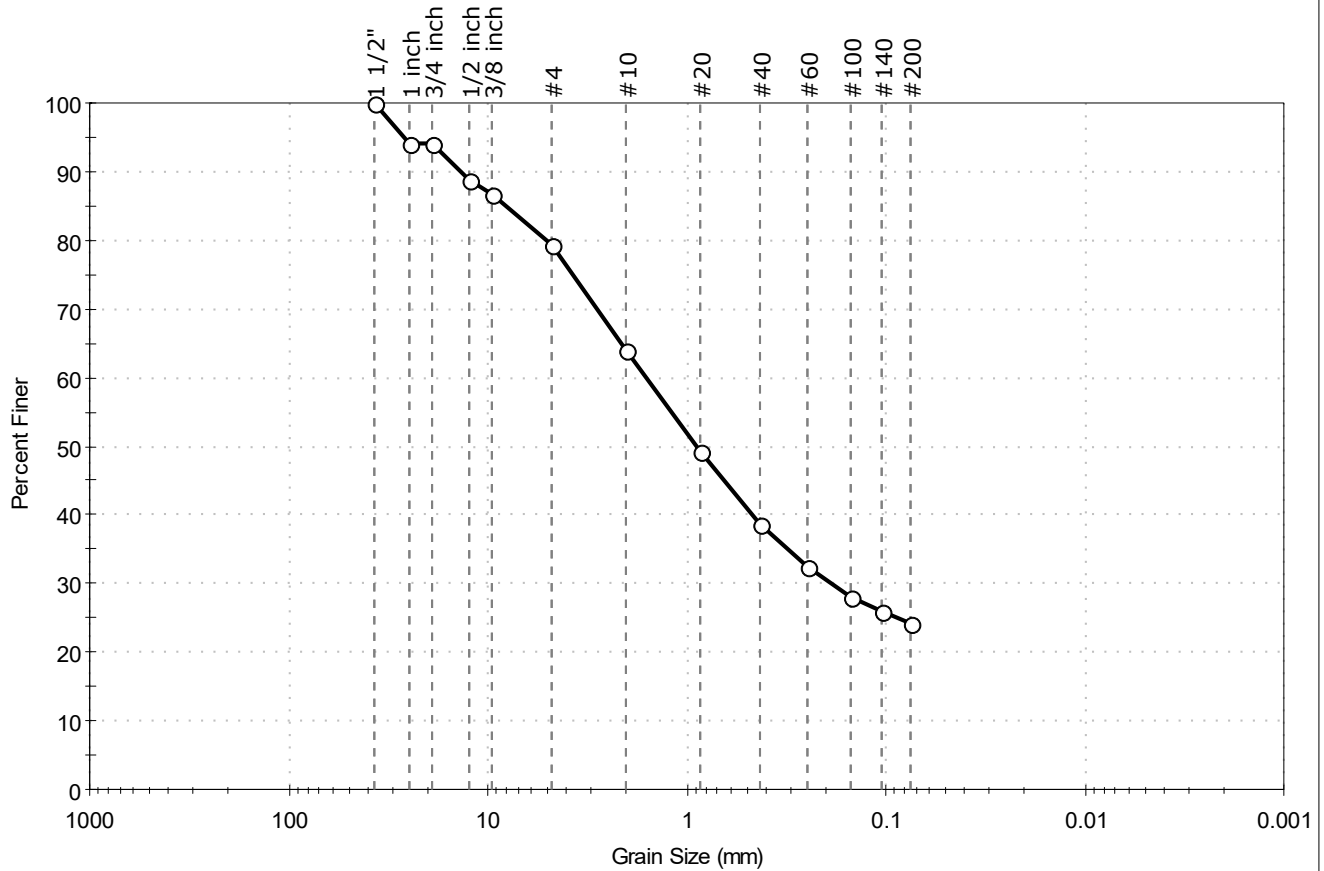
AASHTO Stone Fragments, Gravel and Sand (A-1-a (1))

Sample/Test Description

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

Client: WSP USA, Inc.	Project No: GTX-320898
Project: MEDOT N. Bridge Bundle	
Location: ME	
Boring ID: P8-PMS-102	Sample Type: Jar
Sample ID: S-3(3D)	Test Date: 04/25/25
Depth: 5.0-6.0 ft	Test Id: 812346
Test Comment: ---	
Visual Description: Moist, dark yellowish brown silty sand with gravel	
Sample Comment: ---	

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.5	55.4	24.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2"	37.50	100		
1 inch	25.00	94		
3/4 inch	19.00	94		
1/2 inch	12.50	89		
3/8 inch	9.50	87		
#4	4.75	79		
#10	2.00	64		
#20	0.85	49		
#40	0.42	39		
#60	0.25	32		
#100	0.15	28		
#140	0.11	26		
#200	0.075	24		

Coefficients

$D_{85} = 8.1010 \text{ mm}$ $D_{30} = 0.1890 \text{ mm}$
 $D_{60} = 1.5866 \text{ mm}$ $D_{15} = \text{N/A}$
 $D_{50} = 0.8854 \text{ mm}$ $D_{10} = \text{N/A}$
 $C_u = \text{N/A}$ $C_c = \text{N/A}$

Classification

ASTM N/A

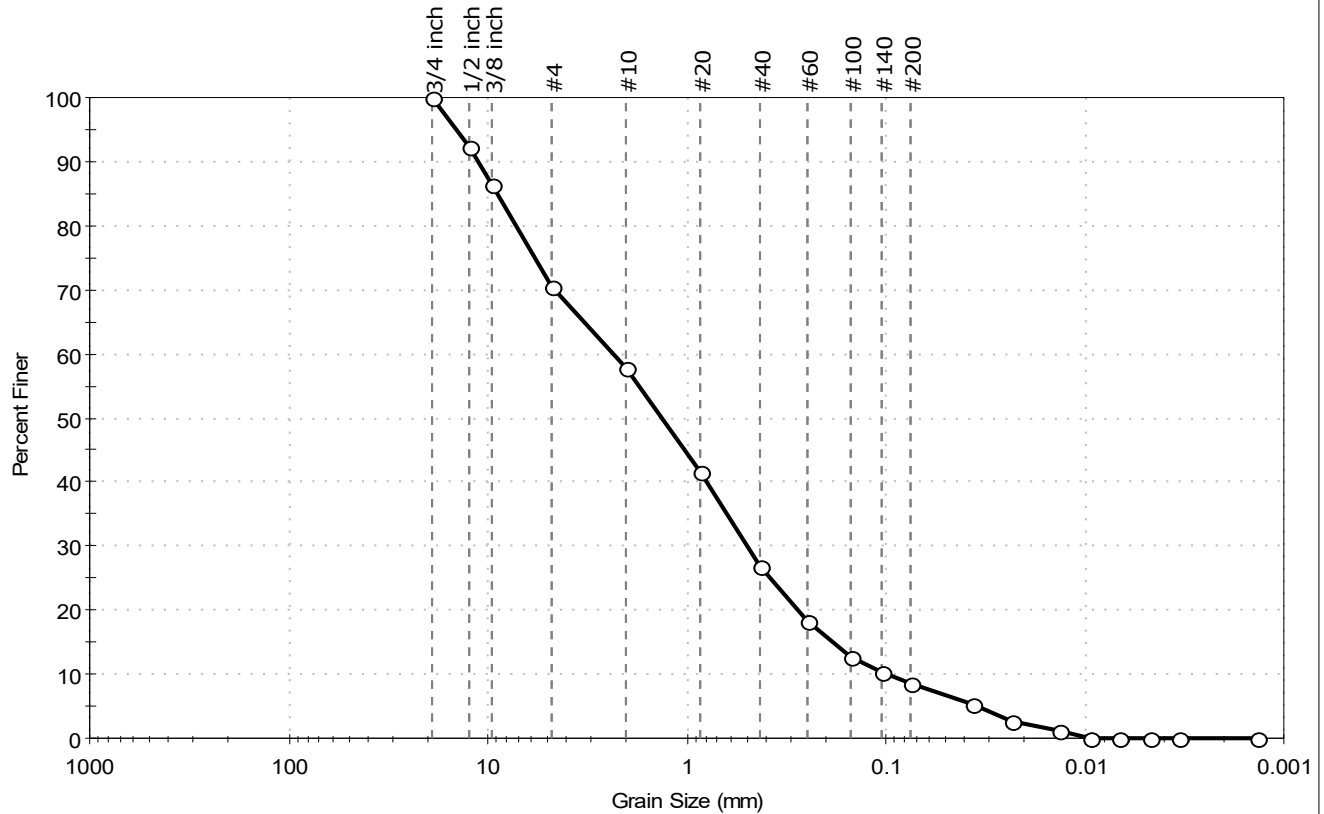
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

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: PB-PMS-102A	Sample Type: Jar
Sample ID: S-1(1D)	Test Date: 04/25/25
Depth: 10.0-12.0 ft	Test Id: 812354
Test Comment: ---	Tested By: ajl
Visual Description: Moist, brown sand with silt and gravel	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	29.4	62.0	8.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	92		
3/8 inch	9.50	86		
#4	4.75	71		
#10	2.00	58		
#20	0.85	42		
#40	0.42	27		
#60	0.25	18		
#100	0.15	13		
#140	0.11	10		
#200	0.075	8.6		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0367	5		
---	0.0232	3		
---	0.0134	1		
---	0.0095	0		
---	0.0067	0		
---	0.0047	0		
---	0.0034	0		
---	0.0014	0		

Coefficients

$D_{85} = 8.9474$ mm $D_{30} = 0.4918$ mm
 $D_{60} = 2.3106$ mm $D_{15} = 0.1839$ mm
 $D_{50} = 1.3222$ mm $D_{10} = 0.1007$ mm
 $C_u = 22.945$ $C_c = 1.039$

