REPORT

Geotechnical Design Report (GDR)

Hogan Road Diverging Diamond Interchange, Bangor Maine (WIN 018595.11)

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1.0 INTRODUCTION AND PROJECT BACKGROUND

This Geotechnical Design Report (GDR) summarizes the results of WSP's geotechnical investigations and recommendations for the US I-95 Exit 187/ Hogan Road Diverging Diamond Interchange in Bangor Maine (MaineDOT WIN 018595.11, formerly identified as WIN 018595.10). The location of US I-95 Exit 187/ Hogan Road is shown in Figure 1 (attached). The purpose of this GDR is to present soils and bedrock information for the proposed diverging diamond interchange obtained from Phase I and Phase II subsurface investigations and laboratory tests; to present recommended geotechnical parameters for the proposed design; and geotechnical design recommendations for the construction of the proposed embankments. mast arms, and overhead sign structures associated with the diverging diamond interchange.

The existing Hogan Road interchange carries traffic in the east and west directions over US I-95 Exit 187 and was constructed in 1960 with traffic signals being added in 1978. Hogan Road was widened in 1983 to accommodate the increase in traffic. This interchange is Maine's busiest diamond interchange and is considered a high crash location. Thus, the replacement design provided in the Preliminary Design Report[1] (PDR) is a diverging diamond interchange to improve traffic flow, decrease the risk of accidents, and provide a safe way for pedestrians to cross I-95.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The Hogan Road DDI project lies in south-central Maine within the Seaboard Lowlands Physiographic Province, characterized by an eroded glacial terrain with occasional low, elongated hills and ridges shaped by glacial ice flowing to the southeast and south.[3] Regional surficial geologic mapping indicates the surficial soils consist of Pleistocene Presumpscot Formation glaciomarine deposits which overlie Pleistocene glacial till deposits.[2] Artificial fill was previously placed in areas west of the Hogan Road Bridge during placement of the culvert crossing of Penjajwoc Stream under I-95 and construction of the northbound off ramp and southbound on ramp at the Hogan Road exit. The glaciomarine deposits generally consist predominantly of silt and fine sand, with local clay and sandy beds deposited on the late-glacial sea floor. In places the glaciomarine deposits may be reworked as sea levels regressed by waves and currents. The glacial till generally consists of a light to dark grey, non-sorted to poorly sorted mixture of clay, silt, sand, pebbles, cobbles, and boulders, with the upper few feet weathered.[3]

2.2 Regional Bedrock Geology

Regional bedrock mapping indicates the bedrock beneath the site consists of rocks of the Medium bedded facies of the Penobscot River Member of the Silurian-aged Bangor Formation of the Vassalboro Group.[4] The lithology consists of medium to dark grey, fine-grained to very fine-grained, metagreywacke (greywacke being a variety of sandstone and the prefix meta indicating it has gone through some degree of metamorphism making it a metasedimentary rock) in beds ranging from 7.5 inches to 12 inches in thickness.[9] The metagreywacke (also called metawacke) is thinly to thickly bedded with parallel and/or ripple cross laminations. The thick beds are typically very fine-grained to fine-grained, and some beds are structureless (exhibit no laminations). Thin beds of

[1] MaineDOT – Highway Preliminary Design Report – Bangor Diverging Diamond Interchange (WIN 18595.10), dated 3/16/2023.

[2] Hildreth, C.T., 2011, Surficial geology of the Veazie quadrangle, Maine: Maine Geological Survey, Open-File Map 11-32, scale 1:24,000.

[3] Hildreth, C.T., 2008. Surficial Geology of the Veazie 7.5-Minute Quadrangle, Penobscot County, Maine. Maine Geological Survey Open File 08-56, 5 p.

[4] Pollock, S.G., 2011, Bedrock geology of the Veazie quadrangle, Maine: Maine Geological Survey, Open-File Map 11-58, scale 1:24,000

slate are also present and clasts of slate up to a few centimeters wide, interpreted as rip-up clasts, are common near the bases of metagreywacke beds, can also occur as isolated clasts within the metagreywacke beds.[8]

3.0 SUBSURFACE INVESTIGATIONS

WSP performed two series of highway borings at the site: 100-series highway borings (HB-HBR-1XX) during the 2022 preliminary geotechnical subsurface investigation, and 200-series highway borings (HB-HBR-2XX) during the 2023 supplemental geotechnical subsurface investigation. WSP subcontracted Seaboard Drilling, LLC (Seaboard) of Bangor, Maine to complete both the 100- and 200-series borings. Boring location coordinates and ground surface elevations are summarized in Table 1 (attached), and boring locations with respect to existing site features are illustrated in Figure 2 (attached) through Figure 7 (attached).

The borings were planned based on WSP's review of the 1980 Soils Report[5] from the Hogan Road bridge widening and Hogan Road Geotechnical Record Plans[6], which provided a preliminary idea of the types of subsurface materials expected, at least near the existing bridges. MaineDOT had performed seven (7) wash borings near the existing bridge in 1979 at the end of each abutment and pier that were used in the design and construction of the 1980's bridge. MaineDOT additionally used the soil information from the 1979 study with simple boring logs and stratigraphy from the 14 borings from the 1958 subsurface investigation to create a subsurface profile in the geotechnical record plans[6]. Subsurface soils at the site consist of brown silty clay fill associated with the original roadway and bridge construction overlying "stiff brown mottled and weathered silty clay over medium to dense brown and/or grey pebbly fine sand and sandy silt with a few stones".[5] Bedrock depth was found to vary from 41 feet below ground surface (bgs) near historic Abutment 1 (adjacent to northbound I-95) to 16 feet bgs near historic Abutment 2 (adjacent to southbound I-95).

3.1 2022 Subsurface Investigation – 100-Series Borings

To plan the 100-series highway boring locations and depths, WSP reviewed the 2021 Progress Print Plans[7], 2022 Progress Print Plans[8], CADD drawings[9] received from MaineDOT on May 18, 2022, a technical presentation[10], and the public meeting record.[11] The 2022 Progress Print Plans[8] included changes to embankment geometries at several ramps and spurs and three retaining walls between 150 feet and 400 feet long associated with the shifted I-95 southbound access ramp embankments, which were abandoned during subsequent ramp realignment efforts.

WSP and our boring subcontractor Seaboard completed the preliminary geotechnical subsurface investigation 100-series borings comprised of sixteen (16) highway borings labelled HB-BHR-101 through HB-BHR-116

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[5] Maine Department of Transportation, Material and Research Division Soils Section. January 1980. Subsurface Investigation for the Proposed Widening of the Hogan Road Bridge over I-95 in Bangor, Project M-149-0(2), Soils Report 80-04. 37 pages.

[6] File: Hogan Road Record Geotech Information.pdf, 5 sheets, provided by Laura Krusinski.

[7] VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated February 9, 2021.

[8] VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated July 7, 2022.

[9] State of Maine Department of Transportation, CADD file containing proposed highway interchange geometry, titled "highway.dgn", received by WSP Golder on May 18, 2022.

[10] VHB for Maine Department of Transportation. May 27, 2020. Bangor, US I-95 Exit 187 Hogan Road Interchange Highway Reconstruction Technical Presentation. 25 slides.

[11] Maine Department of Transportation, Meeting to Discuss the Proposed Improvements to I-95 Exit 187 In Bangor (Hogan Rd), Work Identification Number 018595.00, Held on June 9th, 2016.

between April 26 and May 9, 2022. MaineDOT surveyed the as-drilled boring locations following completion of the drilling program. The boring locations are summarized below:

- Boring HB-BHR-103 is located near the I-95 northbound onramp.
- Borings HB-BHR-101, -102, -104, -105, -106, and -111 are located near the I-95 southbound onramp.
- Borings HB-BHR-107, -108, -109, and -110 are located near I-95 northbound and Hogan Road westbound.
- Borings HB-BHR-112, -113, -114, and -115 are located near the I-95 southbound onramp shoulder.
- Boring HB-BHR-116 is located off-road from the I-95 southbound shoulder.

Seaboard drilled all borings to refusal at between 22 feet and 83 feet bgs, with the exception of HB-BHR-116, which was terminated at 83 feet bgs within a boulder in glacial till. A summary of the testing and rock coring performed is below:

- Standard Penetration Testing (SPT) was performed in all borings.
- *In situ* field vane shear tests were performed in six (6) borings: HB-BHR-102, -105, -107, -111, -113, and -114.
- Undisturbed Shelby tube samples were collected in three (3) borings: HB-BHR-101, -102, and -108.
- For four (4) borings (HB-BHR-102, -105, -108, -111), Seaboard collected 5 feet to 6.7 feet of rock core using NQ2-size (1.98-inch diameter) diamond tipped core barrels in these borings following refusal of either casing, split spoon sampler, or roller bit to advance.

3.2 2023 Subsurface Investigation – 200-Series Borings

To plan the 200-series highway boring locations and depths, WSP reviewed VHB's Preliminary Design Report (PDR)[12] and final PDR plans[13] as well as the results of WSP's Preliminary Geotechnical Design Report (PGDR) and analyses.[14] This review was performed to identify the positions of the overhead sign structures, traffic mast arms, and poles to confirm that each structure had a 100-series or 200-series associated boring for use in design of the foundations.

WSP and our boring subcontractor Seaboard completed the supplemental geotechnical subsurface investigation 200-series borings comprised of nineteen (19) highway borings labelled HB-BHR-201, 201A, -201B, and HB-BHR-202 through HB-BHR-217 between September 17, 2023, and October 2, 2023. MaineDOT surveyed the as-drilled boring locations following completion of the drilling program. The boring locations are summarized below:

- Borings HB-BHR-201, -201A, -201B, and BB-BHR-202 are located on Springer Drive.
- Borings HB-BHR-203 and -204 are located on Bangor Mall Boulevard.

[12] Maine Department of Transportation – Highway preliminary Design Report (PDR) WIN: 18595.10, dated 3/16/2023.

[13] VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated March 16, 2023.

[14] WSP Golder, Preliminary Geotechnical Design Report, Hogan Road Diverging Diamond Interchange, Bangor Maine, WIN 018595.10, dated December 29, 2022.

- Boring HB-BHR-205 is located near the I-95 southbound offramp.
- Borings HB-BHR-206, -207, -208, -209, -210 -211, -212, -213, -214, and -215 are located along the existing Hogan Road eastbound and westbound travel lanes.
- Boring HB-BHR-216 is located on Sylvan Road.
- Boring HB-BHR-217 is located between Ramp B and I-95 southbound.

Seaboard drilled borings HB-BHR-201, -201A, -201B, -202, -203, -204, -205, -210 and -217 to refusal at between 4 feet and 57 feet bgs. Borings HB-BHR-206, -207 were terminated at a depth interpreted as weather rock. Borings HB-BHR-208, -209, -211, -212, -213, -214, -215, -216 were terminated at the planned depth without encountering refusal. A summary of the testing and rock coring performed is below:

- Standard Penetration Testing (SPT) was performed in all borings.
- For eight (8) borings (HB-BHR-201, -201A, -202, -203, -204, -205, -210, and -217), Seaboard collected 1.4 feet to 10 feet of rock core using NQ2-size (1.98-inch diameter) diamond tipped core barrels following refusal of either casing, split spoon sampler or roller bit to advance.

3.3 Subsurface Investigation Methods

WSP's boring subcontractor Seaboard completed both the 100- and 200-series borings using a Diedrich D-50 track-mounted drill rig and the cased wash drilling method. To advance the borings, Seaboard drove 4-inch inside diameter casing in 5-foot lengths using an automatic safety hammer, and the soil in the casing was washed out with a roller bit and water to the depth where samples were subsequently collected. The field program included Standard Penetration Test (SPT) sampling, field vane undrained shear strength data in clay layers, collection of undisturbed Shelby tube samples in clay layers, and rock coring of the underlying bedrock. WSP geotechnical engineers monitored drilling activities, selected sampling intervals, logged subsurface conditions encountered, and obtained soil samples and rock core for use in visual description and subsequent laboratory testing and classification.

Seaboard performed Standard Penetration Testing (SPT) using a calibrated automatic hammer system and standard 2-inch split spoon sampler in accordance with American Society for Testing and Materials (ASTM) D1586 for all borings. Sampling of overburden materials was conducted at approximately 5-foot intervals for all borings. Seaboard drove split spoons 24 inches with a 140-pound hammer dropped 30 inches, and WSP recorded the number of hammer blows required to advance the split spoon sampler through each 6-inch increment. Measured, uncorrected N-values, calculated as the sum of the hammer blows to advance the sampler during the 6-inch to 18-inch interval, are provided in the boring logs in Appendix A. Seaboard provided WSP with the calibrated hammer efficiencies for the drill rigs used during both boring programs. WSP used the average calibrated hammer energy transfer ratio of 91%[15] for the 100-series borings and 92%[16] for the 200-series borings to convert the measured N-values to N_{60} values. Soil samples were collected and stored in sealed glass jars for later evaluation and laboratory testing.

[15] Standard Penetration Test Energy Measurement Calibration, R-2- Diedrich Drill D-50 with Automatic Hammer (S/N: 367), dated September 23, 2021, prepared by Geosciences Testing and Research, Inc.

[16] Standard Penetration Test Energy Measurement Calibration, Tracked Diedrich D-50 with Automatic Hammer (S/N- 367), dated November 4, 2023, prepared by GTR Engineering.

In situ field vane shear tests were performed for cohesive soils at select depths in select 100-series borings in general accordance with the MaineDOT Field Vane Testing Procedure using WSP's Geonor rectangular vane set. For each test or pair of tests, the borehole was advanced to either 6 inches or 1 foot above the desired test depth, and the vane was then advanced the remaining distance by hand, using dedicated vane rods. Upon reaching the desired test depth, Seaboard rotated the vane with a calibrated torque wrench at a rate of approximately 90 degrees per minute to obtain the peak torque, used to estimate the peak undrained shear strength. The vane was then rotated rapidly at the same depth through 10 revolutions to remold the soil. Thereafter, the vane was again rotated with a torque wrench at a rate of approximately 90 degrees per minute to obtain the residual torque, used to estimate the residual undrained shear strength. Results from vane shear testing, including raw torque readings and calculated uncorrected undrained shear strengths, are summarized in Table 2 (attached) and are provided on the boring logs in Appendix A.

Undisturbed 3-inch diameter Shelby tube samples were collected in the cohesive soils at selected intervals within the 100-series borings using a hydraulic piston GUS sampler (Gregory Undisturbed Sampler) in accordance with ASTM D6519. Samples were sealed in the field with wax and transported back to the WSP office for subsequent laboratory testing assignment.

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 100-series and 200-series subsurface investigations. The encountered soils were field characterized 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.

Rock core samples were placed in wooden boxes and transported to the WSP. WSP recorded Total Core Recovery (TCR), Rock Quality Designation (RQD), and coring rates for each core run and are provided in the boring logs in Appendix A. A detailed summary of the rock core quality parameters for the recovered rock core is presented in Table 3 (attached) and photographs of the rock core are presented in Appendix B.

4.0 LABORATORY TESTING PROGRAM

GeoTesting Express (GTX) of Acton, Massachusetts performed laboratory testing on both 100-series and 200-series boring collected soil samples and rock core in accordance with applicable American Society of Testing Materials (ASTM) and American Association of State Highway Transportation Officials (AASHTO) testing procedures. Laboratory tests performed are listed in Table 4-1. Geotechnical laboratory tests were performed on SPT split spoon samples representative of each soil type collected from the borings to assist in soil classification.

At WSP's request, GTX performed X-rays of undisturbed Shelby tube samples collected from the 100-series borings. We used the images to identify potential disturbance zones and select suitable sample intervals for subsequent advanced laboratory consolidation and strength testing.

Measured index and classification test results from soil samples are summarized in Table 4 (attached). Measured consolidation and strength properties are summarized in Table 5 (attached). Soil index and classification testing results are included on the boring logs in Appendix A. Complete laboratory testing results are provided in Appendix C.

Table 4-1: Summary of Laboratory Testing Performed on Soils

Laboratory Test	Standard(s)	100-series No. of Tests Completed	200-series No. of Tests Completed	Total No. of Tests Completed	Purpose
Grain Size (sieve)	ASTM D6913 AASHTO T 88	27	19	65	Index & Classification
Grain Size (sieve and hydrometer)	ASTM D6913/D7928 AASHTO T 88	28	-	28	Index & Classification
Atterberg Limits	ASTM D4318 AASHTO T 89, T 90	34	14	42	Index & Classification
Moisture Content	ASTM D2216 AASHTO T 267	67	28	83	Index & Classification
X-Ray Shelby Tube	ASTM D4452	5*	-	5*	Selection of advanced laboratory test soil samples
1D Incremental Load (IL) Consolidation	ASTM D2435	4*	-	4*	Consolidation& settlement soil properties
1D Constant Rate of Strain (CRS) Consolidation	ASTM D4186	5*	-	5*	Consolidation& settlement soil properties
Direct Simple Shear	ASTM D6528	4*	-	4*	Undrained shear strength

Note: *Includes laboratory tests performed on bridge borings BB-BHR-101 and BB-BHR-103, which are discussed in the corresponding Geotechnical Design Report[17] for Hogan Road Bridge 5823 and Bridge 6787. These data are presented in Table 4 of this report, and results are presented in Appendix C of this report.

5.0 SUBSURFACE CONDITIONS

Soils encountered in the borings generally include asphalt pavement, topsoil, sandy silt, or sand and gravel interpreted as fill; interpreted glaciomarine sediments ranging from silt to very stiff to soft clay; sand and gravel; sand to silty gravel interpreted as glaciofluvial and gravel with sand and silt interpreted as glacial till over bedrock interpreted as metawacke. Detailed descriptions of the soil and bedrock conditions encountered in the borings are provided in the boring logs in Appendix A. The following descriptions summarize the primary stratigraphic units from the existing ground surface to depth.

Asphalt pavement: Asphalt pavement thicknesses observed in borings HB-BHR-112, -113, -115, HB-BHR-201, -201A, -201B, -204, -206, -207, -210, -211, -213, -214, -215, -216 range from approximately 0.2 feet to 0.7 feet. A 1-foot thick section of concrete was encountered in boring BB-BHR-209.

[17] WSP USA, Inc. Geotechnical Design Report (GDR), Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine (WIN 018595.11), Submitted to MaineDOT on December 5, 2025.

Topsoil: Topsoil thicknesses observed in borings HB-BHR-101 through HB-BHR-111, HB-BHR-114, 116, ranged from approximately 0.3 feet to 2.0 feet.

Fill: Existing roadway fill material was encountered beneath the asphalt pavement layer in borings that were drilled within the paved surface, beneath the topsoil layer where borings were drilled off the roadway surface, and below the concrete surface encountered in Boring BB-BHR-209. The layer was encountered in thicknesses ranging from approximately 2.7 feet and 24.8 feet. The thickest fill layers were encountered in borings located on the existing Ramp B (HB-BHR-112, -113, -114) and Hogan Road WB (HB-BHR-209, -210). The fill generally consisted of silt with varying amounts of fine to medium sand and gravel or fine to coarse sand with varying amounts of silt and fine to coarse gravel. Laboratory classifications generally describe the fill layer as A-1-b (0), A-2-4(0), A-4 (0), A-4(2), and A-6 (11) per AASHTO and the measure N_{60} values ranged from 3 to greater than 100 (loose to very dense or soft to hard).

Silt: The silt material was encountered near existing roadway fill areas and is assumed to be fill material from the construction of the existing roadway embankments. The silt was encountered in ten (10) borings, including HB-BHR-103, -107 -205, -206, -207, -209, -211, -213, -215, -217. This layer was encountered at thicknesses ranging between 0.4 feet to 15 feet. The silt generally consists of grey to brown, silt with trace to some clay, trace to some fine to medium sand, and trace to little gravel, Laboratory classifications for the layer are CL-ML per USCS and A-4(0) per AASHTO. The measured N_{60} values range from 6 to 50 (medium stiff to hard).

Stiff to very stiff Clay: Glaciomarine clay was encountered in all borings except the for HB-BHR-101, -201, -201A, -201B, -202, -209, -213, -214, -215, -217. This layer was encountered with thicknesses ranging from 2.8 feet to 17.6 feet. The stiff to very stiff clay generally consists of grey to brown, stiff to very stiff clay, with trace to some silt, trace to some fine to medium sand, and trace to little gravel. Laboratory classifications for the layer are CL per USCS and A-6 (9), A-6 (16), A-6(17), A-6(18), A-6(19) and A-4(0) per AASHTO. Field vane tests could not be performed due to high soil stiffness. The measured N_{60} values range from 9 to 29 (stiff to very stiff).

Soft to medium stiff Clay: Soft to medium stiff glaciomarine clay was encountered in eight (8) borings, including HB-BHR-101, -104, -105, -107, -114, -207, -208, and -212. This layer was encountered with thicknesses ranging from 3.5 feet to 10 feet. The soft to medium stiff clay generally consists of grey to brown, clay with trace to some silt, trace to some coarse sand, and trace to little gravel. Laboratory classifications for the layer are CL and CL-ML per USCS and A-6 (16), A-6(18), and A-4(0) per AASHTO. Field vane test data from these limited depths in limited borings yield undrained shear strengths (s_u) value of 893 pounds per square foot (psf), 1,160 psf and 1,116 psf. Where field vane tests could not be performed due to high soil stiffness, the measured N_{60} values range from 2 to 9 (soft to medium stiff).

Sand and Gravel: A layer of sand and gravel was encountered in twelve (12) borings, including HB-BHR-108, -111, -114, -116, -205, -209, -210, -211, -212, -214, -216, -217. The sand and gravel layer was encountered with thicknesses ranging from 2.0 feet to 12 feet. The sand and gravel generally consist of grey to brown, fine to coarse sand with trace to some fine to coarse gravel, trace to some silt, and trace to some clay. Laboratory classifications for the layer are SC-SM per USCS and A-1-a (0), A-2-4 (0), and A-4 (0) per AASHTO. The measured N_{60} values range from 14 to 86 (medium dense to very dense).

Glacial Till: A layer of silt with sand and gravel interpreted as glacial till was encountered in all borings except HB-BHR-106, 201, -201A, -201B, -202, -203, -211, -212, -216, -217, and BB-BHR-206. The glacial till was encountered, with thicknesses ranging from approximately 2.0 feet and 30.5 feet. The thickest glacial till layers were encountered closest to Ramp B in borings HB-BHR-102, -113, -114, -115, -116. The glacial till generally

consists of silt with trace to some fine to coarse sand with trace to some fine to coarse gravel. Boulders were encountered in HB-BHR-102, -109, -113, -114, -204. Laboratory classifications for the layer are CL-ML, GM, and SC-SM per USCS and A-4(0), A-2-4 (0), and A-1-b (0) per AASHTO and the measured N_{60} values range from 8 to greater than 100 (stiff to hard).

Glaciofluvial: A layer of sand to silty gravel interpreted as glaciofluvial was encountered in borings HB-BHR-114, and -116. The glaciofluvial was encountered at depths between approximately 43.5 feet and 55 feet bgs, with thicknesses ranging between 9.5 feet and 39.5 feet. The glaciofluvial generally consists of brown to grey fine to medium sand with little to some silt, and trace to some fine to coarse gravel. Laboratory classifications for the layer A-2-4 (0) per AASHTO and the measured N_{60} values range from 44 to 66 (dense to very dense). The glaciofluvial layer transitions to bedrock at approximately 64.5 feet to 80 feet bgs in borings HB-BHR-114 and HB-BHR-116, respectively.

Bedrock: The bedrock surface was encountered in all borings except HB-BHR-116, -208, -209, -211, 212, -213, -214, -215, -216 where the borings were terminated at the planned depth in soil, and no refusal was encountered. The bedrock elevation varies by approximately 86.5 feet across site based on the highway borings. The deepest bedrock surface was encountered at approximately Elevation (EL) 28.2 feet (64.5 feet bgs) in boring HB-BHR-114, which is located west of, and adjacent to, the northern abutment of the existing 1983 bridge, and between south bound of I-95 and Ramp B. The shallowest bedrock was encountered in boring HB-BHR-201B, located on the intersection between Springer Drive and Hogan Road west bound at approximately EL 113.6 feet (4 feet bgs). The highest elevation bedrock was encountered was at approximately EL 114.7 feet in boring HB-BHR-202 (4.3 feet bgs).

Rock core was collected in borings HB-BHR-102, -105, -108, -111, -201, -201A, -202, -203, -204, -205, -210, -217 in runs ranging between 1.4 feet to 10 feet in length. The predominant bedrock lithology encountered is grey, very fine grained to fine grained, medium weak strong (R4), fresh (W1) to slightly weathered (W2), metawacke (metasandstone), interpreted to be the Medium Bedded Facies of the Penobscot River Member of the Bangor Formation. The RQD (rock quality designation) ranged from very poor (RQD = 0%) to good (RQD = 70%), and the estimated RMR (rock mass rating) ranged from 41 to 62. Table 3 (attached) provides detailed information about recovery, RQD, RMR, and descriptions of lithology, rock mass, and discontinuities.

Groundwater: WSP measured groundwater levels in all borings, except in HB-BHR-201, -201A, -201B, -202, -203, -204 and -208. The ground water levels were measured upon completion of the boreholes and before the removal of the casing. Groundwater was measured in the borings at depths ranging in depth between approximately 2.6 feet and 12.9 feet bgs. Note that groundwater levels will vary seasonally and that measured values may be affected by water used during drilling.

Possible Soil Contamination: During the drilling of boring HB-BHR-204, WSP identified a strong hydrocarbon odor in the silty sand sample collected at 14.2 feet to 17.4 bgs below a boulder (encountered between 11.4 feet to 14.2 feet bgs) and above weathered bedrock. Our field engineer reported that he and the drill crew identified a hydrocarbon odor during drilling below the boulder even before the sample was recovered. HB-BHR-204 is located on Bangor Mall Road near the intersection of Hogan Road. WSP did not identify olfactory, discoloration, or oil sheen indicators of potential hydrocarbon contamination within the other borings or sample depths; however, in the absence of more rigorous testing, this does not preclude the presence of additional contamination.

6.0 GEOTECHNICAL SITE EVALUATIONS

WSP analyzed the 100-series and 200-series laboratory and field data for soil and rock core samples to determine the geotechnical characteristics of the site, including engineering properties for the soil and bedrock and frost penetration depth which are used as basis of design for the subsequent geotechnical analyses.

6.1 Engineering Design Parameters

WSP evaluated the geotechnical data collected and developed design parameters for the highway extents. Based on our evaluation, the soils and their measured and correlated engineering properties varied between the major design features. Therefore, we developed engineering design properties for each soil layer encountered at the site highway area which are different from the properties developed for the bridge area. Table 6-1 provides a summary of design parameters for the *in situ* soil layers encountered, and Table 6-2 provides a summary of the design parameters for construction fills. The *in situ* soil properties are based on SPT N-value correlations, measured field vane shear data, laboratory index test and classification data presented in Table 4 (attached), and laboratory-measured consolidation and strength properties presented in Table 5 (attached).

Soils were assigned a friction angle were treated as having drained engineering behavior, and soils assigned with an undrained shear strength were treated as having undrained engineering behavior. The upper silt layer had both SPT N-values that could be correlated to drained material properties and measured field vane shear test undrained shear strengths that are related to undrained behavior because much of this layer could be characterized as low plasticity clay (CL), low plasticity silt (ML) or a mixture (CL-ML). Thus, we provide both drained and undrained properties for the silt layer, where we assumed undrained behavior would be predominant below the water table. The silt layer was treated as fully drained for foundation evaluations and was evaluated as both drained and undrained in global stability evaluations. Lastly, while these parameters served as a basis for analyses, adjustments or assumptions may differ for the specific analysis performed.

Table 6-1: Geotechnical Design Parameters for *In Situ* Soils

Soil Layer	Unit weight (pcf)	Friction Angle (degrees)	Overconsolidation Ratio, OCR (-)	Undrained Shear Strength (psf)	Elastic Modulus (ksf)
Fill (Existing)	125	34	-	-	254
Silt (drained)	125	32	-	-	180
Silt (undrained)	125	-	-	1000	-
Soft to medium stiff Clay	121	-	2.6	913	840
Stiff to very stiff Clay	122	-	2.6	1343	1236
Sand and Gravel	130	38	-	-	413
Glacial Till	130	38	-	-	413
Glaciofluvial Deposit	125	44	-	-	677
Bedrock	158	-	-	-	-

Note: Additional geotechnical design parameters for *in situ* soils used in the settlement modeling are provided in the corresponding Geotechnical Design Report[17] for Hogan Road Bridge 5823 and Bridge 6787.

Table 6-2: Geotechnical Design Parameters for Fill Soils

Soil Layer	Unit weight (pcf)	Friction Angle (degrees)	Lateral Earth Pressure at Rest, K_0[1]
Asphalt Pavement	140	45	0.29
Ultra-Lightweight Foamed Glass Aggregate (ULFGA)	18 – 20[2]	35.5[3]	0.42
Granular Borrow	125	32	0.47
Aggregate Subbase	135	36	0.41
Riprap	140	42	0.33

Notes: [1]$K_0 = 1 - \sin\phi$. [2]Upper range of as-compacted ULFGA unit weight based on supplier information. [3]WSP used this friction angle for ULFGA from our interpretation of the minimum friction angle at 10-15% shear strain based on the results of ASTM D3080 Direct Shear test data for both wet-process and dry-process manufactured ULFGA.

6.2 Frost Penetration Depth

WSP estimated an average frost penetration depth for roadway fills using the MaineDOT Bridge Design Guide (BDG).[18] The site has an air design freezing index of approximately 1,726 °F-degree days. For representative roadway coarse-grained fills with 11% water content, we estimate a frost penetration depth of approximately 87.1 inches (7.3 feet) at the site. We recommend foundation structures be located deeper than the frost penetration depth of 7.3 feet.

7.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

This section describes the geotechnical analyses we performed for the embankment fills associated with the proposed realignment of Hogan Road and the ramps and spurs associated with the Hogan Road and I-95 interchange provided in the 2025 Progress Print Plans[19] and the 60% Bridge Plans.[20] These analyses include settlement and global stability of embankments along with mast arm, overhead sign structure, and pole foundation geotechnical and structural design.

7.1 Global Stability Analysis

WSP evaluated global stability for several sections associated with the proposed new highway embankment fills on Hogan Road and widened Ramp B and Ramp C. We selected locations for global stability analyses based on a combination of the tallest and widest fills and the thicknesses of low strength subsurface soil layers. The fills we evaluated were located beyond the two bridges and their approach embankments, as approach embankment and

[18] Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates

[19] VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated March 10, 2025.

[20] VHB for State of Maine Department of Transportation. 60% Westbound Bridge Plans: Bangor, Penobscot County, Hogan Road Bridge over I-95, WIN 018595.10 / FED 1859510. Dated August 30, 2024.

bridge global stability evaluations are discussed in the Bridge Geotechnical Design Report[21]. We evaluated global stability for the following Hogan Road and ramp locations:

- Hogan Road eastbound travel lane embankment at Station (Sta.) 106+00, west of Ramp B
- Hogan Road eastbound travel lane embankment at Sta. 119+00, east of Spur C
- Ramp B embankment at Sta. 808+00 (2:1 slope)
- Ramp B embankment at Sta. 808+00 (4:1 slope)
- Ramp C embankment at Sta. 1007+50

A minimum factor of safety (FS) of 1.3 was used for potential failure surfaces that extended through embankment fills under static conditions. AASHTO 2020[22] Section 11.6.3.7 indicates that where geotechnical parameters and subsurface stratigraphy are well defined, a resistance factor, $\phi = 0.75$ (factor of safety, FS = 1.3) should be used and where geotechnical parameters and subsurface stratigraphy are highly variable or based on limited information a resistance factor $\phi = 0.65$ (FS= 1.5) should be used. We recommend a minimum FS of 1.3 for potential failure surfaces based on the presence of compacted fills from prior embankment construction, because the selected *in situ* soil engineering properties are well defined across the site based on the 100-series and 200-series borings and because engineering properties specified for construction fills have resulted in adequate embankment performance on numerous projects.

We performed global stability analyses using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience[23] for post-construction embankment fill static load conditions transverse to the roadway centerline. We developed the models using the geometry for the embankments from the 2025 Progress Print Plans[19] and design material properties discussed in Section 6.1. Where ULFGA is present in the proposed embankment, we used a lower unit weight of 18 pcf in the analyses, which would generally result in the lower factors of safety for potential global stability failure surfaces. We additionally evaluated the shallow *in situ* silt layer as both drained and undrained with the associated strength properties as discussed in Section 6.1.

We analyzed global stability under static loading conditions using the Spencer and Bishop methods 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. We analyzed the lowest FS within the embankment sections under two conditions: 1) for a potential failure surfaces within the proposed embankment fills alone, and 2) for potential failure surfaces that extend deeper into the existing *in situ* soils below the embankment fills. The full methodology of the analysis, calculations, and locations of the lowest factor of safety potential failure surfaces are presented in Appendix D.

Table 7-1 summarizes the lowest FS results for static loading conditions for proposed embankment fills. Results show that proposed 4:1 slope at Sta. 808+00 left meet the minimum FS = 1.3. However, the 2:1 slopes evaluated at Sta. 106+00, Sta. 119+00, Sta. 808+00 right, and Sta. 1007+50 have FS values just below the minimum recommended FS = 1.3. These shallow potential failure surfaces are contained entirely within the new or existing embankment fills and indicate potential surficial sloughing failures. These 2H:1V slopes will be grubbed and

[21] WSP Geotechnical Design Report (GDR), Hogan Road Bridge, Bangor, Maine (WIN 18595.10) dated October 2025.

[22] AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

[23] Rocscience Inc. Slide2 Software Package Version 9.038, build date February 28, 2025.

infilled to the design slopes with granular borrow with a design unit weight of 125 pcf and friction angle of $\phi = 32^{o}$ per the MaineDOT BDG[18]. The 2H:1V embankment slope angle of 26.5° is less than the angle of internal friction of the embankment fill material (32°); however, this combination of friction angle and slope angle typically results in calculated surficial FS < 1.3. Based on discussions with MaineDOT, we understand that slopes constructed with granular borrow using methods commensurate with MaineDOT Standard Specifications[24] have historically performed adequately.

Table 7-1: Minimum Static Global Stability Factors of Safety for Embankments: Fill Soils

Structure	Station	Silt Strength Condition	Factor of Safety[1,2]
Hogan Road Eastbound Embankment Fill	106+00	Undrained	1.26
		Drained	1.26
Hogan Road Eastbound Embankment Fill	119+00	Undrained	1.25
		Drained	1.25
Ramp B Embankment Fill	808+00 2:1 slope Right	Undrained	1.25
		Drained	1.25
	808+00 4:1 slope Left	Undrained	2.48
		Drained	2.48
Ramp C Embankment Fill	1007+50	Drained	1.25

Notes: [1]Minimum FS = 1.3 recommended for potential failure surfaces under static loading. [2]values less than the minimum recommended values are shown in red.

Table 7-2: Minimum Static Global Stability Factors of Safety For Embankments: Deeper *In situ* Soils

Structure	Station	Silt Condition	Factor of Safety[1]
Hogan Road Eastbound Embankment Fill	106+00	Undrained	1.69
		Drained	1.69
Hogan Road Eastbound Embankment Fill	119+00	Undrained	1.45
		Drained	1.45
Ramp B Embankment Fill	808+00 2:1 slope Right	Undrained	1.41
		Drained	1.40
	808+00 4:1 slope Left	Undrained	2.69
		Drained	2.66
Ramp C Embankment Fill	1007+50	Drained	1.39

Note: [1]Minimum FS = 1.3 recommended for potential failure surfaces under static loading.

[24] State of Maine, Department of Transportation. March 2020. Standard Specifications. State of Maine, Department of Transportation. March 2020. Standard Specifications.

Table 7-2 summarizes the lowest FS results for static loading conditions for potential failure surfaces that extend deeper into *in situ* soils. Stability evaluations of embankments with failure surfaces through *in situ* soils at the locations analyzed all yield adequate factors of safety under static conditions.

In summary, our evaluation of the proposed slopes associated with the Hogan Road and highway access ramps indicate the embankment slopes as proposed have adequate factors of safety against potential global stability failure surfaces. By inspection, we assume the remaining proposed 2H:1V and shallower roadway, ramp, and spur slopes (outside of the proposed bridges and approaches) comprised of embankment fills and underlain by similar *in situ* soils have similarly adequate factors of safety.

7.2 Settlement Analysis

7.2.1 Settlement Model Setup

WSP evaluated the anticipated total and differential settlement expected at the larger embankment fills for the proposed interchange embankment widening and grade increases using the three-dimensional (3D) analysis software *Settle3*[25] by Rocscience. We created two *Settle3* settlement models: an Interchange Model and a Ramp B Model. Refer to the full methodology and basis of analysis, material properties, and model development in Appendix E. A third model for the proposed new Hogan Road westbound was also created and is discussed in the Bridge Geotechnical Design Report[21].

The Interchange Model includes embankment fills for:

- Hogan Road westbound, Sta. 208+50 to Sta. 217+00 (850-foot length)
- Ramp A, Sta. 702+50 to Sta. 706+25 (375-foot length)
- Spur B, Sta. 900+00 to Sta.901+50 (150-foot length)
- Ramp C, Sta. 1006+25 to Sta. 1008+50 (225-foot length)
- Spur C, Sta. 1100+50 to Sta. 1102+00 (150-foot length)
- Hogan Road Eastbound, Sta. 118+75 to Sta. 121+25 (250-foot length)

The Ramp B Model includes embankment fills for:

- Hogan Road eastbound Sta. 103+50 to Sta. 106+25 (275-foot length)
- Ramp B Sta. 801+75 to Sta. 813+75 (1200-foot length)

The *Settle3* models were created using the existing topographic ground surfaces from the 2022 Progress Print Plans[8] and subsurface soil stratigraphy based on the 100-series highway and bridge borings as discussed in the PGDR[14] and updated in the vicinity of the bridges following completion of the 200-series bridge borings. The Interpretive Stratigraphic Profiles (ISPs) from the PGDR[14] used for the *Settle3* models are illustrated in Figure 9 (attached) through Figure 13 (attached), and Figure 8 identifies the locations of these ISP sections within the project plan. Only Figure 10 (attached) has been updated to include the subsurface information from the 200-series bridge borings (BB-BHR-2XX) discussed in the Bridge GDR[21]). Topsoil was assumed to be removed before placement of new embankment fills. Our interpreted engineering properties for the soil layers are described in

[25] Rocscience, Inc. Settle3 software package, version 5.025, build date February 13, 2025.

Section 6.1. We modeled the cohesionless soil layers (existing fills including the silt layer, sand and gravel, glaciofluvial sand, and glacial till) with elastic settlement only. We modeled the cohesive clay layers with immediate settlement, primary consolidation settlement, and secondary compression settlement. Material properties for the clay layers were based on consolidation laboratory test results from undisturbed thin-wall Shelby tube samples from the 100-series borings presented in Table 5 (attached).

The proposed embankment geometry from the 2025 Progress Print Plans[19] and the 60% Bridge Plans[20] was added to the Settle3 models and loading associated with embankment fills was applied above the existing ground surface up to the proposed geometry using the unit weight of the proposed fills. For embankments, we assumed an embankment fill is Granular Borrow with unit weight of 125 pounds pcf. ULFGA was modeled in the embankments when needed for settlement mitigation using a compacted unit weight of 20 pcf. This compacted unit weight is conservative as it represents the upper bound loading that may be applied by the ULFGA.

For each of the *Settle3* models, we evaluated immediate settlement, primary consolidation settlement, secondary compression settlement, and total settlement within *in situ* soils below the existing ground surface only. Note that settlement is not calculated within the planned fill soils in *Settle3*, as these fills are used only to apply load. Based on the coarse-grained nature of the fill soils typically specified, compaction to minimize voids, and immediate settlement upon subsequent load application, settlement of fills is expected to occur during construction and will not influence post construction settlement magnitudes.

Settle3 allows the user to identify settlement for soils below any elevation of inquiry, where a settlement contour plot in plan view from that elevation can be generated. When that elevation is selected, settlement estimated to occur above that elevation of inquiry is omitted from the generated results. Additionally, a section view of settlement contours with depth can be generated from the existing ground surface elevation and a cross-section of interest (i.e., abutment centerline). The section view of settlement allows for a visual representation of the differential settlement that may be expected in the soils below the elevation of inquiry along the cross-section of interest.

7.2.2 Settlement Criteria

The *Settle3* software allows for analysis of settlement with time based on time-rate of consolidation parameters, and specific evaluation times can be set to view resulting settlement in a model. WSP incorporated staged construction of the ramp embankments and bridge abutments into the *Settle3* models to analyze relative time-rates of settlement throughout the interchange. Settlement was evaluated up to 75 years (the anticipated design life of the bridges) after construction of the final ramp. WSP also included intermediate evaluation stages at 5 years and 20 years after construction of each ramp or abutment to compare settlement results against MaineDOT's limiting settlement criteria. These limiting settlement criteria, provided by MaineDOT, are as follows:

- Limiting Settlement for Highway Embankments more than 200 feet from Bridge
  - Less than 4 inches of settlement from 0 years after construction to 20 years after construction
  - Less than an additional 4 inches of settlement from 20 years after construction to 75 years after construction (i.e., less than 8 inches total at 75 years)
- Limiting Settlement for Highway Embankments within 200 feet of Bridge
  - Less than 2 inches of settlement from 0 years after construction to 5 years after construction

- Less than an additional 2 inches of settlement from 5 years after construction to 20 years after construction (i.e., less than 4 inches total at 20 years)
- Less than an additional 2 inches of settlement from 20 years after construction to 75 years after construction (i.e., less than 6 inches total at 75 years)

- Limiting Settlement for Approach Slab
  - Less than 1 inch of settlement from 0 years after construction to 5 years after construction

WSP initially evaluated settlement assuming the embankments would be constructed with Granular Borrow. This resulted in estimated settlement amounts of up to 7 inches in the first 5 years after construction, exceeding the limiting settlement criteria. After discussion with MaineDOT and the project designers, WSP revised the models to incorporate settlement mitigation using staged construction and ULFGA.

In the southwestern area of the modeled interchange, consisting of Spur C Sta. 1101+50 to Sta. 1102+00 and Hogan Road Sta.118+25 to Sta. 120+75, the model predicted that the estimated settlement would occur quickly after embankment construction (i.e., a fast time-rate of settlement). Thus, the proposed settlement mitigation consisted of preloading the area with Granular Borrow for 8 months while construction progressed within the interchange. Thereafter, the embankments could be raised to full height as planned with additional Granular Borrow.