Classification

ASTM N/A

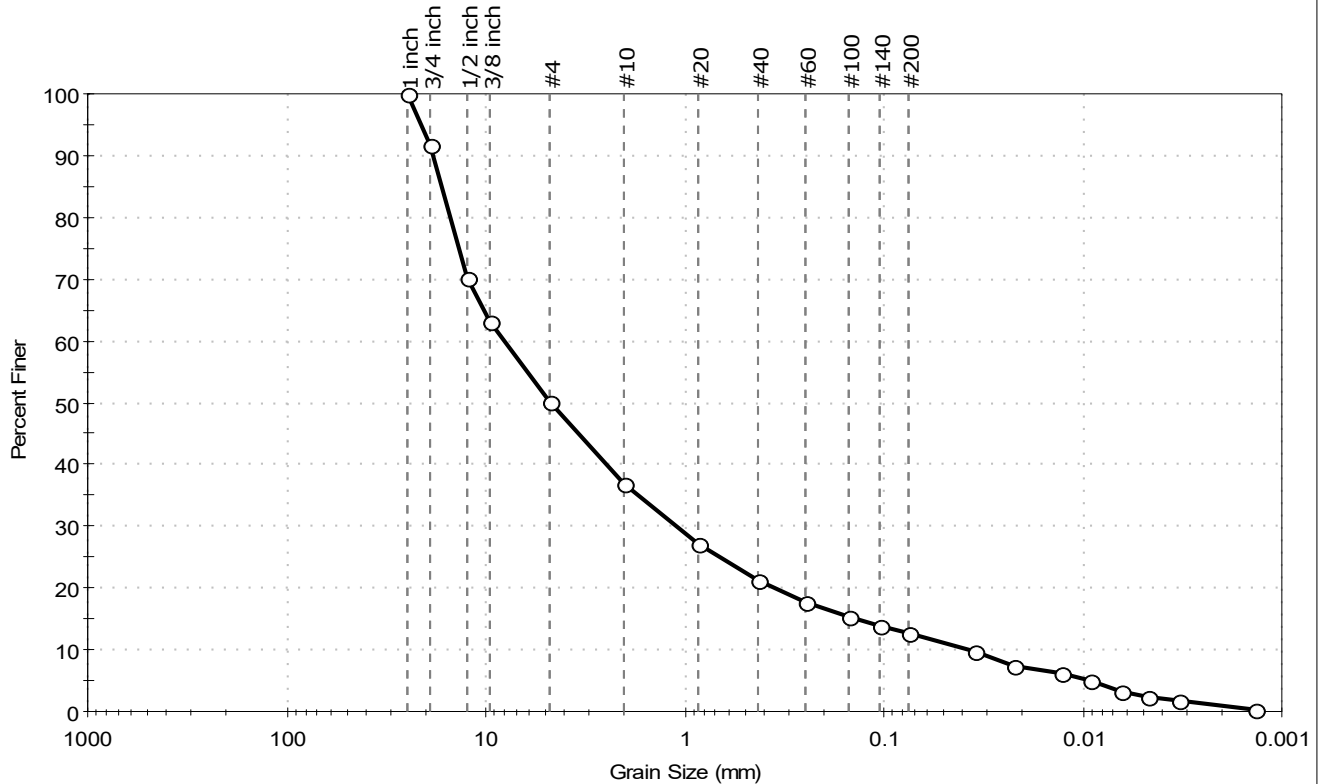
AASHTO Stone Fragments, Gravel and Sand (A-1-b (1))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD
 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: PB-PMS-102A	Sample Type: Jar	Tested By: ajl
Sample ID: S-3(3D)	Test Date: 04/25/25	Checked By: ank
Depth: 15.0-17.0 ft	Test Id: 812355	
Test Comment: ---		
Visual Description: Moist, gray silty gravel with sand		
Sample Comment: ---		

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	50.0	37.4	12.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	92		
1/2 inch	12.50	70		
3/8 inch	9.50	63		
#4	4.75	50		
#10	2.00	37		
#20	0.85	27		
#40	0.42	21		
#60	0.25	18		
#100	0.15	15		
#140	0.11	14		
#200	0.075	13		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0353	10		
---	0.0220	7		
---	0.0129	6		
---	0.0092	5		
---	0.0065	3		
---	0.0047	2		
---	0.0033	2		
---	0.0014	0		

Coefficients

D₈₅ = 16.6312 mm D₃₀ = 1.0971 mm
 D₆₀ = 8.0961 mm D₁₅ = 0.1381 mm
 D₅₀ = 4.7334 mm D₁₀ = 0.0375 mm
 C_u = 215.896 C_c = 3.964

Classification

ASTM N/A

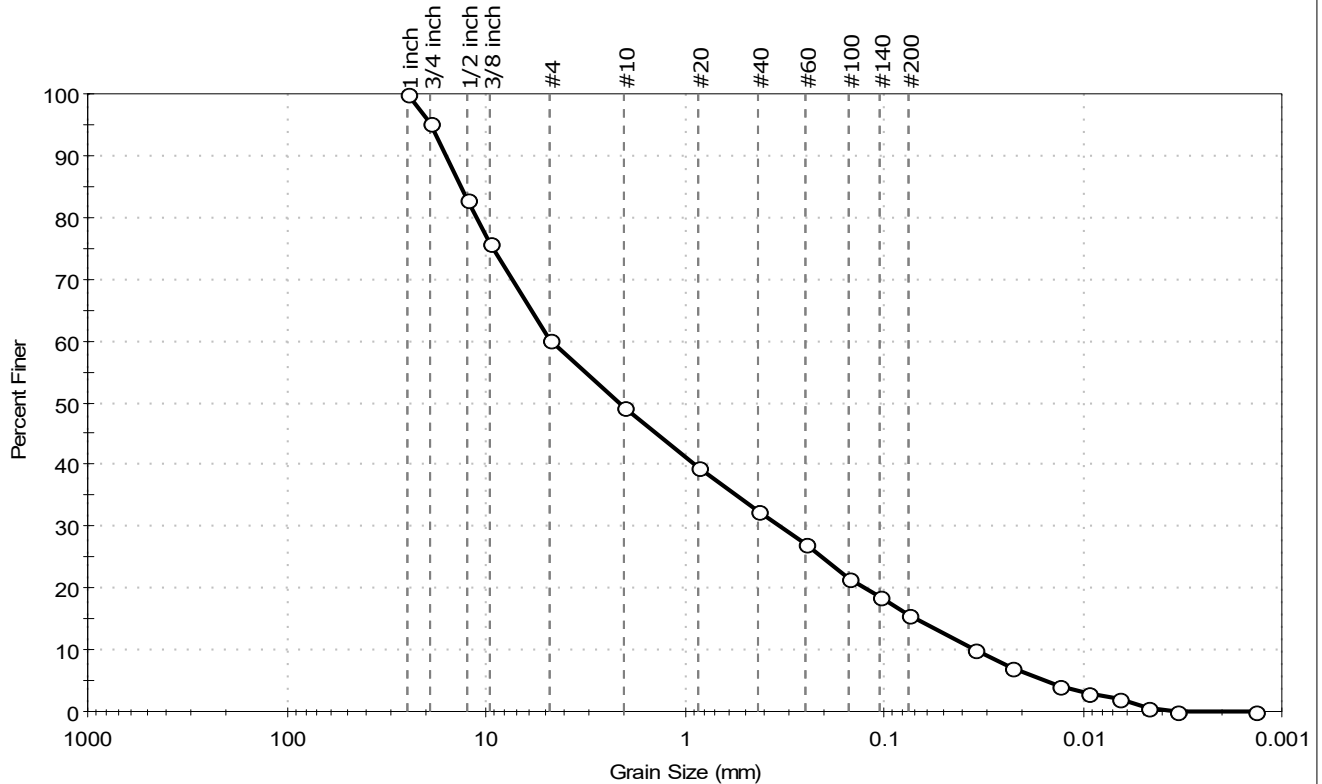
AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD
 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: PB-PMS-102A	Sample Type: Jar
Sample ID: S-4(4D)	Test Date: 04/25/25
Depth: 20.0-22.0 ft	Test Id: 812356
Test Comment: ---	Tested By: ajl
Visual Description: Moist, grayish brown silty sand with gravel	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.7	44.5	15.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	95		
1/2 inch	12.50	83		
3/8 inch	9.50	76		
#4	4.75	60		
#10	2.00	49		
#20	0.85	40		
#40	0.42	32		
#60	0.25	27		
#100	0.15	22		
#140	0.11	19		
#200	0.075	16		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0351	10		
---	0.0227	7		
---	0.0132	4		
---	0.0093	3		
---	0.0066	2		
---	0.0047	1		
---	0.0034	0		
---	0.0014	0		

Coefficients

D₈₅ = 13.4186 mm D₃₀ = 0.3340 mm
 D₆₀ = 4.6712 mm D₁₅ = 0.0678 mm
 D₅₀ = 2.1257 mm D₁₀ = 0.0347 mm
 C_u = 134.617 C_c = 0.688

Classification

ASTM N/A

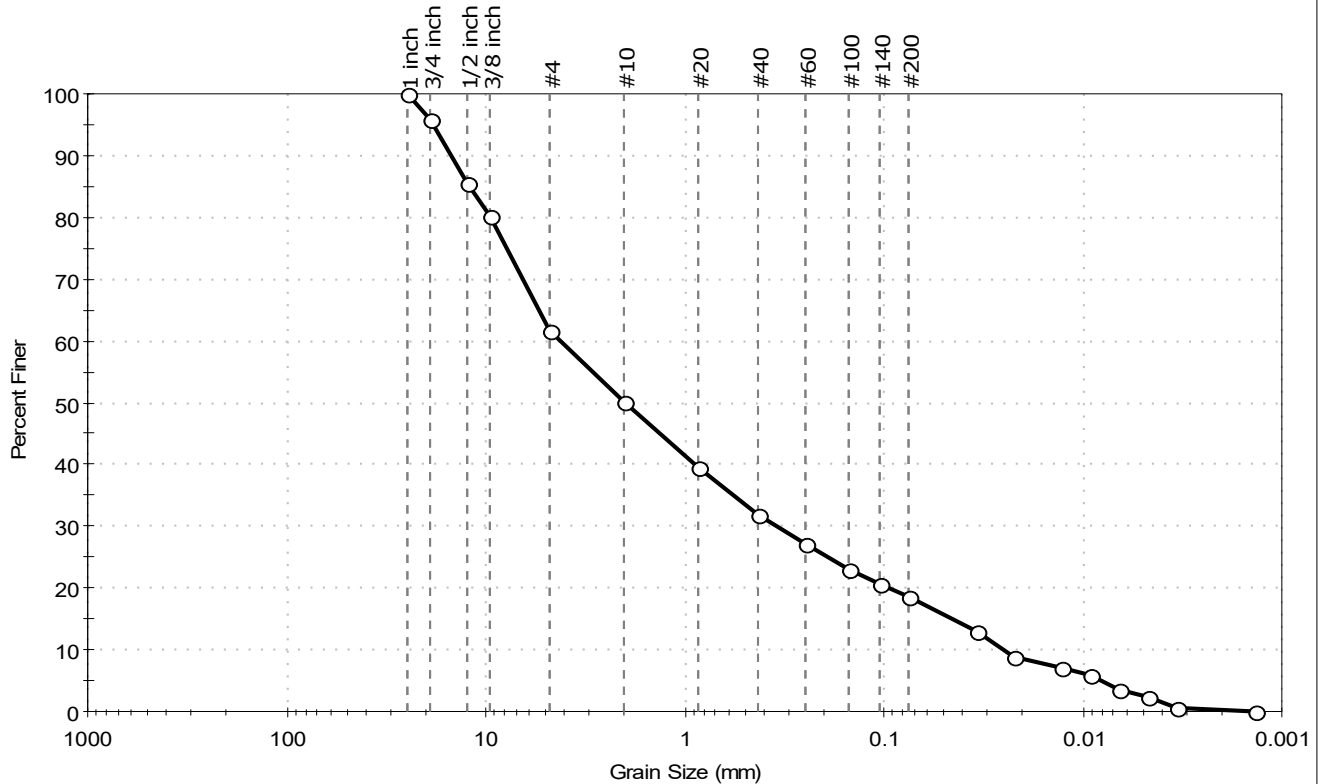
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR
 Sand/Gravel Hardness : HARD
 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: PB-PMS-102A	Sample Type: Jar
Sample ID: S-5(5D)	Test Date: 04/25/25
Depth: 25.0-25.8 ft	Test Id: 812357
Test Comment: ---	Tested By: ajl
Visual Description: Moist, gray silty sand with gravel	Checked By: ank
Sample Comment: ---	