In the northeastern areas of the modeled interchange near the bridge abutments, consisting of Hogan Road Sta. 208+75 to Sta. 210+50 and Sta. 215+00 to Sta. 217+00, Spur B Sta. 900+00 to Sta. 901+25, and Ramp C Sta. 1007+40 to Sta. 1009+00, the proposed construction staging and proposed location of temporary traffic detour ramps meant that there would be insufficient time available for preloading during construction. Instead, the embankments in these areas were proposed to be constructed of ULFGA. The proposed thickness of ULFGA in each embankment was adjusted in the models to provide estimated total settlement at or below the limiting settlement criteria. Refer to the Contract Plans for the extents of the ULFGA for the proposed embankments.

In areas of the modeled interchange beyond the bridge abutments, such as Ramp A and Ramp B, the estimated settlement did not exceed the limiting criteria, and settlement mitigation was not necessary in the models. The proposed embankments for Ramp A and Ramp B were assumed to be constructed of Granular Borrow.

7.2.3 Settlement Results

While the settlement was modeled in 3D, we have strategically selected and provided 2D plan and profile sections of settlement to demonstrate settlements relevant to the proposed structures and embankment fills. Settlement at these locations is from the existing ground surface and includes deformations in the *in situ* materials only. Table 7-3 provides a summary of the settlement estimates for the proposed embankment fills for Ramp A, Ramp B, Spur B, Ramp C, and Spur C from *Settle3*. These settlement estimates include the proposed settlement mitigations, consisting of ULFGA embankments along Hogan Road Westbound, Spur B, and Ramp C, and preloading with Granular Borrow along Spur C and Hogan Road Eastbound.

The settlement values presented in Table 7-3 include estimated total, immediate, and consolidation (primary consolidation and secondary compression) settlement over the life of the project (75 years) for the embankment fills at the ramps and spurs. Due to the staged construction, the settlement values for each ramp or spur are shown at 75 years after construction completion of that ramp or spur. The table presents both the minimum and the maximum settlement estimated along the centerline of the feature. Appendix E provides a detailed basis for

analysis and additional information on settlement modeling inputs and results for both models, including contour plots of settlement at and near each ramp and spur in both plan view and section view to show variation in settlement. These settlement contour plots show that the maximum estimated settlement occurs at Spur C, where *in situ* soils will experience the greatest embankment loading but lightweight fill is not proposed; instead, preloading is proposed to generate this estimated settlement prior to construction completion.

Table 7-3: Settlement Results: Embankments on Ramps and Spurs

Linear Feature		Ramp A	Ramp B	Spur B	Ramp C	Spur C
Elevation (ft)		105.4	98.9	99.3	82.3	82.3
Modeled Feature Length (ft)		373	1475	103	211	435
Total Settlement, S_t (in.)	Max.	2.5	2.0	1.9	1.5	6.6
	Min.	0.3	1.5	0.0	0.3	0.3
Immediate Settlement, S_i (in.)	Max.	1.0	1.8	0.4	0.7	2.9
	Min.	0.0	1.0	0.0	0.1	0.2
Primary Consolidation Settlement, S_c (in.)	Max.	1.4	0.3	1.3	0.7	4.4
	Min.	0.1	0.1	0.0	0.0	0.1
Secondary Compression Settlement, S_s (in.)	Max.	0.2	0.2	0.3	0.2	0.2
	Min.	0.0	0.0	0.0	0.1	0.1

7.3 Geotechnical Design of Substructure Drilled Shafts

WSP designed foundations, specifically drilled shafts, for the traffic mast arms, overhead sign structures (OHSS), and span wire pole structures. VHB provided WSP with mast arm, and span wire pole design loads[26] for eleven (11) mast arms and four (4) span wire poles. VHB also provided design loads[27] and stationing for three (3) overhead sign structures (OHSS). VHB additionally provided minimum dimensions for foundations to accommodate the structures atop the foundations.

The 200-series borings were specifically planned such that a boring was located at or near each substructure location. WSP prepared simplified subsurface soil and geotechnical engineering material property profiles at each of the proposed mast arm, span wire pole, and OHSS foundation locations. These material properties were based on correlations with SPT N_{60} values and engineering judgement and were used for subsequent geotechnical design. The subsurface characteristics and interpreted engineering parameters for each of the substructures are summarized in Appendix F. The design subsurface profiles considered both the presence of cohesive and cohesionless soils (proposed and *in situ* fills, sands, sand and gravel, silt, and silty clay above bedrock) and the location of the bedrock surface where encountered.

[26] VHB Computations, Loads to Overhead Mast Arm Signal/Sign Structures, dated 2/17/25.

[27] VHB Computations, Assumed OHSS Foundation Loads, dated 4/9/2024.

Table 6 (attached) provides structure information related to design of the mast arm structures, span wire poles, and OHSS, respectively. Figure 2 (attached) through Figure 7 (attached) and the Contract Plans show the locations of each of these structures with respect to site features and the 100 series and 200 series borings.

WSP evaluated drilled shaft lengths assuming the top of shaft is at the proposed ground surface, ignoring any reveal height above the ground surface. We assumed the upper 2 feet of soil does not contribute to soil-shaft frictional interaction in the analyses that are subsequently discussed for each for following: axial resistance (Section 7.3.1), lateral resistance (Section 7.3.2), and torsion resistance (Section 7.3.3). The upper 2 feet of soil is incorporated into the lateral resistance and pushover evaluations (Section 7.3.2). The complete analysis methodology and calculations are provided in Appendix F.

7.3.1 Axial Resistance

In accordance with the MaineDOT BDG[18] Section 5.8, the design methods in AASHTO LRFD[22] and FHWA GEC 10[28] were used to evaluate geotechnical resistance for axial compression loads through a combination of side friction and end bearing tip resistance. The nominal axial geotechnical resistance was calculated at each drilled shaft location for the respective soil profile. Contributions from side resistance of the soils in the top two feet of each shaft were neglected in the analysis to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. Below this depth, side and tip resistance were evaluated in accordance with AASHTO LRFD[22] Article 10.8.3.5 for both cohesionless and cohesive soils, and in bedrock. Resistance factors of 0.55 and 0.45 were applied to side resistance in cohesionless/ bedrock and cohesive soils, respectively. Resistance factors of 0.50 and 0.40 were applied to tip resistance in cohesionless/ bedrock and cohesive soils, respectively. Table 7 (attached) presents our recommended values of factored geotechnical resistance in axial compression for each drilled shaft foundation location.

7.3.2 Lateral Resistance

The computer program LPile[29] was used to evaluate the lateral geotechnical resistance of the drilled shaft for each design profile. A summary of the input soil and rock parameters and p-y lateral models used in the LPile analyses is provided in Appendix F.

A pushover analysis was performed with LPile to evaluate lateral geotechnical resistance at the Strength I and Extreme I limit states for each design profile. The shaft was modeled as a simple linear elastic beam with an elastic modulus equal to that of concrete and a moment of inertia equal to that of the uncracked circular cross section. Shaft head deflection was computed for a constant factored axial design load and various multiples of the factored shear and moment design loads, up to $1/\phi$ times the factored shear and moment design loads. A resistance factor (ϕ) of 0.67 was used for Strength I loads and a resistance factor of 0.8 was used for Extreme I loads. In accordance with FHWA GEC 10[28] Section 9.3.3.3.1, the drilled shaft design was considered stable if the analyses converged to a solution with a computed head deflection no larger than 10% of the shaft diameter. Furthermore, WSP designed the drilled shaft foundations so that total drilled shaft rotation at the end of the pushover analysis would result in an estimated vertical movement at the tip of the mast arm of less than 8 inches.

[28] FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.

[29] Ensoft, Inc. LPile software package, version 2022.12.07, release date 11/15/22.

Lateral geotechnical resistance at the Service I limit state was evaluated for each design profile by modeling the shaft as a nonlinear reinforced concrete shaft in flexure. In accordance with FHWA GEC 10[28], a limiting requirement of 0.5 inches was used for the shaft head deflection under Service I loads. Table 7 (attached) presents the computed head deflections under Service I loads for each drilled shaft design profile. In support of WSP's structural design, we provided the maximum bending moment in the drilled shaft for each foundation location at the Strength I, Extreme I, and Service I limits states in Table 8 (attached).

7.3.3 Torsional Resistance

The design method in Florida Department of Transportation (FDOT) Modifications to LRFD[30] and modifications to the FDOT method by Rodriguez et al.[31] and Aguilar et al.[32] were used to evaluate torsional resistance of the drilled shaft for the traffic mast arm, overhead sign structure, and pole foundation design profiles. We assumed that the drilled shaft concrete is in contact with soil and no more than a two-foot length of casing is left in place near the ground surface. The nominal torsional resistance was calculated for a single vertical drilled shaft at each design profile location. Contributions from skin friction resistance of the soils in the top two feet of each shaft were neglected in the analysis to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete. Furthermore, per the FDOT manual a resistance factor of 1.0 was applied to torsional resistance for mast arm structures, and a resistance factor of 0.9 was applied to torsional resistance for OHSS. For the mast arms and OHSS, shaft length was evaluated in 5-foot increments and was controlled by torsional resistance. Torsion demand capacity ratios ranged from 0.10 to 0.60 in soil and 0.04 to 0.09 in rock for mast arms, and 0.61 to 0.83 for OHSS, calculated under the Extreme I load case. Table 7 (attached) presents the results of our torsional resistance analysis for each foundation location.

7.3.4 Summary

Table 7 (attached) presents the factored geotechnical resistance values and lateral deflection at the service limit state for the drilled shaft designs for each structure. As discussed, the design soil profiles were chosen based on engineering judgment to represent subsurface layering and material properties that include fill and *in situ* cohesionless and cohesive soils, bedrock, and proposed loading. Table 8 (attached) presents the maximum bending moment derived from the lateral analyses for each structure. Furthermore, Table 7 (attached) presents WSP's recommended foundation dimensions for each of the mast arms, OHSS, and span wire poles.

8.0 Structural Design of Substructure Drilled Shafts

The structural design of the drilled shafts was completed in accordance with the MaineDOT BDG[18] (Section 5.8) and the design methods in AASHTO LRFD[22]. Drilled shaft loading was extracted from the LPile results and the geotechnical calculations located in Appendix G. The governing demand from limit state Strength I, Extreme I, or Service I was used for the drilled shaft and hammerhead reinforcement design. A resistance factor of 0.9 was used for shear, torsion, and combined axial and flexure resistance at tension-controlled and pure bending regions. A resistance factor of 0.65 was used for combined axial and flexure resistance at compression-controlled regions.

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[30] Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.

[31] Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.

[32] Aguilar, V.J., Stallings, M.J., Anderson, B., and Nowak, A. Auburn University Research Report. Torsional Resistance of Drilled Shaft Foundations. The Auburn University Highway Research Center, October 2018.

Class A concrete with a compressive strength of 5 ksi was assumed for the drilled shaft and hammerhead design. A yield strength of 60 KSI was assumed for steel reinforcement. The drilled shaft diameters and lengths were determined by the geotechnical design. The drilled shafts were evaluated for shear, torsion, and combined axial and moment demand. The reinforcement design was governed by the minimum reinforcement requirements of AASHTO LRFD[22] Section 5.6.4.2 and Section 5.7.2.5. Hammerheads were required at two OHSS drilled shaft locations due to the sign structure anchoring being located outside of the drilled shaft. The hammerhead dimensions were determined based on bolts arrangement information provided by VHB. The hammerheads were evaluated for one-way and two-way shear along with combined axial and moment demand. The reinforcement design was governed by the minimum reinforcement requirement of AASHTO LRFD[22] Section 5.6.4.2. The interface reinforcement was governed by the minimum requirement of AASHTO LRFD[22] Section 5.7.4.2. The full design methodology and calculations are provided in Appendix G.

9.0 GEOTECHNICAL RECOMMENDATIONS

9.1 Embankment Construction

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

Subgrade surfaces for embankments should be prepared in accordance with MaineDOT Specification[24] Section 203.09 for preparation of the embankment area. which requires a firm, unyielding surface. Embankment fill should meet the requirements of MaineDOT Standard Specification[24] Section 703.19 for Granular Borrow. Embankment excavation and fill placement should follow MaineDOT Specification[24] 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 Specifications[24], specifically Section 203.10 through Section 203.12, and 203.17.

If wet subgrade conditions are encountered in abutment and 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 Specifications[24]) to allow proper compaction of overlying fills. If seepages persist over a broad subgrade area, provisions should be made to allow for positive drainage beneath and within the new abutments and 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.

9.2 Ultra-Lightweight Foamed Glass Aggregate (ULFGA)

WSP has identified two manufacturers/distributors of ULFGA within the United States, AeroAggregates of North America, LLC[33] headquartered in Eddystone, PA; and Glavel, Inc.[34] headquartered in Burlington, VT. Both of

[33] AeroAggregates of North America, LLC. Company website. https://www.aerofga.com. Accessed June 2023.

[34] Glavel, Inc. Company website. https://www.glavel.com. Accessed June 2023.

these materials are ultra-lightweight with uncompacted unit weight values generally ranging from 10 pcf to 15 pcf. However, ULFGA from these two manufacturers differ in several ways. With respect to their manufacturing process, AeroAggregates uses a dry manufacturing process with a dry silicon carbide[35] foaming agent and Glavel uses a wet manufacturing process and glycerin, sodium silicate, and water as foaming agents.

Nicks et al (2024)[36] provide comprehensive laboratory-measured parameters and differences between wet-process and dry-process manufactured ULFGA material engineering properties, including gradation, particle breakage, compaction, secondary compression deformation, shear strength, and modulus, all of which differ for a given ULFGA type depending on whether the material is dry or saturated.

ULFGA properties used in these analyses are presented in Table 6-2, which account for an as-compacted unit weight of 20 pcf and shear strength friction angle of 35.5º. These material properties were used in our geotechnical evaluations and are based on laboratory-measured shear strength test data available to, and interpreted by, WSP from both AeroAggregates and Glavel ULFGA products. Special Provision 203 for ULFGA has been created for the project that specifies the ULFGA material properties, specifically unit weight, shear strength, compressive strength, and placement methods for the embankment fills. For extents of the ULFGA required in the proposed embankments, refer to the Contract Plans.

WSP provides the following installation recommendations based on our experience on other projects and manufacturer[37,38,39,40] specifications and recommendations:

- ULFGA placed in embankments should be benched in accordance with MaineDOT Standard Specifications[24] to an equivalent slope of no greater than 1H:1V.
- Where utility poles, guardrail, shallow foundations, and drilled shafts, are to be installed, MaineDOT requires the embedded substructure to have a minimum buffer of conventional fills between them and the ULFGA of at least 1 foot below and 2 feet around the sides of the embedded substructure. Where ULFGA fills are to be placed prior to installation of these types of substructures, a corrugated plastic pipe (CPP) may be installed vertically within the fills and ULFGA compacted around the perimeter. If this method is used, the CPP should be of adequate diameter to allow for compaction of conventional fills within the CPP around the substructure (e.g., guardrail post).
- Compacted ULFGA has permeability values comparable to crushed aggregates and is assumed to be well-drained provided the ULFGA is encapsulated in an appropriate geotextile for drainage design per MaineDOT Standard Specifications[24] and ULFGA does not become clogged due to fines migration from surrounding fill materials.

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[35] AeroAggregates of North America, LLC. Manufacturer's Information TN 106: Manufacturing Quality Control Manual for Production of Ultra-Lightweight Foamed Glass Aggregate, V.1.1. https://www.aerofga.com/technical-library

[36] J. E. Nicks, I. I. Ghaaowd, and M. T. Adams, "Strength-Deformation Characteristics of Two Different Foamed Glass Aggregates under a Static Design Load," Geotechnical Testing Journal Vol. 47, No. 1 (January/February 2024): pp 351–370. https://doi.org/10.1520/GTJ20220258

[37] AeroAggregates of North America, LLC. May 2020. Manufacturers Information Document TN 112: Ultra-lightweight Foamed Glass Aggregate Design and Construction Considerations, Technical Note. https://www.aerofga.com/technical-library

[38] Glavel, Inc. Lightweight Fill Guide. https://www.glavel.com/lightweight-fill-guide.

[39] Glavel, Inc. Commercial Construction. https://www.glavel.com/commercial-construction.

[40] Glavel, Inc. Commercial Construction – Technical Guide. https://6889409.fs1.hubspotusercontent-na1.net/hubfs/6889409/Commercial%20Technical%20Guide%202025.pdf

- A needle-punched nonwoven geotextile in accordance with MaineDOT Standard Specification Item 722.04, should be placed as a separator between the subgrade and the initial lift of ULFGA, between the final lift of ULFGA and the material to be placed above it, and along the side edges of the ULFGA lifts. The geotextile should have a minimum weight of 6 ounces per square yard and a grab tensile strength of 160 lbs per ASTM D4632, with a minimum of 12-inch overlapping at the seams, per Special Provision 203 Ultra-Lightweight Foamed Glass Aggregate.
- Where ULFGA is to be placed, a fine-grained subgrade should be prepared with a smooth-edged bucket to prevent disturbance as much as possible. If organic or otherwise unsuitable materials are identified in areas where ULFGA is to be placed, these materials should be removed and replaced with competent fill such as Common Borrow or Granular Borrow.
- The area to be filled with ULFGA should not have any standing water, including ice, prior to placement of the ULFGA.
- Where groundwater is encountered during excavations for ULFGA placement, the Contractor should provide temporary dewatering of excavations, if needed, such that ULFGA is placed and compacted in-the-dry. Because the unit weight of ULFGA is less than that of water, ULFGA will float when inundated by water. Thus, the Contractor should be prepared to keep the ULFGA positively weighted with ballast if inundation is expected.
- In open areas, ULFGA should be placed in lifts with a maximum thickness of 24 inches and then compacted with a tracked excavator or dozer at ground pressures between 625 psf and 1,025 psf. The number of full passes of the selected equipment required to achieve compaction should be determined in the field.
- For areas not accessible by tracked equipment, ULFGA should be placed in lifts with a maximum thickness of 12 inches compacted using four full passes with a plate compactor weighing up to 250 lbs.
- Heavy construction equipment and temporary traffic, other than for placement and compaction, should not be operated over the ULFGA until 12 inches of cover material and/or crane mats have been placed.
- A minimum of 6 inches of cover material should be placed above ULFGA for final conditions where live loads are not applied (e.g., embankments).

9.3 Drilled Shaft Foundations

We recommend the drilled shaft foundation locations be cleared, grubbed, and stripped of existing vegetation, pavement, and topsoil, and any unsuitable materials exposed at the subgrade level, such as wood, logs, tree stumps, organic silt, peat, soft clay, debris fill, or other materials that may compress, decay or collapse should be removed prior to start of foundation construction. We recommend unsuitable soils be replaced with Granular Borrow materials in accordance with MaineDOT Standard Specifications. [24]

Drilled shaft foundations should be constructed in accordance with MaineDOT Standard Specification[24] Section 626.034 and Special Provision 626 for greater than 60-inch diameter foundations. Foundations should be cast-in-place using temporary casing, if necessary, with no more than 2 feet of permanent casing (e.g., tubular form) below the ground surface. We recommend concrete for drilled shafts be placed (via tremie methods) as soon after excavation as practicable to prevent debris from collecting in the excavated area. The concrete for drilled shafts should be placed in accordance with Section 502.01 as temporary casing is withdrawn to prevent debris from contaminating the foundation and to ensure concrete is cast against the surrounding soil.

9.4 Construction Observation Recommendations

The analyses and recommendations contained herein are based on the known and predictable behavior of properly engineered and constructed foundations and embankments. We recommend observation of the foundation installation and embankment construction by a qualified geotechnical engineer to verify that the procedures and techniques used during construction are in accordance with the recommendations contained herein and in the contract documents. A qualified geotechnical engineer should be present during the following construction activities:

- Construction of embankments.
- Proof rolling subgrade areas beneath pavements.
- Assessment of seepage control requirements for permanent cut slopes.
- Subgrade preparation operations in site areas overlain with ULFGA and embankment fills.
- Assessment of the suitability of excavated soils for reuse as engineered fill.
- Placement, compaction, and testing of ULFGA and embankment fills.
- Preparation of subgrades for bridge abutments.
- Foundation drainage installations.
- Drilled shaft installations.

10.0 REPORT AND EXPLORATION LIMITATIONS

WSP prepared this report for the exclusive use of MaineDOT for specific application to the proposed Hogan Road Diverging Diamond Interchange at I-95 project in Bangor, 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, we 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. In addition, WSP should review the final plans and specifications to evaluate compliance with these recommendations.

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

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Tables

Table 1: Boring Locations
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MAINEDOT WIN 018595.11

Test Boring Designation[1]	As-Drilled Locations[2]				Existing Ground Surface Elevation[2] (ft)	Boring Depth[3] (ft)	Top of Bedrock[3,4] (ft)	Nearby Feature
	Northing	Easting	Stationing	Offset				
HB-BHR-101	486576.621	1739700.751	104+87	26.0 RT	110.3	33.0	30.3	Hogan Rd (EB)
HB-BHR-102	486494.684	1739703.212	105+68	31.2 RT	109.2	51.8	45.5	Hogan Rd (EB)
HB-BHR-103	486383.449	1739898.792	704+44	20.6 LT	106.8	28.0	27.5	Ramp A
HB-BHR-104	486219.248	1739736.001	208+75	23.2 RT	104.2	25.0	24.5	Hogan Rd (WB)
HB-BHR-105	486134.486	1739754.808	209+52	9.4 LT	100.7	29.0	24.0	Hogan RD (WB)
HB-BHR-106	486159.163	1739635.157	804+63	11.0 RT	105.9	18.5	14.5	Ramp B
HB-BHR-107	485563.443	1739838.275	215+33	3.6 LT	88.3	23.5	23.5	Hogan Rd (WB)
HB-BHR-108	485405.013	1739809.715	1007+98	41.2 RT	82.3	34.0	28.6	Ramp C
HB-BHR-109	485212.362	1739903.181	118+72	13.2 RT	78.3	40.0	37.0	Hogan Rd (EB)/ Spur C
HB-BHR-110	485210.377	1739810.637	1100+89	48.4 RT	76.5	22.0	12.0	Spur C
HB-BHR-111	485973.041	1739565.218	806+62	21.2 RT	97.8	41.2	34.5	Ramp B
HB-BHR-112	485832.419	1739547.816	808+01	4.9 LT	97.5	36.0	35.4	Ramp B
HB-BHR-113	485637.994	1739481.375	810+05	14.6 LT	92.2	48.5	47.8	Ramp B
HB-BHR-114	485577.276	1739497.406	810+53	55.1 LT	92.7	67.5	64.5	Ramp B
HB-BHR-115	485372.826	1739331.654	813+08	13.3 LT	96.7	53.5	53.0	Ramp B
HB-BHR-116	485309.571	1739242.155	814+10	23.1 RT	93.5	83.0	NE	Ramp B
HB-BHR-201	487189.137	1739746.201	10+23	80.3 LT	117.7	15.4	5.4	Springer Drive/ 60 ft Mast Arm C-MI
HB-BHR-201A	487188.016	1739744.060	10+24	77.9 LT	117.8	6.5	5.1	Springer Drive/ 60 ft Mast Arm C-MI
HB-BHR-201B	487189.874	1739740.560	10+22	72.8 LT	117.6	4	4	Springer Drive/ 60 ft Mast Arm C-MI
HB-BHR-202	487118.074	1739789.126	11+01	110.2 LT	118.7	9.3	4.3	Springer Drive/ 45 ft Mast Arm D-MI
HB-BHR-203	487072.610	1739610.188	11+15	73.8 RT	114	13.5	8.5	Bangor Mall Blvd/ 60 ft Mast Arm A-MI
HB-BHR-204	487169.516	1739605.640	10+19	61.6 RT	116	24.5	19.5	Bangor Mall Blvd/ 50 ft Mast Arm B-MI
HB-BHR-205	486487.458	1739878.180	703+52	32.8 RT	108.1	32.8	27.8	Ramp A
HB-BHR-206	486285.194	1739807.189	207+89	25.3 LT	118.4	26.3	NE[5]	Hogan Rd (WB)/ 35 ft Mast Arm C-MI
HB-BHR-207	486572.986	1739763.072	104+96	35.1 LT	116.7	25	NE[6]	Hogan Rd (WB)
HB-BHR-208	485512.735	1739866.267	215+93	13.9 LT	90.8	27	NE	Hogan Rd (WB)/ 15 ft Mast Arm D-MI
HB-BHR-209	485412.629	1739959.139	116+70	45.2 LT	105.1	27	NE	Hogan Rd (EB)/ 40 ft Mast Arm A-MI
HB-BHR-210	485339.432	1739884.834	117+49	22.8 RT	102.1	65	57	Hogan Rd (EB)/ Ramp C/Spur C
HB-BHR-211	485225.093	1740037.794	401+70	20.2 RT	98.5	27	NE	87 ft overhead sign structure/ Ramp D
HB-BHR-212	485194.455	1739963.729	119+06	39.3 LT	97.5	27	NE	87 ft overhead sign structure/ Hogan Rd (EB)
HB-BHR-213	484647.909	1740117.969	225+00	15.1 LT	81.5	27	NE	Hogan Rd (WB)/ 55 ft Mast Arm A-MI
HB-BHR-214	484505.412	1740123.448	38+37	27.1 LT	78.8	27	NE	Hogan Rd (WB)/ 60 ft Mast Arm B-MI
HB-BHR-215	484503.667	1740063.128	37+33	32.9 RT	78.6	27	NE	Hogan Rd (EB)/ 55 ft Mast Arm C-MI
HB-BHR-216	484574.735	1740015.232	36+57	78.1 RT	78.2	27	NE	Sylvan Rd/ 30 ft Mast Arm D-MI
HB-BHR-217	485824.994	1739646.308	807+77	100.8 LT	91.4	22.9	17.9	Ramp B

Notes

1. Borings HB-BHR-101 through HB-BHR-116 were drilled by Seaboard Drilling, LLC between April 26, 2022 and May 9, 2022. Borings HB-BHR-201 through HB-BHR-217 were drilled by Seaboard Drilling, LLC between September 17, 2023 and October 2, 2023.
2. As-drilled locations and elevations for borings HB-BHR-101 through HB-BHR-116 are derived from electronic file "Borings.dgn" received by WSP on May 24, 2022 from MaineDOT. As-drilled locations and elevations for borings HB-BHR-201 through HB-BHR-217 are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.
3. Boring logs with descriptions of the soil and rock encountered at the site are provided in Appendix A.
4. NE = Not Encountered
5. Probable weathered rock encountered at the end of boring, based on weathered rock fragments in sample.
6. Probable weathered rock encountered at 23.8 ft bgs based on drilling resistance, advanced boring to 25 ft bgs.

Prepared By: ATM/TJ/DEB
Checked By: FCT/LMP/ATM
Reviewed By: MEL

Table 2: Summary of In-Situ Vane Shear Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Test Number[3]	Test Depth Below Ground Surface (feet)	Approximate Test Elevation (feet)	Vane Size (mm x mm)	Undisturbed[4]		Remolded[4]		Sensitivity, S_t[5]	Comments
						Vane Reading Torque (ft-lbs)	s_u from vane Reading (psf)	Vane Reading Torque (ft-lbs)	s_{u-r} from vane Reading (psf)		
HB-BHR-102	109.2	MV	10.0 - 11.0	99.2 - 98.2	55 x 110	-	-	-	-	-	Could not push into soil
HB-BHR-105	100.7	V1	15.0 - 16.0	85.7 - 84.7	55 x 110	26	1161	8	357	3.3	
HB-BHR-105	100.7	V2	16.0 - 17.0	84.7 - 83.7	55 x 110	25	1116	5	223	5.0	
HB-BHR-107	88.3	V1	15.0 - 16.0	73.3 - 72.3	55 x 110	20	893	5	201	4.4	
HB-BHR-107	88.3	MV	16.0 - 17.0	72.3 - 71.3	55 x 110	-	-	-	-	-	Could not push into soil
HB-BHR-111	97.8	MV	10.0 - 11.0	87.8 - 86.8	55 x 110	-	-	-	-	-	Could not push into soil
HB-BHR-113	92.2	MV	22.0 - 23.0	70.2 - 69.2	55 x 110	-	-	-	-	-	Could not push into soil
HB-BHR-114	92.7	MV	7.00 - 8.00	85.7 - 84.7	55 x 110	-	-	-	-	-	Could not push into soil

Notes:

1. Test boring locations are illustrated in the plans titled "Boring Location Plan" (Draft BLP 11.21.23.pdf).
2. As-drilled elevations for 100-series borings are derived from the survey file received from MaineDOT on May 24, 2022 entitled "Borings.dgn" and for 200-series borings are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.
3. Geonor Rectangular Vane, mm = millimeters
4. s_u and s_{u-r} are the calculated peak and remolded undrained shear strengths, respectively. Conversion from torque readings to shear strengths are based on data from Geonor. For the 55 x 110 mm vane, 1 ft-lb of torque corresponds to 44.7 psf. For the 65 x 130 mm vane, 1 ft-lb of torque corresponds to 27.5 psf.
5. Sensitivity, S_t is calculated as s_u/s_{u-r}
6. Vane shear test results are shown on the boring logs in Appendix A
7. - = no data
8. MV = missed vane

Prepared By: ATM/TJ
Checked By: FCT/LMP
Reviewed By: MEL

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation	Core Size	Existing Ground Surface Elevation[1]	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No.	Midpoint Depth Below Bedrock Surface	Depth Below Ground Surface (ft)			Length	Length		Length		Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
(-)	(in)	(ft)	(-)	(ft)	Start	End	Midpoint	(ft)	(ft)	%	(ft)	%	(-)	(-)	(-)	(-)	
HB-BHR-102	NQ (1.88)	109.2	R1	3.8	46.8	51.8	49.3	5.0	4.1	82%	2.00	40%	Poor	Fresh (W1)	Strong (R4)	55	46.8-51.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite vein (0.1 in thick) present at 48.6 ft bgs; average 2.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
HB-BHR-105	NQ (1.88)	100.7	R1	1.5	24.0	27.0	25.5	3.0	2.2	73%	1.55	52%	Fair	Fresh (W1)	Strong (R4)	61	24-27 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.1 in to 1.0 in thick) present from 24.0 ft to 25.6 ft bgs; average 1.3 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
HB-BHR-105	NQ (1.88)	100.7	R2	4.0	27.0	29.0	28.0	2.0	2.0	100%	0.40	20%	Very Poor	Fresh (W1)	Strong (R4)	50	27-29 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.2 in to 0.5 in thick) present from 28.0 ft to 29.0 ft bgs; average 3.5 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
HB-BHR-108	NQ (1.88)	82.3	R1	1.6	29.0	31.3	30.2	2.3	2.1	91%	0.60	26%	Poor	Fresh (W1)	Strong (R4)	51	29-31.3 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open with grey-brown silt infilling, milky quartz and calcite veins (0.1 in to 0.2 in thick) from 29 ft to 30.5 ft bgs; average 2.6 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
HB-BHR-108	NQ (1.88)	82.3	R2	4.1	31.3	34.0	32.7	2.7	2.7	100%	1.30	48%	Poor	Fresh (W1)	Strong (R4)	55	31.3-34 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 32 ft to 32.5 ft bgs and 33.3 ft bgs; average 2.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
HB-BHR-111	NQ (1.88)	97.8	R1	4.2	36.2	41.2	38.7	5.0	4.8	96%	2.75	55%	Fair	Fresh (W1)	Strong (R4)	62	36.2-41.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderately dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.1 in thick) present at 36.5 ft and 39.8 ft bgs; average 1.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation	Core Size	Existing Ground Surface Elevation[1]	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No.	Midpoint Depth Below Bedrock Surface	Depth Below Ground Surface (ft)			Length	Length		Length		Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
(-)	(in)	(ft)	(-)	(ft)	Start	End	Midpoint	(ft)	(ft)	%	(ft)	%	(-)	(-)	(-)	(-)	
HB-BHR-201	NQ (1.88)	117.7	R1	0.5	5.4	6.4	5.9	1.0	0.3	33%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	41	5.4-6.4 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low to steep angle dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-201	NQ (1.88)	117.7	R2	2.4	6.4	9.1	7.8	2.7	2.6	96%	0.33	12%	Very Poor	Fresh (W1)	Strong (R4)	49	6.4-9.1 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to vertical dipping, close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-201	NQ (1.88)	117.7	R3	5.6	9.1	12.8	11.0	3.7	3.7	100%	2.00	54%	Fair	Fresh (W1)	Strong (R4)	62	9.1-12.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-201	NQ (1.88)	117.7	R4	8.7	12.8	15.4	14.1	2.6	2.6	100%	1.20	46%	Poor	Fresh (W1)	Strong (R4)	57	12.8-15.4 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low to steep angle dipping, close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-201A	NQ (1.88)	117.8	R1	0.7	5.1	6.5	5.8	1.4	1.4	98%	0.67	48%	Poor	Fresh (W1)	Strong (R4)	55	5.1-6.4 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-202	NQ (1.88)	118.7	R1	1.2	4.3	6.7	5.5	2.4	2.4	100%	0.67	28%	Poor	Slightly Weathered (W2)	Strong (R4)	50	4.3-6.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low angle to very steep/vertical dipping, wide, moderate close to close spacing, extremely fractured, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-202	NQ (1.88)	118.7	R2	3.7	6.7	9.3	8.0	2.6	2.4	91%	1.67	64%	Fair	Fresh (W1)	Strong (R4)	62	6.7-9.3 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-203	NQ (1.88)	114.0	R1	2.0	8.5	12.5	10.5	4.0	3.9	98%	0.42	11%	Very Poor	Fresh (W1)	Strong (R4)	50	8.5-12.5 ft: Grey, fine grained to very grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-203	NQ (1.88)	114.0	R2	4.5	12.5	13.5	13.0	1.0	1.0	100%	0.70	70%	Fair	Fresh (W1)	Strong (R4)	60	12.5-13.5 ft: Grey, fine grained to very grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to low angle dipping, close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation	Core Size	Existing Ground Surface Elevation[1]	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No.	Midpoint Depth Below Bedrock Surface	Depth Below Ground Surface (ft)			Length	Length		Length		Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
(-)	(in)	(ft)	(-)	(ft)	Start	End	Midpoint	(ft)	(ft)	%	(ft)	%	(-)	(-)	(-)	(-)	
HB-BHR-204	NQ (1.88)	116.0	R1	N/A	11.2	16.2	13.7	5.0	1.9	38%	N/A	N/A	N/A	N/A	N/A	N/A	11.2-16.2 ft: BOULDER
HB-BHR-204	NQ (1.88)	116.0	R2	3.5	19.5	22.2	20.9	2.7	2.6	96%	0.67	25%	Very Poor	Fresh (W1)	Strong (R4)	52	19.5-22.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep to vertical dipping, close spacing, tight to open, extremely fractured, calcite intrusions, extremely fractured 20.5-22.2 [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-204	NQ (1.88)	116.0	R3	6.0	22.2	24.5	23.4	2.3	2.3	100%	0.40	17%	Very Poor	Fresh (W1)	Strong (R4)	48	22.2-24.5 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low to steep angle dipping, very close to close spacing, tight, frequent calcite intrusions, fractured 23.7-24.4 [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-205	NQ (1.88)	108.1	R1	1.9	27.8	31.6	29.7	3.8	3.0	79%	0.60	16%	Very Poor	Fresh (W1)	Strong (R4)	50	27.8-30.87 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, close spacing, tight, frequent calcite intrusions, chemical weathering in bottom foot of the run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-205	NQ (1.88)	108.1	R2	4.4	31.6	32.8	32.2	1.2	1.1	92%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	47	30.8-32.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, close spacing, open to tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-210	NQ (1.88)	102.1	R1	0.6	57.0	58.3	57.7	1.3	0.7	54%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	42	57-58.3 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-210	NQ (1.88)	102.1	R2	2.1	58.3	59.8	59.1	1.5	0.5	36%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	43	58.3-59.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-210	NQ (1.88)	102.1	R3	5.4	59.8	65.0	62.4	5.2	2.2	42%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	43	59.8-65 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured, frequent calcite veins [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
HB-BHR-217	NQ (1.88)	91.4	R1	1.4	17.9	20.7	19.3	2.8	2.4	86%	1.40	50%	Poor	Fresh (W1)	Strong (R4)	60	17.9-20.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, very close to moderate close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Table 3: Summary of Rock Core Quality
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Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation	Core Size	Existing Ground Surface Elevation[1]	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No.	Midpoint Depth Below Bedrock Surface	Depth Below Ground Surface (ft)			Length	Length		Length		Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
(-)	(in)	(ft)	(-)	(ft)	Start	End	Midpoint	(ft)	(ft)	%	(ft)	%	(-)	(-)	(-)	(-)	
HB-BHR-217	NQ (1.88)	91.4	R2	3.9	20.7	22.9	21.8	2.2	2.1	95%	1.20	55%	Fair	Fresh (W1)	Strong (R4)	62	20.7-22.9 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, very close to close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Notes:

1. As-Drilled elevations for 100-series borings are derived from the survey file received from MaineDOT on May 24, 2022 entitled: "Borings.dgn" and refer to the North American Vertical Datum of 1988 (NAVD88)
   As-Drilled elevations for 200-series borings are derived from electronic file received from MaineDOT on October 13, 2023 entitled: "18595.10.BORINGLOC.200SERIES.10.11.23.csv" and refer to the North American Vertical Datum of 1988 (NAVD88)
2. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
3. 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 versus the run length. Note that vertical discontinuities are not included in determination of RQD.
4. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Willey, 2004 (based on ISRM, 1981).
5. 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 give 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.
6. Bedrock formation name from: Pollock, S.G., 2011. Bedrock Geology of the Veazie Quadrangle, Maine. Maine Geological Survey, Open-File No. 11-58, 1 sheet, scale 1:24,000.
7. ft = feet, in = inches

Prepared by: ATM/LMP
Checked by: FCT/KAR
Reviewed by: JRS/MEL

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Laboratory Testing[4,5,6]						Soil Classification	
					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO[7]	USCS[8]
HB-BHR-101	110.3	1U	10.0 - 12.0	100.3 - 98.3	-	28.0	36	21	15	0.5	-	CL
HB-BHR-101	110.3	3D	12.0 - 14.0	98.3 - 96.3	99.2	27.9	-	-	-	-	A-4 (0)	*CL*
HB-BHR-101	110.3	6D	25.0 - 27.0	85.3 - 83.3	-	9.0	non-plastic			-	-	*ML*
HB-BHR-102	109.2	1U	10.5 - 12.5	98.7 - 96.7	-	26.0	39	22	17	0.2	-	CL
HB-BHR-102	109.2	3D	12.5 - 14.5	96.7 - 94.7	99.4	27.6	39	22	17	0.3	A-6 (18)	CL
HB-BHR-102	109.2	5D	20.0 - 22.0	89.2 - 87.2	43.2	12.7	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-102	109.2	8D	35.0 - 37.0	74.2 - 72.2	47.2	9.4	18	13	5	-0.7	A-4 (0)	SM-SC
HB-BHR-103	106.8	2D	5.0 - 7.0	101.8 - 99.8	99.4	25.4	-	-	-	-	A-4 (0)	*CL*
HB-BHR-103	106.8	3D	10.0 - 11.4	96.8 - 95.4	97.0	24.0	28	17	11	0.6	A-6 (9)	CL
HB-BHR-104	104.2	2D	5.0 - 7.0	99.2 - 97.2	-	25.5	45	23	22	0.1	-	CL
HB-BHR-104	104.2	4D-A	15.0 - 15.9	89.2 - 88.3	99.7	31.6	-	-	-	-	A-4 (0)	*CL*
HB-BHR-105	100.7	2D	5.0 - 7.0	95.7 - 93.7	-	25.2	38	22	16	0.2	-	CL
HB-BHR-105	100.7	4D	15.0 - 17.0	85.7 - 83.7	99.8	29.8	32	19	13	0.8	A-6 (13)	CL
HB-BHR-105	100.7	5D	20.0 - 22.0	80.7 - 78.7	32.6	9.3	-	-	-	-	A-2-4 (0)	*GM-GC*
HB-BHR-106	105.9	3D	10.0 - 12.0	95.9 - 93.9	-	25.6	35	21	14	0.3	-	CL
HB-BHR-107	88.3	3D	10.0 - 12.0	78.3 - 76.3	-	30.0	42	23	19	0.4	-	CL
HB-BHR-107	88.3	4D-A	15.0 - 16.7	73.3 - 71.6	99.7	29.1	-	-	-	-	A-4 (0)	*CL*
HB-BHR-108	82.3	2D	5.0 - 7.0	77.3 - 75.3	-	25.5	37	22	15	0.2	-	CL
HB-BHR-108	82.3	1U	8.0 - 10.0	74.3 - 72.3	-	28.0	35	20	15	0.6	-	CL
HB-BHR-108	82.3	3D-A	10.0 - 11.6	72.3 - 70.7	-	26.5	32	21	11	0.5	-	CL
HB-BHR-108	82.3	4D	15.0 - 17.0	67.3 - 65.3	12.6	6.0	-	-	-	-	A-1-a (0)	*GM-GC*
HB-BHR-108	82.3	6D	25.0 - 27.0	57.3 - 55.3	39.9	11.6	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-109	78.3	2D	5.0 - 7.0	73.3 - 71.3	88.3	27.5	37	18	19	0.5	A-6 (16)	CL
HB-BHR-109	78.3	4D	15.0 - 17.0	63.3 - 61.3	44.8	11.9	-	-	-	-	A-4 (0)	*GM-GC*
HB-BHR-109	78.3	6D	27.0 - 29.0	51.3 - 49.3	25.5	9.1	-	-	-	-	A-2-4 (0)	*SM-SC*

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Laboratory Testing[4,5,6]						Soil Classification	
					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO[7]	USCS[8]
HB-BHR-110	76.5	2D-A	5.0 - 6.9	71.5 - 69.6	-	44.6	69	28	41	0.4	-	CH
HB-BHR-111	97.8	2D	5.0 - 7.0	92.8 - 90.8	-	25.0	39	18	21	0.3	-	CL
HB-BHR-111	97.8	4D	15.0 - 17.0	82.8 - 80.8	54.2	12.5	-	-	-	-	A-4 (0)	*CL*
HB-BHR-111	97.8	5D	20.0 - 22.0	77.8 - 75.8	46.4	9.9	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-112	97.5	2D	5.0 - 7.0	92.5 - 90.5	-	15.9	21	12	9	0.4	-	CL
HB-BHR-112	97.5	4D	15.0 - 17.0	82.5 - 80.5	52.2	14.5	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-112	97.5	5D	20.0 - 22.0	77.5 - 75.5	97.3	21.9	-	-	-	-	A-4 (0)	*CL*
HB-BHR-113	92.2	2D	5.0 - 7.0	87.2 - 85.2	80.7	15.6	33	17	16	-0.1	A-6 (11)	CL
HB-BHR-113	92.2	3D	10.0 - 12.0	82.2 - 80.2	43.7	12.6	-	-	-	-	A-4 (0)	*SM-SC, GM-GC*
HB-BHR-113	92.2	5D	17.0 - 19.0	75.2 - 73.2	36.5	17.8	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-113	92.2	6D	20.0 - 22.0	72.2 - 70.2	77.7	18.1	22	14	8	0.5	A-4 (3)	CL
HB-BHR-113	92.2	8D	27.0 - 29.0	65.2 - 63.2	-	14.1	non-plastic			-	-	*ML*
HB-BHR-113	92.2	9D	30.0 - 32.0	62.2 - 60.2	32.7	8.9	-	-	-	-	A-2-4 (0)	*SM-SC, GM-GC*
HB-BHR-113	92.2	11D	40.0 - 42.0	52.2 - 50.2	49.8	11.3	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-114	92.7	2D	5.0 - 7.0	87.7 - 85.7	67.7	16.5	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-114	92.7	5D	20.0 - 22.0	72.7 - 70.7	71.8	28.3	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-114	92.7	7D	30.0 - 32.0	62.7 - 60.7	39.7	9.2	18	12	6	-0.5	A-4 (0)	SC-SM
HB-BHR-114	92.7	10D	45.0 - 47.0	47.7 - 45.7	51.1	10.3	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-114	92.7	12D	55.0 - 57.0	37.7 - 35.7	31.4	12.2	-	-	-	-	A-2-4 (0)	*SM-SC*

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					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO[7]	USCS[8]
HB-BHR-115	96.7	1D	1.0 - 3.0	95.7 - 93.7	12.9	5.2	-	-	-	-	A-1-b (0)	*SM-SC*
HB-BHR-115	96.7	2D	5.0 - 7.0	91.7 - 89.7	61.0	12.6	21	12	9	0.1	A-4 (2)	CL
HB-BHR-115	96.7	4D	15.0 - 17.0	81.7 - 79.7	98.4	25.8	36	21	15	0.3	A-6 (16)	CL
HB-BHR-115	96.7	7D	30.0 - 32.0	66.7 - 64.7	58.9	12.3	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-115	96.7	9D	40.0 - 42.0	56.7 - 54.7	68.7	12.6	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-115	96.7	11D	50.0 - 52.0	46.7 - 44.7	65.7	13.7	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-116	93.5	2D	5.0 - 7.0	88.5 - 86.5	99.4	21.9	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-116	93.5	4D	15.0 - 17.0	78.5 - 76.5	33.1	10.3	-	-	-	-	A-2-4 (0)	*SM-SC*
HB-BHR-116	93.5	7D	30.0 - 32.0	63.5 - 61.5	68.0	12.2	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-116	93.5	11D	50.0 - 52.0	43.5 - 41.5	33.6	9.4	-	-	-	-	A-2-4 (0)	*SM-SC*
HB-BHR-116	93.5	14D	72.0 - 74.0	21.5 - 19.5	15.4	9.0	-	-	-	-	A-1-b (0)	*SM*
HB-BHR-203	114.0	2D	5.00 - 7.00	109.0 - 107.0	99.0	23.8	31	19	12	0.4	A-6 (12)	CL
HB-BHR-204	116.0	2D	5.00 - 7.00	111.0 - 109.0	-	24.0	37	23	14	0.1	-	CL
HB-BHR-204	116.0	3D	10.00 - 12.00	106.0 - 104.0	52.8	12.4	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-205	108.1	2D	5.00 - 7.00	103.1 - 101.1	99.5	27.7	-	-	-	-	A-4 (0)	*ML*
HB-BHR-205	108.1	3D	10.00 - 12.00	98.1 - 96.1	-	24.8	30	18	12	0.6	-	CL
HB-BHR-206	118.4	3D	10.00 - 12.00	108.4 - 106.4	-	22.7	35	23	12	0.0	-	CL
HB-BHR-206	118.4	5D	20.00 - 22.00	98.4 - 96.4	55.7	14.6	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-207	116.7	2D	5.00 - 7.00	111.7 - 109.7	70.8	-	-	-	-	-	A-4 (0)	*ML*
HB-BHR-207	116.7	4D	15.00 - 17.00	101.7 - 99.7	-	27.8	44	21	23	0.3	-	CL
HB-BHR-208	90.8	2D	5.00 - 7.00	85.8 - 83.8	-	24.5	37	20	17	0.3	-	CL
HB-BHR-208	90.8	4D	15.00 - 17.00	75.8 - 73.8	-	28.2	23	16	7	1.7	-	CL-ML
HB-BHR-209	105.1	2D	5.00 - 7.00	100.1 - 98.1	68.0	10.1	-	-	-	-	A-4 (0)	*ML*
HB-BHR-209	105.1	4D	15.00 - 17.00	90.1 - 88.1	32.2	10.2	-	-	-	-	A 2-4 (0)	*SM-SC*

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Laboratory Testing[4,5,6]						Soil Classification	
					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO[7]	USCS[8]
HB-BHR-210	102.1	2D	5.00 - 7.00	97.1 - 95.1	-	10.5	-	-	-	-	-	-
HB-BHR-210	102.1	4D	15.0 - 17.0	87.1 - 85.1	-	12.0	-	-	-	-	-	-
HB-BHR-210	102.1	6D	25.0 - 27.0	77.1 - 75.1	-	23.6	36	20	16	0.2	-	CL
HB-BHR-210	102.1	9D	40.0 - 42.0	62.1 - 60.1	23.4	6.8	-	-	-	-	A-1-b (0)	GM-GC
HB-BHR-211	98.5	2D	5.0 - 7.0	93.5 - 91.5	81.0	22.1	-	-	-	-	A-4 (0)	*ML*
HB-BHR-211	98.5	4D	15.0 - 17.0	83.5 - 81.5	-	30.1	34	21	13	0.7	-	CL
HB-BHR-211	98.5	5D	20.0 - 22.0	78.5 - 76.5	38.7	11.8	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-212	97.5	2D	5.0 - 7.0	92.5 - 90.5	17.7	-	-	-	-	-	A-1-b (0)	*SM*
HB-BHR-212	97.5	3D	10.0 - 12.0	87.5 - 85.5	-	21.4	27	18	9	0.4	-	CL
HB-BHR-212	97.5	4D	15.0 - 17.0	82.5 - 80.5	97.7	24.7	38	20	18	0.3	A-6 (19)	CL
HB-BHR-212	97.5	5D	20.0 - 22.0	77.5 - 75.5	42.4	11.5	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-213	81.5	2D	5.0 - 7.0	76.5 - 74.5	16.4	-	-	-	-	-	A-1-b (0)	*SM*
HB-BHR-213	81.5	4D	15.0 - 17.0	66.5 - 64.5	-	13.9	18	13	5	0.2	-	CL-ML
HB-BHR-214	78.8	3D	10.0 - 12.0	68.8 - 66.8	43.0	-	-	-	-	-	A-4 (0)	*SM-SC*
HB-BHR-214	78.8	6D	25.0 - 27.0	53.8 - 51.8	58.7	12.8	-	-	-	-	A-4 (0)	*CL-ML*
HB-BHR-215	78.6	2D	5.0 - 7.0	73.6 - 71.6	91.8	17.9	-	-	-	-	A-4 (0)	*ML*
HB-BHR-215	78.6	5D	20.0 - 22.0	58.6 - 56.6	-	9.9	17	12	5	-0.4	-	CL-ML
HB-BHR-216	78.2	3D	10.0 - 12.0	68.2 - 66.2	-	15.6	25	17	8	-0.2	-	CL
HB-BHR-216	78.2	6D	25.0 - 27.0	53.2 - 51.2	30.1	9.7	-	-	-	-	A-2-4 (0)	*SM*

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Laboratory Testing[4,5,6]						Soil Classification	
					Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	AASHTO[7]	USCS[8]

Notes:

1. Test boring locations of the 100 series and 200 series borings are illustrated in the plans titled "Boring Location Plan Sheets 2 through 7".

2. As-drilled elevations for borings HB-BHR-101 through HB-BHR-116 and BB-BHR-101 through BB-BHR-103 are derived from the electronic file "borings.dgn" provided by MaineDOT to WSP Golder on May 24, 2022 and refer to the North American Vertical Datum of 1988 (NAVD88). As-drilled locations and elevations for borings HB-BHR-201 through HB-BHR-217 are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.

4. The particle size testing was done in accordance with ASTM D6913, Standard Test Mehtods for Particle Size Distribution (Gradation) of Soils Using Sieve Analysis and ASTM D7928, Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis

5. Moisture content testing was done in accrodance with ASTM D2216, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

6. Atterberg Limits testing was done in accordance with ASTM D4318, Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

7. 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.

8. USCS classification perormed by GeoTesting Express when ASTM D 4318, ASTM D6913, and ASTM D7928 data is available. USCS performed by WSP based on interpreted laboratory test results are in in italics and based on similar soils in the same or nearby borings and fines type based on field characterization.

9. Complete laboratory soil test results are provided in Appendix C.

Prepared By: LDN
Checked By: DEB
Reviewed By: MEL

Table 5: Summary of Laboratory Soil Engineering Property Testing Results
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 18595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface	Approximate Sample Elevation	Tube X-ray	Test Type[4]	Test No.	Bulk Unit Weight[6]	Void Ratio, e	Estimated In Situ Vertical Effective Stress, σ'_{v0}[7]	Consolidation Vertical Effective Stress, σ'_{vc}[8]	Undrained Shear Strength, s_u[9]	Pre-consolidation Stress, σ'_p[10]	Over-consolidation Ratio, OCR[11]	Recompression Index, C_r[12]	Compression Index, C_c[12]	Coefficient of Consolidation, c_v[13]	Secondary Compression Index, C_a[14]	Secondary Recompression Index, C_{ar}[14]
(-)	(-)	(-)	(feet)	(feet)	(-)	(-)	(-)	(pcf)	(-)	(psf)	(psf)	(psf)	(psf)		(-)	(-)	(ft^2/day)	(-)	(-)
HB-BHR-101	110.3	1U	11.0 - 11.3	99.3 - 99.0	YES	ILC	IP-3	120	0.86	930	-	-	3550	2.0	0.02	0.20	0.4	0.001	0.0002
			11.3 - 11.6	99.0 - 98.7	YES	DSS	DSS-3	121	-	951	925	644	-	-	-	-	-	-	-
			11.5 - 11.8	98.8 - 98.5	YES	CRSC	CRC-3	121	0.87	959	-	-	17100 [15]	15.9 [15]	0.03	0.30	42.0	-	-
HB-BHR-102	109.2	1U	11.3 - 11.6	97.9 - 97.6	YES	ILC	IP-2	124	0.76	1050	-	-	2600	4.0	0.01	0.11	0.8	0.001	0.0003
			11.7 - 12.0	97.5 - 97.2	YES	DSS	DSS-2	124	-	1088	1050	626	-	-	-	-	-	-	-
			12.0 - 12.3	97.2 - 96.9	YES	CRSC	CRC-2	126	0.72	1091	-	-	19750 [15]	17.4 [15]	0.02	0.17	12.6	-	-
HB-BHR-108	82.3	1U	8.4 - 8.7	73.9 - 73.6	YES	ILC	IP-1	122	0.79	657	-	-	5500	5.2	0.01	0.14	0.4	0.001	0.0004
			8.7 - 9.0	73.6 - 73.3	YES	DSS	DSS-1	122	-	688	900.0	688.0	-	-	-	-	-	-	-
			9.5 - 9.8	72.8 - 72.5	YES	CRSC	CRC-1	121	0.88	720	-	-	12250 [15]	14.6 [15]	0.02	0.23	120.0	-	-
BB-BHR-101	99.7	1U	15.3 - 15.3	84.5 - 84.4	YES	ILC	IP-1	121	0.84	1147	-	-	4000	3.5	0.04	0.25	0.19	0.007	0.004
			15.6 - 15.7	84.1 - 84.0	YES	DSS	DSS-1	119	0.87	1167	1175	493	-	-	-	-	-	-	-
			16.6 - 16.7	83.1 - 83.0	YES	CRSC	CRC-1	123	0.82	1225	-	-	3200	2.6	0.01	0.20	5.4	-	-
BB-BHR-103	88.5	1U	15.8 - 15.9	72.7 - 72.6	YES	CRSC	CRC-2	124	0.79	1328	-	-	3500	2.6	0.01	0.20	4.2	-	-

Notes:

1. Test boring locations are illustrated in the plans titled "Boring Location Plan Sheets 2 through 5"
2. As-drilled elevations for borings HB-BHR-101, HB-BHR-102 and HB-BHR-108 and BB-BHR-101 through BB-BHR-103 are derived from the electronic file "borings.dgn" provided by MaineDOT to WSP Golder on May 24, 2022 and refer to the North American Vertical Datum of 1988 (NAVD88).
3. Laboratory testing was performed by Geotesting Express of Acton, MA
4. ILC = Incremental Load Consolidation, CRSC = Constant Rate of Strain Consolidation, DSS = Direct Simple Shear, CAUC = Consolidated Anisotropic Undrained Compression
5. DSS tests were performed using the recompression method, where the sample was consolidated to the estimated in situ effective stress before shear.
6. Bulk unit weight values were measured during engineering property testing. Refer to Appendix C for more detailed laboratory results.
7. In situ vertical effective stress for each sample was estimated using the interpreted engineering profiles, average water table, and estimates or measurements of unit weight for each soil type in the profile.
8. Vertical effective stress state at the end of consolidation, also known as the pre-shear effective stress.
9. Undrained shear strength determined as the peak of the stress vs. strain plot resulting from DSS tests and CAUC test.
10. Preconsolidation stress determind using the Casagrande Construction method on stress vs. void ratio plots from both ILC and CRSC tests.
11. Overconsolidation ratio is OCR = σ'_p/σ'_{v0}
12. Recompression and compression indices were determined by interpretation of stress vs. void ratio plots from ILC and CRSC tests.
13. Coefficient of consolidation values reported are from the ILC and CRSC tests.
14. Secondary compression and recompression indices were determined by interpretation of strain vs. log time plots from ILC testing.
15. CRSC tests yielded noticeably large preconsolidation stresses, thought to be the result of normalized pore pressures that were too low and not in compliance with ASTM D4186. OCR is also affected.
16. Bridge boring data is included and was utilized to better characterize the subsurface conditions.
17. Complete laboratory test results for soil testing are provided in Appendix C.

Prepared By: ATM
Checked By: MA
Reviewed By: MEL

Table 6: Summary of Mast Arm, OHSS, and Span Wire Pole Foundation Locations
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Structure	Description[1]	Station[2]	Offset[2]	Ground Surface Elevation[2] (ft)	
				Proposed	Existing
Mast Arm Foundations					
Pole A-M1	60-foot mast arm	12+33.41	50.3 feet RT	115.0	115.0
Pole B-M1	50-foot mast arm	11+40.93	72.3 feet RT	116.0	116.0
Pole C-M1	60-foot mast arm	11+36.46	61.8 feet LT	118.0	118.0
Pole D-M1	45-foot mast arm	12+31.57	116.7 feet LT	118.7	118.7
Pole C-M1	35-foot mast arm	20+47.11	24.7 feet LT	118.2	118.5
Pole A-M1	40-foot mast arm	29+43.37	35.4 feet RT	107.0	105.5
Pole D-M1	15-foot mast arm	28+20.45	3.1 feet RT	110.6	101.4
Pole D-M2	15-foot mast arm	30+86.82	6.9 feet RT	101.5	94.9
Pole A-M1	55-foot mast arm	37+17.84	48.1 feet RT	81.4	81.4
Pole B-M1	60-foot mast arm	38+66.09	33.6 feet LT	79.0	79.0
Pole C-M1	55-foot mast arm	38+66.80	51.7 feet RT	78.6	78.6
Pole D-M1	30-foot mast arm	37+76.27	83.0 feet RT	77.0	77.0
Overhead Sign Structures (OHSS)					
Pole C-1	45-foot mast arm	104+75.00	24.4 feet LT	115.7	116.6
Pole C-2	35-foot mast arm	900+00.00	12.3 feet LT	114.8	97.6
Pole C-1 (Right)	87-foot mast arm, bridge type	219+50.00	39.2 feet RT	96.9	94.4
Pole C-1 (Left)	87-foot mast arm, bridge type	401+50.00	27.0 feet RT	98.7	97.2
Span Wire Poles					
Pole A	125-foot span wire	15+36.44	71.95 feet LT	115.5	113.3
Pole B	125-foot span wire	15+36.44	52.92 feet RT	112.9	112.4
Pole A	113.5-foot span wire	35+57.60	64.62 feet RT	85.1	79.8
Pole B	113.5-foot span wire	35+57.60	48.35 feet LT	86.1	87.5

Notes:

1. Based on mast arm, OHSS, and span wire pole loading scenario information provided by VHB.
2. Spanwire and mast arm stationing obtained from project centerline. OHSS stationing provided by VHB on OHSS loading summary.

Prepared By: LDN
Checked By: JKW
Reviewed By: CCB

Table 7: Factored Geotechnical Resistance and Lateral Deflections for the Drilled Shafts
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Structure	Location	Shaft Diameter[1] (in)	Shaft Length[1,2] (ft)	Factored Axial Compression Resistance (kips)	Lateral Shaft Head Deflection[3] (in)	Factored Torsional Resistance (kip-ft)
Mast Arm Foundations						
Pole A-M1	12+33.41	48	10.0	4822	0.11	514
Pole B-M1	11+40.93	48	12.5	414	0.08	467
Pole C-M1	11+36.46	48	7.5	2316	0.04	1476
Pole D-M1	12+31.57	48	7.5	2487	0.01	2092
Pole C-M1	20+47.11	48	10.0	97	0.14	177
Pole A-M1	29+43.37	48	10.0	223	0.12	243
Pole D-M1	28+20.45	48	10.0	163	0.09	182
Pole D-M2	30+86.82	48	10.0	218	0.05	208
Pole A-M1	37+17.84	48	10.0	222	0.20	236
Pole B-M1	38+66.09	48	10.0	220	0.25	217
Pole C-M1	38+66.80	48	10.0	187	0.22	218
Pole D-M1	37+76.27	48	10.0	77	0.08	208
Overhead Sign Structures (OHSS)						
Pole C-1	104+75.00	72	20.0	201	0.09	784
Pole C-2	900+00.00	60	15.0	284	0.12	578
Pole C-1 (Right)	219+50.00	60	15.0	147	0.15	-
Pole C-1 (Left)	401+50.00	60	15.0	123	0.12	-
Span Wire Poles						
Pole A	15+36.44	48	10.0	153	0.29	-
Pole B	15+36.44	48	10.0	115	0.24	-
Pole A	35+57.60	48	10.0	219	0.20	-
Pole B	35+57.60	48	10.0	223	0.17	-

Notes:

1. The drilled shaft diameters and lengths are adequate for the Strength I, Extreme I, and Service I limit load cases, considering axial, shear, moment, and torsion loading provided by VHB. See Appendix E- Drilled Shaft Design Calculations, for details.
2. Shaft length assumes the top of the shaft is at the proposed final grade. This length ignores the maximum 3 inches of reveal above ground level allowable in certain conditions noted in MaineDOT Standard Specification Section 626.034 and in MaineDOT Standard Details 626(01) and 626(02). If the location of the shaft allows for VHB to specify a reveal above the ground surface, we recommend the shaft length be increased by 0.5 feet.
3. At the Service Limit State.

Prepared By: LDN
Checked By: JKW
Reviewed By: CCB

Table 8: Maxiumum Bending Moment for Drilled Shafts
Geotechnical Design Report
Hogan Road Diverging Diamond Interchange, Bangor, Maine
MaineDOT WIN 018595.11

Structure	Station	Shaft Diameter[1] (in)	Shaft Length[1,2] (ft)	Load Cases		
				Strength I M_{Max} (kip-ft)	Extreme I M_{max} (kip-ft)	Service I M_{max} (kip-ft)
Mast Arm Foundations						
Pole A-M1	12+33.41	48	10.0	1800.0	2270.2	1608.3
Pole B-M1	11+40.93	48	12.5	1320.0	1922.2	1257.9
Pole C-M1	11+36.46	48	7.5	1800.0	2368.0	1645.0
Pole D-M1	12+31.57	48	7.5	1080.0	1794.0	1048.7
Pole C-M1	20+47.11	48	10.0	600.0	1514.1	741.8
Pole A-M1	29+43.37	48	10.0	720.0	1522.4	867.9
Pole D-M1	28+20.45	48	10.0	120.0	882.4	393.5
Pole D-M2	30+86.82	48	10.0	120.0	511.2	255.6
Pole A-M1	37+17.84	48	10.0	1560.0	1996.7	1341.6
Pole B-M1	38+66.09	48	10.0	1800.0	2138.8	1598.0
Pole C-M1	38+66.80	48	10.0	1560.0	2138.8	1478.4
Pole D-M1	37+76.27	48	10.0	360.0	1283.9	631.2
Overhead Sign Structures (OHSS)						
Pole C-1	104+75.00	72	20.0	2160.0	7742.9	3233.4
Pole C-2	900+00.00	60	15.0	1200.0	4948.4	1947.2
Pole C-1 (Right)	219+50.00	60	15.0	0.0	5954.8	2385.4
Pole C-1 (Left)	401+50.00	60	15.0	0.0	5947.4	2105.6
Span Wire Poles						
Pole A	15+36.44	48	10.0	509.3	2894.2	1377.2
Pole B	15+36.44	48	10.0	511.0	2896.9	1259.0
Pole A	35+57.60	48	10.0	511.3	2638.1	1120.1
Pole B	35+57.60	48	10.0	508.6	2515.5	1118.0

Notes:

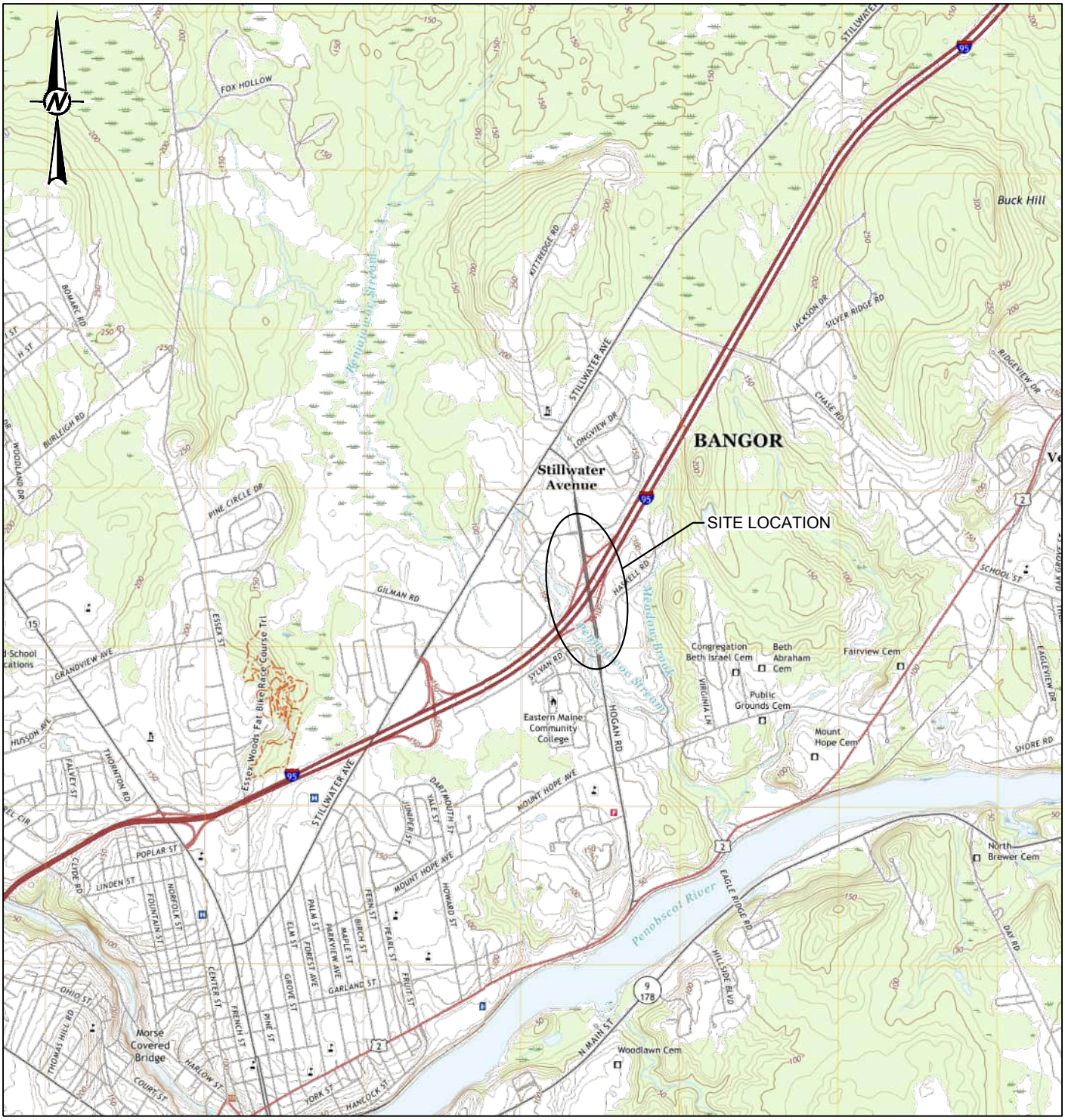
1. The drilled shaft diameters and lengths are adequate for the Strength I, Extreme I, and Service I limit load cases, considering axial, shear, moment, and torsion loading provided by VHB. See Appendix E- Drilled Shaft Design Calculations, for details.

2. Shaft length assumes the top of the shaft is at the proposed final grade. This length ignores the maximum 3 inches of reveal above ground level allowable in certain conditions noted in MaineDOT Standard Specification Section 626.034 and in MaineDOT Standard Details 626(01) and 626(02). If the location of the shaft allows for VHB to specify a reveal above the ground surface, we recommend the shaft length be increased by 0.5 feet.

3. The drilled shaft was modelled as a simple linear elastic beam with an elastic modulus equal to that of concrete and a moment of inertia equal to that of the uncracked circular cross section.

Prepared By: LDN
Checked By: JKW
Reviewed By: CCB

Figures



REFERENCE(S)

BASE MAPS TAKEN FROM U.S.G.S. 7.5 MINUTE QUADRANGLE OF VEAZIE, MAINE DATED 2021
AND U.S.G.S. 7.5 MINUTE QUADRANGLE OF BANGOR, MAINE DATED 2021

0 1500 3000
1" = 3000' FEET

CLIENT
MAINE DEPARTMENT OF TRANSPORTATION
16 STATE HOUSE STATION
AUGUSTA, MAINE 04333-0016

CONSULTANT
wsp

YYYY-MM-DD	2023-12-11
DESIGNED	DEB
PREPARED	AM
REVIEWED	MEL
APPROVED	

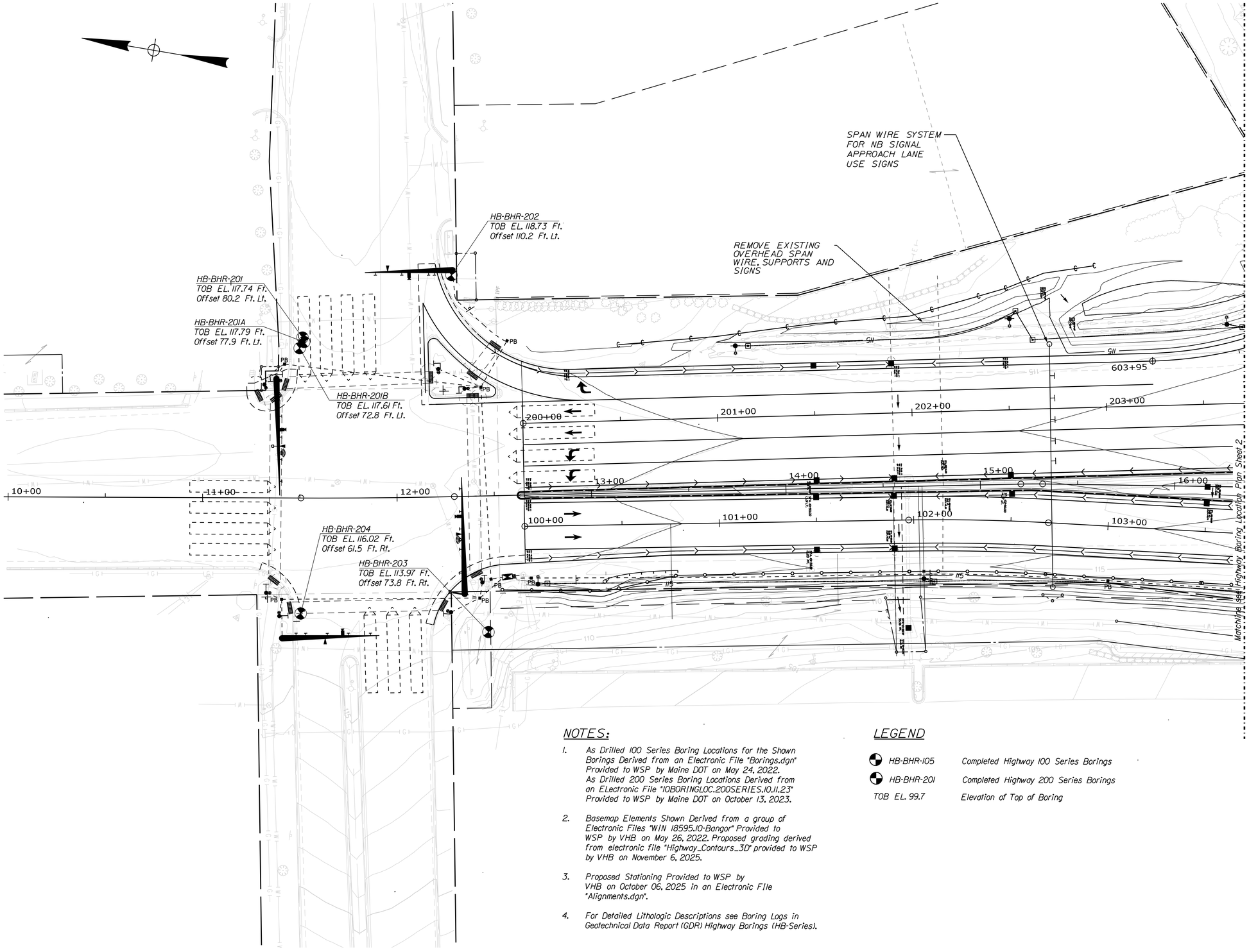
PROJECT
I-95 HOGAN ROAD BRIDGE DIVERGING DIAMOND INTERCHANGE (EXIT 187) BANGOR, MAINE
(WIN 018595.11)

TITLE
SITE LOCATION MAP

PROJECT NO.	CONTROL	REV.	SHEET
31404817.003	0001-001	0	1

Last Edited By: amorales Date: 2023-12-11 Time:8:37:27 AM | Printed By: AMorales Date: 2023-12-11 Time:8:38:30 AM
Path: \\golder.gds\complexdata\office\manchester\SIM\MEDOT\HoganRd\99_PROJ\31404817.003_hoganrd\0001_geotechdesignreport40_PROD\ | File Name: 31404817003-0001-001.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
0 1in



NOTES:

1. As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022. As Drilled 200 Series Boring Locations Derived from an ELectronic File "10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.
3. Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic FIle "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

BRIDGE NO. 5823

WIN 018595.11

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

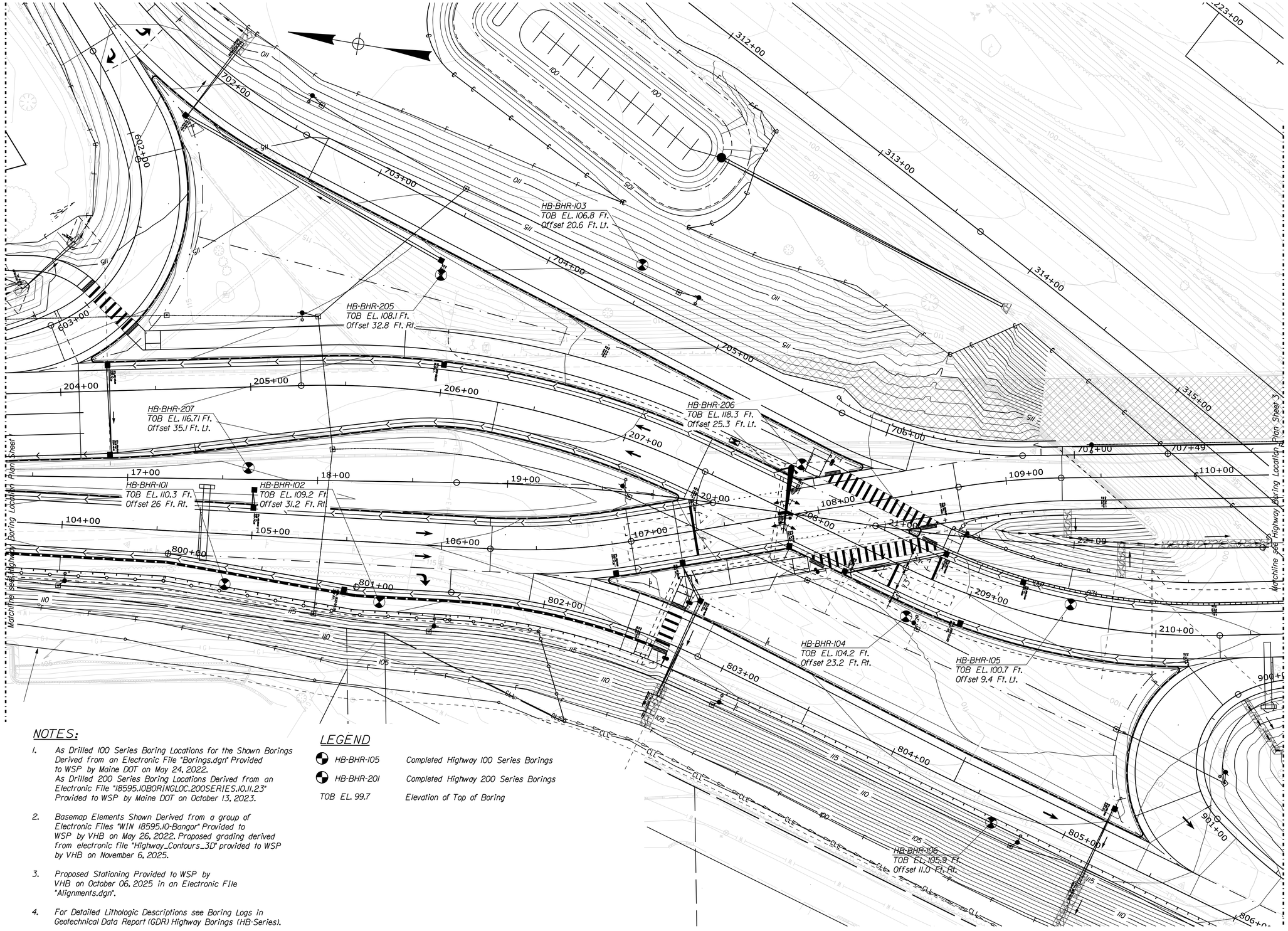
HIGHWAY BORING LOCATION PLAN SHEET 1

SHEET NUMBER

2

Filename: ...\BRIDGE\MSTA\hw_borplan_01.dgn Division: BRIDGE Username: Date:11/7/2025

Filename: ...\BRIDGE\MSTA\hw_borplan_02.dgn Division: BRIDGE Username: Date:11/7/2025



NOTES:

1. As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022.
   As Drilled 200 Series Boring Locations Derived from an Electronic File "18595.10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.
3. Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic File "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE NO. 5823

BRIDGE PLANS

PROJ. MANAGER	M. LANDON	BY	DATE
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

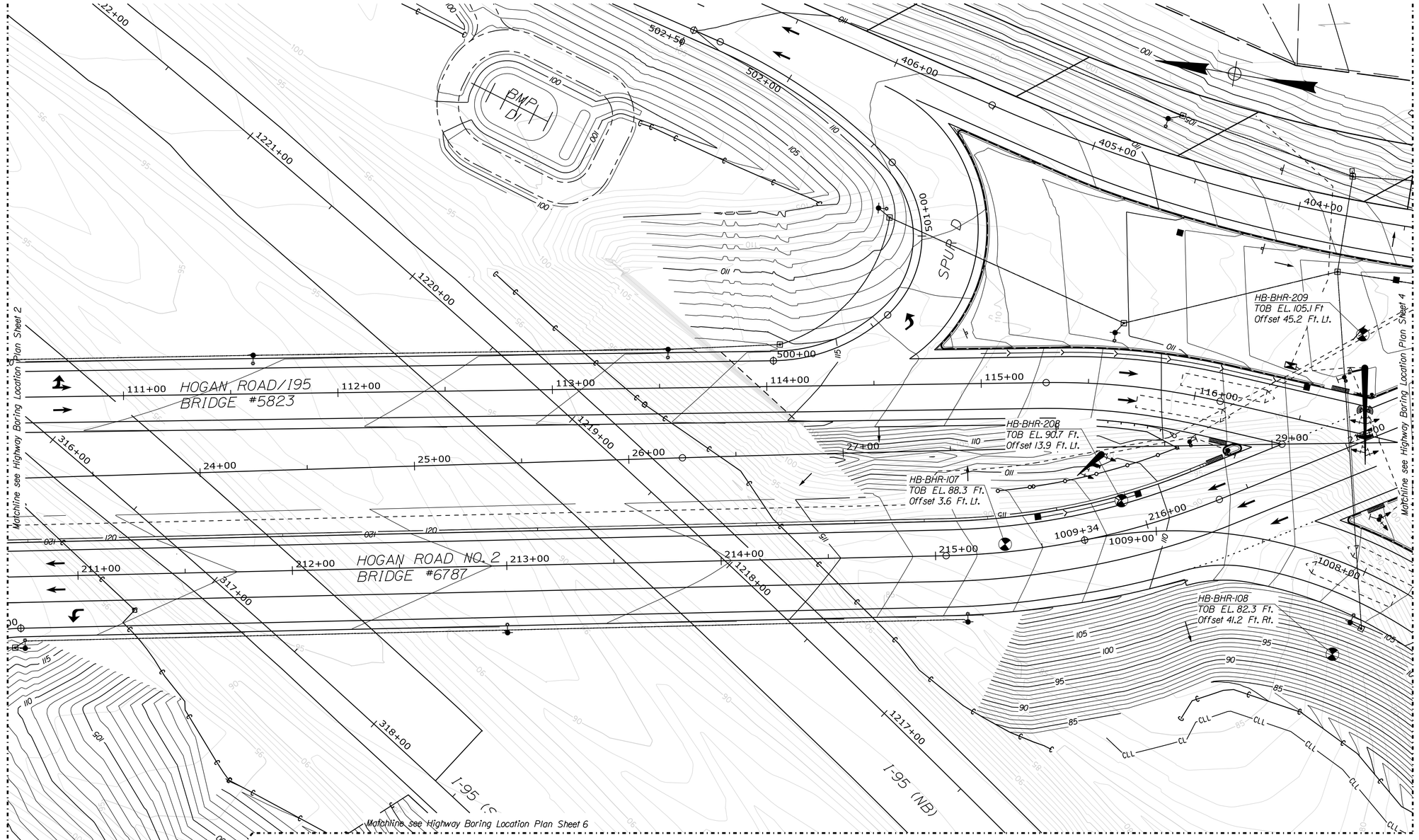
DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

HIGHWAY BORING LOCATION PLAN SHEET 2

SHEET NUMBER

3



NOTES:

1. *As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022. As Drilled 200 Series Boring Locations Derived from an Electronic File "18595.10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.*
2. *Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.*
3. *Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic File "Alignments.dgn".*
4. *For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).*

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

BRIDGE NO. 5823

WIN 018595.11

BRIDGE PLANS

PROJ. MANAGER	M. LANDON	BY	DATE
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

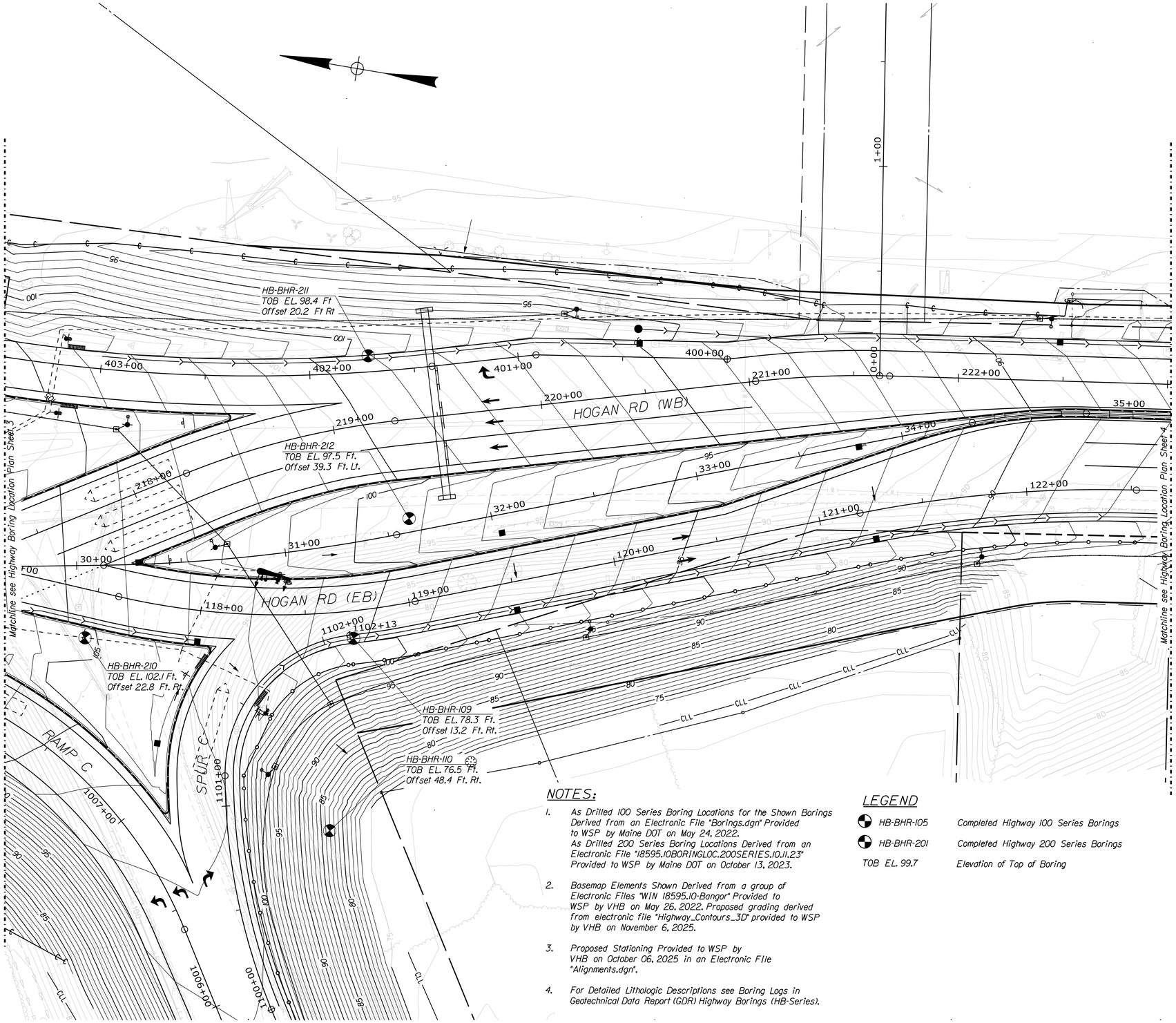
HIGHWAY BORING LOCATION PLAN SHEET 3

SHEET NUMBER

4

4 OF 13

Filename: ...\BRIDGE\MSTA\hw_borplan_03.dgn Division: BRIDGE Username: Date:11/7/2025