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	38.2	43.2	18.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	96		
1/2 inch	12.50	86		
3/8 inch	9.50	80		
#4	4.75	62		
#10	2.00	50		
#20	0.85	40		
#40	0.42	32		
#60	0.25	27		
#100	0.15	23		
#140	0.11	21		
#200	0.075	19		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0342	13		
---	0.0225	9		
---	0.0130	7		
---	0.0093	6		
---	0.0065	4		
---	0.0047	2		
---	0.0034	1		
---	0.0014	0		

Coefficients

D₈₅ = 12.1047 mm D₃₀ = 0.3419 mm
 D₆₀ = 4.1671 mm D₁₅ = 0.0460 mm
 D₅₀ = 1.9825 mm D₁₀ = 0.0255 mm
 C_u = 163.416 C_c = 1.100

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
 Dispersion Device : Apparatus A - Mech Mixer
 Dispersion Period : 1 minute
 Est. Specific Gravity : 2.65
 Separation of Sample: #200 Sieve

APPENDIX D

Calculations



CALCULATIONS

Date:	7/28/2025	Made by:	JKW
Project No.:	US0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Frost Penetration Depth - Bridge Structures in Fill	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

OBJECTIVE

Determine the depth of frost penetration at the project site.

REFERENCES

1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. WSP Interpreted Subsurface Profile (Figure 3, Preliminary Geotechnical Design Report).
3. WSP geotechnical 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, Loose Fill, and Glacial Till materials around the existing bridge, 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 Fill, Loose Fill, and Glacial Till 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 Prentiss Township, Maine. Using the Maine Design Freezing Index Map (Reference 1, Figure 5-1), the maximum design freezing index at the site location is interpolated to be 1,900 degree-days.

Based on Reference 2, the proposed culvert will be founded within the proposed fill (Granular and Gravel Borrow), existing fill, loose fill, and glacial till, which are coarse-grained soils.

Based on References 3 and 4, the values of water contents in the upper coarse granular fill are: 9.0%, 15.6% and 17.2%, 30.2% in the Loose Fill, and 14.9% and 10.9% in the Glacial Till. Average water content is 16.3% for these samples.

From Reference 1, Table 5-1, with design freezing index = 1,900, coarse-grained soil, and $w = 16.3\%$.

Frost Penetration = 82.5 inches 6.9 feet

CONCLUSIONS

The depth of frost penetration at the proposed culvert site is estimated to be 6.9 feet.



CALCULATIONS

Date:	6/23/2025	Made by:	JKW
Project No.:	US0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Lateral Earth Pressure	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

OBJECTIVE

Determine lateral earth pressure acting on the proposed culvert and wingwalls.

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. Mattagodus Brook Bridge #5185 Material Properties.
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
$\phi_{\text{Loose Fill}} =$	28	degrees
$\phi_{\text{Glacial Till}} =$	36	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
0.53	0.36	2.77	Loose Fill - In Situ
0.41	0.26	3.85	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 and wingwall system proposed for this project, an at-rest lateral earth pressure coefficient of 0.47 is recommended for design.



CALCULATIONS

Date:	7/8/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Modulus of Subgrade Reaction	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

OBJECTIVE

Determine the modulus of subgrade reaction in the glacial till deposit.

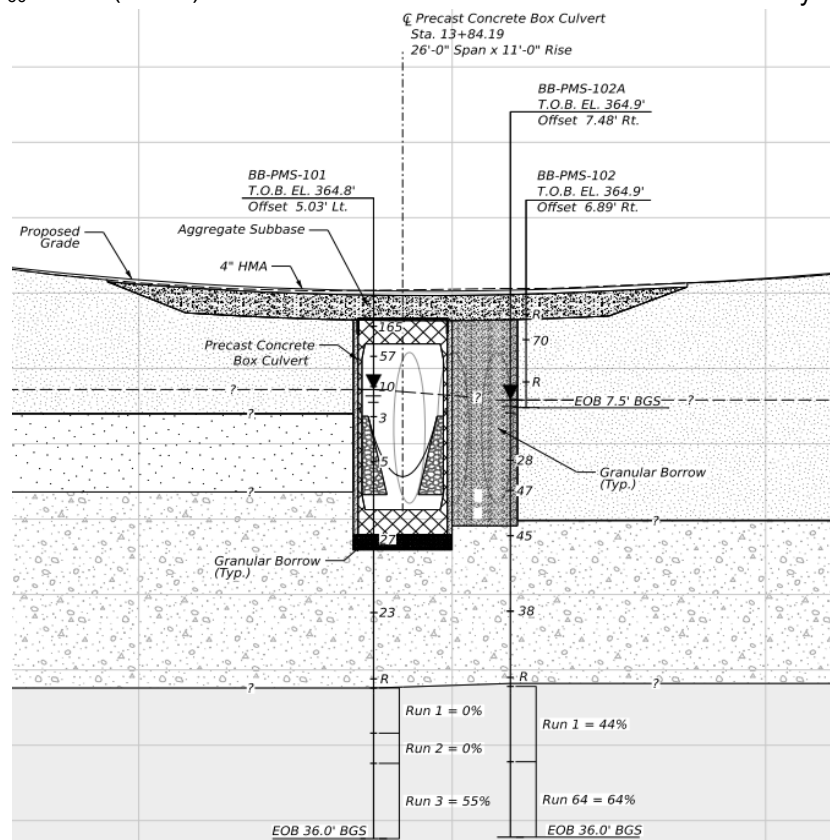
REFERENCES

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. GeoTesting Express laboratory testing results (Appendix C, Geotechnical Design Report).
4. WSP Geotechnical boring logs (Appendix A, Geotechnical Design Report).
5. WSP interpreted subsurface profile (Figure 3, Geotechnical Design Report).
6. WSP. 2025. Mattagodus Brook Bridge #5185 Material Properties.

ASSUMPTIONS

1. The average design N_{60} value for the Glacial Till Deposit. (Ref. 6)
2. Density based on N_{60} value. (Ref. 6)

$N_{60} = 23$
Density = Medium Dense



Interpretive Subsurface Profile (Ref 5)



CALCULATIONS

Date:	7/8/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Modulus of Subgrade Reaction	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

CALCULATION

Determine Modulus of Subgrade Reaction.

Determine Modulus of Subgrade Reaction (K_{VI}) using Figure 8-3 below (Ref. 2). Based on WSP boring logs (Ref. 4), WSP material properties (Ref. 6), and laboratory testing results (Ref. 3), the relative density was calculated using Equation 18.14 of Reference 1 (below). The calculated relative density ranged from 55.5% to 74.5% with an average of 66.2%. The average relative density shows the glacial till to be in the dense range of the chart and the coarse grained soil curve was used to determine (K_{VI}).

$$D_r(\%) = \left[\frac{N_{60} \left(0.23 + \frac{0.06}{D_{50}} \right)^{1.7}}{9} \left(\frac{98}{\sigma'_o} \right) \right]^{0.5} (100) \quad (\text{Ref. 1, Eq 18.14})$$

Sample No.

BB-PMS-101, S-7

$d_{50} = 1.72 \text{ mm}$

$\sigma'_o = 1697 \text{ psf}$

81.3 kN/m^2

$N_{60} = 22$

$D_r(\%) = 55.5$

BB-PMS-102A, S-3

$d_{50} = 4.7334 \text{ mm}$

$\sigma'_o = 1404 \text{ psf}$

67.2 kN/m^2

$N_{60} = 38$

$D_r(\%) = 74.5$

BB-PMS-102A, S-4

$d_{50} = 2.1257 \text{ mm}$

$\sigma'_o = 1742 \text{ psf}$

83.4 kN/m^2

$N_{60} = 36$

$D_r(\%) = 68.6$

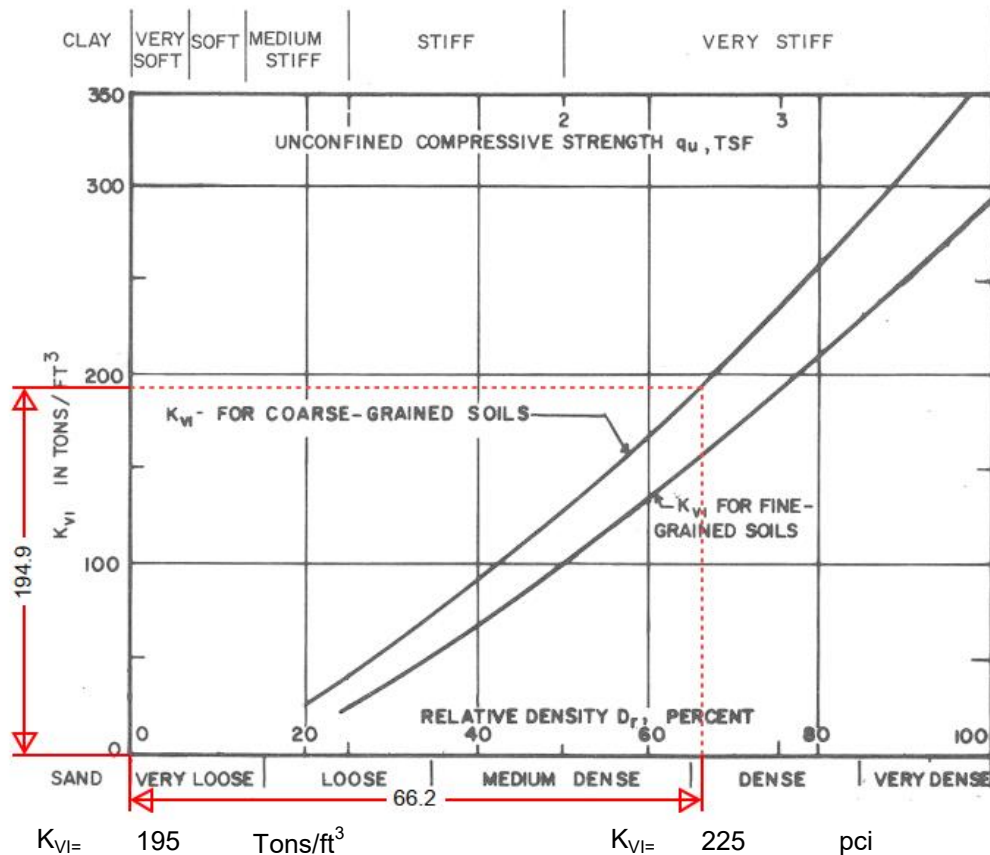
Average $D_r(\%) = 66.2$



CALCULATIONS

Date:	7/8/2025	Made by:	RJN
Project No.:	0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Modulus of Subgrade Reaction	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

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



CONCLUSIONS

The recommended modulus of subgrade reaction is $K = 225$ pci as determined using FHWA GEC-6.