NOTES:

1. As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022.
   As Drilled 200 Series Boring Locations Derived from an Electronic File "18595.10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.
3. Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic File "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

BRIDGE NO. 5823 | **WIN 018595.11** | BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

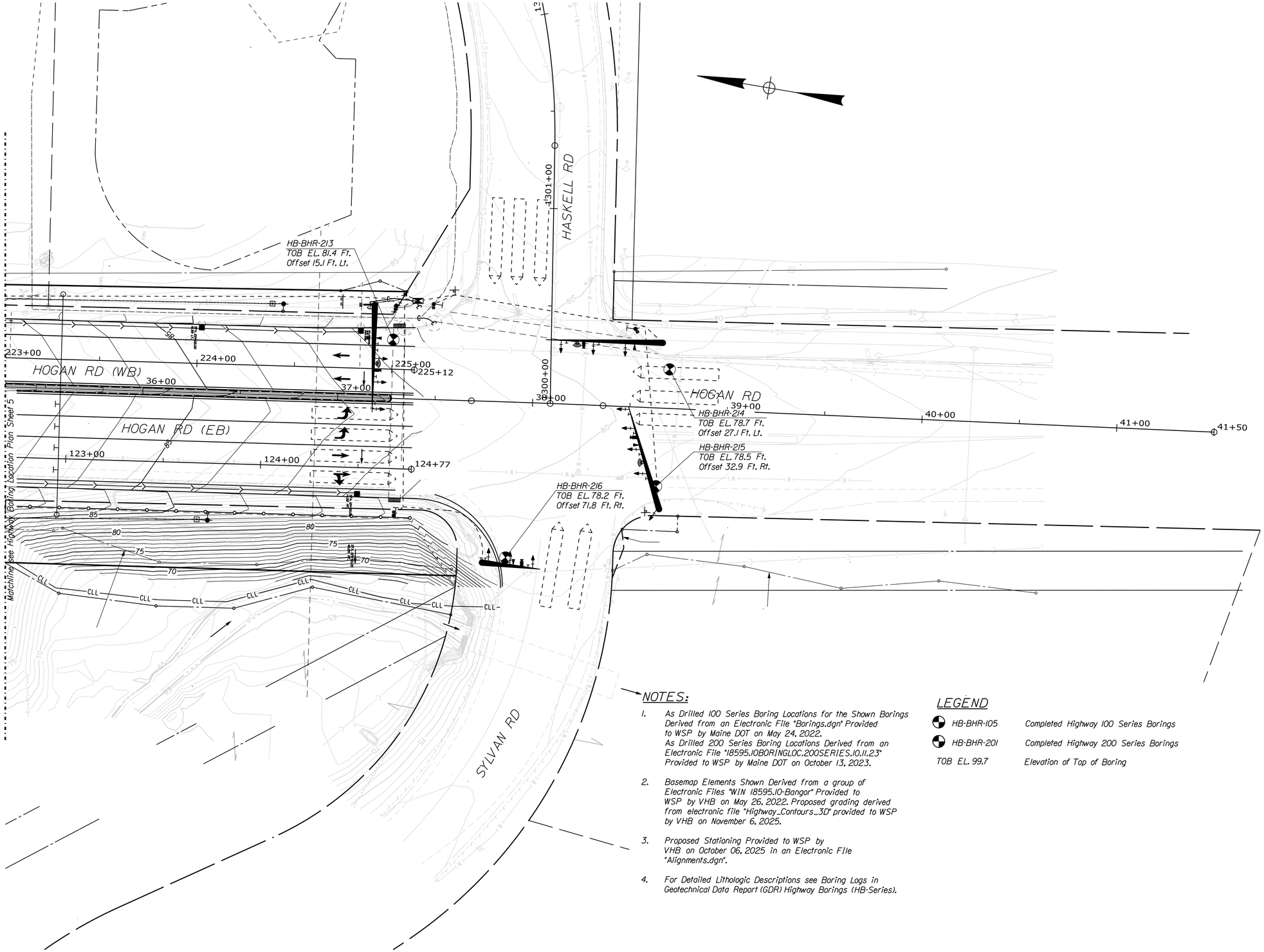
I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

HIGHWAY BORING LOCATION PLAN SHEET 4

SHEET NUMBER

5

Filename: ...\BRIDGE\MSTA\hw_borplan_04.dgn Division: BRIDGE Username: Date:11/7/2025



NOTES:

1. As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022.
   As Drilled 200 Series Boring Locations Derived from an Electronic File "18595.10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.
3. Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic File "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE NO. 5823

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

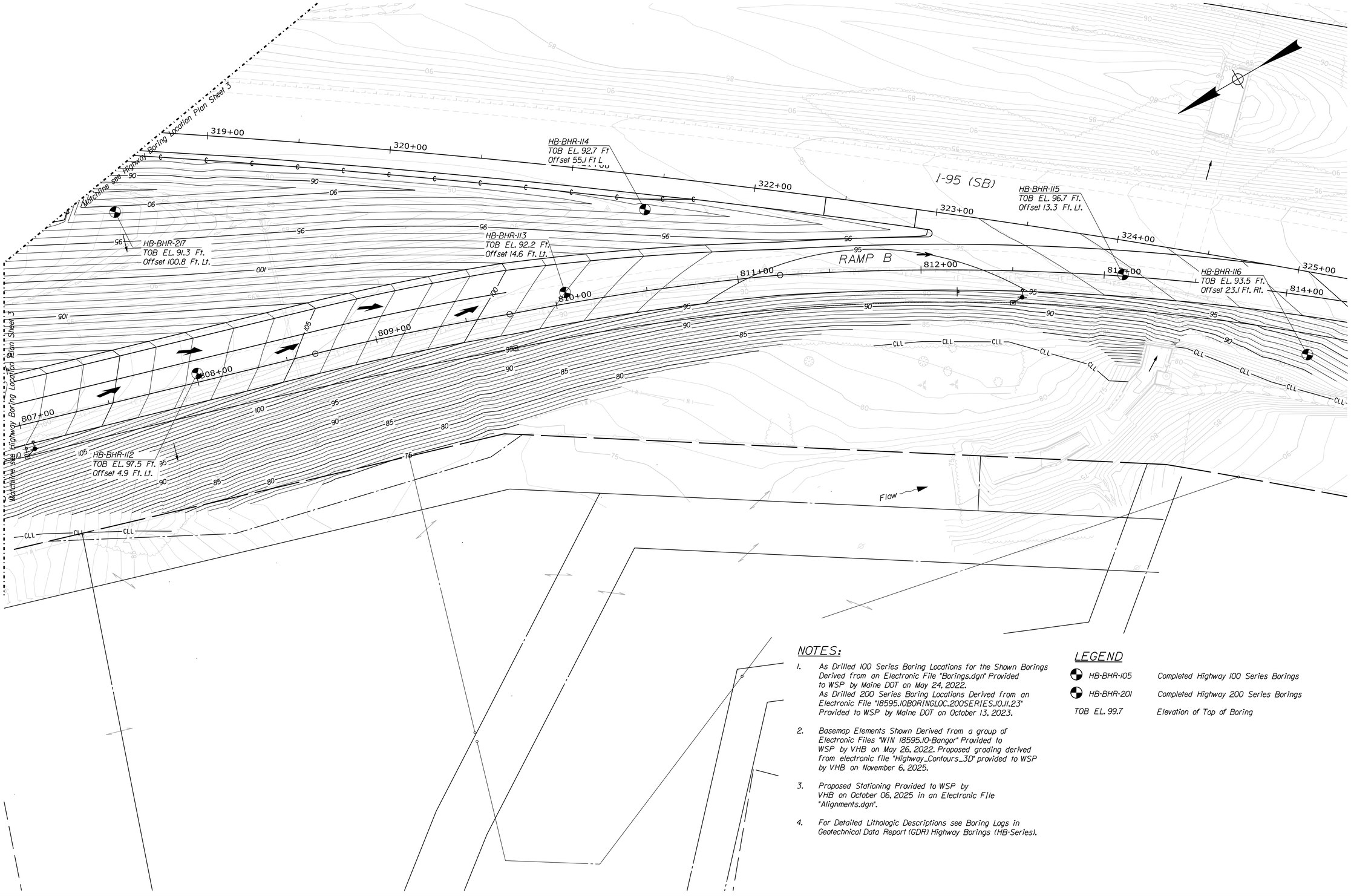
HIGHWAY BORING LOCATION PLAN SHEET 5

SHEET NUMBER

6

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Filename: ...\BRIDGE\MSTA\hw_borplan_06.dgn Username: Division: BRIDGE Date:11/7/2025



NOTES:

1. As Drilled 100 Series Boring Locations for the Shown Borings Derived from an Electronic File "Borings.dgn" Provided to WSP by Maine DOT on May 24, 2022.
   As Drilled 200 Series Boring Locations Derived from an Electronic File "18595.10BORINGLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT on October 13, 2023.
2. Basemap Elements Shown Derived from a group of Electronic Files "WIN 18595.10-Bangor" Provided to WSP by VHB on May 26, 2022. Proposed grading derived from electronic file "Highway_Contours_3D" provided to WSP by VHB on November 6, 2025.
3. Proposed Stationing Provided to WSP by VHB on October 06, 2025 in an Electronic File "Alignments.dgn".
4. For Detailed Lithologic Descriptions see Boring Logs in Geotechnical Data Report (GDR) Highway Borings (HB-Series).

LEGEND

HB-BHR-105	Completed Highway 100 Series Borings
HB-BHR-201	Completed Highway 200 Series Borings
TOB EL. 99.7	Elevation of Top of Boring

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE NO. 5823 — BRIDGE PLANS

PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MEL	WEG	10/2023
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2	-	-	-
DESIGN3-DETAILED3	-	-	-
REVISIONS 1	-	-	-
REVISIONS 2	-	-	-
REVISIONS 3	-	-	-
REVISIONS 4	-	-	-
FIELD CHANGES	-	-	-

BY | DATE

SIGNATURE

P.E. NUMBER

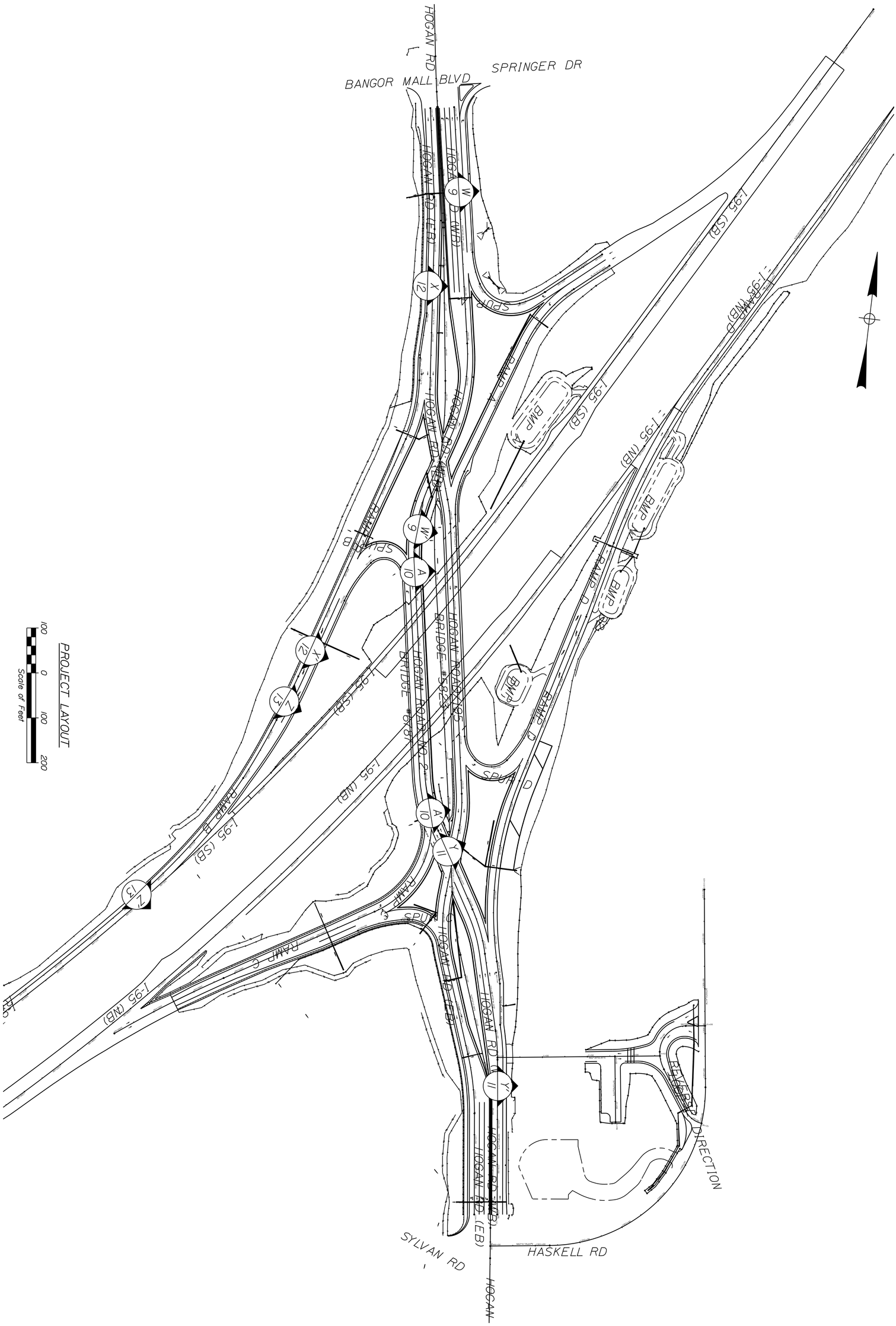
DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT #5823 (EXIT 187)
BANGOR MAINE

HIGHWAY BORING LOCATION PLAN SHEET 6

SHEET NUMBER

7



SHEET NUMBER 8	I-95 HOGAN ROAD DIVERGING DIAMOND INTERCHANGE (EXIT 187) BANGOR MAINE	M. LANDON		
8 OF 13	PROJECT LAYOUT	DESIGN-DETAILED: MEL / DJH / 10/2025		
		CHECKED-REVIEWED: DEB / MEL / 10/2025	SIGNATURE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
		DESIGN2-DETAILED2		018595.11
		DESIGN3-DETAILED3	P.E. NUMBER	
		REVISIONS 1		
		REVISIONS 2		
		REVISIONS 3	DATE	
		REVISIONS 4		
		FIELD CHANGES		

BRIDGE No. 5823 WIN 018595.11 BRIDGE PLANS

W
NORTH

W'
SOUTH

203+00 204+00 205+00 206+00 207+00 208+00 209+00

SPUR A MATCH

CROSSOVER

HB-BHR-101
TOB EL. 110.3 Ft
Offset 26 Ft R

HB-BHR-102
TOB EL. 109.2 Ft
Offset 31.2 Ft R

HB-BHR-104
TOB EL. 104.2 Ft
Offset 23.2 Ft R

EOB 33.0' BGS

EOB 25.0' BGS

ROD = 40%

EOB 51.8' BGS

Matchline see Interpretive Subsurface Profile Sheet 2 of 13

LEGEND

- Topsoil
- Asphalt
- Grey-Brown, Medium Stiff, Sandy Silt, Trace Gravel (Fill)
- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Sand and Gravel) Esker Deposit
- Proposed Bridge Substructure and Superstructure
- Grey-Brown, Soft to Very Stiff, Low Plasticity, Silt and Clay, Trace Sand (Silty Clay) Sediments
- Grey, Medium Dense to Very Dense, Gravelly Sand, Some Silt/Clay (Glacial Till)
- Grey, Fine Grained to Very Fine Grained Metawacke (Bedrock)
- Existing Ground Surface
- Interpreted Ground Water Surface

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line

- Top of Boring
- 30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
- VI-3378 /1965 — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
- MV — Unsuccessful Field Vane Shear Test Attempt
- R — Refusal
- IU — Thin Wall Tube Sample
- —? — Strata Interface
- RQD = 40% — Rock Core Run Number and Rock Quality Designation (RQD)
- EOB — End of Boring Depth Below Ground Surface (BGS)

INTERPRETIVE SUBSURFACE PROFILE
HOGAN ROAD WB SECTION W-W'

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

NOTE:

1. See Boring Location Plan Sheet 2 of 13 For Notes.

Username:
Date:10/15/2025
Division: BRIDGE
Filename: ...\Superceded\GeoProfile1.dgn

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN
018595.11

BRIDGE No. 5823

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	MA		09/05/22
CHECKED-REVIEWED	DEB	MEL	10/2025
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD DIVERGING
DIAMOND INTERCHANGE (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE W-W'

SHEET NUMBER

9

9 OF 13

Filename: ...\MSTA\GeoProfile_5Span_wb.dgn Username: Division: BRIDGE Date:10/14/2025

A NORTH — A' SOUTH

209+00 | 210+00 | 211+00 | 212+00 | 213+00 | 214+00 | 215+00 | 216+00

℄ Brg. Abutment 1 Sta. 210+50.00
℄ PIER 1 Sta. 211+24.40
℄ PIER 2 Sta. 212+24.61
℄ PIER 3 Sta. 213+24.81
℄ PIER 4 Sta. 214+25.02
℄ Brg. Abutment 2 Sta. 215+00.00

HB-BHR-105 EL. 100.7 Ft Offset 9.4 Ft Lt
BB-BHR-101 EL. 99.7 Ft. Offset 17.8 Ft. Lt.
BB-BHR-201 EL. 97.3 Ft. Offset 53.8 Ft. RT.
BB-BHR-202 EL. 91.5 Ft. Offset 9.7 Ft. Lt.
BRP-BHR-207 EL. 89.9 Ft. Offset 37.2 Ft. Rt.
BRP-BHR-208 EL. 93.0 Ft. Offset 6.3 Ft. Lt.
BB-BHR-102 EL. 92.4 Ft. Offsett 49.0 Ft. Lt.
BB-BHR-203 EL. 91.4 Ft. Offsett 37.2 Ft. Rt.
BRP-BHR-209 EL. 90.6 Ft. Offsett 22.2 Ft. Lt.
BB-BHR-204 EL. 88.9 Ft. Offsett 45.9 Ft. Rt.
BB-BHR-205 EL. 87.8 Ft. Offsett 4.1 Ft. Lt.
BB-BHR-103 EL. 88.5 Ft. Offsett 11.9 Ft. Lt.
HB-BHR-107 EL. 88.3 Ft. Offsett 3.6 Ft. Lt.
BB-BHR-206 EL. 87.4 Ft. Offsett 14.5 Ft. Rt.
BRP-BHR-210 EL. 84.8 Ft. Offsett 26.8 Ft. Rt.

Proposed Hogan Road West Bound
Proposed Superstructure
Proposed Abutment on Pile (Typ.)
Approach Slab (Typ.)
Proposed Pier on Piles (Typ.)
Aggregate Subbase
Granular Borrow
FGA
Existing Grade
Silt
Fill
Stiff to Very Stiff Clay
Medium Stiff to Soft Clay
Glacial Till
Weathered Bedrock
Approximate Limits of I-95 SB
Approximate Limits of I-95 NB
Stiff to Very Stiff Silty Clay (Clay)
Medium Stiff to Soft Low Plasticity, Silt and Clay, Trace Sand (Clay)
Metawacke

EOB 29' BGS
EOB 34.9' BGS
EOB 35.0' BGS
EOB 31.2' BGS
EOB 26.7' BGS
EOB 29.7' BGS
EOB 36.2' BGS
EOB 39.6' BGS
EOB 30.0' BGS
EOB 38.0' BGS
EOB 28.8' BGS
EOB 23.5' BGS
EOB 30.8' BGS

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring Probe
30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
VI-3378/1965 — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
MV — Missed Vane
WOH — Weight of Hammer
R — Refusal
UI Thin Wall Tube Sample
—?— Strata Interface
Run 1 = 92% Rock Core Run Number and Rock Quality Designation (RQD)
EOB — End of Boring Depth Below Ground Surface (BGS)

LEGEND
Topsoil
Asphalt
Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)
Granular Burrow
Aggregate Subbase
Olive-Brown, Medimum Stiff to Very Stiff Silt, Trace Fine Sand and Gravel, Slightly Plastic (Silt)
Foamed Glass Aggregate (FGA)
Crushed Stone for Slope Protection
Grey-Brown, Medimum Stiff to Soft Low Plasticity, Silt and Clay, Trace Sand (Clay)
Stiff to Very Stiff Silty Clay (Clay)
Grey, Medimum Dense to very Dense Gravelly Sand, some Silt/Clay (Glacial Till)
Weathered Bedrock
Grey, Fine Grained to Very Fine Grained Metawacke
Proposed Substructure and Superstructure
Existing Ground Surface
Interpreted Ground Water Surface

NOTE:
1. See Boring Location Plan Sheet 2 of 13 For Notes.

INTERPRETIVE SUBSURFACE PROFILE
HOGAN ROAD WB SECTION A-A'

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE PLANS

BRIDGE No. 5823

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	NVW	5/2024
CHECKED-REVIEWED	WEG	10/2025
DESIGN2-DETAILED2	DEB	
DESIGN3-DETAILED3		
REVISIONS 1	XXX	
REVISIONS 2	XXX	
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER
XXX
XXX

DATE

I-95 HOGAN ROAD BRIDGE
REPLACEMENT *5823 (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE A-A'

SHEET NUMBER
10
10 OF 13

Filename: ...\Superceded\GeoProfile3.dgn Username:
Division: BRIDGE
Date:10/15/2025

X NORTH — X' SOUTH

216+00 217+00 218+00 219+00 220+00 221+00 222+00

RAMP C MATCH

CROSSOVER

RAMP D MATCH

HB-BHR-108
TOB EL. 82.3 Ft
Offset 41.2 Ft R

HB-BHR-109
TOB EL. 78.3 Ft
Offset 13.2 Ft R

Matchline see Interpretive Subsurface Profile Sheet 2

8 20 IU 14 58 33 23

ROD= 26%
ROD = 48%

EOB 34.0' BGS

EOB 40.0' BGS

18 5 15 21 45 142 R

INTERPRETIVE SUBSURFACE PROFILE
HOGAN ROAD WB SECTION X-X'

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

LEGEND

- Topsoil
- Asphalt
- Grey-Brown, Medium Stiff, Sandy Silt, Trace Gravel (Fill)
- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Sand and Gravel) Esker Deposit
- Proposed Bridge Substructure and Superstructure
- Grey-Brown, Soft to Very Stiff, Low Plasticity, Silt and Clay, Trace Sand (Silty Clay) Sediments
- Grey, Medium Dense to Very Dense, Gravelly Sand, Some Silt/Clay (Glacial Till)
- Grey, Fine Grained to Very Fine Grained Metawacke (Bedrock)
- Existing Ground Surface
- Interpreted Ground Water Surface

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring
30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
VI-3378 /1965 — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
MV — Unsuccessful Field Vane Shear Test Attempt
R — Refusal
IU Thin Wall Tube Sample
—?— Strata Interface
ROD = 26%
Rock Core Run Number and Rock Quality Designation (RQD)
EOB — End of Boring Depth Below Ground Surface (BGS)

NOTE:
1. See Boring Location Plan Sheet 2 of 13 For Notes.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11
BRIDGE No. 5823

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	MA	09/05/22
CHECKED-REVIEWED	DEB	10/2025
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD DIVERGING
DIAMOND INTERCHANGE (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE X-X'

SHEET NUMBER
11
11 OF 13

Filename: ...\Superceded\GeoProfile4.dgn Username: Division: BRIDGE Date:10/15/2025

Y NORTH

Y' SOUTH

800+00 801+00 802+00 803+00 804+00 805+00 806+00

130 125 120 115 110 105 100 95 90 85 80 75 70 65 60 55 50 45 40

SPUR B MATCH

HB-BHR-102
EL. 109.2 Ft
Offset 31.2 Ft R

HB-BHR-104
EL. 104.2 Ft
Offset 23.2 Ft R

HB-BHR-106
EL. 105.9 Ft
Offset 11.0 Ft R

HB-BHR-111
EL. 97.8 Ft
Offset 21.2 Ft R

8 20 MV 17 51 47 24 20 35 Boulder

8 24 11 3 74

3 11 26 EOB 18.5' BGS

5 20 15 76 36 33 123 R

MATCH HOGAN ROAD (EB)

RQD = 40%

EOB 51.8' BGS

EOB 25.0' BGS

RQD = 55%

EOB 41.2' BGS

Matchline see Interpretive Subsurface Profile Sheet 5

LEGEND

- Topsoil
- Asphalt
- Grey-Brown, Medium Stiff, Sandy Silt, Trace Gravel (Fill)
- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Sand and Gravel) Esker Deposit
- Proposed Bridge Substructure and Superstructure
- Grey-Brown, Soft to Very Stiff, Low Plasticity, Silt and Clay, Trace Sand (Silty Clay) Sediments
- Grey, Medium Dense to Very Dense, Gravelly Sand, Some Silt/Clay (Glacial Till)
- Grey, Fine Grained to Very Fine Grained Metawacke (Bedrock)
- Existing Ground Surface
- Interpreted Ground Water Surface

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring
30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
VI-3378 /1965 — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
MV — Unsuccessful Field Vane Shear Test Attempt
R — Refusal
IU — Thin Wall Tube Sample
—? — Strata Interface
RQD = 40% Rock Core Run Number and Rock Quality Designation (RQD)
EOB — End of Boring Depth Below Ground Surface (BGS)

NOTE:
1. See Boring Location Plan Sheet 2 of 13 For Notes.

INTERPRETIVE SUBSURFACE PROFILE
RAMP B SECTION Y-Y'

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE No. 5823

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	MA	09/05/22
CHECKED-REVIEWED	DEB	10/2025
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1	MEL	
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD DIVERGING
DIAMOND INTERCHANGE (EXIT 187)
BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE Y-Y'

SHEET NUMBER

12

Filename: ...\Superceded\GeoProfile5.dgn Division: BRIDGE Username: Date:10/15/2025

Z NORTH

Z' SOUTH

807+00 808+00 809+00 810+00 811+00 812+00 813+00 814+00

MATCH I-95 SB

GORE

Matchline see Interpretive Subsurface Profile Sheet 4

HB-BHR-112
EL. 97.5 Ft
Offset 4.9 Ft L

HB-BHR-113
EL. 92.2 Ft
Offset 14.6 Ft L

HB-BHR-114
EL. 92.7 Ft
Offset 55.1 Ft L

HB-BHR-115
EL. 96.7 Ft
Offset 13.3 Ft L

BB-BHR-116
TOB EL. 93.5 Ft
Offset 23.1 Ft R

EOB 36.0' BGS

EOB 48.5' BGS

EOB 67.5' BGS

EOB 53.5' BGS

EOB 83' BGS

NOTE:

1. See Boring Location Plan Sheet 2 of 13 For Notes.

LEGEND

- Topsoil
- Asphalt
- Grey-Brown, Medium Stiff, Sandy Silt, Trace Gravel (Fill)
- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Sand and Gravel) Esker Deposit
- Proposed Bridge Substructure and Superstructure
- Grey-Brown, Soft to Very Stiff, Low Plasticity, Silt and Clay, Trace Sand (Silty Clay) Sediments
- Grey, Medium Dense to Very Dense, Gravelly Sand, Some Silt/Clay (Glacial Till)
- Grey, Fine Grained to Very Fine Grained Metawacke (Bedrock)
- Grey, Very Dense, Fine to Course Sand, some Silt, Little Gravel (Glaciafluvial Sand)
- Existing Ground Surface
- Interpreted Ground Water Surface

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring
30 SPT: N60 - Value (Corrected for Hammer Efficiency)
VI-3378 /1965 Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
MV Unsuccessful Field Vane Shear Test Attempt
R Refusal
IU Thin Wall Tube Sample
—? Strata Interface
ROD= 40% Rock Core Run Number and Rock Quality Designation (RQD)
EOB End of Boring Depth Below Ground Surface (BGS)

INTERPRETIVE SUBSURFACE PROFILE
RAMP B SECTION Z-Z'

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE No. 5823

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	MEL	09/05/22
CHECKED-REVIEWED	DEB	10/2025
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

I-95 HOGAN ROAD DIVERGING DIAMOND INTERCHANGE (EXIT 187) BANGOR MAINE

INTERPRETIVE SUBSURFACE PROFILE Z-Z'

SHEET NUMBER

13

13 OF 13

APPENDIX A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, Gravelly sands, little or no fines
			SP	Poorly-graded sands, Gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.
			OL	Organic silts and organic Silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. Sandy, Clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N_{60}-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N_{60}-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD\ (\%) = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
-tightness (tight, open, or healed)
-infilling (grain size, color, etc.)
Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Sample Container Labeling Requirements:

WIN
Bridge Name / Town
Boring Number
Sample Number
Sample Depth
Blow Counts
Sample Recovery
Date
Personnel Initials

May 2024

Table A-2

Classification of Rock Material Strengths[1]

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.

[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: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-101
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	110.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (08:55); 5/6/22	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486576.621, E: 1739700.751	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	4.0 ft on 5/6/22 at 10:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10.8	0.00 - 2.00	1-2-2-3	4	6	OPEN	109.9		(TOPSOIL) 0.4 Brown, moist, medium stiff, Sandy SILT, some gravel, trace grass roots in top 0.4 ft (FILL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
5	2D	21/12	5.00 - 6.75	1-1-2-5	3	5	26			Grey, moist, soft, Sandy SILT, some gravel (FILL).	
							34				
							40				
							59	102.3		8.0 Grey-brown, silt and clay present in wash water starting at 8 feet bgs.	
							78				
10	1U	24/24	10.00 - 12.00	Piston sampler			28			Olive, moist, CLAY, moderately plastic (CLAY).	WC = 28% LL = 36 PL = 21 PI = 15 LI = 0.5 CL
							37				
	3D	24/24	12.00 - 14.00	2-2-2-4	4	6	49			Brown, wet, medium stiff, CLAY (CLAY). q_p = 1 ksf (Pocket Penetrometer), T_v = 716 psf	WC = 27.9% Fines = 99.2% A-4 (0)
							71				
							95	95.8		14.5	
15	4D	24/14.4	15.00 - 17.00	21-20-22-17	42	64	OPEN			Grey-brown, wet, hard, Sandy SILT, little gravel (GLACIAL TILL).	
20	5D	24/7.2	20.00 - 22.00	22-19-12-13	31	47				Grey-brown, wet, hard, Sandy SILT, little gravel (GLACIAL TILL).	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 15 ft bgs

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-101
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	110.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (08:55); 5/6/22	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486576.621, E: 1739700.751	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	4.0 ft on 5/6/22 at 10:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/9.6	25.00 - 27.00	11-13-13-15	26	39				Grey-brown, wet, hard, Sandy SILT, little gravel, non-plastic (GLACIAL TILL).	WC = 9.0%
30	7D	3.6/3.6	30.00 - 30.30	100/4"	R			80.0		Grey-brown, wet, very dense, GRAVEL, some fine to coarse sand, trace silt (GLACIAL TILL).	
										30.3	
										Top of Probable Bedrock Elev. 80.0 ft Advanced with roller bit through probable bedrock from 30.3 to 33 ft bgs.	
								77.3		33.0	
										Bottom of Exploration at 33.0 feet below ground surface. Boring backfilled with bentonite chips to ground surface.	
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 15 ft bgs

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-102
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	109.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (11:00); 5/9/22 (08:35)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486494.684, E: 1739703.212	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.7 ft on 5/6/22 at 14:20
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/0	0.00 - 2.00	3-2-3-3	5	8	OPEN	108.2		NO RECOVERY Advanced with solid stem augers from 0 ft to 5 ft bgs.	
										1.0	
5	2D	24/6	5.00 - 7.00	2-10-3-3	13	20	15			Grey, wet, medium dense, fine to coarse Gravelly SAND, some silt (FILL).	
							19				
							31				
							53	101.2		8.0	
							72			Silt and clay present in wash water starting at 8 feet bgs.	WC = 26% LL = 39 PL = 39 PI = 17 LI = 0.2 CL
10	1U	24/24	10.50 - 12.50	Would Not Push			MV			Olive, moist, CLAY, moderately plastic (CLAY).	
				Piston Sampler			OPEN				
	3D	24/24	12.50 - 14.50	4-4-7-7	11	17				Grey-brown, wet, very stiff, CLAY, moderately plastic (CLAY). q_p = 4.1 tsf (pocket penetrometer), T_v = 716 psf	WC = 27.6% Fines = 99.4% LL = 39 PL = 22 PI = 17 LI = 0.3 A-6 (18), CL
15	4D	24/24	15.00 - 17.00	2-5-29-25	34	51	32			4DA, Top 12 in: Grey-brown, wet, hard, CLAY (CLAY). q_p = 3.6 tsf, T_v = 633 psf	
							34	93.2		16.0 4DB, Bottom 12 in: Brown, wet, hard, Sandy SILT, little gravel (GLACIAL TILL) Advanced with roller bit ahead of casing from 15 ft to 20 ft bgs.	
							40				
							49				
							57				
20	5D	24/15.6	20.00 - 22.00	8-9-22-16	31	47	34			Grey-brown, wet, hard, fine to coarse Silty SAND, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 20 ft to 25 ft bgs.	WC = 12.7% Fines = 43.2% A-4 (0)
							41				
							52				
							66	86.2		23.0 Drilled past cobbles from 23 ft to 24 ft bgs.	
							74	85.2		24.0	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 15 ft bgs

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-102
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	109.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (11:00); 5/9/22 (08:35)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486494.684, E: 1739703.212	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.7 ft on 5/6/22 at 14:20
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/1.2	25.00 - 27.00	9-8-8-8	16	24	19			Grey, wet, medium dense, GRAVEL, some fine to coarse sand, little silt (GLACIAL TILL). Advanced with roller bit ahead of casing from 25 ft to 30 ft bgs.	
							27				
							27				
							30				
							42				
30	7D	24/7.2	30.00 - 32.00	4-6-7-15	13	20	26			Grey, wet, medium dense, fine to coarse SAND, some silt, little fine to coarse gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 30 ft to 35 ft bgs.	
							28				
							31				
							32				
							36				
35	8D	24/12	35.00 - 37.00	9-14-9-13	23	35	OPEN			Grey, wet, dense, fine to coarse Silty- Clayey SAND, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 35 ft to 40 ft bgs.	WC = 9.4% Fines = 47.2% LL = 18 PL = 13 PI = 5 LI = -0.7 A-4 (0), SM-SC
40							NQ2	69.2		40.0 — Cored through boulder from 40 ft to 43 ft bgs.	
								66.2		43.0 — Grey sand & gravel present in drill cuttings from 43 to 45.5 ft bgs.	
45								63.7		45.5 — Top of Bedrock Elev. 63.7 ft. Advanced with roller bit through bedrock from 45.5 to 46.8 ft bgs.	
	R1	60/49.2	46.80 - 51.80	RQD = 40%						Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite vein (0.1 in thick) present at 48.6 ft bgs; average 2.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock mass Quality = Poor	
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 15 ft bgs

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-102
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	109.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (11:00); 5/9/22 (08:35)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486494.684, E: 1739703.212	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.7 ft on 5/6/22 at 14:20
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								57.4		82% Recovery Rock Core Rate (min:sec) 46.8-47.8 ft (2:58) 47.8-48.8 ft (2:54) 48.8-49.8 ft (2:41) 49.8-50.8 ft (2:12) 50.8-51.8 ft (2:21) 51.8 **Bottom of Exploration at 51.8 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 15 ft bgs

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-103
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	106.8	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/27/22 (14:35); 4/27/22 (15:25)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486383.449, E: 1739898.792	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.2 ft on 4/27/22 at 15:30
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20.4	0.00 - 2.00	WOH/6"-1-3-3	4	6	OPEN			Grey-brown, moist, medium stiff, SILT, some fine to coarse sand, trace gravel, trace grass roots (TOPSOIL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
								104.8		2.0	
5	2D	24/24	5.00 - 7.00	3-5-6-8	11	17	27			Grey-brown, wet, very stiff, CLAY (CLAY).	WC = 25.4% Fines = 99.4% A-4 (0)
							28				
							30				
							52				
							85				
10	3D	24/24	10.00 - 12.00	1-4-17-16	21	32	OPEN			3DA, Top 16.8 in: Grey-brown, wet, hard, CLAY, slightly plastic (CLAY). q_p = 1.6 ksf (Pocket Penetrometer), T_v = 531 psf 3DB, Bottom 7.2 in: Grey-brown, wet, hard, CLAY (CLAY).	WC = 24.0% Fines = 97.0% LL = 28 PL = 17 PI = 11 LI = 0.6 A-6 (9), CL
15	4D	24/24	15.00 - 17.00	13-18-27-15	45	68		91.8		15.0 Grey-brown, wet, hard, SILT, some fine to medium sand, trace gravel (GLACIAL TILL).	
20	5D	24/24	20.00 - 22.00	6-7-7-8	14	21				Grey, wet, very stiff, sandy SILT, trace gravel (GLACIAL TILL).	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 10 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-103
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	106.8	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/27/22 (14:35); 4/27/22 (15:25)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486383.449, E: 1739898.792	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.2 ft on 4/27/22 at 15:30
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	22.8/16.8	25.00 - 26.90	9-9-58-100/5"	67	101				Grey, wet, very dense, fine to coarse Silty SAND, some gravel (GLACIAL TILL).	
								79.3		27.5	
								78.8		Top of Probable Bedrock Elev. 79.3 ft. Casing refusal at 27.5 feet bgs; rock chips present in drill cuttings from 27.5 to 28 feet bgs.	
										28.0 **Bottom of Exploration at 28.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 10 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-104
WIN: 018595.11

Driller:	S. W. Cole	**Elevation (ft.)**	104.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (07:45); 5/2/22 (09:05)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486219.248, E: 1739736.001	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	8.8 ft on 5/2/22 at 09:10
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/7.2	0.00 - 2.00	1-2-3-4	5	8	OPEN	103.6		Brown, moist, medium stiff, Sandy SILT, trace gravel, trace grass roots (TOPSOIL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
										0.6	
										Sand, silt, and gravel mixture in drill cuttings from 0.6 to 4.0 ft bgs (FILL).	
								100.2		4.0	
5	2D	24/14.4	5.00 - 7.00	5-6-10-10	16	24	20			Grey-brown, wet, very stiff, CLAY, moderately plastic (CLAY).	WC = 25.5% LL = 45 PL = 23 PI = 22 LI = 0.1 CL
							25				
							54			Switched to cased wash drilling at 7 ft bgs.	
							61			Water level taken after installing casing to 10 ft bgs and before drilling with water was started.	
							64				
10	3D	24/24	10.00 - 12.00	3-3-4-4	7	11	72			Grey-brown, wet, stiff, CLAY (CLAY). q_p = 2.4 ksf (Pocket Penetrometer), T_v = 716 psf	
							78				
							104				
							91				
							99				
15	4D	24/24	15.00 - 17.00	2-1-1-2	2	3	34	89.2		15.0	
							38			4DA, Top 22.8 in: Grey, wet, soft, CLAY (CLAY). q_p = 0.4 ksf (Pocket Penetrometer), T_v = 315 psf	WC = 31.6% Fines = 99.7% A-4 (0)
							51			4DB, Bottom 1.2 in: Grey-brown, wet, soft, Clayey SILT (CLAY).	
							54	85.7		18.5	
							62				
20	5D	24/8.4	20.00 - 22.00	15-18-31-36	49	74	OPEN			5DA, Top 3.6 in: Brown, wet, hard, SILT, little fine sand, trace gravel (GLACIAL TILL). 5DB, Bottom 4.8 in: Grey-brown, wet, very dense, fine to coarse Gravelly SAND, some silt (GLACIAL TILL).	
25								79.7		24.5	

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading taken on 5/2/22 at 09:10 was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-104
WIN: 018595.11

Driller:	S. W. Cole	**Elevation (ft.)**	104.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (07:45); 5/2/22 (09:05)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486219.248, E: 1739736.001	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	8.8 ft on 5/2/22 at 09:10
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								79.2		Top of Probable Bedrock at Elev. 79.7 ft. Advanced with roller bit through probable bedrock from 24.5 to 25 ft bgs. **Bottom of Exploration at 25.0 feet below ground surface.**	
								25.0		**Bottom of Exploration at 25.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading taken on 5/2/22 at 09:10 was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-105
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	100.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/29/22 (11:00); 4/29/22 (12:50)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486134.486, E: 1739754.808	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.8 ft on 4/29/22 at 12:55
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/19.2	0.00 - 2.00	1-1-3-6	4	6	OPEN	100.3		(TOPSOIL) 0.4	
										Grey-brown, moist, medium stiff, SILT, some fine to medium sand, trace grass roots in top 0.4 ft (FILL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
								97.2		3.5	
5	2D	24/24	5.00 - 7.00	4-5-6-8	11	17	34			Grey-brown, moist, very stiff, CLAY, trace fine sand, moderately plastic (CLAY) q_p = 8.2 ksf (pocket penetrometer), T_v = 921 psf	WC = 25.2% LL = 38 PL = 22 PI = 16 LI = 0.2 CL
							35				
							38				
							40				
							45				
10	3D	24/24	10.00 - 12.00	1-2-3-3	5	8	40	90.7		10.0 Grey-brown, wet, medium stiff, CLAY (CLAY). q_p = 2.5 ksf (Pocket Penetrometer), T_v = 716 psf	
							45				
							49				
							55				
							51				
15	4D	24/24	15.00 - 17.00	WOH/12"-1-1	--		OPEN			Grey, wet, stiff, CLAY, moderately plastic (CLAY).	WC = 29.8% Fines = 99.8% LL = 32 PL = 19 PI = 13 LI = 0.8 A-6 (13), CL
	V1			S_u=1161/357 psf						55x100 mm vane raw torque readings: V1: 26/8 ft-lbs	
	V2			S_u=1116/223 psf						V2: 25/5 ft-lbs	
								82.2		18.5	
20	5D	24/10.8	20.00 - 22.00	10-18-19-14	37	56	52			Grey, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, some silt (GLACIAL TILL).	WC = 9.3% Fines = 32.6% A-2-4 (0)
							54				
							55				
							57				
								76.7		24.0	
25	R1	36/26.4	24.00 - 27.00	RQD = 52%			56			Top of Bedrock at Elev. 76.7 ft.	