CALCULATIONS

Date:	7/31/2025	Made by:	JKW
Project No.:	US0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Global Stability Analysis	Reviewed by:	MEL
Project:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

OBJECTIVE

Calculate global factor of safety for the embankments at Bridge No. 5185 Station 14+10 (Crossing Mattagodus Brook). The slope stability evaluations were made for failure surfaces with a minimum depth of 1 foot below ground surface, and surfaces with a minimum depth of 5 feet below ground surface to evaluate deeper failure surfaces.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 10th Ed, 2024.
2. State of Maine Department of Transportation. Progress Print: Br 5185 Cross Sections, WIN 028262.00. Dated July 7, 2025.
3. GeoTesting Express laboratory testing results, included as Appendix C and summarized in Table 4 and Table 5 of the Geotechnical Design Report.
4. WSP boring logs (Appendix A, Geotechnical Design Report).
5. Rocscience Inc. Slide2 software package, version 9.038 64 bit, build date February 28, 2025.
6. WSP, Material properties table named "NBB Bridge 5185 Geotechnical Material Properties.xlsx"
7. MaineDOT Bridge Design Guide, prepared by Guertin Elkerton & Associates, August 2003, Updated 2018

ATTACHMENTS

1. Material Properties table.
2. Slide output figures.

ASSUMPTIONS

1. The load applied by the road and traffic for final design conditions is modeled as a 2 foot equivalent load of soil (Reference 1, Table 3.11.6.4-1). Soil unit weight taken as 125 pcf (Reference 7, Section 3.6.8): 2 feet x 125 pcf (Granular Borrow) = 250 psf.
2. A minimum target factor of safety (FS) of $FS \geq 1.3$ for static loading is recommended for potential failure surfaces per AASHTO LRFD and MaineDOT Bridge Design Guide is recommended for all potential failure surfaces.
3. Circular surfaces were analyzed using the Spencer method and auto refine search. Non-circular surfaces were analyzed using the Spencer method and cuckoo search; the search algorithm that produced a lower factor of safety was selected.
4. Surficial failure surfaces evaluate the global minimum FS with surfaces deeper than 1 foot below ground surface. Deep failure surfaces evaluation include the global minimum FS with surfaces deeper than 5 feet below ground surface.
5. The existing stratigraphy and water table surfaces are estimated from samples and groundwater measurements encountered during drilling outlined in Reference 4. Stratigraphy data from Reference 4 was drawn over the roadway cross-section in Reference 2 for interpretive subsurface profile.
6. The existing grading, proposed grading, and construction design features are taken from Reference 2.
7. Infinite Strength conditions with sliding allowed along the layer boundary was assumed for the bedrock layers.
8. Void filled rip rap will be used at the surface of the proposed slope and extend a vertical depth of 3 feet. The remainder of the slope will be filled using granular borrow as the proposed slope fill to cover on the front slope of the existing fill.



CALCULATIONS

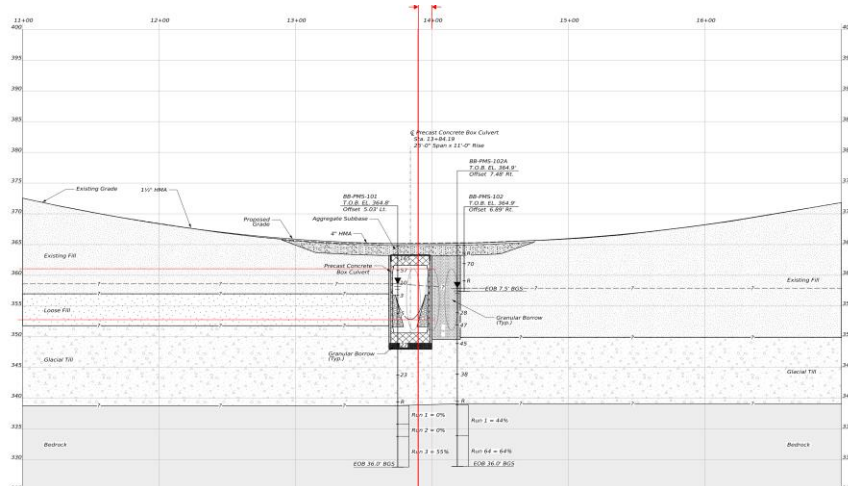
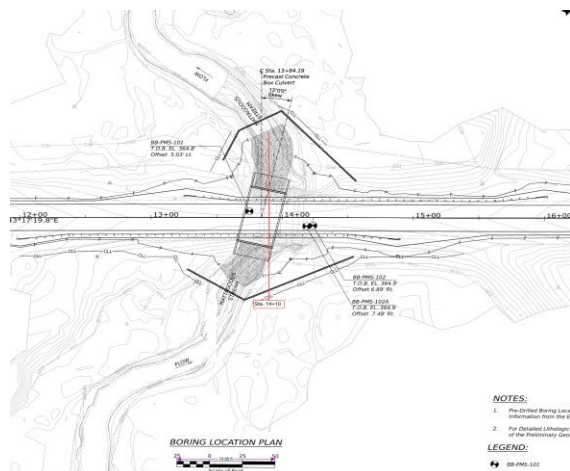
Date: 7/31/2025
Project No.: US0040947.9162 / WIN 027532.00
Subject: Global Stability Analysis
Project: Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME

Made by: JKW

Checked by: LDN

Reviewed by: MEL

9. The area modeled at Sta. 14+10, is located where the existing culverts are in place. The Interpretive Subsurface Profile shows the top of the existing culvert at elevation 361 feet and the existing culvert extends to elevation 351 feet. The existing culverts are being removed and the area is being infilled with Granular Borrow, and is shown as granular borrow in the models.



10. The interpretive subsurface cross section for Sta. 14+10 is based on that developed for Sta. 14+00 for the centerline of the culvert (Figure A.0, attached). The stratigraphy within the Sta. 14+10 cross section shows both Fill and Loose Fill in brown and yellow. The Fill and Loose Fill area are where the existing culvert is located, these materials are removed during the culvert removal process and replaced with granular borrow. The glacial till stratigraphy shown in the models is from the Interpretive Subsurface Cross Section created at Sta. 14+00.

CALCULATION

1. Determine input parameters to build the soil model in Slide.

The material parameters selected for use in the Slide models are shown in the table below.

- The friction angle parameters for the existing fill, loose fill, and glacial till layers are based on empirical correlation to the average of the N_{60} -values encountered in nearby borings for each layer (Ref. 4) using Peck, Hanson, & Thornburn (1974) from Wolff (1989). Friction Angle = $27.1 + 0.3 \cdot N_{60} - 0.00054 \cdot (N_{60})^2$
- The unit weight parameters for the existing fill, loose fill, glacial till, and bedrock layers were selected based on local engineering experience (Ref. 6).
- The friction angle and unit weight parameters for the MaineDOT Standard Specification construction materials (Granular Borrow and Gravel Borrow) were selected from Table 3-3 of Ref. 7.
- Where it applies, structures such as sidewalks, jersey barriers, and underdrain systems shown on the cross sections were not included in the Slide2 models for the slope stability analysis.



CALCULATIONS

Date:	7/31/2025	Made by:	JKW
Project No.:	US0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Global Stability Analysis	Reviewed by:	MEL
Project:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

Table 1: In Situ Soil Properties (Ref. 6)

Station Number	Borings Present	Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle ¹ (°)
14+10	BB-PMS-101, BB-PMS-102, and BB-PMS-102A	Existing Fill	125	Mohr-Coulomb	0	35
		Loose Fill	115	Mohr-Coulomb	0	28
		Glacial Till	130	Mohr-Coulomb	0	36

Note: ¹Friction angles for the loose fill and glacial till were selected based off values from WSP boring logs (Reference 4) and calculated in WSP's material properties table (Reference 6).

Table 2: In Situ Rock Material

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)
Bedrock	165	Infinite Strength	-	-

Table 3: Construction Materials

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle ¹ (°)
Pavement	140	Mohr-Coulomb	0	45
Granular Borrow	125	Mohr-Coulomb	0	32
Aggregate Subbase (Gravel Borrow)	135	Mohr-Coulomb	0	36
Void Filled Rip Rap	135	Mohr-Coulomb	0	40

Note: ¹Friction angles for the granular borrow and gravel borrow fill materials were selected based off values provided in Table 3-3 of the MaineDOT Bridge Design Guide (Ref. 7).

Table 4: Proposed Geometry and Stratigraphy

Station Number	Proposed Structure	Attachment
14+10	NBB Bridge 5185 Roadway Fill	A.0

Station Selection: This station and slopes were selected because they show the greatest extent of fill placement, steepest slope angles, and subsurface soil layering and material properties that are likely to result in low global stability factors of safety.

Subsurface soils: The layering used for the Slide2 models were determined using information from nearby borings and are shown in attachment A.0 as shown in Table 4. Where layer changes were not observed within a split spoon sample or in drill cuttings, wash water color, or drill rig behavior, layer changes were assumed to occur at the start point of the bottom sample, based on engineering interpretation. For sections showing two borings, outside the bounds of the two borings, layer boundaries were assumed to stay at constant elevations equal to the elevation of the boundary at the nearest of the two borings. Layer boundaries were linearly interpolated in the area between borings.

Water Table: The water tables used in the Slide2 models were estimated from groundwater measurements made during the subsurface investigation and interpreted based on the model stratigraphy and geometry.



CALCULATIONS

Date:	7/31/2025	Made by:	JKW
Project No.:	US0040947.9162 / WIN 027532.00	Checked by:	LDN
Subject:	Global Stability Analysis	Reviewed by:	MEL
Project:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

2. Use the soil layer parameters from Table 1 to analyze the slope stability with Slide2.

The soil layer properties above were used to analyze the following scenario at two different locations: Averill Road Bridge No. 5185 14+10 North and South Embankment Fills.

Results of the stability analyses are summarized as follows: Table 6: Surficial failures in fill under static loading, Table 7: Deep failures within in situ soil under static loading.

Table 6: Surficial Failures Through the Proposed Fill - Static Loading

Structure	Station	Analysis Method	Factor of Safety	Figure # (Attach. 1)
14+10 Embankment Fill	14+10 (South Slope)	Circular - Spencer ¹	1.46	-
		Non Circular - Spencer ¹	1.42	A.1
	14+10 (North Slope)	Circular - Spencer ¹	1.41	-
		Non Circular - Spencer ¹	1.42	A.2

Note: ¹Factor of safety for surficial failures deeper than 1 foot. Shallower surfaces were not included in the analyses.

Table 7: Deeper failures surfaces - Static Loading

Structure	Station	Analysis Method	Factor of Safety	Figure # (Attach. 1)
14+10 Embankment Fill	14+10 (South Slope)	Circular - Spencer ¹	1.73	-
		Non Circular - Spencer ¹	1.69	A.3
	14+10 (North Slope)	Circular - Spencer ¹	1.48	-
		Non Circular - Spencer ¹	1.47	A.4

Note: ¹Factor of safety for failures deeper than 5 feet. Shallower surfaces were not included in the analyses.

CONCLUSIONS

Static analyses; shallow failure surfaces: The proposed north and south embankment fills at Station 14+10 produced global stability FS > 1.3 (1.3 is the minimum recommended) for potential slope failures in the proposed fill areas under static conditions.

Static analyses; deeper failure surfaces: The surfaces deeper than 5 feet analyzed, the proposed north and south embankment fills at Station 14+10 produced global stability FS > 1.3 (1.3 is the minimum recommended) for potential slope failures through the proposed fill soils and fill and in situ soils under static conditions.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu	Ru Value
Pavement		140	Mohr- Coulomb	0	45		None			0
Aggregate subbase		135	Mohr- Coulomb	0	36		Water Table	Custom	1	
Void Filled Rip Rap		135	Mohr- Coulomb	0	40		Water Table	Custom	1	
Slope Fill (Granular Borrow)		125	Mohr- Coulomb	0	32		Water Table	Custom	1	
Fill (Existing)		125	Mohr- Coulomb	0	35		Water Table	Custom	1	
Loose Fill		115	Mohr- Coulomb	0	28		Water Table	Custom	1	
Glacial Till		130	Mohr- Coulomb	0	36		Water Table	Custom	1	
Bedrock		165	Infinite Strength			Yes	None			0



Project

North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine

Analysis Description

Material Properties Table

Drawn By

JKW

Checked By

LDN

Reviewed By

MEL

Scale

1:122

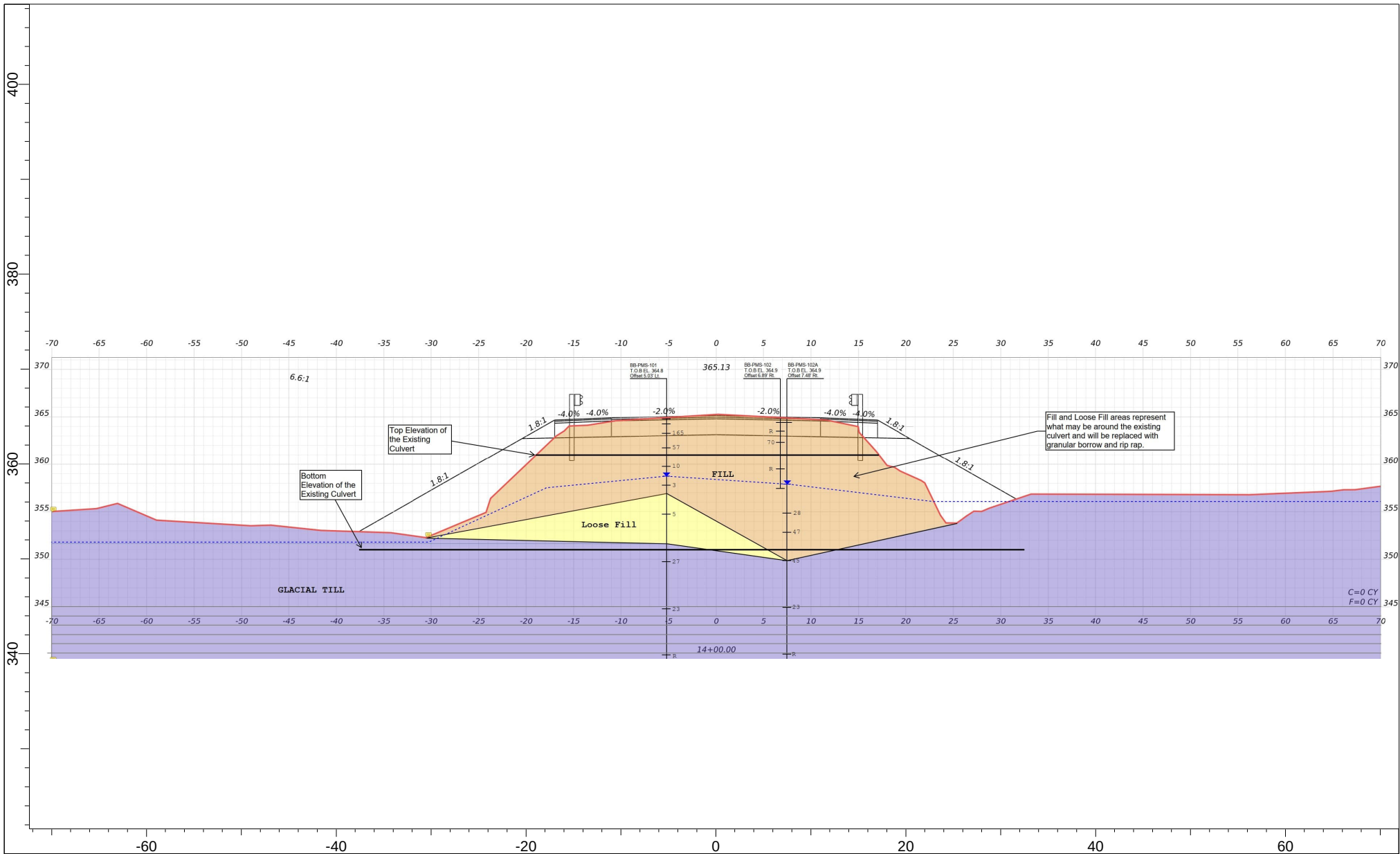
Date

7/30/2025

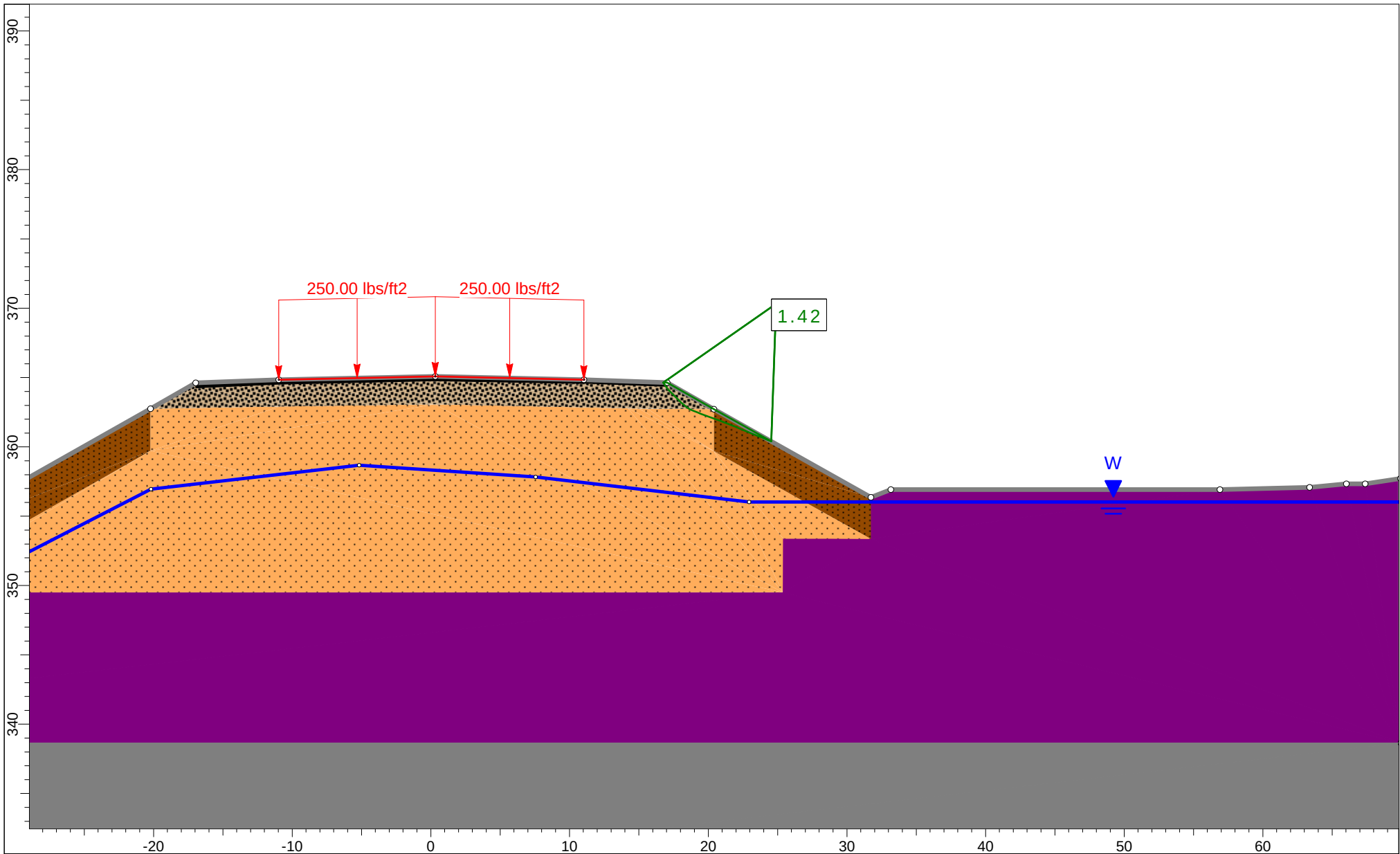
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14+10.00.slmd