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane shear test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 24 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-105
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	100.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/29/22 (11:00); 4/29/22 (12:50)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486134.486, E: 1739754.808	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.8 ft on 4/29/22 at 12:55
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ2			R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.1 in to 1.0 in thick) present from 24.0 ft to 25.6 ft bgs; average 1.3 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Fair	
	R2	24/24	27.00 - 29.00	RQD = 20%						73% Recovery	
										Rock Core Rate (min:sec)	
								71.7		24-25 ft (3:32)	
30										25-26 ft (3:41)	
										26-27 ft (3:28)	
										R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.2 in to 0.5 in thick) present from 28.0 ft to 29.0 ft bgs; average 3.5 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Very Poor	
35										100% Recovery	
										Rock Core Rate (min:sec)	
										27-28 ft (2:51)	
										28-29 ft (2:48)	
										29.0	
										Bottom of Exploration at 29.0 feet below ground surface.	
										Boring backfilled with bentonite chips to ground surface.	
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane shear test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 24 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-106
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	105.9	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/6/22 (7:30); 5/6/22 (8:25)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 486159.163, E: 1739635.157	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.6 ft on 5/6/22 at 8:30
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.00 - 2.00	1-1-1-2	2	3	OPEN	105.2		Brown, moist, soft, Sandy SILT, trace gravel, trace grass roots in top 0.7 ft (TOPSOIL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
										0.7	
5	2D	24/16.8	5.00 - 7.00	7-3-4-3	7	11	21			Grey, moist, loose, Sandy SILT, trace gravel (FILL).	
							33	98.9			
							48			7.0	
							72				
							87				
10	3D	24/24	10.00 - 12.00	4-7-10-10	17	26	81			Grey-brown, wet, very stiff, CLAY, trace sand, moderately plastic (CLAY). q_p = 5.1 tsf (Pocket Penetrometer), T_v = 858 psf	WC = 25.6% LL = 35 PL = 21 PI = 14 LI = 0.3 CL
							101				
							125				
							166				
							OPEN	91.4		14.5	
15										Top of Probable Bedrock Elev. 91.4 ft. Casing refusal at 14.5 feet bgs. Advanced with roller bit through probable bedrock from 14.5 to 18.5 feet bgs.	
								87.4		18.5	
20										**Bottom of Exploration at 18.5 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 14.5 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-107
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	88.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/26/22 (12:50); 4/26/22 (14:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485563.443, E: 1739838.257	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.4 ft on 4/26/22 at 13:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14.4	0.00 - 2.00	1-2-2-4	4	6	OPEN	87.8		(TOPSOIL) 0.5	
										Grey-brown, moist, medium stiff, SILT, little fine to medium sand, little gravel, trace grass roots in top 0.5 ft (SILT). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
								84.8		3.5	
5	2D	24/24	5.00 - 7.00	4-5-8-7	13	20	34			Grey-brown, moist, very stiff, CLAY, trace fine to medium sand, trace gravel (CLAY).	
							33				
							54				
							51				
							56				
10	3D	24/24	10.00 - 12.00	2-2-3-4	5	8	29	78.3		10.0	
							41			Grey-brown, wet, medium stiff, CLAY, moderately plastic (CLAY). q_p = 3 ksf (Pocket Penetrometer), T_v = 737 psf	WC = 30.0% LL = 42 PL = 23 PI = 19 LI = 0.4 CL
							49				
							58				
							66				
15	4D	24/24	15.00 - 17.00	WOR/24"	--		48			4DA, Top 20.4 in: Grey, wet, medium stiff, CLAY (CLAY).	WC = 29.1% Fines = 99.7% A-4 (0)
	V1			S_u=893/201 psf			59			55x100 mm vane raw torque readings: V1: 20/4.5 ft-lbs MV: Failed vane, would not push to 17 ft bgs.	
							84	71.6		16.7	
							105			4DB, Bottom 3.6 in: Brown, wet, Sandy SILT (GLACIAL TILL).	
							121				
20	5D	24/20.4	20.00 - 22.00	10-7-9-44	16	24	OPEN			Grey, wet, medium dense, fine to coarse Gravelly SAND, little silt (apparent fractured rock in tip of spoon) (GLACIAL TILL).	
	6D	0/0	23.50 - 23.50	50/0"	R			64.8		NO RECOVERY. Split spoon bounced for 50 blows and no penetration at 23.5 ft bgs. Top of Probable Bedrock at Elev. 64.8 ft.	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane shear test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made during drilling with bottom of casing at 15 ft bgs.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-107
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	88.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/26/22 (12:50); 4/26/22 (14:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485563.443, E: 1739838.257	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.4 ft on 4/26/22 at 13:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										23.5	
										Bottom of Exploration at 23.5 feet below ground surface. Boring backfilled with bentonite chips to ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. "-" Sample interval disturbed by vane shear test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made during drilling with bottom of casing at 15 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-108
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	82.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/27/22 (07:30); 4/27/22 (10:10)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485405.013, E: 1739809.715	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.6 ft on 4/27/22 at 08:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/7.2	0.00 - 2.00	2-2-3-3	5	8	OPEN	81.7		(TOPSOIL) 0.6 Brown, wet, loose, Silty fine SAND, trace roots (FILL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
								78.8		3.5	
							22				
5	2D	24/24	5.00 - 7.00	4-5-8-8	13	20	23			Grey-brown, moist, very stiff, CLAY, trace fine sand, moderately plastic (CLAY). q_p = 7.2 ksf (pocket penetrometer), T_v = 1022 psf	WC = 25.5% LL = 37 PL = 22 PI = 15 LI = 0.2 CL
							24				
							26				
	1U	24/18	8.00 - 10.00	Piston Sampler			OPEN			Olive, moist, CLAY, moderately plastic (CLAY).	WC = 28% LL = 35 PL = 20 PI = 15 LI = 0.6 CL
10	3D	24/24	10.00 - 12.00	2-1-8-24	9	14	49			3DA, Top 19.2 in: Grey-brown, wet, stiff, CLAY, moderately plastic (CLAY). q_p = 5.1 ksf (pocket penetrometer), T_v = 716 psf	WC = 26.5% LL = 32 PL = 21 PI = 11 LI = 0.5 CL
							54	70.7		11.6	
							85			3DB, Bottom 4.8 in: Grey, wet, medium dense, Sandy GRAVEL, some silt (SAND and GRAVEL).	
							112				
							116				
15	4D	24/15.6	15.00 - 17.00	13-23-15-9	38	58	OPEN			Grey, wet, very dense, fine GRAVEL, some sand, little silt (SAND and GRAVEL).	WC = 6.0% Fines = 12.6% A-1-a (0)
								63.8		18.5	
20	5D	24/8.4	20.00 - 22.00	4-4-18-8	22	33	50			Grey, wet, dense, fine to coarse SAND, some silt, some gravel, little clay (GLACIAL TILL).	
							53				
							140				
							149				
25							150				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level was taken during drilling with casing at 15 ft bgs, prior to obtaing sample 4D, with a stabilization time of 20 minutes.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-108
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	82.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/27/22 (07:30); 4/27/22 (10:10)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485405.013, E: 1739809.715	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.6 ft on 4/27/22 at 08:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	9-9-6-6	15	23	38			Grey, wet, medium dense, fine to coarse SAND, some silt, some gravel, little clay (GLACIAL TILL).	WC = 11.6% Fines = 39.9% A-4 (0)
							54				
							73				
							103	53.7		Casing refusal encountered at 28.6 feet bgs. 28.6	
	R1	27.6/25.2	29.00 - 31.30	RQD = 26%			NQ2			Top of Bedrock Elev. at 53.7 ft. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open with grey-brown silt infilling, milky quartz and calcite veins (0.1 in to 0.2 in thick) from 29 ft to 30.5 ft bgs; average 2.6 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 91% Recovery Rock Core Rate (min:sec) 29-30 ft (2:24) 30-31 ft (2:36) 31-31.3 ft (2:14)
30											
	R2	32.4/32.4	31.30 - 34.00	RQD = 48%							
								48.3		R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 32 ft to 32.5 ft bgs and 33.3 ft bgs ; average 2.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 31.3-32 ft (2:38) 32-33 ft (2:31) 33-34 ft (2:24) 34.0	
35										**Bottom of Exploration at 34.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level was taken during drilling with casing at 15 ft bgs, prior to obtaing sample 4D, with a stabilization time of 20 minutes.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-109
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	78.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/9/22 (9:50); 5/9/22 (11:20)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485212.362, E: 1739903.181	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	6.3 ft on 5/9/22 at 11:25
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/9.6	0.00 - 2.00	1-1-2-4	3	5	OPEN	77.5		(TOPSOIL) 0.8	
										Brown, moist, soft, SILT, some fine to medium sand, trace grass roots (FILL). Advanced with solid stem augers from 0 ft to 5 ft bgs.	
								74.8		3.5	
5	2D	24/24	5.00 - 7.00	5-6-6-9	12	18	24			Grey-brown, moist, very stiff, CLAY, moderately plastic (CLAY). q_p = 7.7 tsf, T_v = 409 psf	WC = 27.5% Fines = 88.3% LL = 37 PL = 18 PI = 19 LI = 0.5 A-6 (16), CL
							30				
							42	70.8		7.5	
							56			Drilled past cobbles from 7.5 to 9.5 feet bgs.	
							109	68.8		9.5	
10	3D	24/10.8	10.20 - 12.20	5-5-5-7	10	15	OPEN	68.1		Advanced with a roller bit past a boulder from 9.5 ft to 10.2 ft bgs. 10.2 Grey, wet, medium dense, fine GRAVEL, some fine to coarse sand, little silt, little clay (GLACIAL TILL).	
15	4D	24/6	15.00 - 17.00	4-7-7-11	14	21				Grey, wet, medium dense, fine GRAVEL, some fine to coarse sand, little silt, little clay (GLACIAL TILL).	WC = 11.9% Fines = 44.8% A-4 (0)
20	5D	24/7.2	20.00 - 22.00	10-9-21-20	30	45				Grey, wet, dense, fine GRAVEL, some fine to coarse sand, little silt, little clay (GLACIAL TILL).	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 9.5 ft bgs.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-109
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	78.3	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/9/22 (9:50); 5/9/22 (11:20)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485212.362, E: 1739903.181	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	6.3 ft on 5/9/22 at 11:25
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								53.3		25.0 Advanced with a roller bit past a boulder from 25 ft to 27 ft bgs.	
	6D	24/19.2	27.00 - 29.00	35-43-51-68	94	142		51.3		27.0 Grey-brown, wet, very dense, fine to coarse SAND, some silt, little fine gravel, trace clay (GLACIAL TILL).	WC = 9.1% Fines = 25.5% A-2-4 (0)
30											
35	7D	2.4/0	35.00 - 35.20	100/2"	R			44.3		34.0 Advanced with a roller bit past cobbles from 34 ft to 37 ft bgs. NO RECOVERY. Split spoon bounced on cobbles.	
								41.3		37.0 Top of Probable Bedrock Elev. 41.3 ft. Advanced with roller bit through probable bedrock from 37 ft to 40 ft bgs.	
40								38.3		40.0 **Bottom of Exploration at 40.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 9.5 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-110
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	76.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/9/22 (11:50); 5/9/22 (12:45)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485210.377, E: 1739810.637	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.1 ft on 5/9/22 at 12:50
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16.8	0.00 - 2.00	1-1-3-4	4	6	OPEN	76.1		(TOPSOIL) 0.4 Grey-brown, moist, medium stiff, SILT, some fine to medium sand, little gravel, trace grass roots in top 0.4 ft (FILL).	
								73.0		3.5	
5	2D	24/24	5.00 - 7.00	1-2-6-7	8	12	22			2DA, Top 22.8 in: Grey, moist, stiff, CLAY, trace fine to medium sand, trace gravel, highly plastic (CLAY). q_p = 4.3 ksf, T_v = 511 psf	WC = 44.6% LL = 69 PL = 28 PI = 41 LI = 0.4 CH
							26				
							44	69.6		6.9 2DB, bottom 1.2 in: Grey, moist, medium stiff, Gravelly SILT, little fine to medium sand (GLACIAL TILL).	
							50				
							65				
10	3D	24/13.2	10.00 - 12.00	11-16-28-21	44	67	OPEN			Grey, wet, very dense, GRAVEL, some fine to coarse sand, trace silt (GLACIAL TILL).	
								64.5		12.0 Top of Bedrock Elev. 64.5 ft. Advanced with roller bit threough bedrock from 12 ft bgs to 22 ft bgs.	
15											
20								54.5		22.0 **Bottom of Exploration at 22.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 10 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-111
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	97.8	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/5/22 (10:25); 5/5/22 (15:40)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485973.041, E: 1739565.218	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.7 ft on 5/6/22 at 7:00
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16.8	0.00 - 2.00	1-1-2-2	3	5	OPEN	97.0		(TOPSOIL) 0.8	
										Brown, moist, soft, SILT, some fine to medium sand, trace gravel, trace grass roots in top 0.8 ft (FILL).	
								94.3		3.5	
5	2D	24/22.8	5.00 - 7.00	4-5-8-10	13	20	41			Grey-brown, moist, very stiff, CLAY, trace gravel, trace fine to coarse sand, moderately plastic (CLAY). q_p = 5.6 ksf, T_v = 1063 psf	WC = 25.0% LL = 39 PL = 18 PI = 21 LI = 0.3 CL
							54				
							79				
							88				
							94				
10	3D	24/24	10.00 - 12.00	3-5-5-7	10	15	64			Grey-brown, wet, stiff, CLAY, trace fine to medium sand (CLAY).	
	MV			Would Not Push			84			Failed 55 x 110 mm vane, would not push to 11 ft bgs.	
							99				
							119				
							131				
15	4D	24/18	15.00 - 17.00	20-23-27-26	50	76	71	82.8		15.0	
										Brown, wet, hard, Sandy CLAY, trace gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 15 to 20 ft bgs.	WC = 12.5% Fines = 54.2% A-4 (0)
							94				
							104				
							141				
							150				
20	5D	24/16.8	20.00 - 22.00	5-11-13-10	24	36	68			Grey, wet, medium dense, fine to coarse Clayey SAND, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 20 to 25 ft bgs.	WC = 9.9% Fines = 46.4% A-4 (0)
							89				
							105				
							118				
25							120				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 15.5 hours after completion of drilling with bottom of casing at 34.5 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-111
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	97.8	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/5/22 (10:25); 5/5/22 (15:40)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485973.041, E: 1739565.218	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.7 ft on 5/6/22 at 7:00
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/13.2	25.00 - 27.00	7-9-13-20	22	33	62			Grey, wet, dense, fine to coarse Clayey SAND, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 25 to 30 ft bgs.	
							83				
							88				
							91				
							131				
30	7D			11-42-39-89	81	123	133			Grey, wet, very dense, fine to coarse Clayey SAND, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 30 to 33 ft bgs.	
							148				
							162				
							OPEN			Casing crimped at 33 ft bgs preventing further advancement of the broing. Boring offset 3 feet to the south and blind drilled to 33 ft bgs.	
								63.3		34.5 Top of Bedrock Elev. 63.3 ft.	
35		2.4/0	36.00 - 36.20	100/2"	R					Fracture or soft zone of bedrock from 35.5 to 36.2 ft bgs.	
	R1	60/57.6	36.20 - 41.20	RQD = 55%			NQ2			NO RECOVERY. Split spoon bouncing on rock R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderately dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.1 in thick) present at 36.5 ft and 39.8 ft bgs; average 1.2 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Fair 96% Recovery Rock Core Rate (min:sec) 36.2 - 37.2 ft (2:11) 37.2 - 38.2 ft (2:24) 38.2 - 39.2 ft (2:39) 39.2 - 40.2 ft (3:14) 40.2 - 41.2 ft (3:08)	
40								56.6		41.2 **Bottom of Exploration at 41.2 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 15.5 hours after completion of drilling with bottom of casing at 34.5 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-112
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	97.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/28/22 (7:45); 4/28/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485832.419, E: 1739547.816	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.0 ft on 4/28/22 at 10:05
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15.6	0.70 - 2.70	8-10-15-11	25	38	OPEN	96.8		ASPHALT (0.7)	
										Grey-brown, moist, dense, fine to coarse SAND, some silt, little gravel (FILL).	
5	2D	24/22.8	5.00 - 7.00	12-10-14-12	24	36	28			Grey-brown, moist, very hard, CLAY, some fine to coarse sand, little gravel, slightly plastic (FILL).	WC = 15.9% LL = 21 PL = 12 PI = 9 LI = 0.4 CL
							42				
							48				
							98			Drilled past cobbles from 8.5 ft to 9.5 ft bgs.	
							104				
10	3D	24/2.4	10.00 - 12.00	10-6-3-6	9	14	25			Brown, wet, loose, GRAVEL, some fine to coarse sand, trace silt (FILL). Gravel lodged in tip of split spoon. Advanced with roller bit ahead of casing from 10 to 15 ft bgs.	
							31				
							34				
							41				
							45				
15	4D	24/12	15.00 - 17.00	3-5-4-3	9	14	30	82.5		(15.0) Brown, wet, stiff, CLAY, some fine to coarse sand, some gravel (CLAY).	WC = 14.5% Fines = 52.2% A-4 (0)
							44				
							56				
							55				
							62				
20	5D	24/24	20.00 - 22.00	2-4-5-7	9	14	OPEN			Grey, wet, stiff, CLAY, trace fine sand (CLAY). q_p = 4.6 ksf, T_v = 409 psf	WC = 21.9% Fines = 97.3% A-4 (0)
								74.5		(23.0) Advanced past cobbles from 23 ft to 24 ft bgs.	
								73.5		(24.0)	
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-112
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	97.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/28/22 (7:45); 4/28/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485832.419, E: 1739547.816	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.0 ft on 4/28/22 at 10:05
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/16.8	25.00 - 27.00	16-12-6-9	18	27				Grey-brown, wet, very stiff, Sandy SILT, little gravel (GLACIAL TILL).	
30	7D	24/14.4	30.00 - 32.00	4-5-30-18	35	53				Grey, wet, hard, Sandy SILT, little gravel (GLACIAL TILL).	
35	8D	4.8/4.8	35.00 - 35.40	100/5"	R			62.1		Grey, wet, very dense, GRAVEL, little fine to medium sand, little silt, fractured bedrock in tip of the spoon (GLACIAL TILL).	
								61.5		35.4 Top of Probable Bedrock Elev. 62.1 ft. Advanced with roller bit through probable bedrock from 35.4 ft to 36 ft bgs. 36.0 **Bottom of Exploration at 36.0 feet below ground surface.** Boring backfilled with bentonite chips to 1.7 ft bgs, clean gravel from 1.7 ft to 0.7 ft bgs, and cold patch from 0.7 ft bgs to ground surface.	
40											
45											
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-113
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/28/22 (10:30); 4/28/22 (13:15)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485638.994, E: 1739481.375	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft on 4/28/22 at 13:15
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/9.6	0.50 - 2.50	5-7-7-6	14	21	OPEN	91.7		ASPHALT 0.5	
										Brown, moist, medium dense, Silty fine to coarse SAND, little gravel (FILL).	
5	2D	24/21.6	5.00 - 7.00	2-2-5-5	7	11	22			Grey-brown, moist, stiff, CLAY, little fine to coarse sand, trace gravel, moderately plastic (FILL).	WC = 15.6% Fines = 80.7% LL = 33 PL = 17 PI = 16 LI = -0.1 A-6 (11), CL
							24				
							25				
							36				
							46				
10	3D	24/7.2	10.00 - 12.00	4-3-2-2	5	8	24			Brown, wet, loose, Clayey-Silty SAND, some gravel (FILL).	WC = 12.6% Fines = 43.7% A-4 (0)
							31				
							33				
							41				
							44				
15	4D	24/1.2	15.00 - 17.00	2-3-5-7	8	12	25			Grey-brown, wet, medium dense, Clayey-Silty SAND (FILL).	
							28				
	5D	24/8.4	17.00 - 19.00	4-5-4-4	9	14	40			Grey-brown, wet, medium dense, fine to coarse Clayey-Silty SAND, little gravel (FILL).	WC = 17.8% Fines = 36.5% A-4 (0)
							49				
							55	72.7		19.5	
20	6D	24/14.4	20.00 - 22.00	3-3-3-5	6	9	31			Grey, wet, stiff, CLAY, little gravel, trace sand, slightly plastic (CLAY). q_p = 1.6 ksf, T_v = 817 psf	WC = 18.1% Fines = 77.7% LL = 22 PL = 14 PI = 8 LI = 0.5 A-4 (3), CL
							33				
	MV			Would Not Push			45			Failed 55 x 110 mm vane, would not push to 23 ft bgs.	
							46				
25							51				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made at the completion of drilling with bottom of casing at 40 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-113
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/28/22 (10:30); 4/28/22 (13:15)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485638.994, E: 1739481.375	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft on 4/28/22 at 13:15
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/0	25.00 - 27.00	4-3-6-5	9	14	46			NO RECOVERY	
							50				
								65.2		27.0	
	8D	24/9.6	27.00 - 29.00	2-2-3-4	5	8	59			Grey, wet, medium stiff, SILT, some gravel, trace sand, non-plastic (GLACIAL TILL). q_p = 4.9 ksf, T_V = 531 psf	WC = 14.1%
							64				
							88				
30	9D	24/7.2	30.00 - 32.00	9-10-10-11	20	30	46			Grey, wet, medium dense, fine to coarse SAND, some gravel, some silty clay (GLACIAL TILL).	WC = 8.9% Fines = 32.7% A-2-4 (0)
							48				
							81				
							82				
							86				
35	10D	24/16.8	35.00 - 37.00	14-13-15-17	28	42	30			Grey, wet, dense, Silty-Clayey SAND, little gravel (GLACIAL TILL).	
							109				
							103				
							110				
							131				
40	11D	24/12	40.00 - 42.00	40-11-13-28	24	36	OPEN			Grey, wet, dense, Silty-Clayey SAND, some gravel (GLACIAL TILL).	WC = 11.3% Fines = 49.8% A-4 (0)
45										Drilled through boulder from 45 ft to 46.6 ft bgs.	
	12D	9.6/9.6	47.00 - 47.80	20-100/4"	R					Grey, wet, dense, Silty-Clayey SAND, little gravel (GLACIAL TILL).	
								44.4		47.8	
								43.7		Top of Probable Bedrock at Elev. 44.4 ft. Advanced with a roller bit through probable bedrock from 47.8 ft to 48.5 ft bgs.	
50										48.5	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-113
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.2	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	4/28/22 (10:30); 4/28/22 (13:15)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485638.994, E: 1739481.375	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft on 4/28/22 at 13:15
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										**Bottom of Exploration at 48.5 feet below ground surface.** Boring backfilled with bentonite chips to 1.5 ft bgs, clean gravel from 1.5 ft to 0.5 ft bgs, and cold patch from 0.5ft gs to ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made at the completion of drilling with bottom of casing at 40 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-114
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/4/22 (12:20); 5/5/22 (9:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485577.276, E: 1739497.406	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.5 ft on 5/5/22 at 9:05
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14.4	0.00 - 2.00	4-9-15-12	24	36	OPEN	92.1		Brown, moist, medium dense, fine to coarse SAND, some silt, some gravel (TOPSOIL).	
										0.6	
5	2D	24/12	5.00 - 7.00	1-2-2-3	4	6	21			Grey, wet, medium stiff, Clayey SILT, some fine to coarse sand, trace gravel (FILL). q_p = 2.9 ksf Torvane not completed due to amount of gravel in the sample.	WC = 16.5% Fines = 67.7% A-4 (0)
							26				
	MV			Would Not Push			36			Failed 55 x 110 mm vane, would not push to 8 ft bgs.	
							45				
							46				
10	3D	24/7.2	10.00 - 12.00	4-8-11-8	19	29	21			Grey-brown, wet, very stiff, Clayey SILT, some fine to coarse sand, little gravel (FILL). q_p = 7.8 ksf, T_v = 368 psf	
							24				
							29				
							31				
							90				
15	4D	24/2.4	15.00 - 17.00	5-6-7-8	13	20	28			Grey, wet, very stiff, SILT, little gravel, little fine to coarse sand (FILL). Advanced with roller bit ahead of casing from 15 ft to 20 ft bgs.	
							31				
							44			Significant water loss occurred while drilling from 15 ft to 20 ft bgs.	
								74.7		18.0	
							124				
							158				
20	5D	24/8.4	20.00 - 22.00	7-2-2-2	9	14	OPEN			Grey, wet, medium stiff, Clayey SILT, little fine to coarse sand, trace fine gravel, trace wood (0.5-inch diameter root) (SILT). q_p = 2.6 ksf Torvane not completed due to amount of gravel in the sample.	WC = 28.3% Fines = 71.8% A-4 (0)
25											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-114
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/4/22 (12:20); 5/5/22 (9:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485577.276, E: 1739497.406	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.5 ft on 5/5/22 at 9:05
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	4-4-4-5	8	12				Grey-brown, wet, stiff, Clayey SILT, some fine to coarse sand, trace gravel (SILT). q_p = 6.5 ksf, T_V = 429 psf	
30	7D	24/6	30.00 - 32.00	6-10-9-10	19	29		62.7		30.0 — Grey, wet, medium dense, Clayey-Silty SAND, little gravel, slightly plastic (SAND and GRAVEL). q_p = 8.0 ksf, T_V = 593 psf	WC = 9.2% Fines = 39.7% LL = 18 PL = 12 PI = 6 LI = -0.5 A-4 (0), SC-SM
35	8D	10.8/7.2	35.00 - 35.90	12-100/5"	R			59.2		33.5 — Grey, wet, hard, SILT, some fine to coarse sand, little gravel (GLACIAL TILL). Advanced with roller bit through a boulder from 35.9 ft to 38.2 ft bgs.	
40	9D	24/19.2	40.00 - 42.00	22-18-17-20	35	53				Grey, wet, hard, SILT, some fine to coarse sand, little gravel (GLACIAL TILL).	
45	10D	24/19.2	45.00 - 47.00	5-6-14-13	20	30				Grey, wet, very stiff, Clayey SILT, some gravel, some fine to coarse sand (GLACIAL TILL).	WC = 10.3% Fines = 51.1% A-4 (0)
50											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 20 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-114
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	92.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/4/22 (12:20); 5/5/22 (9:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485577.276, E: 1739497.406	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.5 ft on 5/5/22 at 9:05
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/21.6	50.00 - 52.00	20-22-45-52	67	101				Grey-brown, wet, hard, Gravelly-Clayey SILT, some fine to coarse sand (GLACIAL TILL).	
55	12D	24/20.4	55.00 - 57.00	63-62-74-82	141	213		37.7		55.0 Brown, wet, very dense, fine to coarse SAND, some clayey silt, little gravel (GLACIOFLUVIAL).	WC = 12.2% Fines = 31.4% A-2-4 (0)
60	13D	3.6/3.6	60.00 - 60.30	100/4"	R					Grey, wet, very dense, Silty GRAVEL, some fine to coarse sand (GLACIOFLUVIAL).	
65								28.2		64.5 Top of Probable Bedrock at Elev. 28.2 ft. Advanced with a roller bit through probable bedrock from 64.5 ft to 67.5 ft bgs.	
70										**Bottom of Exploration at 67.5 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-115
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	96.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (9:37); 5/2/22 (13:37)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485372.826, E: 1739331.654	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.8 ft on 5/2/22 at 13:42
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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WC = Water Content, percent
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13.2	0.70 - 2.70	6-9-8-9	17	26	OPEN	96.0		ASPHALT 0.7	
										Brown, moist, medium dense, fine to coarse SAND, some fine gravel, little silt (FILL).	WC = 5.2% Fines = 12.9% A-1-b (0)
										Drilled past cobbles from 3.5 ft to 4.0 ft bgs.	
5	2D	24/21.6	5.00 - 7.00	4-3-4-5	7	11	21			Brown, moist, stiff, CLAY, some fine to medium sand, trace gravel, slightly plastic (FILL).	WC = 12.6% Fines = 61.0% LL = 21 PL = 12 PI = 9 LI = 0.1 A-4 (2), CL
							25				
							29				
							32				
							51				
10	3D	24/8.4	10.00 - 12.00	3-2-2-3	4	6	24			Grey, wet, medium stiff, Gravelly CLAY, some fine to medium sand (FILL).	
							33				
							33	84.2		12.5	
							41				
							46				
15	4D	24/24	15.00 - 17.00	4-5-8-9	13	20	43			Grey-brown, wet, very stiff, CLAY, slightly plastic (CLAY). q_p = 5.9 ksf, T_v = 491 psf	WC = 25.8% Fines = 98.4% LL = 36 PL = 21 PI = 15 LI = 0.3 A-6 (16), CL
							44				
							95				
							104				
							128				
20	5D	24/24	20.00 - 22.00	4-4-5-6	9	14	64			Grey-brown, wet, stiff, CLAY (CLAY). q_p = 3.9 ksf, T_v = 838 psf	
							101				
							114				
							347	73.7		23.0	
25							184				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-115
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	96.7	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (9:37); 5/2/22 (13:37)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485372.826, E: 1739331.654	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.8 ft on 5/2/22 at 13:42
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/12	25.00 - 27.00	9-13-17-16	30	45	69			Grey, wet, hard, SILT, some gravel, little fine to coarse sand (GLACIAL TILL).	
							82				
							90				
							96				
							114				
30	7D	24/15.6	30.00 - 32.00	7-10-13-13	23	35	38			Grey, wet, hard, SILT, some fine to coarse sand, little gravel (GLACIAL TILL). Advanced with roller bit ahead of casing from 30ft to 35 ft bgs.	WC = 12.3% Fines = 58.9% A-4 (0)
							41				
							56				
							81				
							103				
35	8D	24/8.4	35.00 - 37.00	26-19-27-20	46	70	46			Grey, wet, hard, SILT, some fine to coarse sand, some gravel (GLACIAL TILL). Advanced with a roller bit ahead of casing from 35 ft to 40 ft bgs.	
							58				
							83				
							109				
							124				
40	9D	24/21.6	40.00 - 42.00	12-20-25-18	45	68	58			Grey, wet, hard, SILT, some fine to coarse sand, trace gravel (GLACIAL TILL). Advanced with a roller bit ahead of casing from 40 ft to 45 ft bgs.	WC = 12.6% Fines = 68.7% A-4 (0)
							102				
							123				
							138				
							141				
45	10D	24/20.4	45.00 - 47.00	5-6-9-10	15	23	81			Grey, wet, very stiff, SILT, little gravel, little fine to coarse sand (GLACIAL TILL). Advanced with a roller bit ahead of casing from 45 ft to 50 ft bgs.	
							102				
							118				
							127				
50							181				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 50 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-115
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	96.7	**Auger ID/OD:**	4.25 in
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Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (9:37); 5/2/22 (13:37)	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	N: 485372.826, E: 1739331.654	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.8 ft on 5/2/22 at 13:42
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WC = Water Content, percent
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Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/19.2	50.00 - 52.00	33-34-36-36	70	106	OPEN			Grey, wet, hard, SILT, some fine to coarse sand, little fine gravel (GLACIAL TILL).	WC = 13.7% Fines = 65.7% A-4 (0)
								43.2		Top of Probable Bedrock Elev. 43.7 ft. Advanced with a roller bit through probable bedrock from 53 ft to 53.5 ft bgs.	
										53.5 **Bottom of Exploration at 53.5 feet below ground surface.** Boring backfilled with bentonite chips to 1.7 ft bgs, clean gravel from 1.7 ft to 0.7 ft bgs, and cold patch from 0.7 ft bgs to ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 5 minutes after completion of drilling with bottom of casing at 50 ft bgs.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-116
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	93.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (15:00); 5/4/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485309.571, E: 1739242.201	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	12.7 ft on 5/4/22 at 10:35
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10.8	0.00 - 2.00	1-4-5-3	9	14	OPEN	92.8		Brown, moist, stiff, SILT, some fine to coarse sand, little gravel, trace grass roots in top 0.7 ft (TOPSOIL).	
										0.7	
5	2D	24/19.2	5.00 - 7.00	5-6-7-7	13	20	56			Grey-brown, moist, very stiff, Silty CLAY, trace fine to coarse sand, trace fine gravel (FILL).	WC = 21.9% Fines = 99.4% A-4 (0)
							84				
							118				
							113	85.0		8.5	
							125				
10	3D	24/24	10.00 - 12.00	4-4-6-7	10	15	48			Grey-brown, wet, stiff, Silty CLAY (CLAY). q_p = 4.6 ksf, T_v = 511 psf	
							68				
							102	81.0		12.5	
							121				
							124				
15	4D	24/9.6	15.00 - 17.00	16-15-16-12	31	47	56			Grey-brown, wet, dense, fine to coarse SAND, some silt, little fine gravel (SAND and GRAVEL).	WC = 10.3% Fines = 33.1% A-2-4 (0)
							68				
							89				
							102	75.0		18.5	
							108				
20	5D	24/13.2	20.00 - 22.00	6-6-12-11	18	27	OPEN			Grey, wet, very stiff, Clayey SILT, some fine to coarse sand, little gravel (GLACIAL TILL). Advanced with a roller bit ahead of casing from 20 ft to 25 ft bgs.	
							118				
							124				
25							121				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 35 minutes after completion of drilling with bottom of casing at 70 ft bgs.

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-116
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	93.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (15:00); 5/4/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485309.571, E: 1739242.201	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	12.7 ft on 5/4/22 at 10:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/16.8	25.00 - 27.00	15-11-15-17	26	39	65			Grey, wet, hard, Clayey SILT, some fine to coarse sand, little gravel (GLACIAL TILL). Advanced with a roller bit ahead of casing through 25 ft to 30 ft bgs.	
							71				
							79				
							88				
							92				
30	7D	24/14.4	30.00 - 32.00	7-12-17-14	29	44	54			Grey, wet, hard, Clayey SILT, some fine to medium sand, trace fine gravel (GLACIAL TILL). Advanced with a roller bit ahead of casing from 30 ft to 35 ft bgs.	WC = 12.2% Fines = 68.0% A-4 (0)
							69				
							74				
							84				
							96				
35	8D	24/19.2	35.00 - 37.00	14-15-17-28	32	48				Grey, wet, hard, Clayey SILT, little gravel, little fine to coarse sand (GLACIAL TILL). Advanced with a roller bit ahead of casing from 35 ft to 40 ft bgs.	
40	9D	4.8/4.8	40.00 - 40.40	100/5"	R		77			Grey, wet, hard, Clayey SILT, some gravel, little fine to coarse sand (GLACIAL TILL). Advanced with a roller bit ahead of casing from 40 ft to 45 ft bgs. Drilled past cobbles from 40.4 to 41 ft bgs.	
							92				
							85				
							89	50.0		43.5	
							91				
45	10D	24/21.6	45.00 - 47.00	17-19-22-22	41	62	110			Brown, wet, very dense, fine to medium SAND, little silt, silt lens from 46.4 ft to 46.5 ft bgs) (GLACIOFLUVIAL).	
							106				
							113				
							114				
50							121				

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 35 minutes after completion of drilling with bottom of casing at 70 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-116
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	93.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (15:00); 5/4/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485309.571, E: 1739242.201	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	12.7 ft on 5/4/22 at 10:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/19.2	50.00 - 52.00	17-39-55-79	94	142	101			Brown, wet, very dense, fine to medium SAND, some silt and clay some fine gravel (GLACIOFLUVIAL).	WC = 9.4% Fines = 33.6% A-2-4 (0)
							150				
							121				
							127				
							137				
55	12D	14.4/14.4	55.00 - 56.20	37-71-100/2"	R		151			Brown, wet, very dense, fine to medium SAND, some fine gravel, little silt and clay (GLACIOFLUVIAL).	
							NQ2	37.0		Advanced with a roller bit through a boulder from 56.2 ft to 56.5 ft bgs. 56.5	
										Used NQ2 barrel to core through a boulder from 56.5 ft to 58.6 ft bgs.	
							OPEN	34.9		58.6	
60										Core barrel advanced to 61 ft bgs after breaking through boulder. Telescoped 3 inch diameter casing through the boulder to 61 ft bgs.	
65	13D	9.6/9.6	65.00 - 65.80	55-100/4"	R					Brown, wet, very dense, Gravelly fine to coarse SAND, little silt (GLACIOFLUVIAL).	
							NQ2	24.0		69.5	
70										Advanced with a roller bit past a boulder from 69.5 ft to 71.5 ft bgs.	
							OPEN	22.0		71.5	
	14D	24/13.2	72.00 - 74.00	23-46-57-81	103	156				Brown, wet, very dense, Gravelly fine to coarse SAND, little silt (GLACIOFLUVIAL).	WC = 9.0% Fines = 15.4% A-1-b (0)
75											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 35 minutes after completion of drilling with bottom of casing at 70 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-116
WIN: 018595.11

Driller:	S.W. Cole	**Elevation (ft.)**	93.5	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Andrew Martin	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	5/2/22 (15:00); 5/4/22 (10:00)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485309.571, E: 1739242.201	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	12.7 ft on 5/4/22 at 10:35
Hammer Efficiency Factor: 0.908		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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N-uncorrected = Raw Field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
LL = Liquid Limit
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C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75											
								15.5		78.0	
							NQ2			Advanced with a roller bit past cobbles from 78 ft to 79 ft bgs.	
								14.5		79.0	
80								13.5		80.0	
										Used NQ2 core barrel to core through a boulder from 80 ft to 82.6 ft bgs.	
								10.5		83.0	
										Bottom of Exploration at 83.0 feet below ground surface. Bottom of casing crimped at 83 ft bgs preventing further advancement of the boring. Boring backfilled with bentonite chips to ground surface.	
85											
90											
95											
100											

Remarks:

1. Hammer Efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Energy Measurement Calibration: R-2- Diedrich Drill D-50 with Automatic Hammer (S/N:367)" by Geoscience Testing and Research Inc., dated 9/23/2021. 2. As-drilled boring locations and ground surface elevations were provided by MaineDOT. 3. Water level reading was made 35 minutes after completion of drilling with bottom of casing at 70 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-201
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	117.7	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:27 PM 9/21/23 - 10:47 PM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 487189.137, E: 1739746.201	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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)
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Hammer Efficiency Factor = Rig Specific Annual Calibration Value
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WC = Water Content, percent
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
				Sample Information							
0							SSA	117.2		ASPHALT 0.5	
	1D	24/12	1.00 - 3.00	13-16-17-50/4"	33	50				Brown, dry, very dense, fine to coarse Gravelly SAND, trace to little silt, well-graded (FILL).	
								113.7		4.0	
5										(WEATHERED BEDROCK)	
	R1	12/3.6	5.40 - 6.40	RQD = 0%			NQ2	112.3		5.4	
	R2	32.4/31.2	6.40 - 9.10	RQD = 12%						Auger refusal. Top of Bedrock at Elev. 112.3 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low to steep angle dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 33% Recovery Rock Core Rate (min:sec) 5.4-6.4 ft (4:37)	
10	R3	44.4/44.4	9.10 - 12.80	RQD = 54%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to vertical dipping, close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 96% Recovery Rock Core Rate (min:sec) 6.4-7.4 ft (5:21) 7.4-8.4 ft (3:17) 8.4-9.1 ft (2:39)	
	R4	31.2/31.2	12.80 - 15.40	RQD = 46%							
15								102.3		R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Fair 100% Recovery Rock Core Rate (min:sec) 9.1-9.4 ft (0:25) 9.4-10.4 ft (2:11) 10.4-11.4 ft (2:56) 11.4-12.4 ft (1:53) 12.4-12.8 ft (1:22)	
20										R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low to steep angle dipping, close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Fair 100% Recovery Rock Core Rate (min:sec)	
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 10+23.9, Offset 80.3 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-201
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	117.7	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:27 PM 9/21/23 - 10:47 PM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 487189.137, E: 1739746.201	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										12.8-13.4 ft (1:49) 13.4-14.4 ft (2:29) 14.4-15.4 ft (3:04) 15.4 **Bottom of Exploration at 15.4 feet below ground surface.** Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-201A
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	117.8	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:26 PM 9/21/23 - 9:27 PM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	NX/NQ
Boring Location:	N: 487188.016, E: 1739744.060	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	117.3		ASPHALT 0.5 Brown, dry, very dense, fine to coarse Gravelly SAND, trace to little silt, well-graded (FILL).	
5	R1	16.8/16.5	5.10 - 6.50	RQD = 48%			NQ2	112.7 111.3		5.1 Top of Bedrock at Elev. 112.7 feet. Auger refusal. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, close, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 98% recovery Rock Core Rate (min:sec) 5.1-6.1 ft (3:12) 6.1-6.4 ft (2:18) 6.5 **Bottom of Exploration at 6.5 feet below ground surface.** Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
10											
15											
20											
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 10+24.69, Offset 77.9 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-201B
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	117.6	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:08 PM 9/21/23 - 8:26 PM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	NX/NQ
Boring Location:	N: 487189.874, E: 1739740.560	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	117.1		ASPHALT 0.5	
										Brown, dry, very dense, fine to coarse Gravelly SAND, trace to little silt, well-graded (FILL).	
								113.6		**Bottom of Exploration at 4.0 feet below ground surface.** 4.0	
5										Top of Bedrock at Elev. 113.6 feet. Auger refusal. Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
10											
15											
20											
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 10+22.26, Offset 72.8 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-202
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	118.7	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	2:50 AM 9/22/23 - 3:35 AM 9/22/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	NQ/NX
Boring Location:	N: 487118.074, E: 1739789.126	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.00 - 2.00	1-7-10-9	17	26	SSA	118.2		(TOPSOIL)	
										0.5	
										Brown, dry, medium dense, fine to medium SAND, some silt, little fine gravel (FILL).	
	R1	28.8/28.8	4.30 - 6.70	RQD = 28%			NQ2	114.4		4.3	
5										Auger refusal.	
										Top of Bedrock at Elev. 114.7 feet.	
										R1:Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low angle to very steep/vertical dipping, wide, moderate close to close spacing, extremely fractured, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Poor	
										100% Recovery	
										Rock Core Rate (min:sec)	
										4.3-5.3 ft (2:31)	
										5.3-6.3 ft (2:00)	
										6.3-6.7 ft (1:02)	
	R2	31.2/28.8	6.70 - 9.30	RQD = 64%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Fair	
										91% Recovery	
										Rock Core Rate (min:sec)	
										6.7-7.3 ft (0:45)	
										7.3-8.3 ft (1:26)	
										8.3-9.3 ft (1:37)	
10								109.4		9.3	
										Bottom of Exploration at 9.3 feet below ground surface.	
										Boring backfilled with bentonite chips to match existing surface.	
15											
20											
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 11+01.4, Offset 110.2 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-203
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	114	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	1:40 AM 9/22/23 - 2:39 AM 9/22/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	NQ/NX
Boring Location:	N: 487072.610, E: 1739610.188	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	1-1-2-2	3	5	SSA	113.8		(TOPSOIL)	
										0.3 Brown, moist, loose, Silty fine to medium SAND (FILL).	
5	2D	24/24	5.00 - 7.00	5-8-10-10	18	28		109.0		5.0 Olive, moist, very stiff, CLAY, moderately plastic (CLAY). q_p = 6.0 ksf (pocket penetrometer)	WC = 23.8% Fines = 99.0% LL = 31 PL = 19 PI = 12 LI = 0.4 A-6 (12), CL
							NQ2	106.2		7.8	
	R1	48/46.8	8.50 - 12.50	RQD = 11%				105.5		Auger refusal. (WEATHERED BEDROCK) 8.5 Top of Bedrock at Elev. 105.5 feet. R1: Grey, fine grained to very grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep angle dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 98% Recovery Rock Core Rate (min:sec) 8.5-9.5 ft (3:16) 9.5-10.5 ft (2:50) 10.5-11.5 ft (2:46) 11.5-12.5 ft (4:07)	
10	R2	12/12	12.50 - 13.50	RQD = 70%				100.5		R2: Grey, fine grained to very grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to low angle dipping, close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Fair 100% Recovery Rock Core Rate (min:sec) 8.5-9.5 ft (3:21)	
15										13.5 **Bottom of Exploration at 13.5 feet below ground surface.** Boring backfilled with bentonite chips to match existing surface.	
20											
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 11+15.3, Offset 73.8 feet R.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-204
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	116	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:56 PM 9/21/23 - 1:33 AM 9/22/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 487169.516, E: 1739605.640	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	115.4		ASPHALT 0.6	
	1D	24/8	1.00 - 3.00	11-11-7-5	18	28				Brown, dry, medium dense, fine to coarse SAND, little silt, little fine gravel (FILL).	
5	2D	24/23	5.00 - 7.00	4-6-7-7	13	20		111.0		5.0 Olive with brown mottled, moist, very stiff, CLAY, moderately plastic (CLAY). q_p = 7.0 to 4.5 ksf (pocket penetrometer)	WC = 24.0% LL = 37 PL = 23 PI = 14 LI = 0.1 A-6 (15), CL
10	3D	14.4/6	10.00 - 11.20	20-41-50/2"	R		2			Brown, wet, hard, Sandy CLAY, little fine gravel, rock fragments in the tip of spoon (GLACIAL TILL).	WC = 12.4% Fines = 52.8% A-4 (0)
	R1	60/22.8	11.20 - 16.20				53	104.8		11.2 Roller cone refusal. R1: 3-foot boulder	
							28/3"				
							OPEN				
15											
	4D	14/6	16.20 - 17.37	18-19-50/2"	R			99.8		16.2 Brown, wet, very dense, fine to coarse SAND, some silt, some fine gravel, strong petroleum-like odor, weathered rock fragments in tip of spoon (GLACIAL TILL).	
							NQ2	98.6		17.4 Auger refusal. Top of Bedrock at Elev. 98.6 feet. (WEATHERED BEDROCK)	
	R2	32.4/31.2	19.50 - 22.20	RQD = 25%				96.5		19.5 R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep to vertical dipping, close spacing, tight to open, extremely fractured, calcite intrusions, extremely fractured 20.5-22.2 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 96% Recovery Rock Core Rate (min:sec) 19.5-20.5 ft (2:19) 20.5-21.5 ft (2:51) 21.5-22.2 ft (3:14)	
20											
	R3	27.6/27.6	22.20 - 24.50	RQD = 17%							
25								91.5			