Figure 1

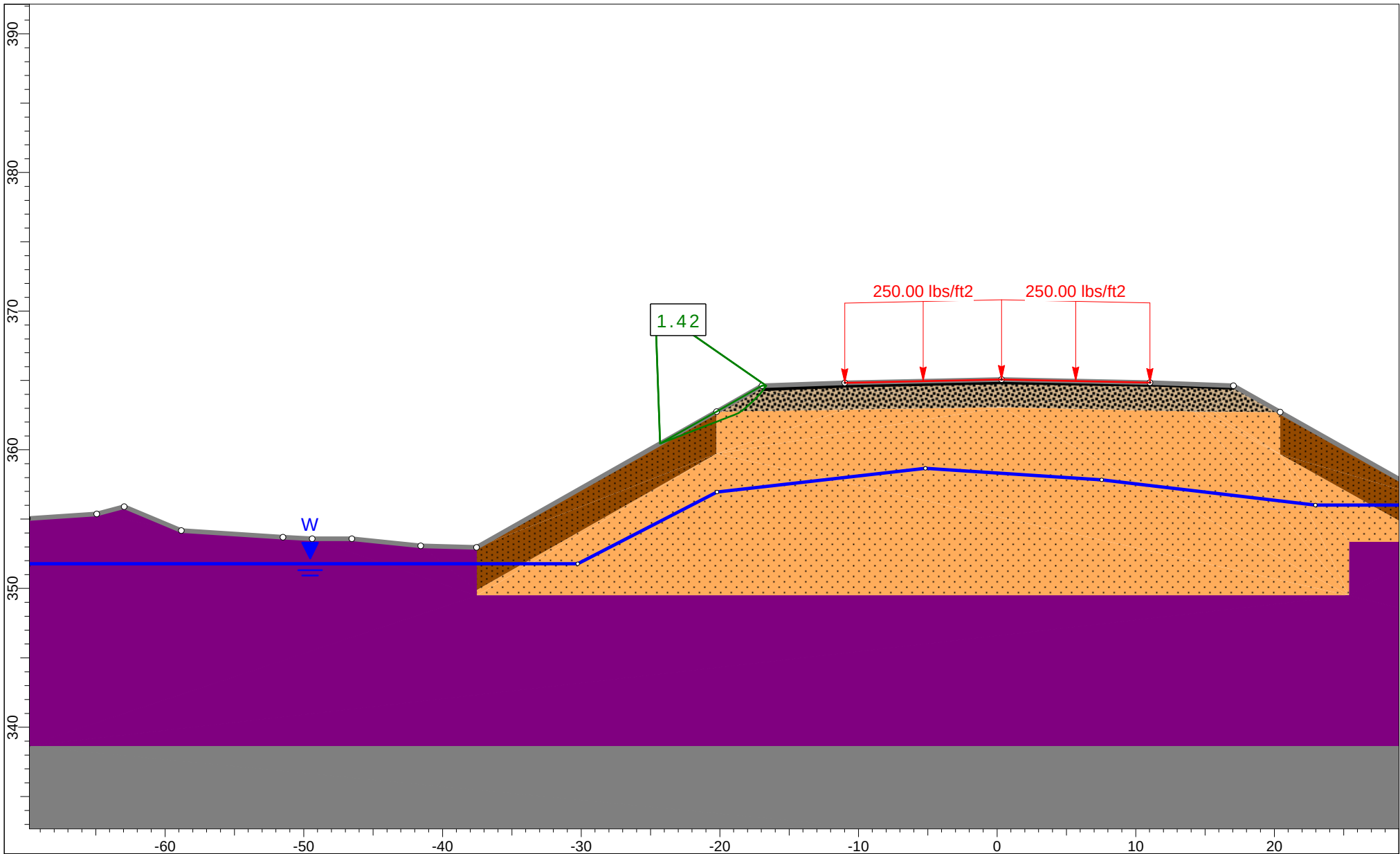


Project		North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine			
Analysis Description		Proposed Geometry			
Drawn By	JKW	Checked By	-	Reviewed By	MEL
Date	8/11/2025	File Name	14+10.00.slmd	Scale	1:168
					Figure A.0

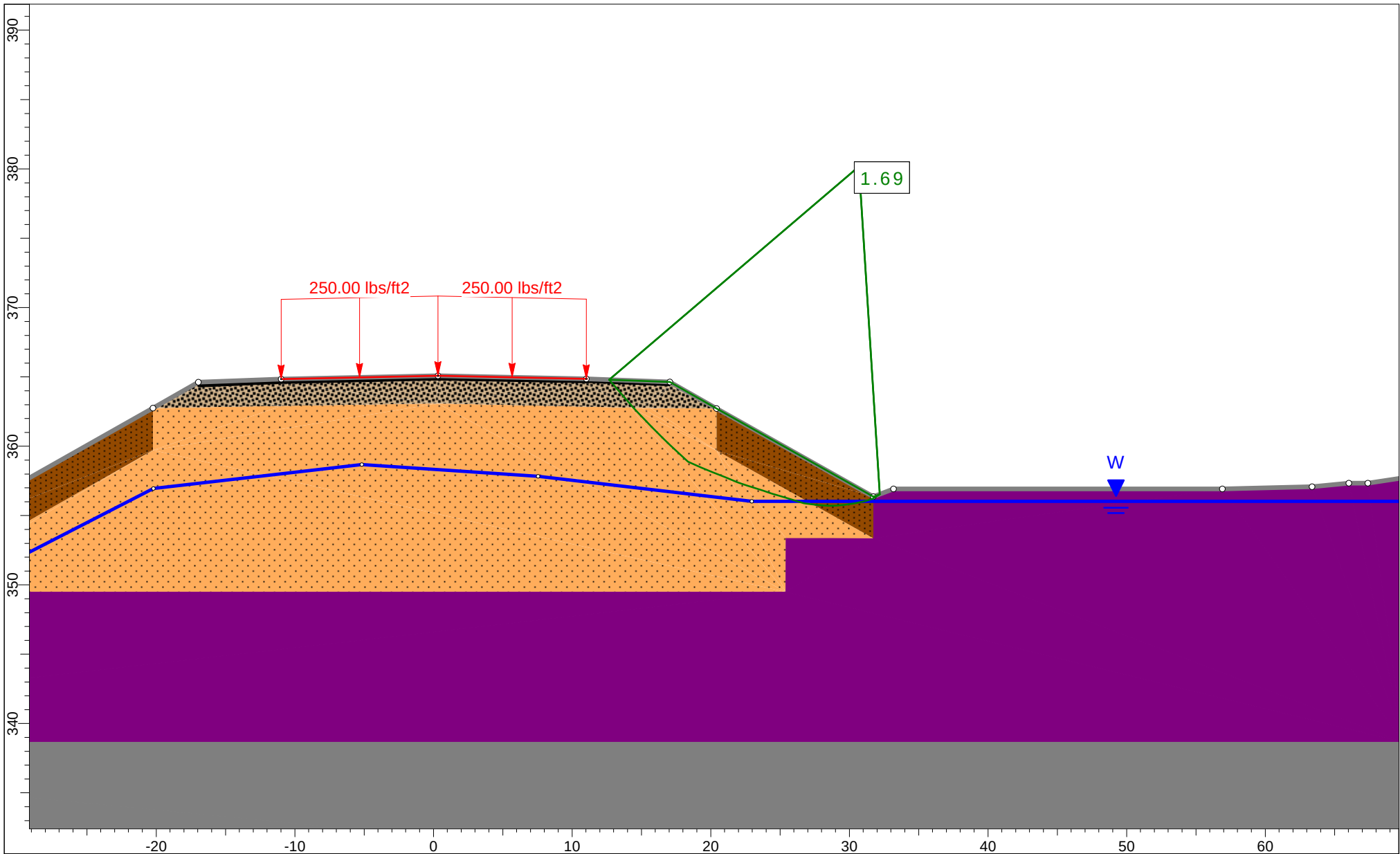


SLIDEINTERPRET 9.038

Project					
North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine					
Analysis Description					
Sta. 14+10.00 Noncircular Surficial Surfaces - South Slope					
Drawn By	JKW	Checked By	LDN	Reviewed By	MEL
Date	7/31/2025	File Name	14+10.00.slmd		
Scale					1:115
					Figure A.1



Project					
North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine					
Analysis Description					
Sta. 14+10.00 Noncircular Surficial Surfaces - North Slope					
Drawn By	JKW	Checked By	LDN	Reviewed By	MEL
Date	7/31/2025	File Name	14+10.00.slmd		
Scale					1:115
					Figure A.2