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 10+19.0, Offset 61.6 feet R.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-204
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	116	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:56 PM 9/21/23 - 1:33 AM 9/22/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 487169.516, E: 1739605.640	**Casing ID/OD:**	4.0/4.5 in and 3.0/3.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low to steep angle dipping, very close to close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 22.2-22.5 ft (1:12) 22.5-23.5 ft (2:24) 23.5-24.5 ft (3:08)	
										24.5 **Bottom of Exploration at 24.5 feet below ground surface.** Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-205
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	108.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:43 AM 9/25/23 - 10:46 AM 9/25/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486487.458, E: 1739878.180	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.4 ft on 9/25/23 at 10:45
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.00 - 2.00	1-1-4-2	5	8	SSA	107.6		(TOPSOIL) 0.5 Olive, moist, medium stiff, SILT, trace fine sand, trace fine gravel, slightly plastic (FILL).	
5	2D	24/23	5.00 - 7.00	4-6-7-8	13	20		103.1		5.0 Olive with grey mottled, moist, very stiff, CLAY (CLAY). q_p = 5.0 ksf to 7.0 ksf (pocket penetrometer).	WC = 27.7% Fines = 99.5% A-4 (0)
10	3D	24/24	10.00 - 12.00	2-3-3-3	6	9	6			Olive, moist to wet, stiff, CLAY, moderately plastic (CLAY). q_p = 2.5 ksf (pocket penetrometer).	WC = 24.8% LL = 30 PL = 18 PI = 12 LI = 0.6 CL
							13				
							50				
							112				
							56				
15	4D	24/12	15.00 - 17.00	15-19-17-21	36	55	24	93.1		15.0 Brown, wet, very dense, fine to coarse SAND, some silt, little fine gravel (SAND with GRAVEL).	
							50				
							60				
							60				
							43				
20	5D	24/12	20.00 - 22.00	13-13-12-14	25	38	32	88.1		20.0 Olive to Brown, wet, hard, SILT, some fine to medium sand, some fine to coarse gravel, slightly plastic (GLACIAL TILL).	
							32				
							51				
							55				
25							40				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 703+51.4, Offset 32.8 feet R.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-205
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	108.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:43 AM 9/25/23 - 10:46 AM 9/25/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 486487.458, E: 1739878.180	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.4 ft on 9/25/23 at 10:45
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/13	25.00 - 27.00	7-10-6-10	16	24	41			Grey, wet, very stiff, SILT, some fine to medium SAND, some fine to coarse gravel, slighty plastic (GLACIAL TILL).	
							38				
	R1	45.6/36	27.80 - 31.60	RQD = 16%			50/1" NQ2	80.3		27.8	
										Top of Bedrock at Elev. 80.31 feet. Roller cone refusal encountered at 27.8 feet bgs. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, close spacing, tight, frequent calcite intrusions, chemical weathering in bottom foot of the run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 79% Recovery Rock Core Rate (min:sec) 27.8-28.8 ft (2:05) 28.8-29.8 ft (3:33) 29.8-30.8 ft (2:53) 30.8-31.6 ft (2:28)	
30											
	R2	14.4/13.2	31.60 - 32.80	RQD = 0%							
								75.3		R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, close spacing, open to tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 92% Recovery Rock Core Rate (min:sec) 31.6-31.8 ft (0:25) 31.8-32.8 ft (3:25)	
35										32.8	
										Bottom of Exploration at 32.8 feet below ground surface. Boring backfilled with bentonite chips to match existing surface.	
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-206
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	118.4	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:48 PM 9/20/23 - 11:47 PM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 486285.194, E: 1739807.189	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.8 ft on 9/20/23 at 23:45
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	117.9		ASPHALT 0.5	
	1D	24/15	1.00 - 3.00	19-20-20-17	40	61				Brown, dry, very dense, fine to coarse Gravelly SAND, little silt (FILL).	
5	2D	24/12	5.00 - 7.00	3-3-7-4	10	15				Olive, moist, stiff, SILT, little fine gravel (FILL).	
10	3D	24/14	10.00 - 12.00	3-3-4-4	7	11	11	107.4		11.0 Grey, moist to wet, stiff, CLAY, moderately plastic (CLAY).	WC = 22.7% LL = 35 PL = 23 PI = 12 LI = 0 CL
							16				
							20				
							27				
15	4D	24/18	15.00 - 17.00	5-6-12-15	18	28	37			Olive to grey mottled, wet, very stiff, CLAY (CLAY). q_p > 10 ksf (pocket penetrometer).	
							63				
							81				
							88				
							45				
20	5D	24/10	20.00 - 22.00	15-36-16-21	52	80	OPEN			Olive to grey, wet, hard, CLAY, some fine to coarse sand, some fine to coarse gravel, fractured rock at 20.5 feet bgs (CLAY).	WC = 14.6% Fines = 55.7% A-4 (0)
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 207+89, Offset 25.3 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-206
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	118.4	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:48 PM 9/20/23 - 11:47 PM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 486285.194, E: 1739807.189	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.8 ft on 9/20/23 at 23:45
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	16/8	25.00 - 26.33	23-23-52/4"	R			92.1		Olive, wet, hard, CLAY, fractured rock in the tip of spoon (CLAY).	
										26.3	
										Bottom of Exploration at 26.3 feet below ground surface. Possible weathered bedrock. Boring backfilled with bentonite chips, and cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-207
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	116.7	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:06 PM 9/20/23 - 10:36 PM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 486572.986, E: 1739763.072	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.5 ft on 9/20/23 at 22:18
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	116.0		ASPHALT 0.7	
	1D	24/14	1.00 - 3.00	21-25-70-46	95	145				Brown, dry, very dense, fine to coarse SAND, some fine to coarse gravel, little silt, asphalt fragments (FILL).	
5	2D	24/19	5.00 - 7.00	41-23-15-11	38	58				Olive, dry, hard, SILT, some fine to coarse sand, trace fine gravel, non-plastic (FILL).	Fines = 70.8% A-4 (0)
10	3D	24/22	10.00 - 12.00	3-4-5-6	9	14	11	106.7		10.0 Olive to grey, moist, stiff, CLAY, trace organics (roots) (CLAY).	
							23				
							53				
							54				
15							36				
	4D	24/22	15.00 - 17.00	3-5-5-7	10	15	32			Olive, wet, stiff, CLAY, moderately plastic (CLAY). q_p = 6.0 ksf to 6.5 ksf (pocket penetrometer)	WC = 27.8% LL = 44 PL = 21 PI = 23 LI = 0.3 CL
							39				
							52				
							68				
							42				
20	5D	24/24	20.00 - 22.00	2-2-3-6	5	8	66	96.7		20.0 Olive, wet, medium stiff, Silty CLAY (CLAY). q_p = 2.5 ksf to 3.5 ksf (pocket penetrometer)	
							55				
							100/9				
							OPEN	93.0		23.8 23.75 feet bgs possible weathered rock.	
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 104+96, Offset 35.1 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-207
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	116.7	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:06 PM 9/20/23 - 10:36 PM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 486572.986, E: 1739763.072	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.5 ft on 9/20/23 at 22:18
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
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MV = Unsuccessful Field Vane Shear Test Attempt
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WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								91.7		25.0- **Bottom of Exploration at 25.0 feet below ground surface.** **Bottom of Exploration at 25.0 feet below ground surface.** Boring backfilled with bentonite chips, and cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-208
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	90.8	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:13 AM 10/02/23 - 9:14 AM 10/02/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485512.735, E: 1739866.267	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/5	0.00 - 2.00	1-3-4-6	7	11	SSA	90.6		(TOPSOIL) 0.3	
										Olive, dry, stiff, SILT, little fine to medium sand, trace fine gravel, trace organics (roots), non-plastic (FILL).	
5	2D	24/24	5.00 - 7.00	5-7-7-7	14	21		85.8		5.0	WC = 24.5% LL = 37 PL = 20 PI = 17 LI = 0.3 CL
										Olive with brown, moist, very stiff, CLAY, moderately plastic (CLAY). q_p = 5.0 ksf to 5.5 ksf (pocket penetrometer)	
10	3D	24/24	10.00 - 12.00	2-2-3-4	5	8	9	80.8		10.0	
							14			Olive, wet, medium stiff, Silty CLAY, moderately plastic (CLAY). q_p = 2.5 to 3.5 ksf (pocket penetrometer)	
							22				
							24				
							24				
15	4D	24/24	15.00 - 17.00	1-2-1-3	3	5	17			Grey, wet, soft, Silty CLAY, slightly plastic (CLAY). q_p < 2.0 ksf (pocket penetrometer)	WC = 28.2% LL = 23 PL = 16 PI = 7 LI = 1.7 CL-ML
							16				
							35				
							50				
							28				
20	5D	24/12	20.00 - 22.00	6-5-7-8	12	18	30	70.8		20.0	
							49			Grey, wet, medium dense, fine to medium SAND, some silt, some fine gravel (GLACIAL TILL).	
							51				
							71				
25							35				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 215+93, Offset 13.9 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-208
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	90.8	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:13 AM 10/02/23 - 9:14 AM 10/02/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485512.735, E: 1739866.267	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	-
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D		25.00 - 27.00	11-14-11-22	25	38	OPEN	63.8		Grey, wet, dense, fine to medium SAND, some silt, some fine gravel (GLACIAL TILL).	
										27.0	
										Bottom of Exploration at 27.0 feet below ground surface. Not refusal encounter. Boring backfilled with bentonite chips to match existing surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-209
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	105.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	12:11AM 9/21/23 - 1:35AM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485412.629, E: 1739959.139	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.8 ft on 9/12/23 at 01:23
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			CONCRETE	
								104.1		1.0	
	1D	24/15	1.00 - 3.00	17-14-14-15	28	43				Brown, dry, dense, fine to coarse SAND, some fine to coarse gravel, little silt (FILL).	
5	2D	24/20	5.00 - 7.00	15-15-14-17	29	44				Olive, dry, hard, SILT, some fine to coarse sand, trace fine gravel (FILL).	WC = 10.1% Fines = 68.0% A-4 (0)
10	3D	24/13	10.00 - 12.00	9-11-11-14	22	34	22			Olive with grey, dry, hard, SILT, little fine gravel (FILL).	
							42				
							98				
							106				
							42				
15	4D	24/14	15.00 - 17.00	3-7-30-57	37	57	25			Grey, wet, very dense, fine to coarse SAND, some silt, some fine to coarse gravel (FILL).	WC = 32.0% Fines = 10.2% A-2-4 (0)
							62				
							92				
							44				
							34				
20	5D	24/22	20.00 - 22.00	6-8-11-11	19	29	12	85.1		20.0	
										Olive to grey mottled, wet, very stiff, Clayey SILT (SILT). q_p = 7.0 to 8.0 ksf (pocket penetrometer)	
							40				
							57				
							70				
25							67				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
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4. Station 116+70, Offset 45.2 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-209
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	105.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	12:11AM 9/21/23 - 1:35AM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485412.629, E: 1739959.139	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.8 ft on 9/12/23 at 01:23
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/17	25.00 - 27.00	10-22-25-22	47	72	OPEN	80.1		Brown, wet, very dense, Silty fine to medium SAND, little fine gravel (SAND with GRAVEL). — 25.0	
								78.1		**Bottom of Exploration at 27.0 feet below ground surface.** — 27.0	
										Not refusal encounter.	
										Boring backfilled with bentonite chips up to bottom of concrete, and cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-210
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	102.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:00 PM 9/17/23 - 4:40 AM 9/18/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485339.432, E: 1739884.834	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.6 ft on 9/18/23 at 03:45
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.20 - 2.20	12-10-10-12	20	31	SSA	101.9		ASPHALT 0.2	
										Brown, dry, dense, fine to medium SAND, some fine to coarse gravel, little silt (FILL).	
5	2D	24/15	5.00 - 7.00	10-11-17-17	28	43				Brown to grey, moist, dense, fine to medium SAND, some silt, trace fine gravel (FILL).	WC = 10.5%
10	3D	24/20	10.00 - 12.00	8-9-15-16	24	37		92.1		10.0	
										Olive to brown with orange, dry to moist, hard, Clayey SILT, trace fine sand, trace fine gravel (SILT).	
15	4D	24/17	15.00 - 17.00	15-13-9-11	22	34	89			Olive to brown, dry to moist, hard, Clayey SILT, trace fine sand, trace fine gravel (SILT).	WC = 12.0%
							75				
							78				
							96				
							99				
20	5D	24/22	20.00 - 22.00	7-8-10-12	18	28	97	82.1		20.0	
							135			Olive, dry to moist, very stiff, CLAY, trace fine sand (CLAY).	
							147				
							153				
25							75				

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-210
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	102.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:00 PM 9/17/23 - 4:40 AM 9/18/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485339.432, E: 1739884.834	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.6 ft on 9/18/23 at 03:45
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	3-7-9-12	16	24	147			Olive to grey, wet, very stiff, CLAY, moderately plastic (CLAY). q_p = 8.0 ksf, 8.0 ksf, and 8.5 ksf (pocket penetrometer)	WC = 23.6% LL = 36 PL = 20 PI = 16 LI = 0.2 CL
							163				
							178				
							156				
							70				
30	7D	24/13	30.00 - 32.00	16-35-36-35	71	109	152	72.1		30.0 Grey, wet, very dense, fine to coarse Gravelly SAND, little silt, trace clay (GLACIAL TILL).	
							137				
							131				
							118				
							85				
35	8D	24/16	35.00 - 37.00	4-5-4-6	9	14	125			Grey, wet, medium dense, fine to medium SAND, trace silt (GLACIAL TILL).	
							116				
							99				
							101				
							90				
40	9D		40.00 - 42.00	6-11-12-35	23	35	140			Grey, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, some silt (GLACIAL TILL).	WC = 6.8% Fines = 23.4% A-1-b (0)
							129				
							125				
							132				
							141				
45	10D	24/16	45.00 - 47.00	18-16-16-16	32	49	133			Grey, wet, dense, fine to coarse SAND, little silt (GLACIAL TILL).	
							137				
							145				
							149				
50							163				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-210
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	102.1	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	9:00 PM 9/17/23 - 4:40 AM 9/18/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485339.432, E: 1739884.834	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.6 ft on 9/18/23 at 03:45
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/22	50.00 - 52.00	10-22-23-37	45	69	171			Grey, moist to wet, hard, SILT, trace fine gravel (GLACIAL TILL).	
							173				
							188				
							182				
							143				
55	12D	24/13	55.00 - 57.00	42-63-50/2"	R		275			Grey to brown, wet, very dense, fine to coarse Gravelly SAND, little silt (GLACIAL TILL).	
							100/1				
								45.1		57.0	
	R1	15.6/8.4	57.00 - 58.30	RQD = 0%			NQ2			Top of Bedrock at Elev. 45.1 feet. Roller cone refusal at 57 feet bgs. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh to slightly weathered; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 54% Recovery Rock Core Rate (min:sec) 57-58 ft (3:23) 58-58.3 ft (1:28)	
	R2	18/6	58.30 - 59.80	RQD = 0%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 36% Recovery Rock Core Rate (min:sec) 58.3-59 ft (2:28) 59-59.8 ft (3:54)	
	R3	62.4/26.4	59.80 - 65.00	RQD = 0%						R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 42% Recovery Rock Core Rate (min:sec) 59.8-60 ft (0:24) 60-61 ft (3:15) 61-62 ft (3:20) 62-63 ft (2:57) 63-64 ft (3:13) 64-65 ft (2:42)	
60											
								37.1		65.0	
65										**Bottom of Exploration at 65.0 feet below ground surface.** Backfilled with bentonite chips to match existing ground surface.	
70											
75											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-211
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	98.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	2:34 AM 9/20/23 - 3:56 AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485225.093, E: 1740037.794	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	12.9 ft on 9/20/23 at 03:48
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	97.8		ASPHALT 0.7	
	1D	24/14	1.00 - 3.00	10-14-10-10	24	37				Brown, dry, dense, fine to coarse Gravelly SAND, little silt (FILL).	
5	2D	24/12	5.00 - 7.00	9-7-6-7	13	20		93.5		5.0 Olive, moist, very stiff, SILT, little fine to coarse sand, trace fine gravel, metal fragment at 6.5 feet bgs (SILT). q_p = 8.0 ksf to 9.0 ksf (pocket penetrometer)	WC = 22.1% Fines = 81.0% A-4 (0)
10	3D	24/22	10.00 - 12.00	2-3-7-7	10	15	4			Grey to olive mottled, wet, stiff, SILT (SILT).	
							17				
							28				
							39				
							26				
15	4D	24/24	15.00 - 17.00	2-3-3-4	6	9	15	83.5		15.0 Olive with grey, wet, stiff, CLAY, moderately plastic (CLAY). q_p = 3.0 ksf (pocket penetrometer)	WC = 30.1% LL = 34 PL = 21 PI = 13 LI = 0.7 CL
							29				
							40				
							42				
							63				
20	5D	24/12	20.00 - 22.00	15-13-11-20	24	37	12	78.5		20.0 Olive, to brown, wet, dense, fine to medium Silty SAND, little fine to coarse gravel (SAND with GRAVEL).	WC = 11.8% Fines = 38.7% A-4 (0)
							49				
							66				
							98				
25							77				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 401+70, Offset 20.2 feet R.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-211
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	98.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	2:34 AM 9/20/23 - 3:56 AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485225.093, E: 1740037.794	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	12.9 ft on 9/20/23 at 03:48
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/16	25.00 - 27.00	14-14-15-15	29	44	OPEN	71.5		Brown to olive, wet, dense, Silty SAND, little fine gravel (SAND with GRAVEL).	
										27.0	
										Bottom of Exploration at 27.0 feet below ground surface. No refusal encounter. Boring backfilled with bentonite chips, and cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-212
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	97.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	1:48 AM 9/21/23 - 3:20 AM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485194.455, E: 1739963.729	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	11.6 ft on 9/21/23 at 02:59
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15	0.00 - 2.00	4-9-8-8	17	26	SSA	97.3		(TOPSOIL) 0.3 Brown, dry, medium dense, fine to coarse Gravelly SAND, some silt (FILL).	
5	2D	24/12	5.00 - 7.00	6-12-9-8	21	32				Brown, moist to wet, dense, fine to coarse SAND, some fine to coarse gravel, little silt (FILL).	Fines = 17.7% A-1-b (0)
10	3D	24/14	10.00 - 12.00	1-1-4-8	5	8	5	87.5		10.0 Grey, wet, medium stiff, CLAY, slightly plastic (CLAY).	WC = 21.4% LL = 27 PL = 18 PI = 9 LI = 0.4 CL
							14				
							40				
							47				
							46				
15	4D	24/16	15.00 - 17.00	9-8-11-14	19	29	48	82.5		15.0 Olive and grey mottled, wet, very stiff, CLAY, trace sand, moderately plastic (CLAY). q_p = 9.5 ksf (pocket penetrometer)	WC = 24.7% Fines = 97.7% LL = 38 PL = 20 PI = 18 LI = 0.3 A-6 (19), CL
							46				
							61				
							75				
							51				
20	5D	24/16	20.00 - 22.00	6-18-25-21	43	66	71	77.5		20.0 Brown, wet, very dense, fine to medium Silty SAND, some fine to coarse gravel (SAND with GRAVEL).	WC = 11.5% Fines = 42.4% A-4 (0)
							99				
							127				
							143				
25							88				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 119+06, Offset 39.3 feet L.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-212
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	97.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	1:48 AM 9/21/23 - 3:20 AM 9/21/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 485194.455, E: 1739963.729	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	11.6 ft on 9/21/23 at 02:59
Hammer Efficiency Factor: 0.918		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/14	25.00 - 27.00	20-21-15-12	36	55	OPEN	70.5		Brown, wet, very dense, fine to coarse Silty SAND, some fine to coarse gravel (SAND with GRAVEL).	
										27.0	
										Bottom of Exploration at 27.0 feet below ground surface. Boring backfilled with bentonite chips to match existing surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-213
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	81.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:06PM 9/19/23 - 11:25AM 9/19/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484647.909, E: 1740117.969	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	7.6 ft on 9/19/23 at 22:21
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
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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
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T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	81.0		ASPHALT 0.5	
	1D	24/14	1.00 - 3.00	15-14-21-19	35	54				Brown, dry, very dense, fine to coarse Gravelly SAND, little silt (FILL).	
5	2D	24/8	5.00 - 7.00	9-9-6-5	15	23				Brown, dry, medium dense, fine to coarse Gravelly SAND, little silt (FILL).	Fines = 16.4% A-1-b (0)
10	3D	24/14	10.00 - 12.00	9-13-17-21	30	46	29	71.5		10.0 Olive, wet, hard, Clayey SILT, trace fine sand, trace fine gravel (SILT).	
							35				
							70				
							74				
							49				
15	4D	24/9	15.00 - 17.00	11-6-5-6	11	17	15			Grey, wet, very stiff, Clayey SILT, trace fine to coarse sand, trace fine gravel, slightly plastic (SILT). q_p = 7.0 ksf (pocket penetrometer)	WC = 13.9% LL = 18 PL = 13 PI = 5 LI = 0.2 CL-ML
							41				
							28				
							37				
							43				
20	5D	24/6	20.00 - 22.00	8-6-6-7	12	18	26			Grey, wet, very stiff, Clayey SILT, trace fine to coarse sand, trace fine gravel, slightly plastic (SILT). q_p = 8.0 ksf (pocket penetrometer)	
							30				
							51				
							89				
25							42/8"				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 225+00.3, Offset 15.1 feet L.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-213
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	81.5	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	10:06PM 9/19/23 - 11:25AM 9/19/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484647.909, E: 1740117.969	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	7.6 ft on 9/19/23 at 22:21
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D		25.00 - 27.00	9-16-30-31	46	70	OPEN	54.5		Grey, wet, hard, Clayey SILT, trace fine to coarse sand, trace fine gravel, slightly plastic (SILT).	
										27.0 **Bottom of Exploration at 27.0 feet below ground surface.** No refusal encountered. Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-214
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.8	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:40PM 9/19/23 - 9:52PM 9/19/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484505.412, E: 1740123.448	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	8.2 ft on 9/19/23 at 21:39
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	78.4		ASPHALT 0.4	
	1D	24/10	1.00 - 3.00	6-12-8-8	20	31				Brown, dry, dense, fine to medium SAND, little silt, fine gravel (FILL).	
5	2D	24/22	5.00 - 7.00	8-6-9-13	15	23				Olive, moist, very stiff, SILT (FILL).	
10	3D	24/20	10.00 - 12.00	9-11-11-13	22	34	24	68.8		10.0 Brown, wet, dense, fine to medium Silty SAND, trace gravel (SAND with GRAVEL).	Fines = 43.0% A-4 (0)
							30				
							56				
							60				
							26				
15	4D	24/14	15.00 - 17.00	5-12-11-10	23	35	20	63.8		15.0 Olive to grey, wet, hard, Clayey SILT (GLACIAL TILL).	
							28				
							45				
							101				
							54				
20	5D	24/7	20.00 - 22.00	10-13-14-26	27	41	OPEN			Grey, wet, hard, Clayey SILT, fine to medium sand, fine gravel (GLACIAL TILL).	
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 38+37.9, Offset 27.1 feet L.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-214
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.8	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	8:40PM 9/19/23 - 9:52PM 9/19/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484505.412, E: 1740123.448	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	8.2 ft on 9/19/23 at 21:39
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D			17-21-38-60	59	90				Grey, wet, hard, Clayey SILT, some fine to medium sand, little fine gravel, rock fragments at bottom of sample (GLACIAL TILL).	WC = 12.8% Fines = 58.7% A-4 (0)
								51.8		27.0	
										Bottom of Exploration at 27.0 feet below ground surface. No refusal encountered. Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-215
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.6	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	1:05AM 9/20/23 - 2:20AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484503.667, E: 1740063.128	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.6 ft on 9/20/23 at 02:04
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	78.0		ASPHALT 0.6	
	1D	24/8	1.00 - 3.00	19-15-14-10	29	44				Brown, dry, dense, fine to coarse SAND, little silt, rounded to sub angular gravel (FILL).	
5	2D	24/14	5.00 - 7.00	6-6-6-7	12	18		73.6		5.0 Olive with grey, dry, very stiff, SILT, trace sand, trace fine gravel, slightly plastic (SILT). q_p = 7.0 to 8.0 ksf (pocket penetrometer)	WC = 17.9% Fines = 91.8% A-4 (0)
10	3D	24/18	10.00 - 12.00	19-15-18-13	33	50	40			Olive with grey and orange, moist, hard, SILT, little fine to medium sand, little fine to coarse gravel (SILT).	
							50				
							44				
							31				
15	4D	24/15	15.00 - 17.00	12-10-11-12	21	32	20			Olive and grey, wet, hard, SILT, little fine rounded gravel (SILT). q_p = 7.0 to 8.0 ksf (pocket penetrometer)	
							23				
							33				
							135				
							37				
20	5D		20.00 - 22.00	7-7-7-8	14	21	OPEN	58.6		20.0 Grey, wet, very stiff, clayey SILT, little fine to coarse rounded gravel, slightly plastic (GLACIAL TILL).	WC = 9.9% LL = 17 PL = 12 PI = 5 LI = -0.4 CL-ML
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 37+33.7, Offset 32.9 feet R.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-215
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.6	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	1:05AM 9/20/23 - 2:20AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	N/A
Boring Location:	N: 484503.667, E: 1740063.128	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.6 ft on 9/20/23 at 02:04
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/10	25.00 - 27.00	25-24-30-32	54	83		51.6		Brown with grey, wet, very dense, fine to coarse SAND, some fine to coarse gravel, little silt (GLACIAL TILL).	
										27.0	
										Bottom of Exploration at 27.0 feet below ground surface.	
										No refusal encountered.	
										Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-216
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.2	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	11:38PM 9/19/23 - 12:52AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 484574.735, E: 1740015.232	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.6 ft on 9/20/23 at 00:39
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	77.7		ASPHALT 0.5	
	1D	24/12	1.00 - 3.00	10-10-7-6	17	26				Brown, dry, medium dense, fine to coarse SAND, little fine to coarse gravel, little silt (FILL).	
5	2D		5.00 - 7.00	4-6-10-8	16	24				Olive, moist, very stiff, SILT, little fine gravel, non-plastic (FILL).	
10	3D	24/12	10.00 - 12.00	3-3-3-6	6	9	6	68.2		10.0 Grey, moist, stiff, CLAY, trace fine gravel, non-plastic, wood fragments at 11 ft bgs (CLAY).	WC = 15.6% LL = 25 PL = 17 PI = 8 LI = -0.2 CL
							20				
							17				
							19				
							8				
15	4D	24/8	15.00 - 17.00	6-29-25-9	54	83	21	63.2		15.0 Grey, wet, very dense, fine to coarse Gravelly SAND, trace silt (SAND with GRAVEL).	
							109				
							82				
							44				
							30				
20	5D	24/4	20.00 - 22.00	53-19-10-9	29	44	50			Grey, wet, dense, fine to coarse Gravelly SAND, trace silt (poor recovery; same as above) (SAND with GRAVEL).	
							81				
							73				
							83				
25							40				

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 36+57.6, Offset 78.2 feet R.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-216
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	78.2	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	11:38PM 9/19/23 - 12:52AM 9/20/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 484574.735, E: 1740015.232	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	10.6 ft on 9/20/23 at 00:39
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D		25.00 - 27.00	10-12-10-10	22	34	OPEN	51.2		Brown, wet, dense, fine to medium SAND, some fine gravel, some silt (SAND with GRAVEL).	WC = 9.7% Fines = 30.1% A-2-4 (0)
										27.0	
										Bottom of Exploration at 27.0 feet below ground surface. No refusal encountered. Boring backfilled with bentonite chips to bottom of pavement then cold patch to top of pavement.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc., dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-217
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.4	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	7:47AM 9/26/23 - 10:17 AM 9/26/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485824.994, E: 1739646.308	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	7.6 ft on 9/26/23 at 8:09
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	3-3-3-5	6	9	SSA	90.7		(TOPSOIL) 0.7 Brown, dry, stiff, SILT, some fine to medium sand, trace organics (roots) (SILT).	
5	2D	24/24	5.00 - 7.00	4-5-6-6	11	17				Olive, moist, very stiff, Clayey SILT, slightly plastic (SILT). q_p = 4.0 ksf to 6.0 ksf (pocket penetrometer)	
10	3D	24/2	10.00 - 12.00	16-28-28-17	56	86		81.4		10.0 Brown, wet, very dense, fine to coarse SAND, little silt, trace fine gravel, poorly graded (SAND with GRAVEL).	
15	4D	24/15	15.00 - 17.00	35-23-11-20	34	52	151 OPEN			Brown, wet, hard, fine to coarse SAND, little silt, weathered rock fragments (SAND with GRAVEL).	
	R1	33.6/28.8	17.90 - 20.70	RQD = 50%			NQ2	73.5		17.9 Roller cone refusal. Top of Bedrock at Elev. 73.5 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, very close to close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER BANGOR FORMATION] Rock Mass Quality = Poor 86% Recovery Rock Core Rate (min:sec) 17-18.9 ft (2:17) 18.9-19.9 ft (2:56) 19.9-20.7 ft (2:32)	
20	R2	26.4/25.2	20.70 - 22.90	RQD = 55%				68.5		R2: Grey, fine grained to very fine grained, METAWACKE	
25											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.
4. Station 807+77.6, Offset 100.8 feet L.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Diverging Diamond Interchange (Exit 187)
Location: Bangor, Maine

Boring No.: HB-BHR-217
WIN: 018595.11

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.4	**Auger ID/OD:**	N/A (ID) 4.5-in (OD)
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs / 30 in
Date Start/Finish:	7:47AM 9/26/23 - 10:17 AM 9/26/23	**Drilling Method:**	Solid Stem Auger / Drive and Wash	**Core Barrel:**	1.98 in I.D. - NQ2
Boring Location:	N: 485824.994, E: 1739646.308	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	7.6 ft on 9/26/23 at 8:09
Hammer Efficiency Factor:	0.918	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										[metasandstone], strong, fresh; discontinuities low angle to steep dipping, wide spacing, open, extremely fractured frequent calcite veins [MEDIUM BEDDED FACIES, PENOBSCOT RIVER] Mass Quality = Fair 95% Recovery Rock Core Rate (min:sec) 20.7-20.9 ft (0:24) 20.9-21.9 ft (2:45) 20.9-21.9 ft (2:45) 21.9-22.9 ft (2:18)	
30										22.9 **Bottom of Exploration at 22.9 feet below ground surface.** Boring backfilled with bentonite chips to match existing surface.	
35											
40											
45											
50											

Remarks:

1. Hammer efficiency factor provided by S.W. Cole and taken from "Standard Penetration Test Measurement Calibration: Track Rig-Diedrich D-50 with automatic Hammer (S/N - 367)" by Geoscience Testing and Research Inc. , dated 11/04/2022.
2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.2csv" received by WSP on October 13, 2023.

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

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

APPENDIX B

Rock Core Photographs

APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BB-BHR-101	R1	24.9 - 27.7	2.5 / 2.8	89	1.10 / 2.8	39	METAWACKE	1	4/29/2022
	R2	27.7 - 28.7	0.9 / 1.0	90	0.00 / 1.0	0	METAWACKE	1	4/29/2022
	R3	28.7 - 33.7	4.7 / 5.0	94	2.45 / 5.0	49	METAWACKE	1,2	4/29/2022
	R4	33.7 - 34.9	1.2 / 1.2	100	0.55 / 1.2	46	METAWACKE	3	4/29/2022
HB-BHR-105	R1	24.0 - 27.0	2.2 / 3.0	73	1.55 / 3.0	52	METAWACKE	4	4/29/2022
	R2	27.0 - 29.0	2.0 / 2.0	100	0.40 / 2.0	20	METAWACKE	4	4/29/2022



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

Prepared By: MA
Checked By: ATM
Reviewed By: JRS

APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
HB-BHR-111	R1	36.2 - 41.2	4.8 / 5.0	96	2.75 / 5.0	55	METAWACKE	1	5/5/2022
HB-BHR-102	R1	46.8 - 51.8	4.1 / 5.0	82	2.00 / 5.0	40	METAWACKE	1	5/9/2022



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

Prepared By: MA
Checked By: ATM
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APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BB-BHR-103	R1	23.0 - 26.6	3.5 / 3.6	97	0.45 / 3.6	13	METAWACKE	1	4/26/2022
	R2	26.6 - 28.6	2.0 / 2.0	100	0.00 / 2.0	0	METAWACKE	1, 2	4/26/2022
	R3	28.6 - 32.0	3.4 / 3.4	100	0.67 / 3.4	20	METAWACKE	2	4/26/2022
	R4	32.0 - 33.4	1.4 / 1.4	100	0.65 / 1.4	46	METAWACKE	2, 3	4/26/2022
	R5	33.4 - 34.8	1.0 / 1.4	71	0.90 / 1.4	64	METAWACKE	3	4/26/2022
	R6	34.8 - 38.0	3.2 / 3.2	100	0.60 / 3.2	19	METAWACKE	3	4/26/2022
HB-BHR-108	R1	29.0 - 31.3	2.1 / 2.3	91	0.60 / 2.3	26	METAWACKE	4	4/27/2022
	R2	31.3 - 34.0	2.7 / 2.7	100	1.30 / 2.7	48	METAWACKE	4	4/27/2022



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

Prepared By: MA
Checked By: ATM
Reviewed By: JRS

APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
HB-BHR-210	R1	57.0 - 58.3	0.7 / 1.3	54	0.00 / 1.3	0	METAWACKE	1	9/18/2023
	R2	58.3 - 59.8	0.5 / 1.5	36	0.00 / 1.5	0	METAWACKE	1	9/18/2023
	R3	59.8 - 65.0	2.2 / 5.2	42	0.00 / 5.2	0	METAWACKE	1	9/18/2023
HB-BHR-201A	R1	5.1 - 6.5	1.4 / 1.4	98	0.67 / 1.4	48	METAWACKE	2	9/21/2023
HB-BHR-201	R1	5.4 - 6.4	0.3 / 1.0	33	0.00 / 1.0	0	METAWACKE	2	9/21/2023
	R2	6.4 - 9.1	2.6 / 2.7	96	0.33 / 2.7	12	METAWACKE	2	9/21/2023
	R3	9.1 - 12.8	3.7 / 3.7	100	2.00 / 3.7	54	METAWACKE	3	9/21/2023
	R4	12.8 - 15.4	2.6 / 2.6	100	1.20 / 2.6	46	METAWACKE	3, 4	9/21/2023



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: LMP
Checked By: KAR
Reviewed By: MEL

APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
HB-BHR-204	R1	11.2 - 16.2	1.9 / 5.0	38	N/A / N/A	N/A	BOULDER	1	9/21/2023
	R2	19.5 - 22.2	2.6 / 2.7	96	0.67 / 2.7	25	METAWACKE	1	9/21/2023
	R3	22.2 - 24.5	2.3 / 2.3	100	0.40 / 2.3	17	METAWACKE	2	9/21/2023
HB-BHR-203	R1	8.5 - 12.5	3.9 / 4.0	98	0.42 / 4.0	11	METAWACKE	2, 3	9/22/2023
	R2	12.5 - 13.5	1.0 / 1.0	100	0.70 / 1.0	70	METAWACKE	3	9/22/2023
HB-BHR-202	R1	4.3 - 6.7	2.4 / 2.4	100	0.67 / 2.4	28	METAWACKE	3, 4	9/22/2023
	R2	6.7 - 9.3	2.4 / 2.6	91	1.67 / 2.6	64	METAWACKE	4	9/22/2023



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: LMP
Checked By: KAR
Reviewed By: MEL

APPENDIX B
ROCK CORE PHOTOGRAPHS
HOGAN ROAD DIVERGING DIAMOND INTERCHANGE, BANGOR, MAINE
MAINEDOT WIN 18595.11

Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
HB-BHR-205	R1	27.8 - 31.6	3.0 / 3.8	79	0.60 / 3.8	16	METAWACKE	1	9/25/2023
	R2	31.6 - 32.8	1.1 / 1.2	92	0.00 / 1.2	0	METAWACKE	1	9/25/2023
BB-BHR-201	R1	25.0 - 27.0	1.7 / 2.0	85	0.00 / 2.0	0	METAWACKE	2	9/25/2023
	R2	27.0 - 30.5	2.5 / 3.5	71	0.00 / 3.5	0	METAWACKE	2	9/25/2023
	R3	30.5 - 31.4	0.6 / 0.9	67	0.00 / 0.9	0	METAWACKE	3	9/25/2023
	R4	31.4 - 35.0	3.6 / 3.6	100	0.75 / 3.6	21	METAWACKE	3	9/25/2023
HB-BHR-217	R1	17.9 - 20.7	2.4 / 2.8	86	1.40 / 2.8	50	METAWACKE	4	9/26/2023
	R2	20.7 - 22.9	2.1 / 2.2	95	1.20 / 2.2	55	METAWACKE	4	9/26/2023



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: LMP
Checked By: KAR
Reviewed By: MEL

APPENDIX C

Laboratory Testing Results

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669087		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-101	3D	12-14 ft	Moist, olive clay	27.9
HB-BHR-101	6D	25-27 ft	Moist, olive silt with gravel	9.0
HB-BHR-102	3D	12-14 ft	Moist, pale olive clay	27.6
HB-BHR-102	5D	20-22 ft	Moist, olive silty sand with gravel	12.7
HB-BHR-102	8D	35-37 ft	Moist, olive gray silty, clayey sand	9.4
HB-BHR-103	2D	5-7 ft	Moist, grayish brown clay	25.4
HB-BHR-103	3D	10-11.4 ft	Moist, brown clay	24.0
HB-BHR-104	2D	5-7 ft	Moist, olive clay with gravel	25.5
HB-BHR-104	4D- A	15-15.9 ft	Moist, gray clay	31.6

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669075		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-105	5D	20-22 ft	Moist, gray silty gravel with sand	9.3
HB-BHR-105	2D	5-7 ft	Moist, olive clay	25.2
HB-BHR-105	4D	15-17 ft	Moist, gray clay	29.8
HB-BHR-106	3D	10-12 ft	Moist, olive clay	25.6
HB-BHR-107	3D	10-12 ft	Moist, light olive brown clay	30.0
HB-BHR-107	4D- A	15-16.7 ft	Moist, gray clay	29.1
HB-BHR-108	2D	5-7 ft	Moist, olive clay	25.5
HB-BHR-108	3D- A	10-11.6 ft	Moist, olive brown clay	26.5
HB-BHR-108	4D	15-17 ft	Moist, brown silty gravel with sand	6.0
HB-BHR-108	6D	25-27 ft	Moist, gray silty sand with gravel	11.6

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669064		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-109	2D	5-7 ft	Moist, olive gray clay	27.5
HB-BHR-109	4D	15-17 ft	Moist, gray silty gravel with sand	11.9
HB-BHR-109	6D	27-29 ft	Moist, gray silty sand with gravel	9.1
HB-BHR-110	2DA	5-6.9 ft	Moist, dark gray clay	44.6
HB-BHR-111	2D	5-7 ft	Moist, light olive brown clay	25.0
HB-BHR-111	4D	15-17 ft	Moist, light olive brown sandy silt	12.5
HB-BHR-111	5D	20-22 ft	Moist, light gray silty sand	9.9
HB-BHR-112	2D	5-7 ft	Moist, brown clay	15.9
HB-BHR-112	4D	15-17 ft	Moist, light olive brown sandy silt with gravel	14.5
HB-BHR-112	5D	20-22 ft	Moist, light brownish gray silt	21.9