Project

North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine

Analysis Description

Sta. 14+10.00 Noncircular Deep Surfaces - South Slope

Drawn By

JKW

Checked By

LDN

Reviewed By

MEL

Scale

1:115

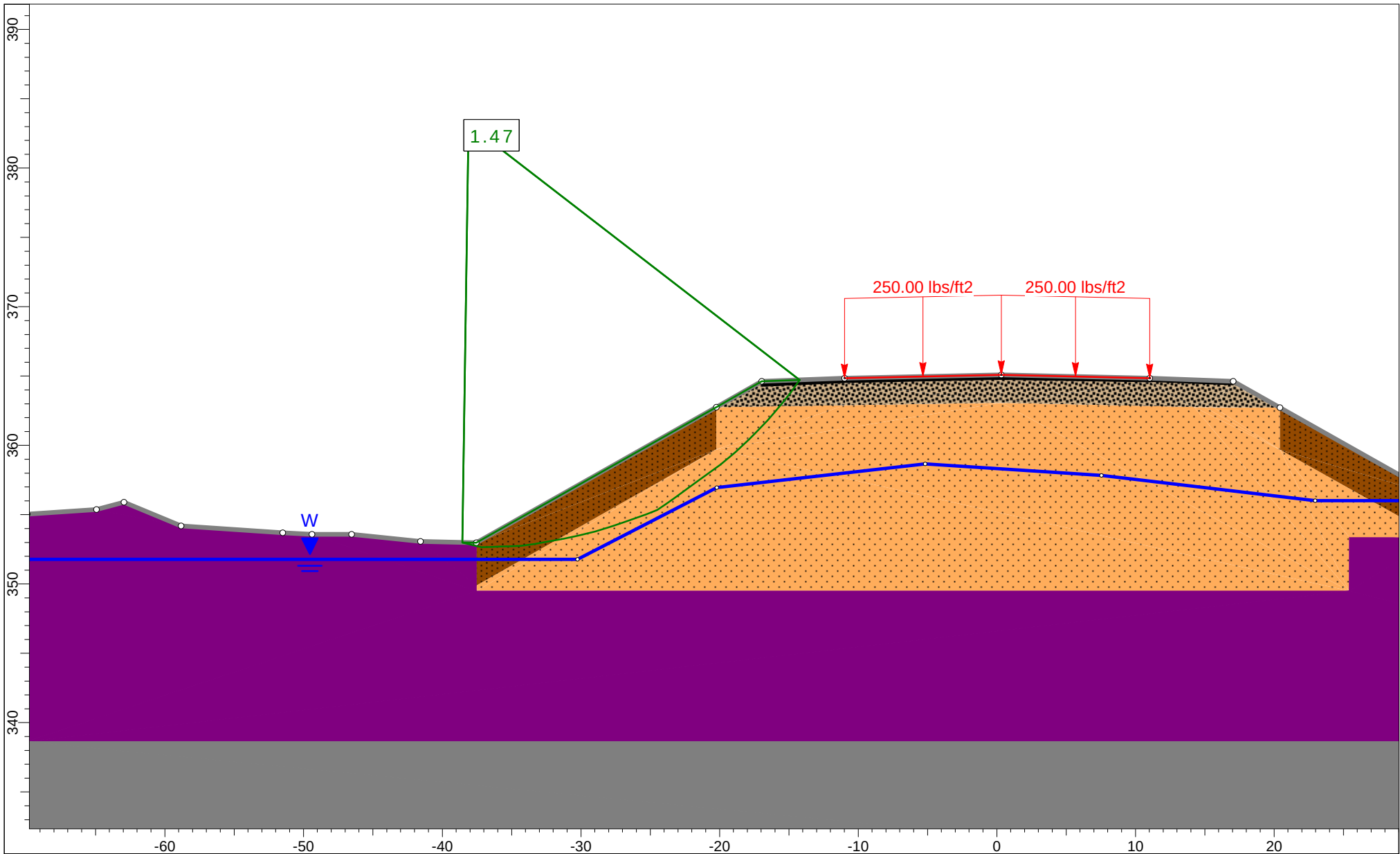
Date

7/31/2025

File Name

14+10.00.slmd

Figure A.3



Project

North Bridge Bundle Mattagodus Brook Bridge #5185, Prentiss, Maine

Analysis Description

Sta. 14+10.00 Noncircular Deep Surfaces - North Slope

Drawn By

JKW

Checked By

LDN

Reviewed By

MEL

Scale

1:115

Date

7/31/2025

File Name

14+10.00.slmd

Figure A.4



CALCULATIONS

Date:	7/8/2025	Made by:	LDN
Project No.:	US0040947.9162 / WIN 026532.00	Checked by:	RJN
Subject:	Bearing Resistance at Proposed Culvert	Reviewed by:	MEL
Project Title:	Averill Road Bridge No. 5185, Crossing Mattagodus Stream, Prentiss TWP T7 R3 NBPP, ME		

OBJECTIVE

Calculate the bearing resistance of the proposed culvert for the strength limit state and the service limit state for the highest anticipated position of the groundwater table which corresponds to the Q50 flooding event. The bearing resistance is calculated at the culvert inlet, where the depth of footing embedment is shallower than at the culvert center. The bearing pressure acting on the culvert footing is estimated at the roadway centerline, where the fill load above the culvert is greatest. □

REFERENCES

1. WSP interpreted subsurface profile (Figure 3, Preliminary Geotechnical Design Report, dated August 2025).
2. WSP geotechnical test boring logs (Appendix A, Geotechnical Design Report).
3. State of Maine Department of Transportation, Mattagodus Brook Bridge #5185 over Mattagodus Stream, Prentiss TWP T7 R3 NBPP, Maine, Dated March 27, 2026 (PS&E).
4. AASHTO LRFD Bridge Design Specifications, 10th Edition, 2024.
5. FHWA. 2002. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054.
6. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates

ASSUMPTIONS

1. The friction angle for each soil layer beneath and above the proposed culvert, based on correlation to the average of the N_{60} -values encountered in all borings for each layer (Reference 2) is:

ϕ_{Fill}	=	35	degrees
$\phi_{\text{Loose Fill}}$	=	28	degrees
$\phi_{\text{Glacial Till}}$	=	36	degrees
$\phi_{\text{Granular Borrow}}$	=	32	degrees
2. The friction angle for the proposed fill beneath the culvert (Reference 6), based on MaineDOT Standard Practice, are:
3. The proposed finish grade elevation at the headwall (STA 13+89), proposed upstream culvert invert elevation (Reference 3, Appendix A, Sheet 5), and proposed centerline culvert invert elevation (Reference 1):

El. finish grade at culvert headwall	=	363.8	ft
El. proposed invert at culvert inlet	=	350.8	ft
El. proposed invert at roadway centerline	=	350.6	ft
4. Based on Reference 3, Appendix A, Sheet 5, the proposed culvert floor thickness and the proposed thickness of excavation/fill beneath the culvert are estimated to be:

Culvert floor thickness	=	1.75	ft
Excavation/fill thickness beneath culvert	=	1.0	ft
5. The water table elevation corresponding to the Q_{50} flooding scenario reported on Reference 3, Hydraulic Report, Table 2, is:

WL elev.	=	360.9	ft
----------	---	-------	----
6. The material type for the proposed fill beneath the culvert is assumed to be granular borrow for structures (Ref. 3, Page 20) with an angle of internal friction of 32 degrees (Ref. 6). However, because this layer is limited to 1.0 foot thick, bearing resistance is based on the Glacial Till soil below this layer.
7. For the surcharge calculation it is assumed the Fill properties as representative for the soil above the footing embedment depth, neglecting the contribution of the Glacial Till. This assumption will give a conservative estimate of the surcharge term since the friction angle and unit weight of the Fill are lower than that of the Glacial Till.
8. Scour of soils at the inflow and outflow of the culvert would reduce the depth of embedment. However, the project team is designing the inflow and outflow soils and underlying rip rap apron to prevent scour, and thus the scour depth of embedment reduction is ignored.



CALCULATIONS

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9. Additional culvert dimensions:

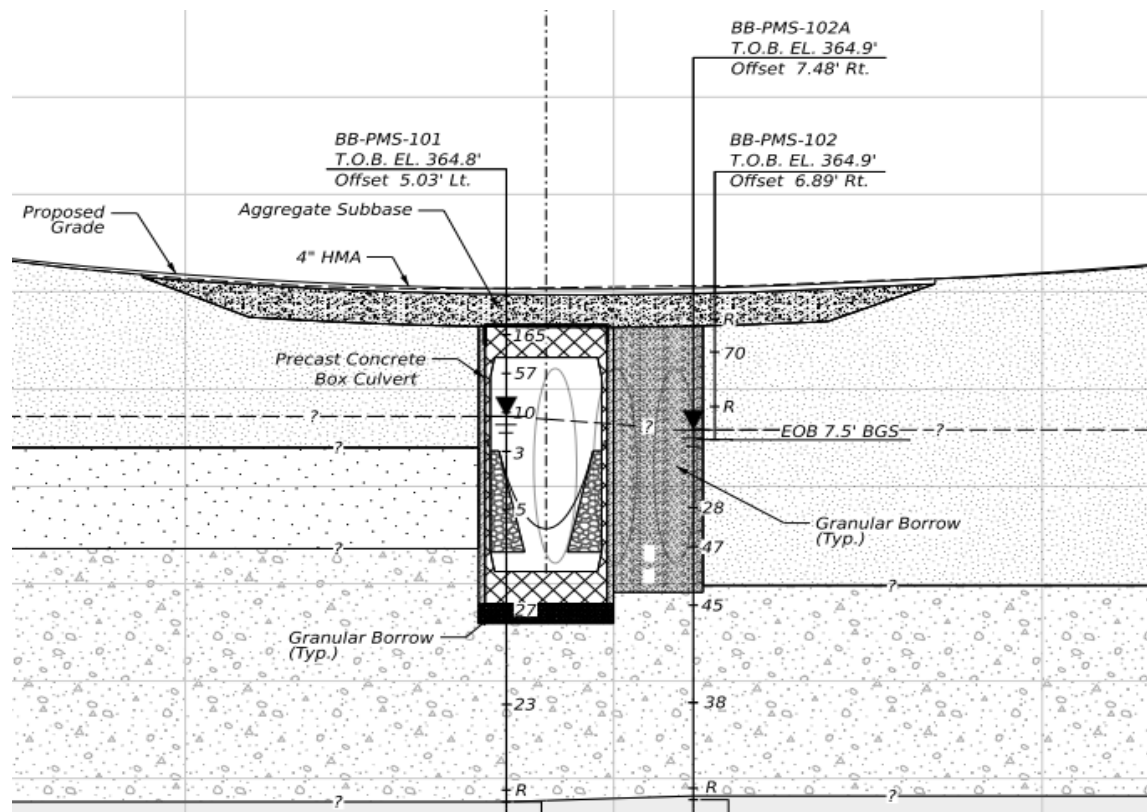
Length =	70	ft
Clear Span =	26	ft
Rise =	11	ft
Wall thickness =	1.25	ft
Top and Bottom Thickness =	1.75	ft

10. The unit weight parameters for construction materials (asphalt pavement, concrete, subbase aggregate, proposed stream bed special fill & stream channel rock) were selected based on local engineering experience. The unit weights for granular borrow and gravel borrow, 125 pcf and 135 pcf respectively, are taken from MaineDOT BDG (Reference 6, Table 3-3). "Subbase Aggregate" is given the properties of gravel borrow.

11. The load applied by the road and traffic for final design conditions is modeled as a linearly interpolated equivalent load of soil based on the distance between the surface of backfill, taken at top of pavement, to the bottom of the structure (Reference 4, Table 3.11.6.4-1) at the centerline (Reference 1).

12. Proposed Stream Bed Special Fill & Stream Channel Rock (Culvert Infill) thickness ranges from approximately 2 ft to 4 ft in thickness (Reference 1). A thickness of 3.25 ft will be used for estimating the bearing load, representing a conservative estimate of the layer thickness. Water thickness is taken from headwater elevation at Q_{50} to the estimated top of the proposed culvert infill thickness (Reference 1).