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669067		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-113	6D	20-22 ft	Moist, light gray clay with sand	18.1
HB-BHR-113	8D	27-29 ft	Moist, gray clay	14.1
HB-BHR-113	9D	30-32 ft	Moist, light olive brown silty sand with gravel	8.9
HB-BHR-113	11D	40-42 ft	Moist, gray silty sand with gravel	11.3
HB-BHR-113	2D	5-7 ft	Moist, olive brown clay with sand	15.6
HB-BHR-113	3D	10-12 ft	Moist, olive silty sand with gravel	12.6
HB-BHR-113	5D	17-19 ft	Moist, olive silty sand with gravel	17.8

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669055		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-114	2D	5-7 ft	Moist, light olive brown sandy silt	16.5
HB-BHR-114	5D	20-22 ft	Moist, olive brown silt with sand	28.3
HB-BHR-114	7D	30-32 ft	Moist, gray silty, clayey sand with gravel	9.2
HB-BHR-114	10D	45-47 ft	Moist, light gray gravelly silt with sand	10.3
HB-BHR-114	12D	55-57 ft	Moist, brown silty sand	12.2

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669057		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-115	4D	15-17 ft	Moist, light olive brown clay	25.8
HB-BHR-115	7D	30-32 ft	Moist, light gray sandy silt	12.3
HB-BHR-115	9D	40-42 ft	Moist, gray sandy silt	12.6
HB-BHR-115	11D	50-52 ft	Moist, gray sandy silt	13.7
HB-BHR-115	1D	1-3 ft	Moist, brown silty sand with gravel	5.2
HB-BHR-115	2D	5-7 ft	Moist, light olive brown sandy clay	12.6

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/23/22	Checked By:	bfs
Depth :	---	Test Id:	669046		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-116	2D	5-7 ft	Moist, brown clay	21.9
HB-BHR-116	4D	15-17 ft	Moist, light olive brown silty sand	10.3
HB-BHR-116	7D	30-32 ft	Moist, gray sandy clay	12.2
HB-BHR-116	11D	50-52 ft	Moist, light olive brown silty sand with gravel	9.4
HB-BHR-116	14D	72-74 ft	Moist, brown silty sand with gravel	9.0

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/27/22	Checked By:	bfs
Depth :	---	Test Id:	669834		

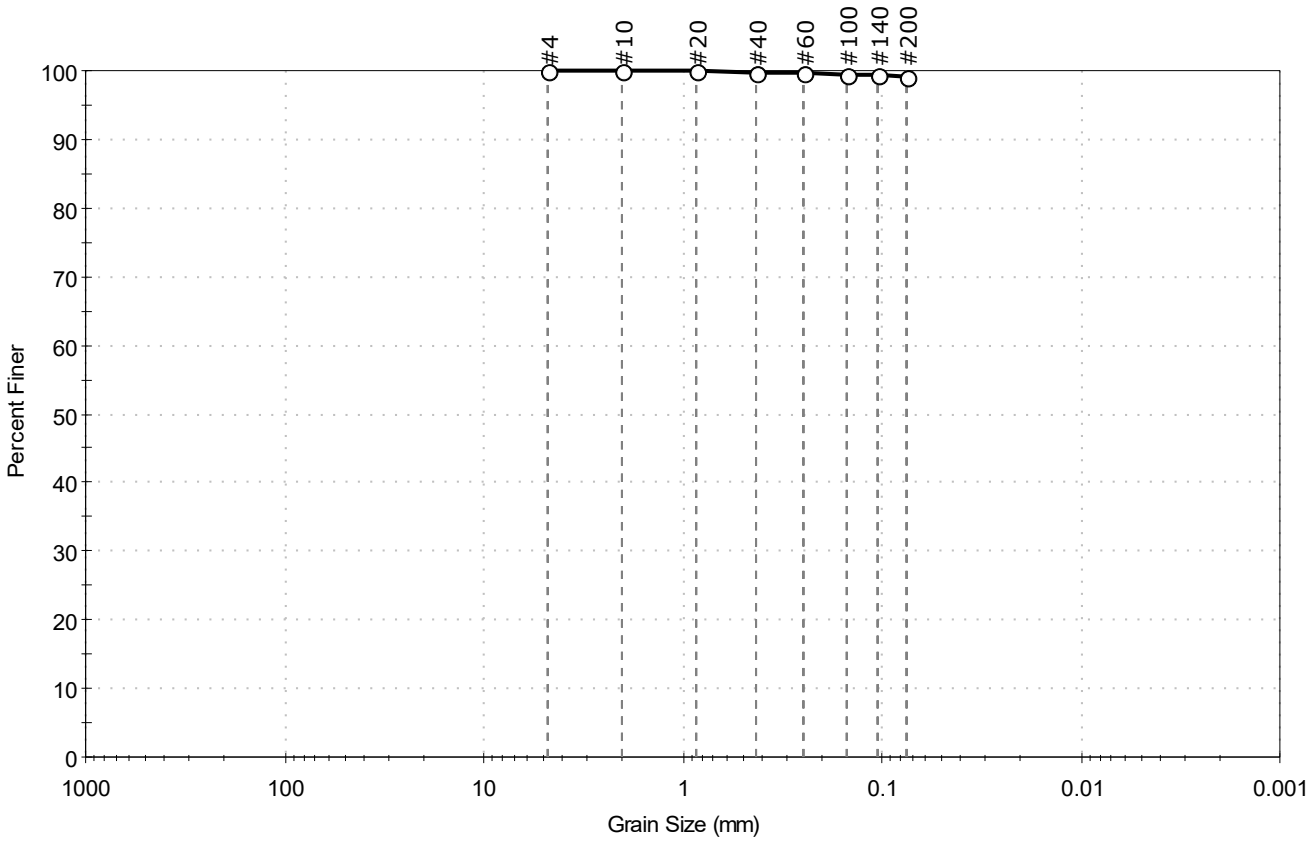
Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content,%	Ash Content,%	Organic Matter,%
HB-BHR-101	1U	10-12 ft	Moist, olive clay	28	99.0	1.0
HB-BHR-102	1U	10.5-12.5 ft	Moist, olive clay	26	98.9	1.1
HB-BHR-108	1U	8-10 ft	Moist, olive clay	28	98.8	1.2

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass;
dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-101	Sample Type:	jar	Tested By:	ckg
Sample ID:	3D	Test Date:	05/23/22	Checked By:	bfs
Depth :	12-14 ft	Test Id:	669116		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.8	99.2

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

Coefficients

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

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

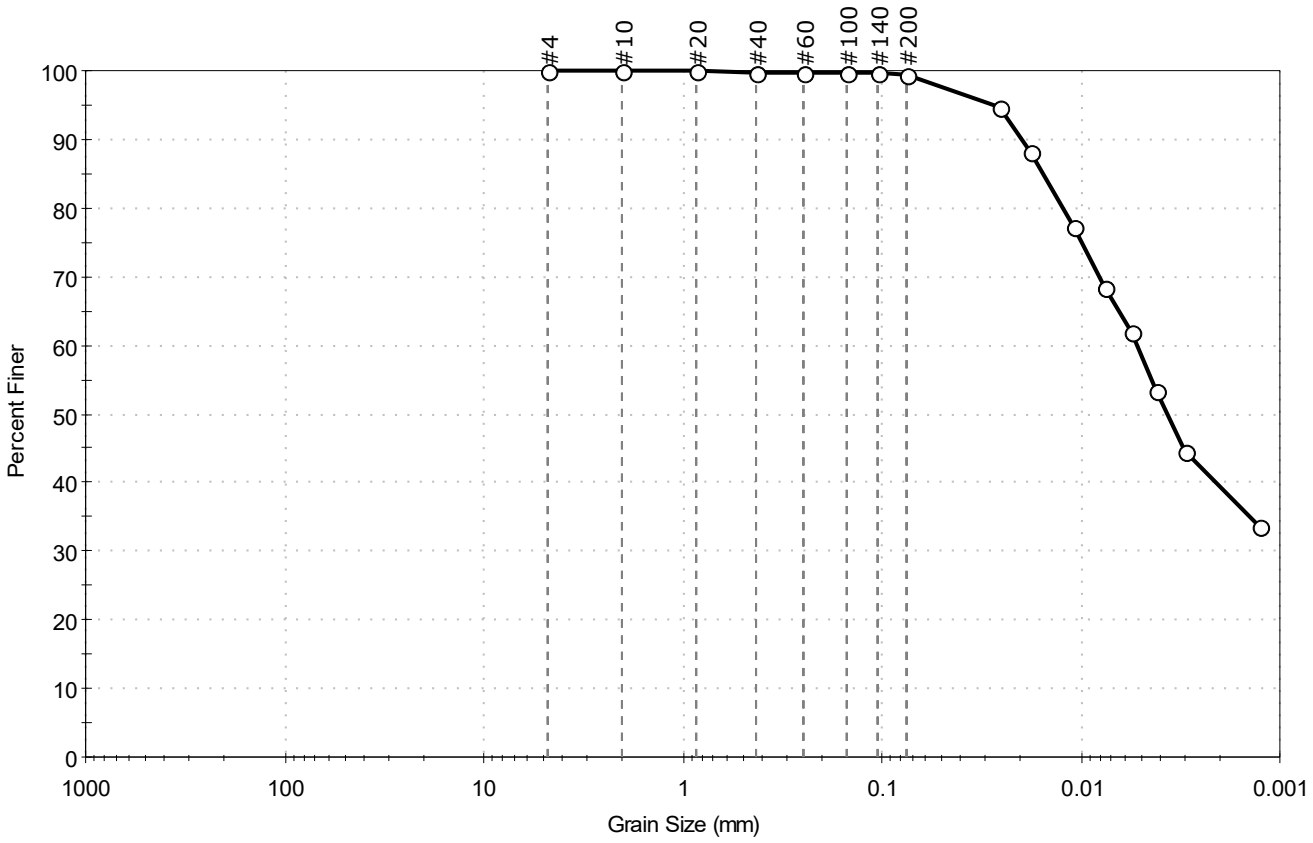
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	3D	Test Date:	05/31/22	Checked By:	bfs
Depth :	12-14 ft	Test Id:	669124		
Test Comment:	---				
Visual Description:	Moist, pale olive clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.6	99.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	100		
#200	0.075	99		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0255	95		
---	0.0180	88		
---	0.0108	77		
---	0.0076	68		
---	0.0056	62		
---	0.0041	53		
---	0.0030	45		
---	0.0013	34		

Coefficients

D_{85} = 0.0156 mm	D_{30} = N/A
D_{60} = 0.0052 mm	D_{15} = N/A
D_{50} = 0.0037 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (18))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

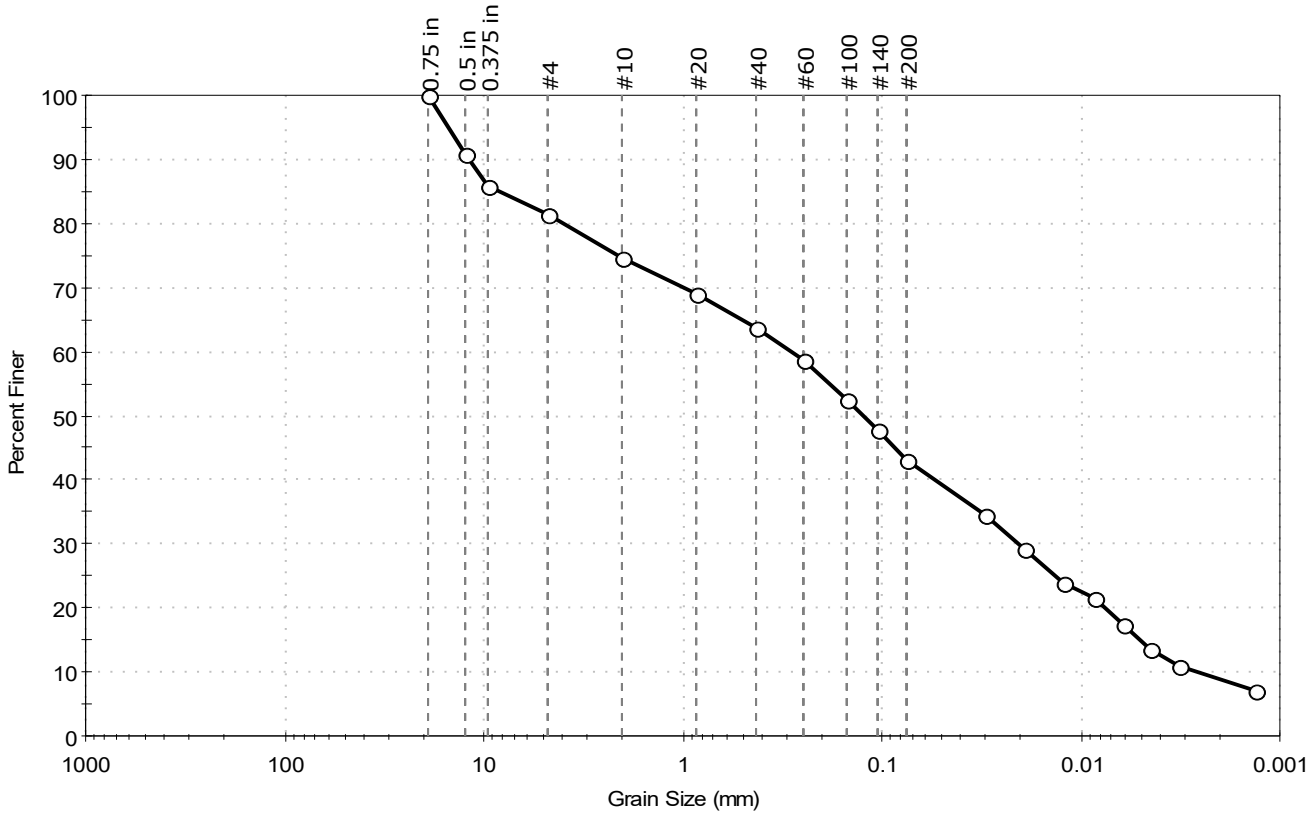
Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/31/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669125		
Test Comment:	---				
Visual Description:	Moist, olive silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



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

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	91		
0.375 in	9.50	86		
#4	4.75	81		
#10	2.00	75		
#20	0.85	69		
#40	0.42	64		
#60	0.25	59		
#100	0.15	53		
#140	0.11	48		
#200	0.075	43		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0300	34		
---	0.0192	29		
---	0.0122	24		
---	0.0086	21		
---	0.0061	18		
---	0.0045	14		
---	0.0032	11		
---	0.0013	7		

Coefficients

D_{85} =8.2702 mm	D_{30} =0.0205 mm
D_{60} =0.2834 mm	D_{15} =0.0050 mm
D_{50} =0.1248 mm	D_{10} =0.0026 mm
C_u =109.000	C_c =0.570

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

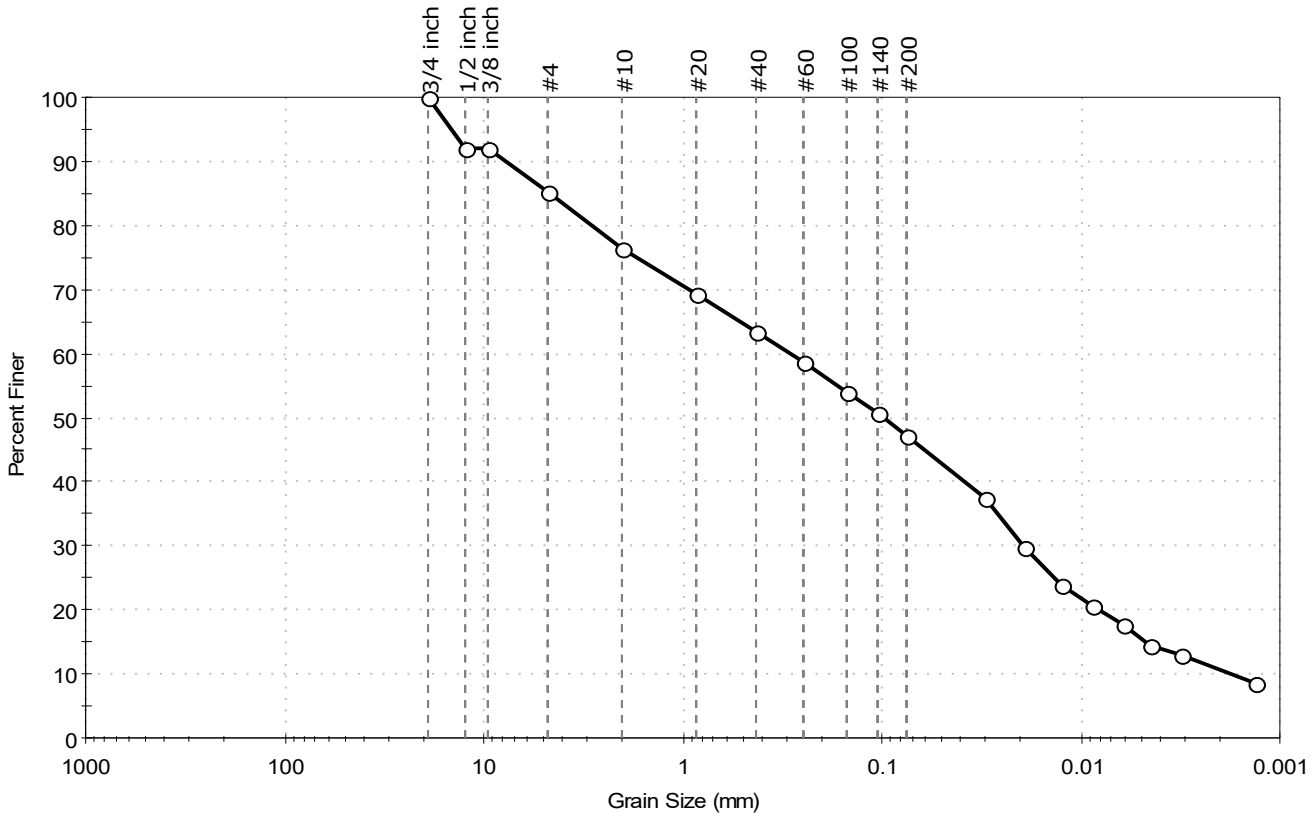
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	8D	Test Date:	05/31/22	Checked By:	bfs
Depth :	35-37 ft	Test Id:	669126		
Test Comment:	---				
Visual Description:	Moist, olive gray silty, clayey sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
–	14.7	38.1	47.2

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	92		
#4	4.75	85		
#10	2.00	77		
#20	0.85	69		
#40	0.42	63		
#60	0.25	59		
#100	0.15	54		
#140	0.11	51		
#200	0.075	47		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0304	38		
---	0.0194	30		
---	0.0124	24		
---	0.0088	21		
---	0.0062	18		
---	0.0045	15		
---	0.0032	13		
---	0.0013	8		

Coefficients

D_{85} =4.5957 mm	D_{30} =0.0196 mm
D_{60} =0.2912 mm	D_{15} =0.0047 mm
D_{50} =0.0994 mm	D_{10} =0.0018 mm
C_u =161.778	C_c =0.733

Classification

ASTM Silty, Clayey SAND (SC-SM)

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

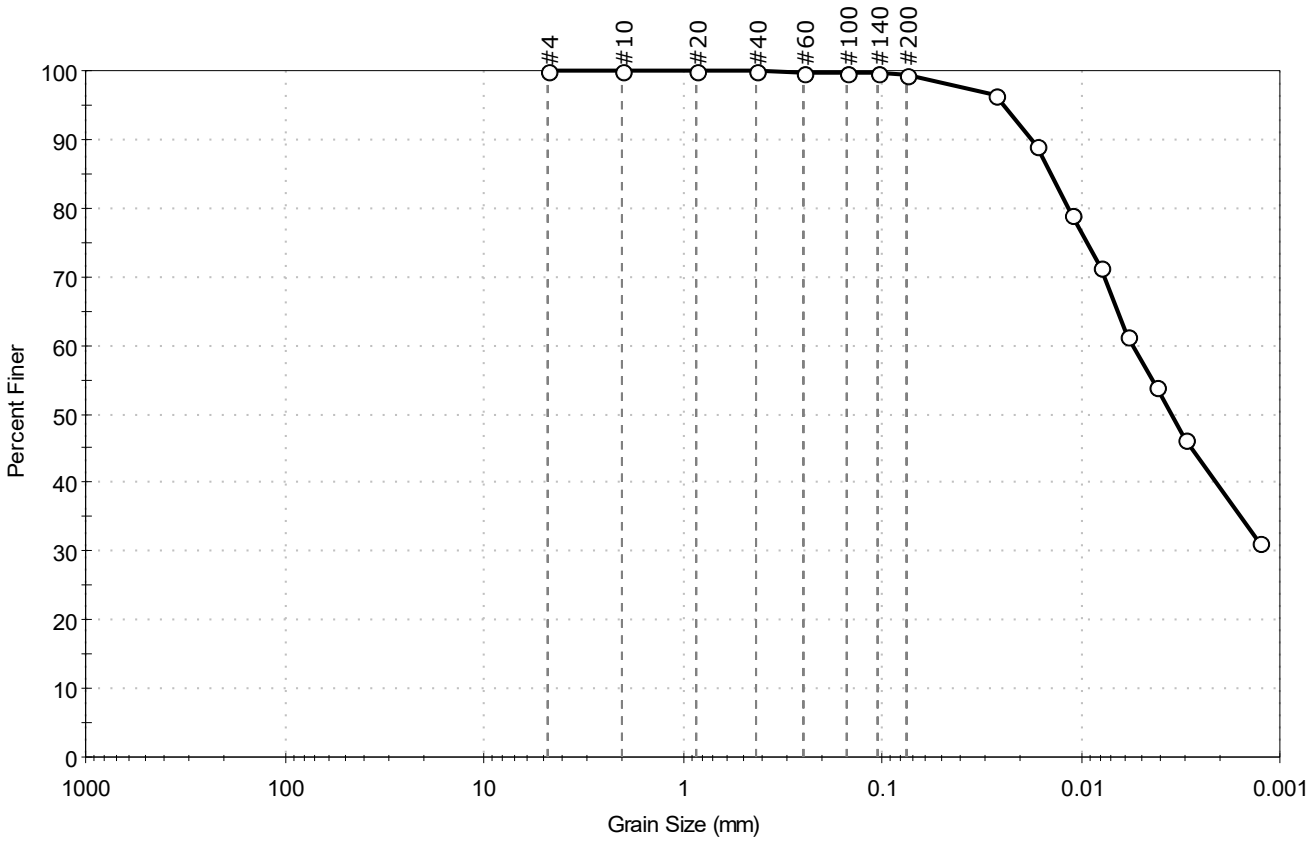
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/31/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669127		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.6	99.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	100		
#200	0.075	99		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0272	97		
---	0.0167	89		
---	0.0111	79		
---	0.0079	72		
---	0.0058	61		
---	0.0042	54		
---	0.0030	46		
---	0.0013	31		

Coefficients

D_{85} = 0.0142 mm	D_{30} = N/A
D_{60} = 0.0054 mm	D_{15} = N/A
D_{50} = 0.0035 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

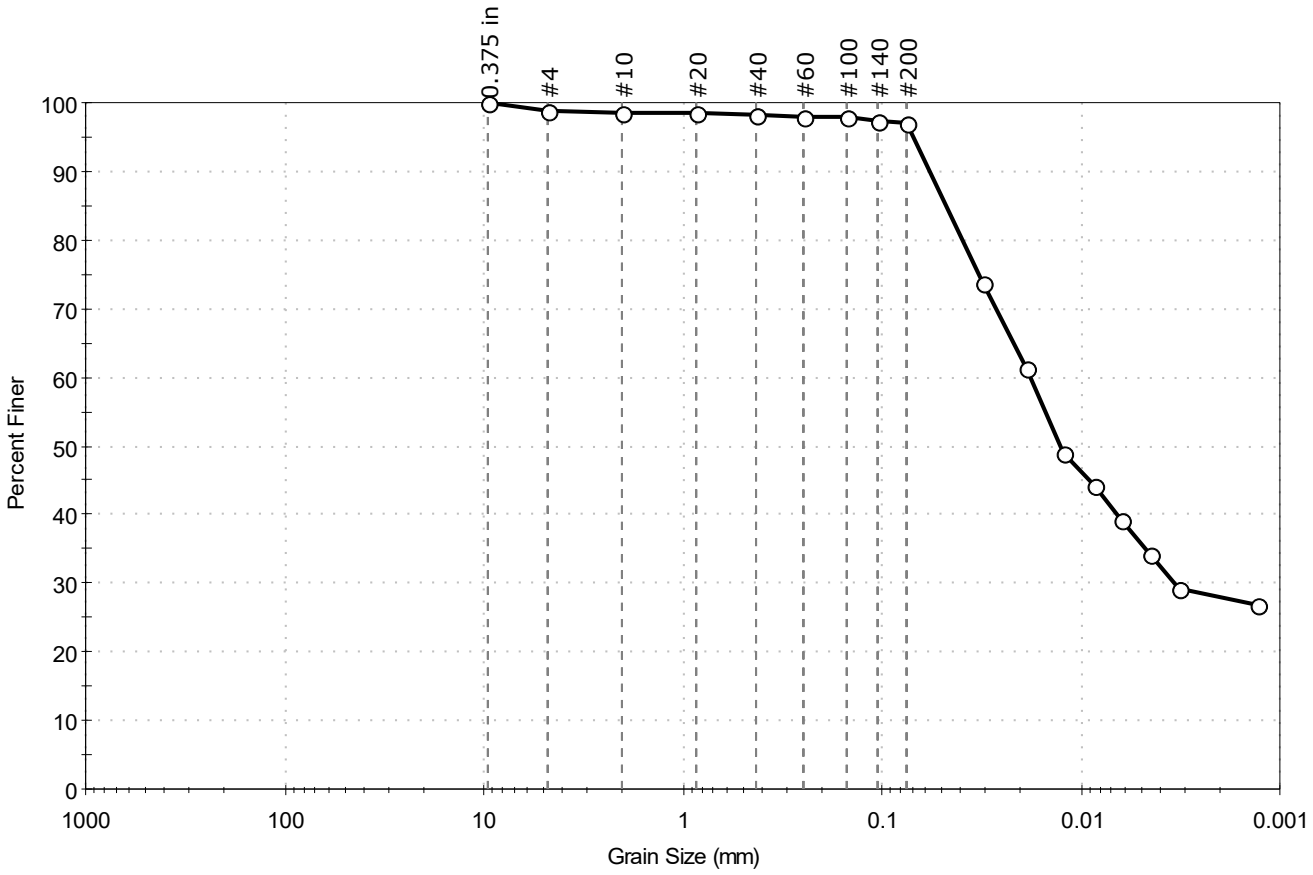
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	3D	Test Date:	05/27/22	Checked By:	bfs
Depth :	10-11.4 ft	Test Id:	669128		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.2	1.8	97.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	99		
#20	0.85	98		
#40	0.42	98		
#60	0.25	98		
#100	0.15	98		
#140	0.11	97		
#200	0.075	97		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0309	74		
---	0.0189	61		
---	0.0122	49		
---	0.0085	44		
---	0.0063	39		
---	0.0045	34		
---	0.0032	29		
---	0.0013	27		

Coefficients

D_{85} = 0.0474 mm	D_{30} = 0.0033 mm
D_{60} = 0.0179 mm	D_{15} = N/A
D_{50} = 0.0126 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (9))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

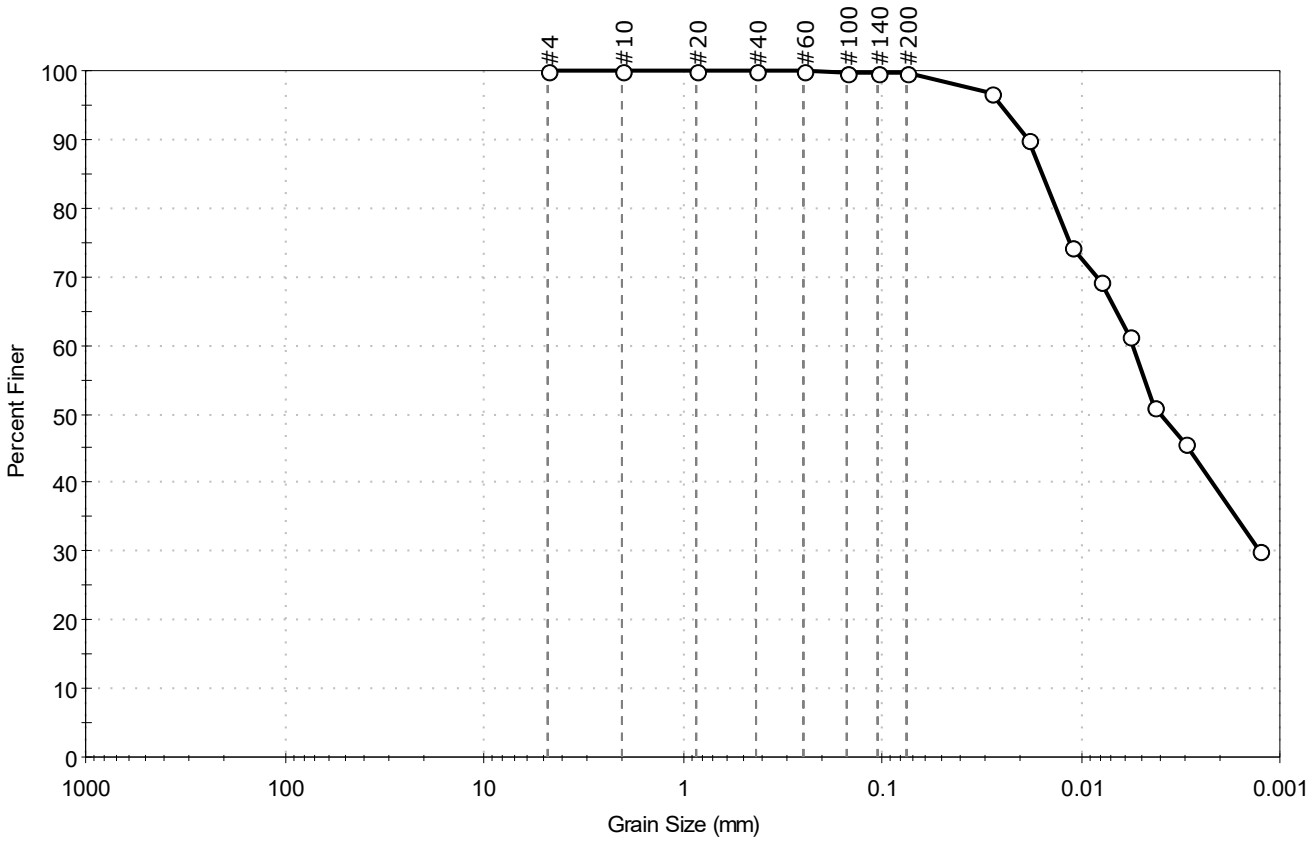
Separation of Sample: #200 Sieve

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GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-104	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D-A	Test Date:	05/31/22	Checked By:	bfs
Depth :	15-15.9 ft	Test Id:	669129		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.3	99.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	100		
#200	0.075	100		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0284	97		
---	0.0182	90		
---	0.0113	74		
---	0.0079	69		
---	0.0057	61		
---	0.0043	51		
---	0.0030	46		
---	0.0013	30		

Coefficients

D_{85} = 0.0156 mm D_{30} = N/A
D_{60} = 0.0055 mm D_{15} = N/A
D_{50} = 0.0040 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

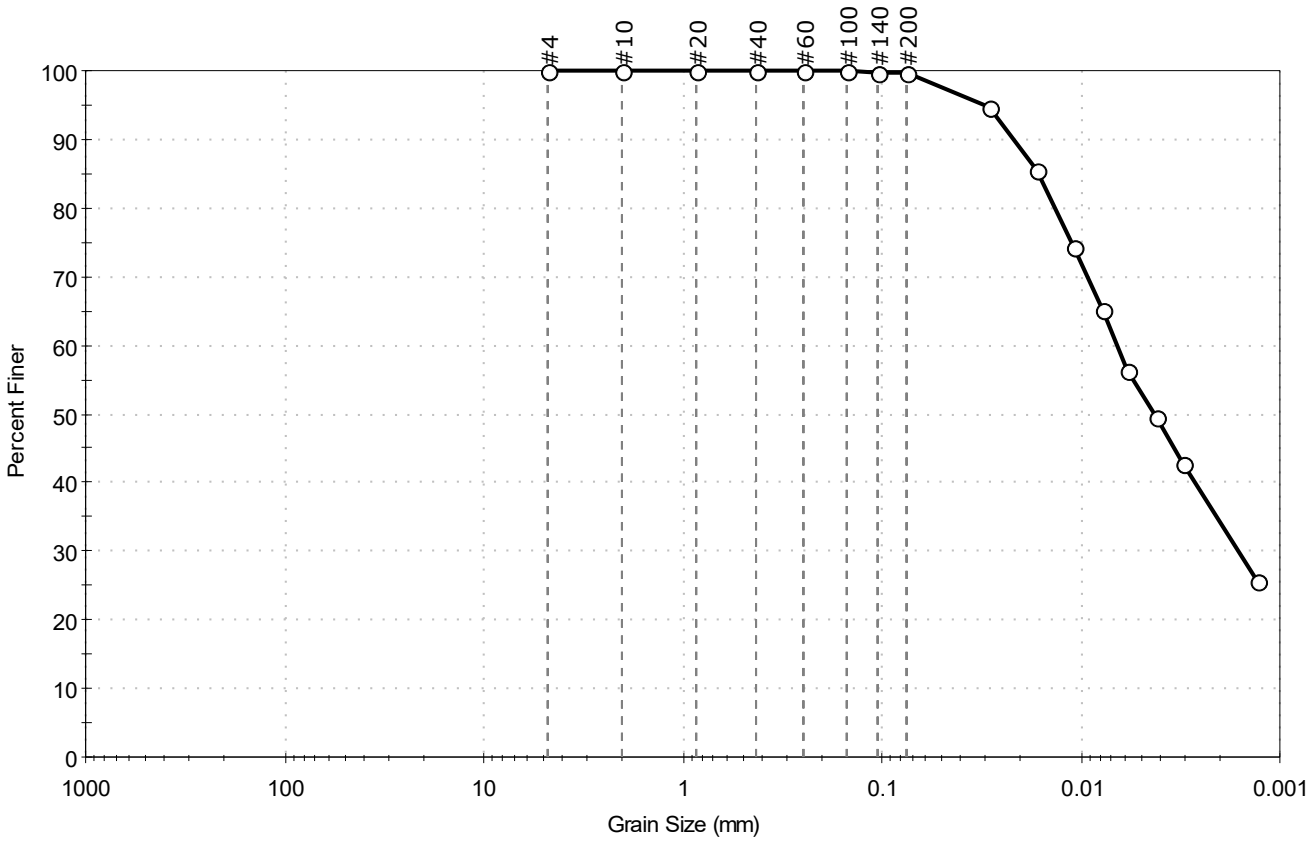
AASHTO Silty Soils (A-4 (0))

Sample/Test Description

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

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/27/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669130		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.2	99.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	100		
#200	0.075	100		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0286	95		
---	0.0169	86		
---	0.0110	74		
---	0.0078	65		
---	0.0058	56		
---	0.0042	49		
---	0.0031	43		
---	0.0013	26		

Coefficients

D_{85} = 0.0164 mm	D_{30} = 0.0016 mm
D_{60} = 0.0066 mm	D_{15} = N/A
D_{50} = 0.0043 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (13))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

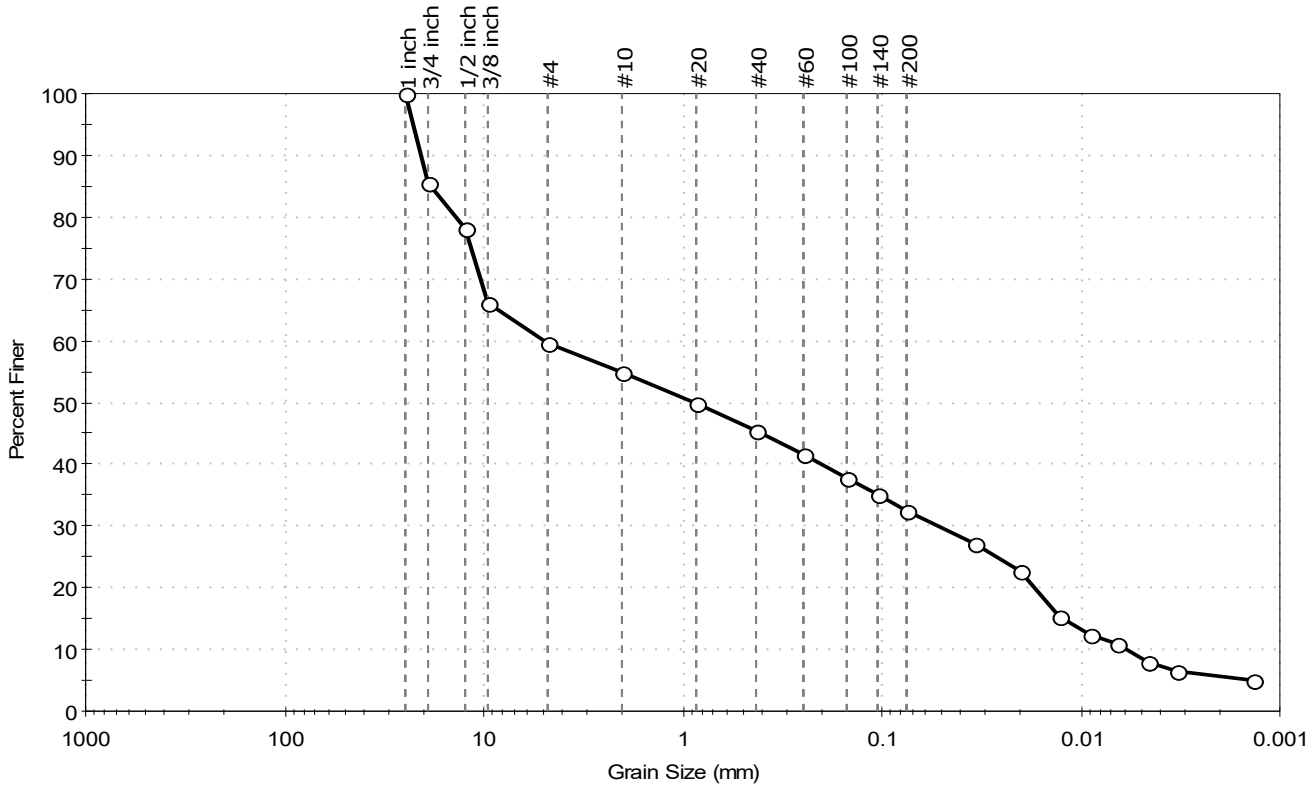
Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/31/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669117		
Test Comment:	---				
Visual Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	40.5	26.9	32.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	85		
1/2 inch	12.50	78		
3/8 inch	9.50	66		
#4	4.75	59		
#10	2.00	55		
#20	0.85	50		
#40	0.42	45		
#60	0.25	42		
#100	0.15	38		
#140	0.11	35		
#200	0.075	33		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0339	27		
---	0.0201	23		
---	0.0129	15		
---	0.0089	12		
---	0.0065	11		
---	0.0047	8		
---	0.0033	7		
---	0.0014	5		

Coefficients

D_{85} = 18.5288 mm D_{30} = 0.0518 mm
D_{60} = 5.0390 mm D_{15} = 0.0123 mm
D_{50} = 0.8891 mm D_{10} = 0.0058 mm
C_u = 868.793 C_c = 0.092

Classification

ASTM N/A

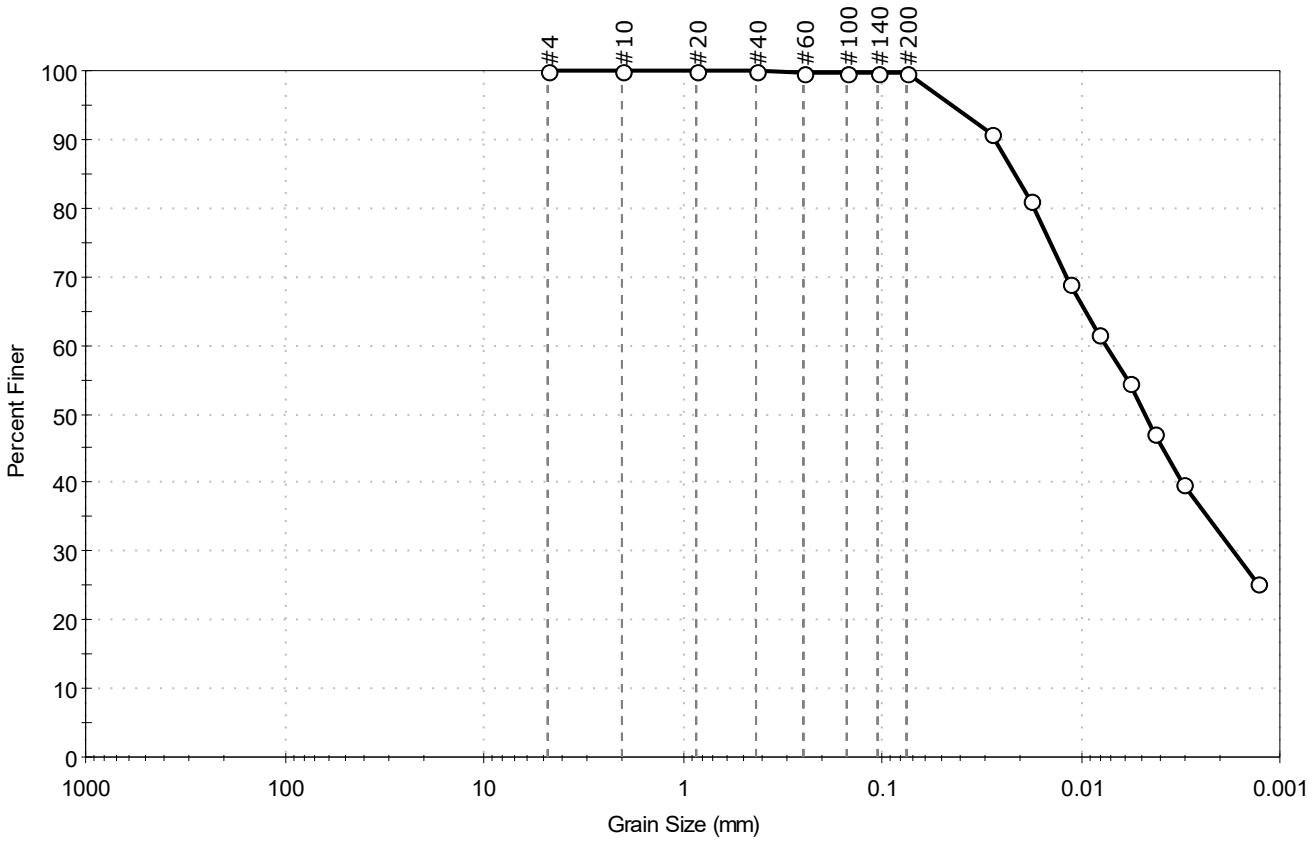
AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