Interpretive Subsurface Profile:



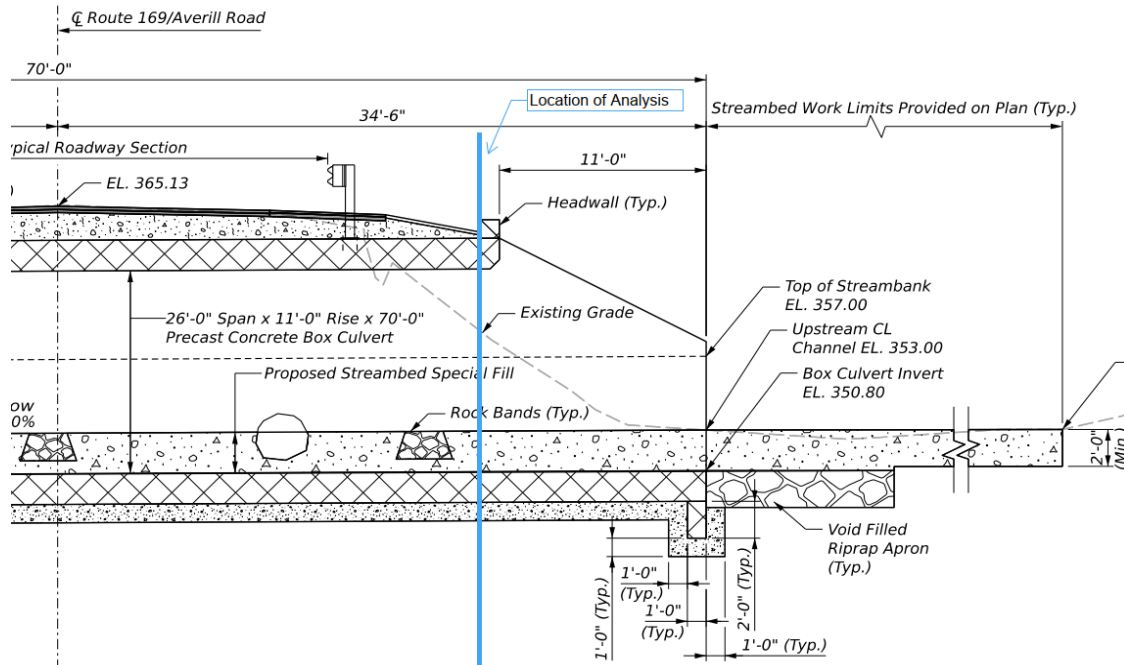
(Reference 1)



CALCULATIONS

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Location of Analysis:



(Reference 3, Appendix A, Sheet 5)



CALCULATIONS

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CALCULATION

A. Determine the bearing resistance at the strength limit state. Use Article 10.6.3, Strength Limit Design, from the AASHTO LRFD Bridge Design Specifications.

Based on Reference 1 and Assumption 6, the proposed culvert will be founded above the Glacial Till layer, and below the

1. Calculate the nominal bearing resistance.

$$q_n = cN_{cm} + \gamma_q D_f N_{qm} C_{wq} + 0.5 \gamma_f B N_{\gamma m} C_{w\gamma} \quad (\text{Ref. 4, Eqn. 10.6.3.1.2a-1})$$

where:

c = cohesion, ksf	$c = 0$ ksf
γ_q = total unit weight of soil above the footing - Fill unit weight, kcf	$\gamma_q = 0.125$ kcf
γ_f = total unit weight of soil below the footing - Glacial Till unit weight, kcf	$\gamma_f = 0.130$ kcf
D_f = footing embedment depth, ft	$D_f = 14.8$ ft (Assump. 8)
$N_{qm} = N_q s_q d_q i_q$ (Ref. 4, Eqn. 10.6.3.1.2a-3)	$N_{qm} = 42.9$
N_q = surcharge factor - Fill	$N_q = 33.3$ (Table 10.6.3.1.2a-1)
$s_q = 1 + \frac{B}{L} \tan \phi_f$ (Ref. 4, Table 10.6.3.1.2a-3)	$s_q = 1.3$
B = footing width, ft	$B = 28.5$ ft (Ref. 3, Page 20)
L = footing length, ft	$L = 70$ ft (Ref. 3, Page 20)
$d_q = 1 + 2 \tan \phi_f (1 - \sin \phi_f)^2 \arctan \left(\frac{D_f}{B} \right)$	$d_q = 1.0$ (Eqn. 10.6.3.1.2a-10)

i_q = load inclination factor (neglected as per Ref. 4, Article C10.6.3.1.2a)

Since in Reference 4, C_{wq} and $C_{w\gamma}$ are determined based on D_w using Table 10.6.3.1.2a-2, which does not contain the value of D_w determined in this analysis, the equations in Reference 5 will be used instead.

$C_{wq} = 0.5 + 0.5 \left(\frac{D_w}{D_f} \right) \leq 1.0$ (Ref. 5, Eqn. 5-10)	$C_{wq} = 0.50$
D_w = groundwater depth, ft	$D_w = 0.0$ ft
$N_{\gamma m} = N_{\gamma} s_{\gamma} i_{\gamma}$ (Ref. 4, Eqn. 10.6.3.1.2a-4)	$N_{\gamma m} = 47.1$
N_{γ} = unit weight factor - Glacial Till	$N_{\gamma} = 56.3$ (Table 10.6.3.1.2a-1)
$s_{\gamma} = 1 - 0.4 \frac{B}{L}$ (Ref. 4, Table 10.6.3.1.2a-3)	$s_{\gamma} = 0.84$
i_{γ} = load inclination factor (neglected as per Ref. 4, Article C10.6.3.1.2a)	
$C_{w\gamma} = 0.5 + 0.5 \left(\frac{D_w}{1.5B_f + D_f} \right) \leq 1.0$ (Ref. 5, Eqn. 5-9)	$C_{w\gamma} = 0.50$
	$q_n = 83.2$ ksf



CALCULATIONS

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2. Apply resistance factor.

$$q_R = \phi_b q_n \quad (\text{Ref. 4, Eqn. 10.6.3.1.1-1})$$

ϕ_b = resistance factor specified in LRFD 10.5.5.2.2 $\phi_b = 0.45$ (Table 10.5.5.2.2-1)
 Semi-empirical methods, all soils

$q_R = 37.4 \text{ ksf}$

B. Determine the bearing resistance at the service limit state.

Use AASHTO LRFD (Ref. 4) Table C10.6.2.5.1-1 to determine the presumptive bearing resistance at the service limit state.

Type of Bearing Material:	Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)
Consistency in Place:	Medium dense to very dense	Medium dense to dense
Bearing Resistance Recommended Value of Use:	5 ksf	10 ksf

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

$$\text{Resistance factor for the service limit state} = 1.0 \quad (\text{Ref 4 Article 10.5.5.1})$$

$$\text{Factored bearing resistance} = 5.0 \text{ ksf}$$

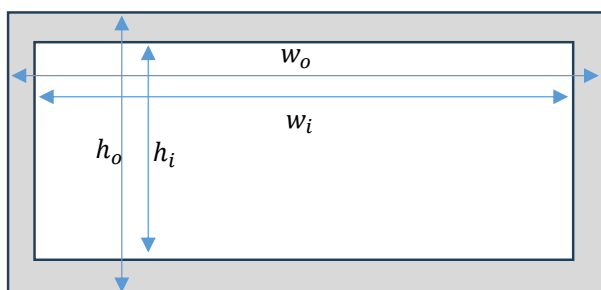


CALCULATIONS

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C. Determine the Estimated Factored Bearing Load at the Culvert Location

γ_{Conc}	Concrete unit weight, kcf	$\gamma_{\text{Conc}} = 0.150$	kcf	(Assump. 10)
γ_{Pave}	Pavement unit weight, kcf	$\gamma_{\text{Pave}} = 0.140$	kcf	(Assump. 10)
γ_{Agg}	Subbase Aggregate unit weight, kcf	$\gamma_{\text{Agg}} = 0.135$	kcf	(Assump. 10)
γ_f	Granular Borrow unit weight, kcf	$\gamma_f = 0.125$	kcf	(Assump. 10)
γ_w	Water unit weight, kcf	$\gamma_w = 0.062$	kcf	
γ_{Infill}	Culvert Infill unit weight, kcf	$\gamma_{\text{Infill}} = 0.140$	kcf	(Assump. 10)



Culvert Width, w_o	=	28.5	ft	(Ref. 3)
Culvert Span, w_i	=	26	ft	(Ref. 3)
Culvert Height, h_o	=	14.5	ft	(Ref. 3)
Culvert Rise, h_i	=	11	ft	(Ref. 3)

$$A_i = 286 \text{ ft}^2$$

$$A_o = 413.25 \text{ ft}^2$$

$$\text{Cross-sectional Area of Culvert, } A_T = 127.25 \text{ ft}^2$$

$$\text{Linear weight of Culvert} = 19.1 \text{ k/ft}$$

$$\text{Culvert Bearing Pressure} = 669.7 \text{ psf}$$

Thickness along centerline (Reference 1):

Thickness of Pavement	=	0.3	ft	(Reference 1)
Thickness of Subbase Aggregate	=	1.5	ft	(Reference 1)
Thickness of Water Above stream bed special fill, Q50	=	7.0	ft	(Reference 1)
Thickness of Culvert Infill	=	3.25	ft	(Assumption 12)
Live Load Pressure from Vehicular Traffic	=	297.5	psf	(Assumption 11)
Pressure from Pavement	=	42.0	psf	
Pressure from Subbase Aggregate	=	202.5	psf	
Pressure from Water, taken at Q_{50}	=	435.4	psf	
Pressure from Culvert Infill	=	455.0	psf	

Load Factors (Reference 4, Section 3.4.1)

Dead Load of Structural Components and Nonstructural Attachments = DC, Strength 1, Maximum	=	1.25	(Table 3.4.1-2)
Vertical Pressure from Dead Load of Earth Fill = EV, Strength 1, Maximum	=	1.30	(Table 3.4.1-2)
Live Load Surcharge = LS, Strength 1	=	1.75	(Table 3.4.1-1)
Water Load and Stream Pressure = WA, Strength 1	=	1.00	(Table 3.4.1-1)
All Loads, Service 1	=	1.00	(Table 3.4.1-1)



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	Strength 1	Service 1
Factored Culvert Bearing Pressure (DC), psf =	837.2	669.7
Factored Live Load Pressure from Vehicular Traffic (LS), psf =	520.6	297.5
Factored Pressure from Pavement (EV), psf =	54.6	42.0
Factored Pressure from Subbase Aggregate (EV), psf =	263.3	202.5
Factored Pressure from Water, taken at Q_{50} (WA), psf =	435.4	435.4
Factored Pressure from Culvert Infill (EV), psf =	591.5	455.0
Total Factored Estimated Bearing Load at Culvert, ksf =	2.7	2.1

Factored Bearing Resistance at Strength 1 =	37.4	ksf	(From Part A)
Factored Bearing Resistance at Service 1 =	5.0	ksf	(From Part B)

Demand Capacity Ratio, Strength 1 =	0.07	OK
Demand Capacity Ratio, Service 1 =	0.42	OK

CONCLUSIONS

For the proposed culvert, the recommended nominal bearing resistance is 83.2 ksf for bearing on the glacial till layer. A resistance factor of 0.45 is recommended for use at the strength limit state and a resistance factor of 1.0 is recommended for use at the service limit state. The factored bearing resistance for the strength limit state is estimated to be 37.4 ksf and the factored presumptive bearing resistance for the service limit state is estimated to be 5 ksf. A demand capacity ratio of 0.07 was calculated at Strength 1 and 0.42 at Service 1 under an estimated bearing load at the culvert's location.