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

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-107	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D-A	Test Date:	05/31/22	Checked By:	bfs
Depth :	15-16.7 ft	Test Id:	669118		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.3	99.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	100		
#140	0.11	100		
#200	0.075	100		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0281	91		
---	0.0180	81		
---	0.0114	69		
---	0.0081	62		
---	0.0058	54		
---	0.0043	47		
---	0.0031	40		
---	0.0013	25		

Coefficients

D_{85} = 0.0216 mm D_{30} = 0.0017 mm
D_{60} = 0.0075 mm D_{15} = N/A
D_{50} = 0.0048 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

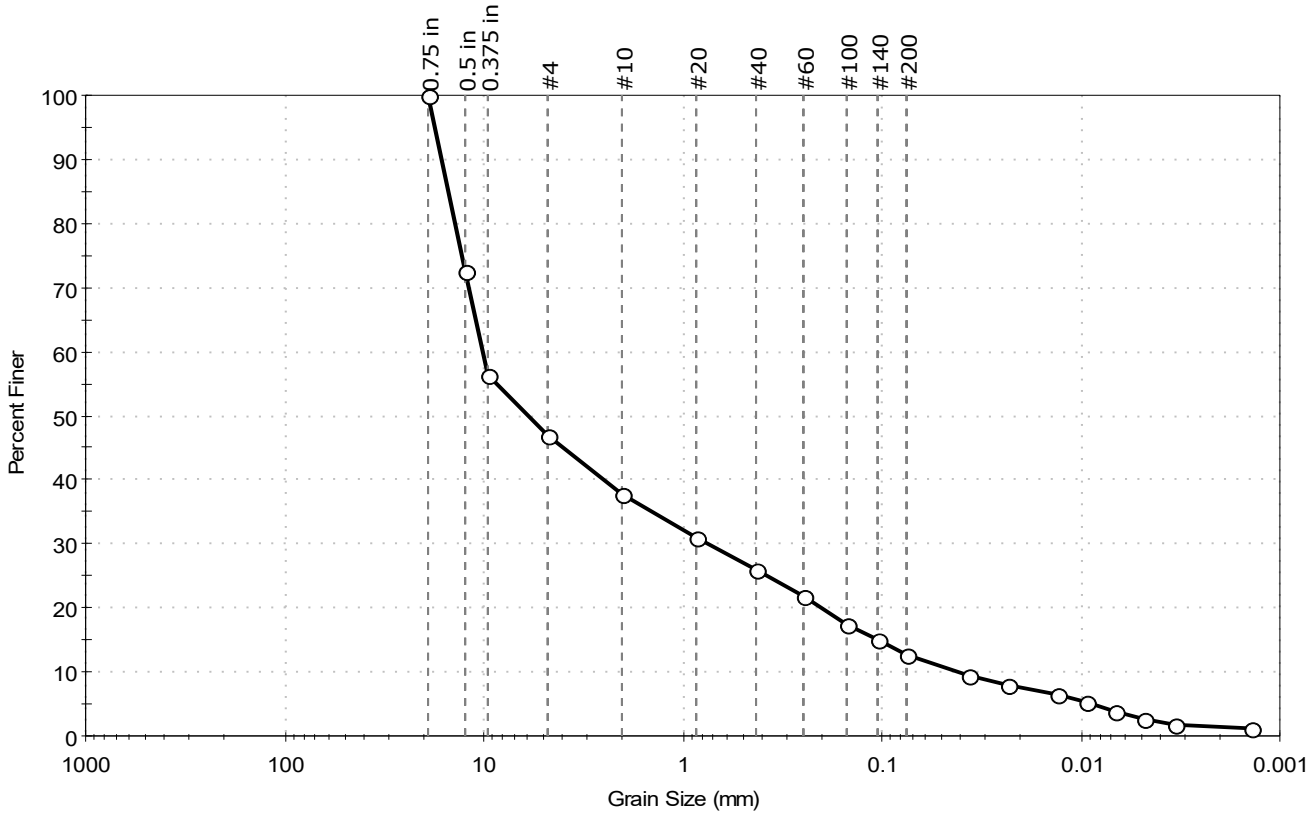
AASHTO Silty Soils (A-4 (0))

Sample/Test Description

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

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-108	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/27/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669119		
Test Comment:	---				
Visual Description:	Moist, brown silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
–	53.1	34.3	12.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	73		
0.375 in	9.50	56		
#4	4.75	47		
#10	2.00	38		
#20	0.85	31		
#40	0.42	26		
#60	0.25	22		
#100	0.15	17		
#140	0.11	15		
#200	0.075	13		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0368	9		
---	0.0233	8		
---	0.0132	7		
---	0.0095	5		
---	0.0068	4		
---	0.0048	3		
---	0.0034	2		
---	0.0014	1		

Coefficients

D_{85} = 15.1205 mm
D_{30} = 0.7521 mm
D_{60} = 10.1266 mm
D_{15} = 0.1067 mm
D_{50} = 5.9895 mm
D_{10} = 0.0427 mm
C_u = 237.157
C_c = 1.308

Classification

ASTM N/A

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

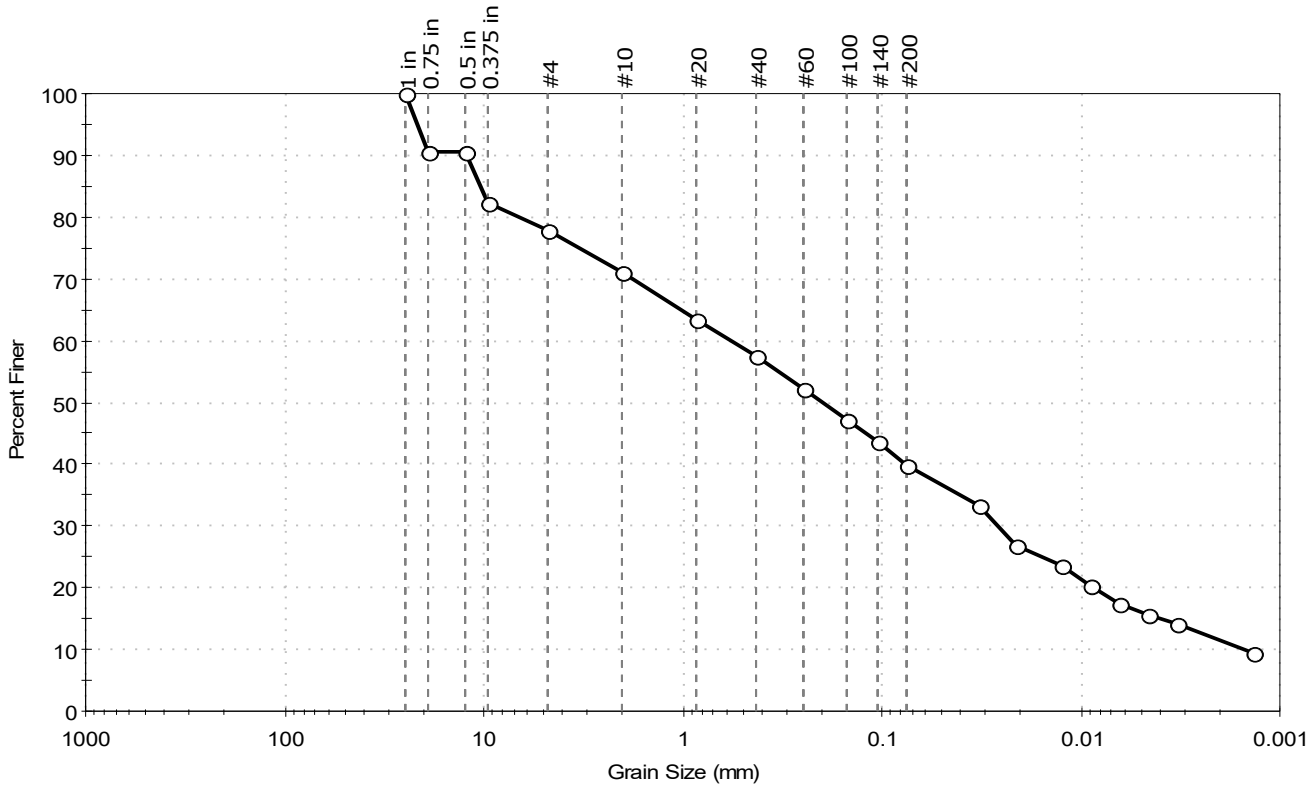
Sample/Test Description

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

GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-108	Sample Type:	jar	Tested By:	ckg
Sample ID:	6D	Test Date:	05/27/22	Checked By:	bfs
Depth :	25-27 ft	Test Id:	669120		
Test Comment:	---				
Visual Description:	Moist, gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	22.2	37.9	39.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	90		
0.5 in	12.50	90		
0.375 in	9.50	82		
#4	4.75	78		
#10	2.00	71		
#20	0.85	64		
#40	0.42	57		
#60	0.25	52		
#100	0.15	47		
#140	0.11	44		
#200	0.075	40		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0322	33		
---	0.0211	27		
---	0.0124	24		
---	0.0089	20		
---	0.0064	17		
---	0.0046	16		
---	0.0033	14		
---	0.0014	9		

Coefficients

D_{85} = 10.4141 mm D_{30} = 0.0260 mm
D_{60} = 0.5690 mm D_{15} = 0.0040 mm
D_{50} = 0.1983 mm D_{10} = 0.0015 mm
C_u = 379.333 C_c = 0.792

Classification

ASTM N/A

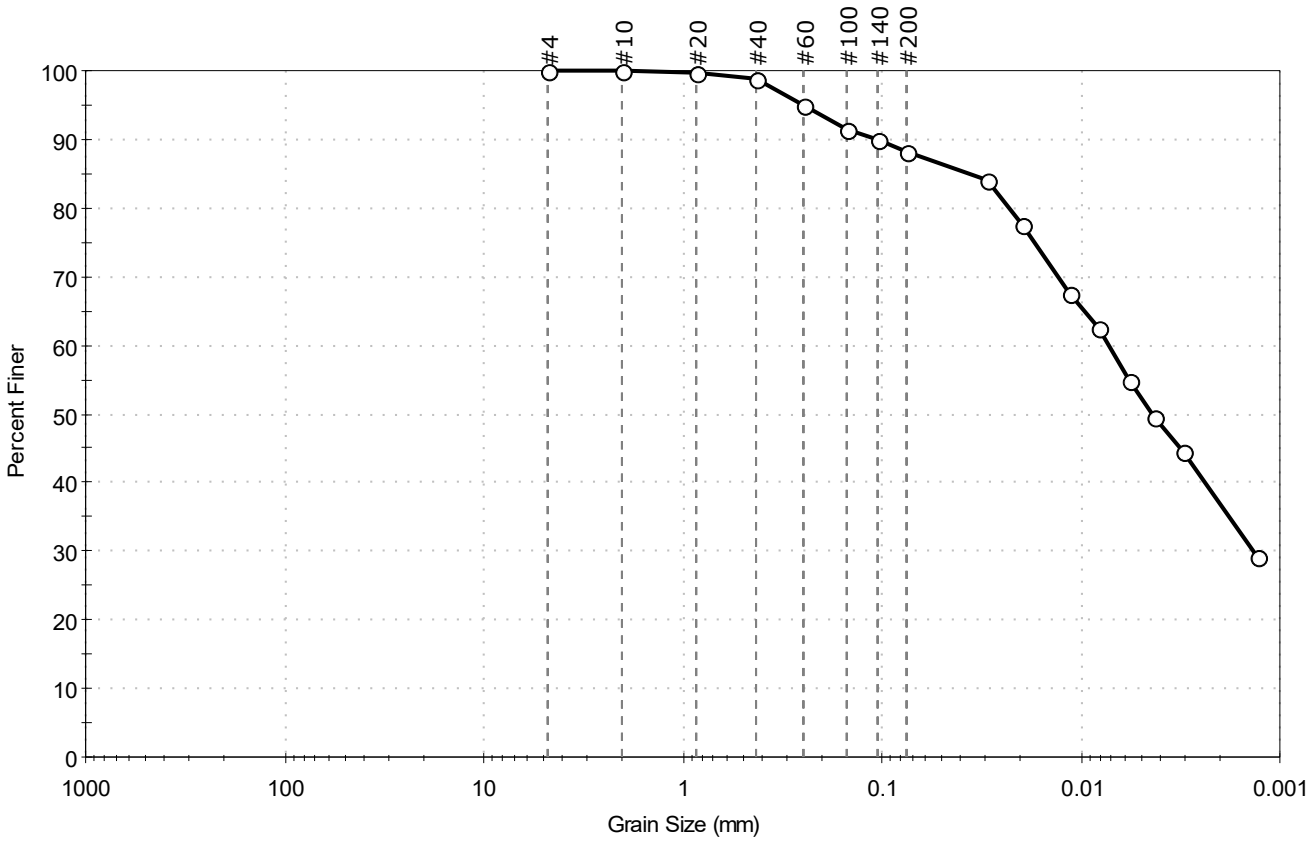
AASHTO Silty Soils (A-4 (0))

Sample/Test Description

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

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-109	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/27/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669121		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	11.7	88.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	99		
#60	0.25	95		
#100	0.15	92		
#140	0.11	90		
#200	0.075	88		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0296	84		
---	0.0195	78		
---	0.0113	68		
---	0.0082	62		
---	0.0057	55		
---	0.0043	50		
---	0.0031	45		
---	0.0013	29		

Coefficients

D_{85} = 0.0362 mm	D_{30} = 0.0013 mm
D_{60} = 0.0073 mm	D_{15} = N/A
D_{50} = 0.0043 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (16))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

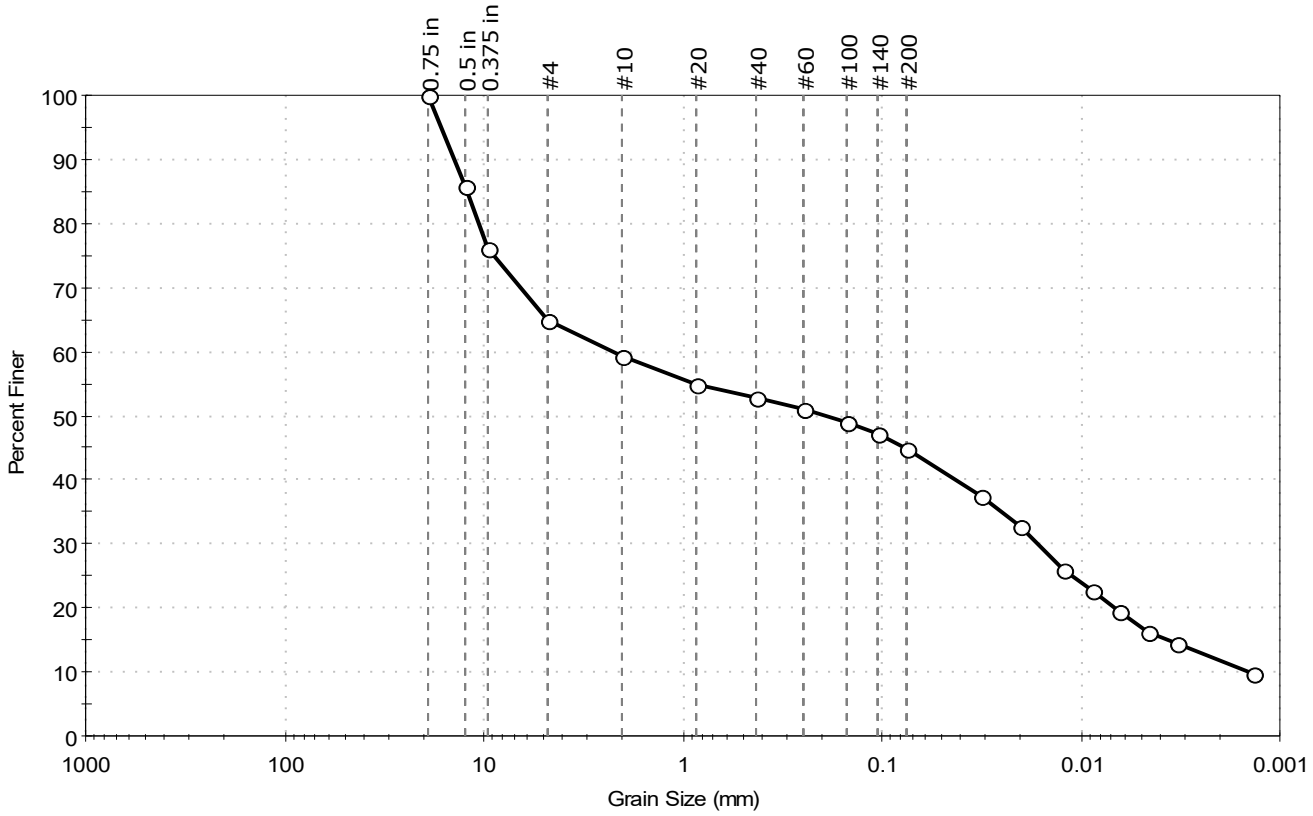
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-109	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/27/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669122		
Test Comment:	---				
Visual Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
–	35.0	20.2	44.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	86		
0.375 in	9.50	76		
#4	4.75	65		
#10	2.00	59		
#20	0.85	55		
#40	0.42	53		
#60	0.25	51		
#100	0.15	49		
#140	0.11	47		
#200	0.075	45		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0322	38		
---	0.0203	33		
---	0.0122	26		
---	0.0089	23		
---	0.0064	19		
---	0.0046	16		
---	0.0033	15		
---	0.0014	10		

Coefficients

D_{85} =12.2054 mm	D_{30} =0.0165 mm
D_{60} =2.2507 mm	D_{15} =0.0036 mm
D_{50} =0.1941 mm	D_{10} =0.0014 mm
C_u =1607.643	C_c =0.086

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

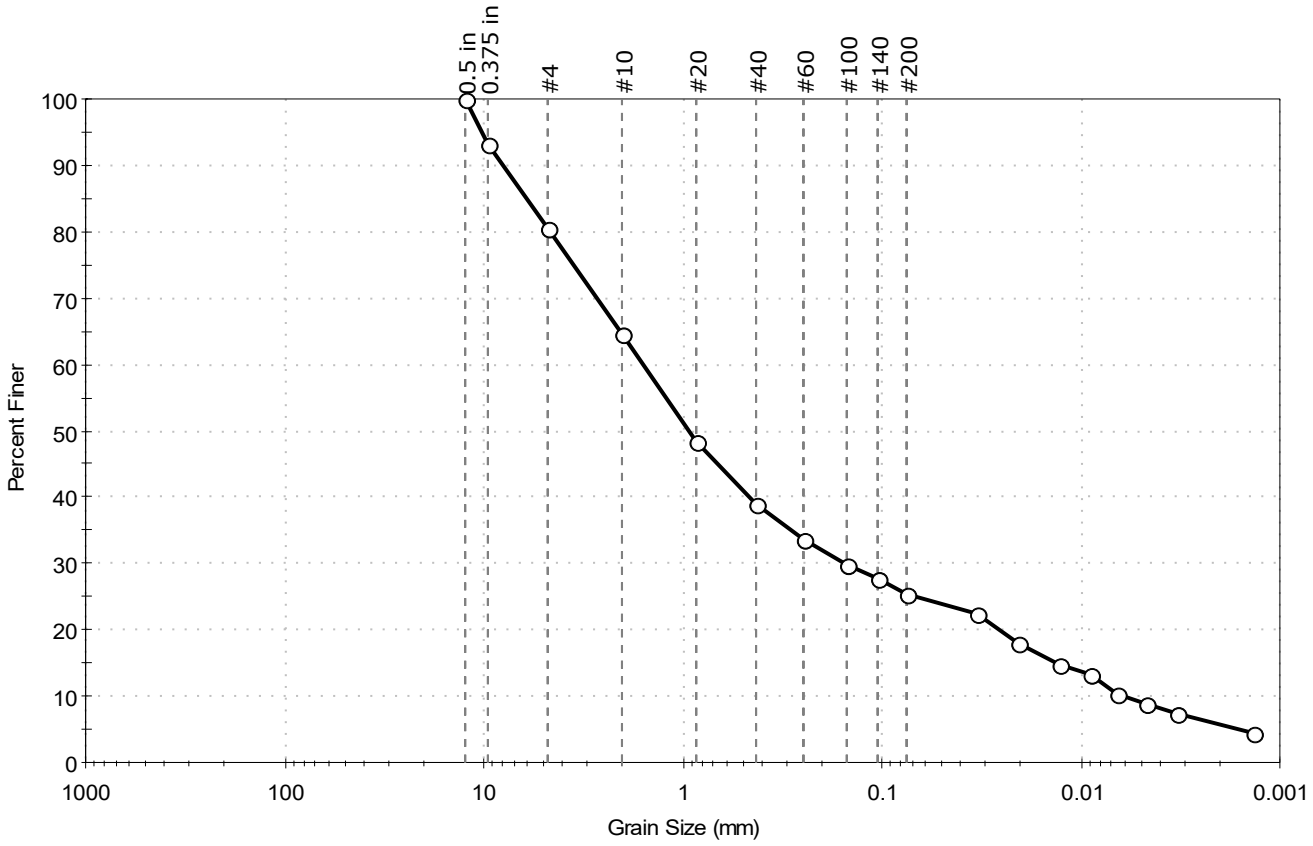
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-109	Sample Type:	jar	Tested By:	ckg
Sample ID:	6D	Test Date:	05/27/22	Checked By:	bfs
Depth :	27-29 ft	Test Id:	669123		
Test Comment:	---				
Visual Description:	Moist, gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	19.6	54.9	25.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	93		
#4	4.75	80		
#10	2.00	65		
#20	0.85	48		
#40	0.42	39		
#60	0.25	34		
#100	0.15	30		
#140	0.11	28		
#200	0.075	25		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0333	22		
---	0.0209	18		
---	0.0130	15		
---	0.0091	13		
---	0.0066	10		
---	0.0047	9		
---	0.0033	7		
---	0.0014	4		

Coefficients

D_{85} =6.0913 mm	D_{30} =0.1512 mm
D_{60} =1.5661 mm	D_{15} =0.0132 mm
D_{50} =0.9288 mm	D_{10} =0.0061 mm
C_u =256.738	C_c =2.393

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

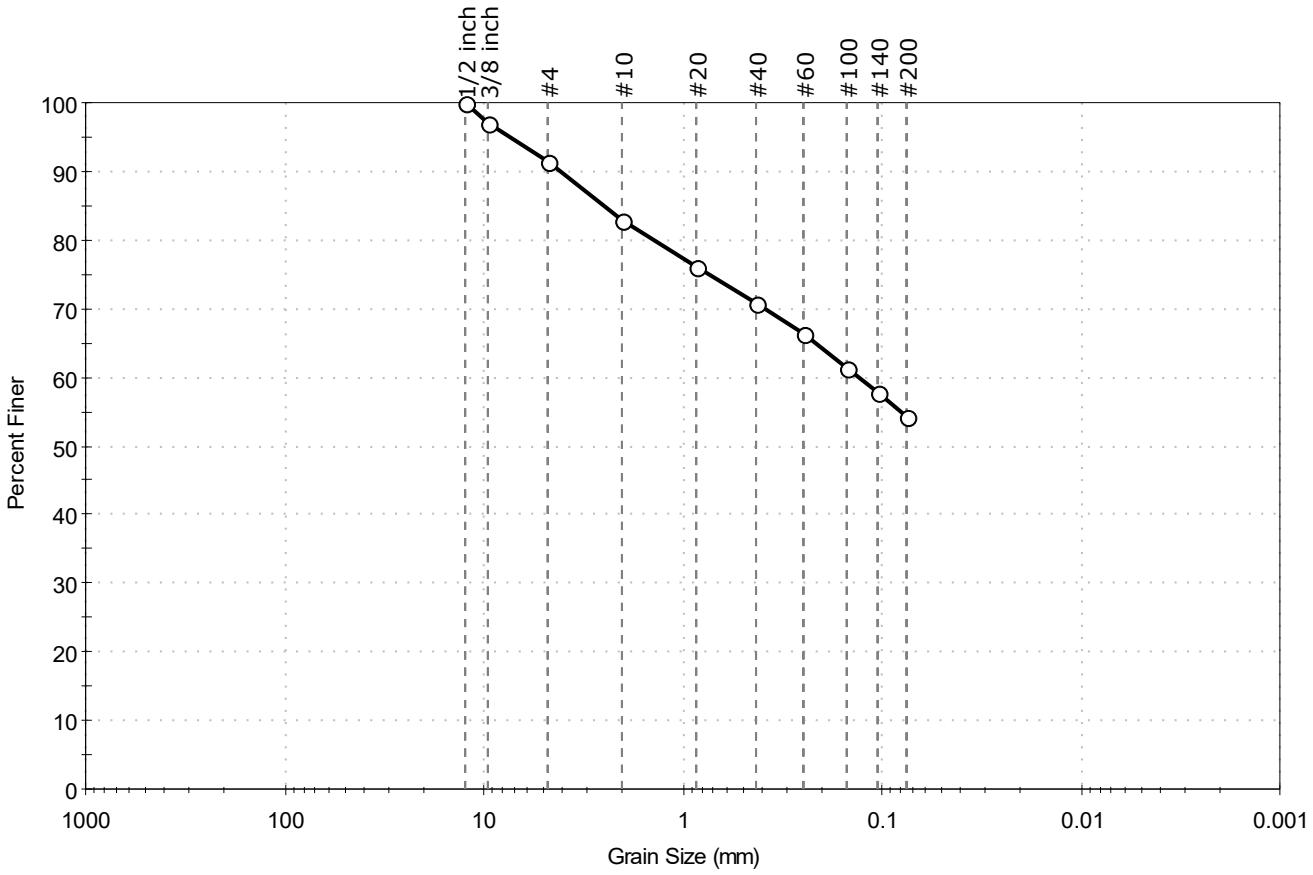
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-111	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/20/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669109		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	8.4	37.4	54.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.50	100		
3/8 inch	9.50	97		
#4	4.75	92		
#10	2.00	83		
#20	0.85	76		
#40	0.42	71		
#60	0.25	66		
#100	0.15	61		
#140	0.11	58		
#200	0.075	54		

Coefficients

D_{85} = 2.4698 mm	D_{30} = N/A
D_{60} = 0.1307 mm	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

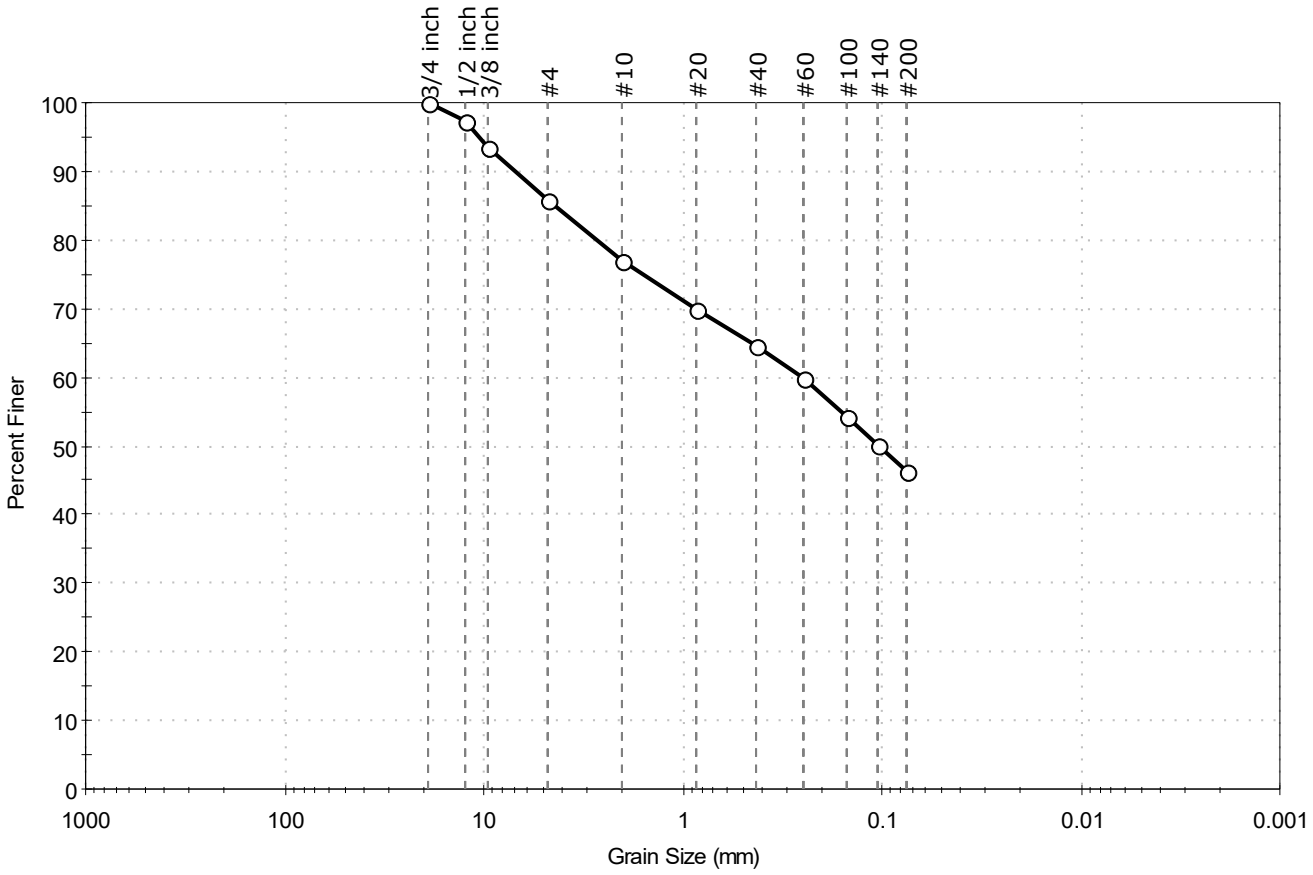
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-111	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/20/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669110		
Test Comment:	---				
Visual Description:	Moist, light gray silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.0	39.6	46.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	97		
3/8 inch	9.50	94		
#4	4.75	86		
#10	2.00	77		
#20	0.85	70		
#40	0.42	65		
#60	0.25	60		
#100	0.15	54		
#140	0.11	50		
#200	0.075	46		

Coefficients

D_{85} = 4.3228 mm D_{30} = N/A

D_{60} = 0.2563 mm D_{15} = N/A

D_{50} = 0.1059 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

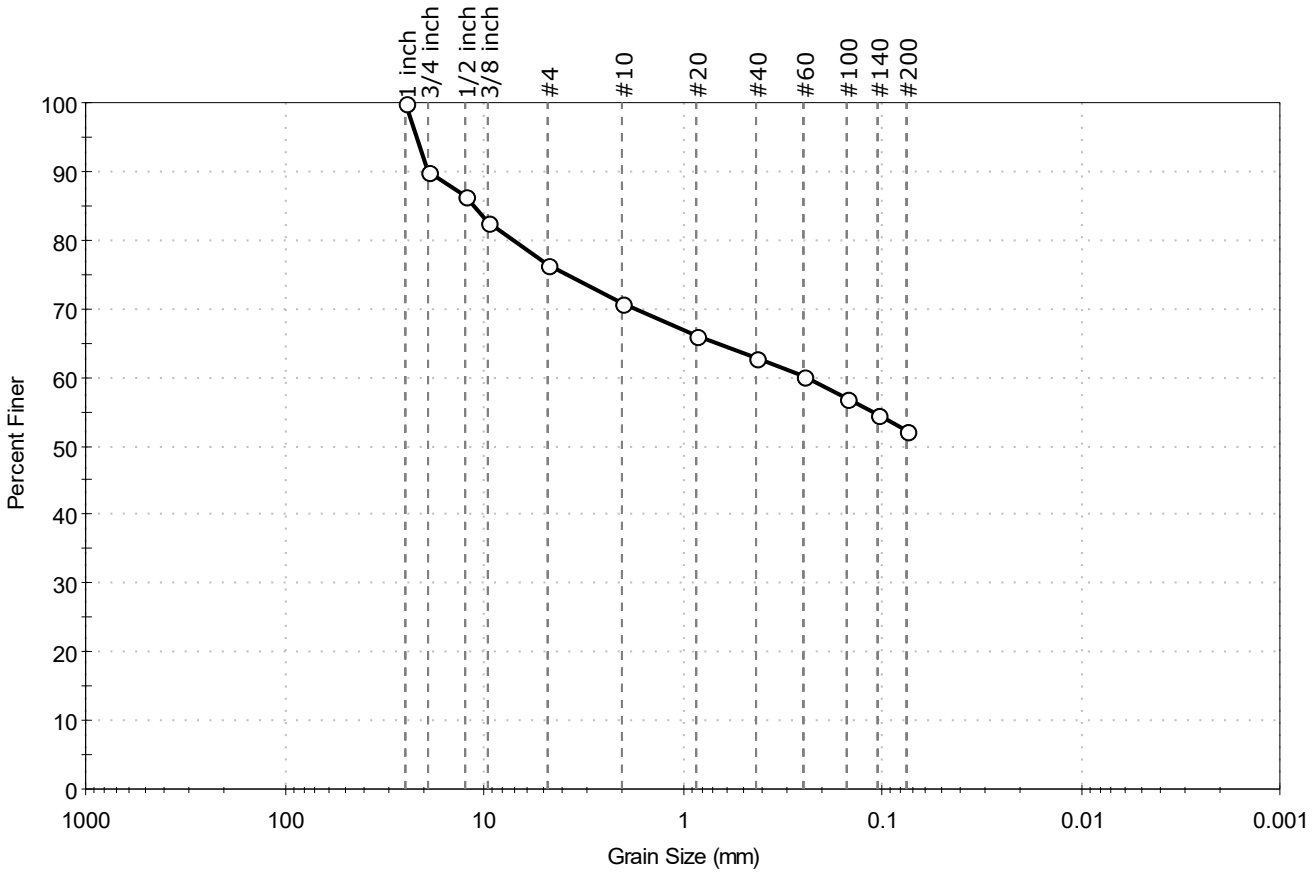
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-112	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/20/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669111		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy silt with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	23.7	24.1	52.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	90		
1/2 inch	12.50	86		
3/8 inch	9.50	83		
#4	4.75	76		
#10	2.00	71		
#20	0.85	66		
#40	0.42	63		
#60	0.25	60		
#100	0.15	57		
#140	0.11	55		
#200	0.075	52		

Coefficients

D_{85} = 11.2813 mm D_{30} = N/A

D_{60} = 0.2444 mm D_{15} = N/A

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

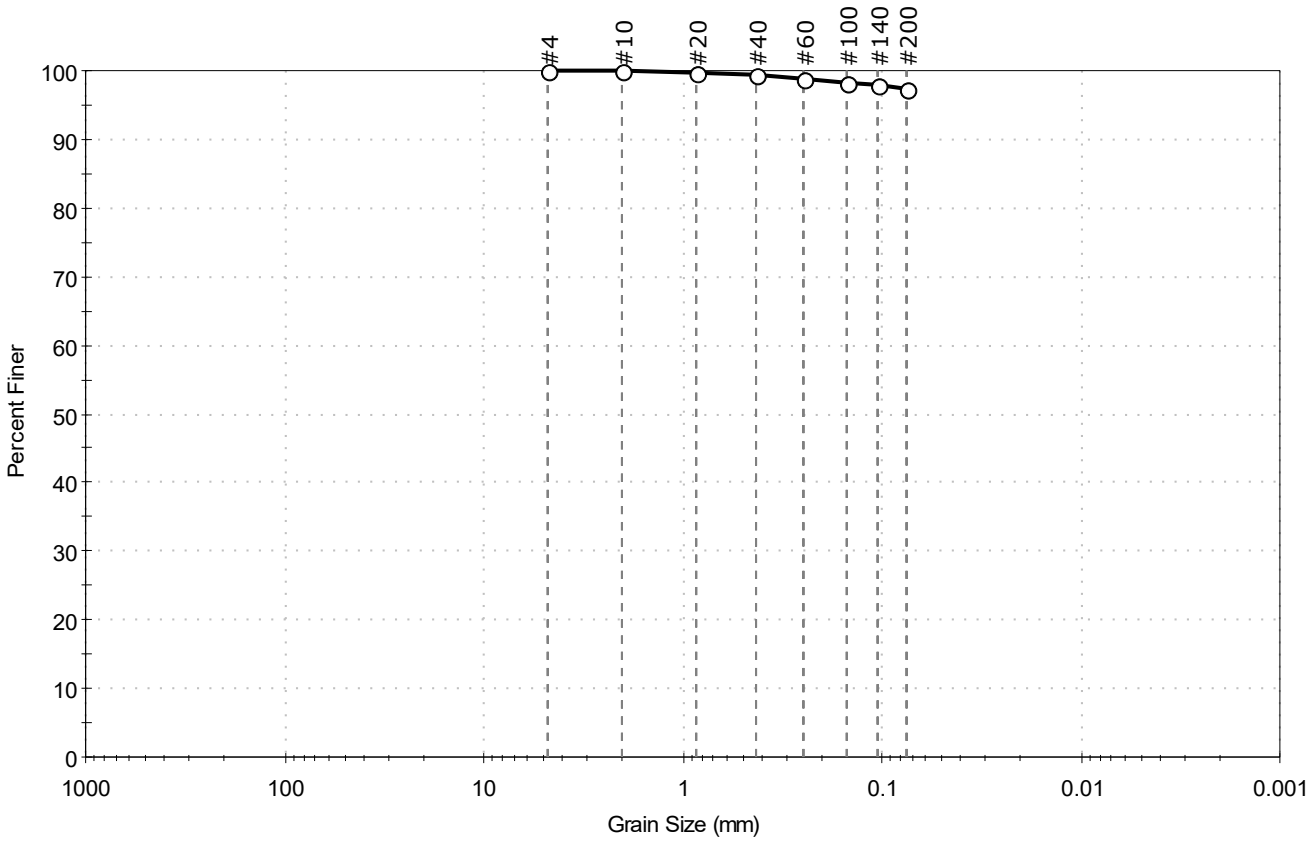
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-112	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/20/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669112		
Test Comment:	---				
Visual Description:	Moist, light brownish gray silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	2.7	97.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	99		
#60	0.25	99		
#100	0.15	98		
#140	0.11	98		
#200	0.075	97		

Coefficients

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

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

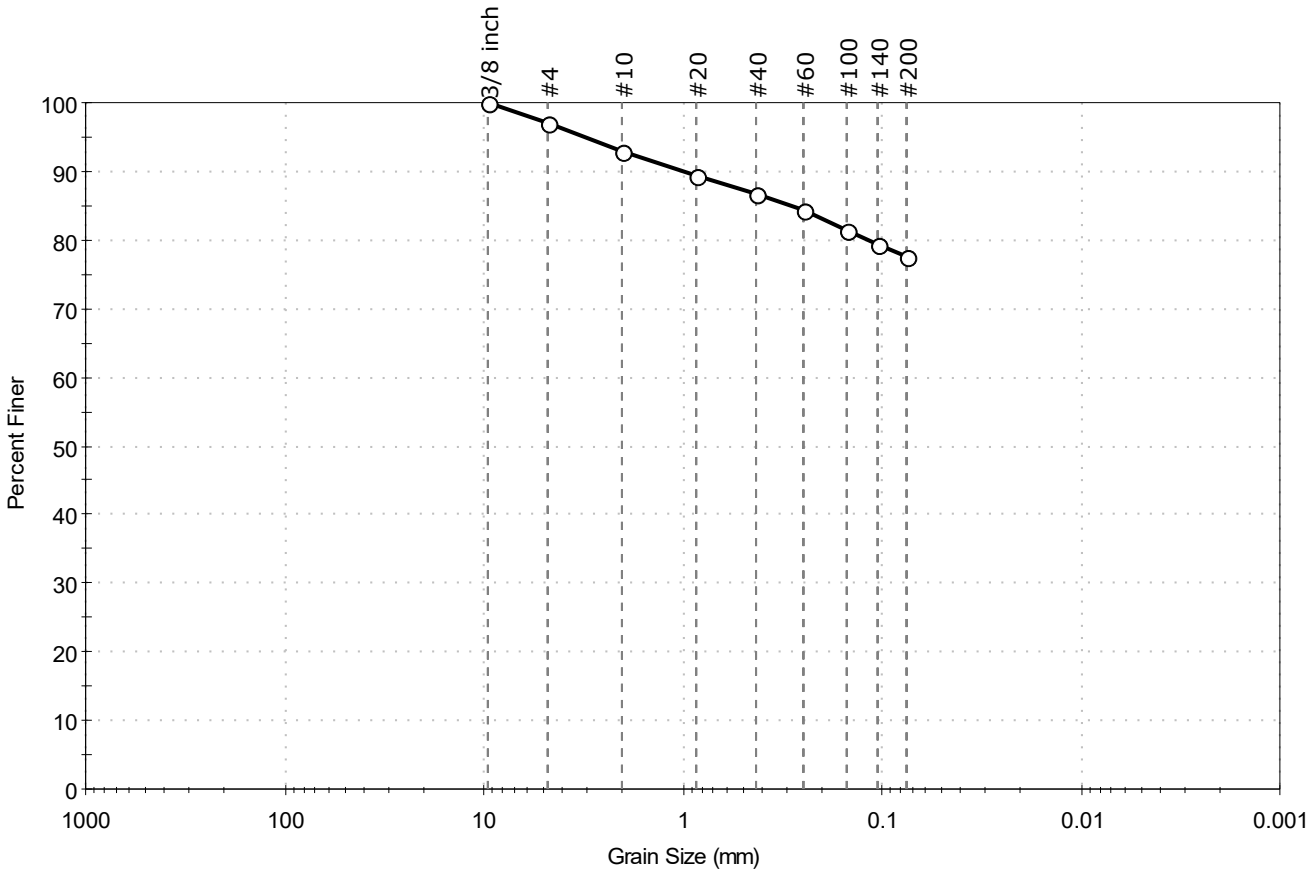
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	6D	Test Date:	05/20/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669099		
Test Comment:	---				
Visual Description:	Moist, light gray clay with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.0	19.3	77.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	97		
#10	2.00	93		
#20	0.85	90		
#40	0.42	87		
#60	0.25	84		
#100	0.15	81		
#140	0.11	79		
#200	0.075	78		

Coefficients

D_{85} = 0.2950 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY with Sand (CL)

AASHTO Silty Soils (A-4 (3))

Sample/Test Description

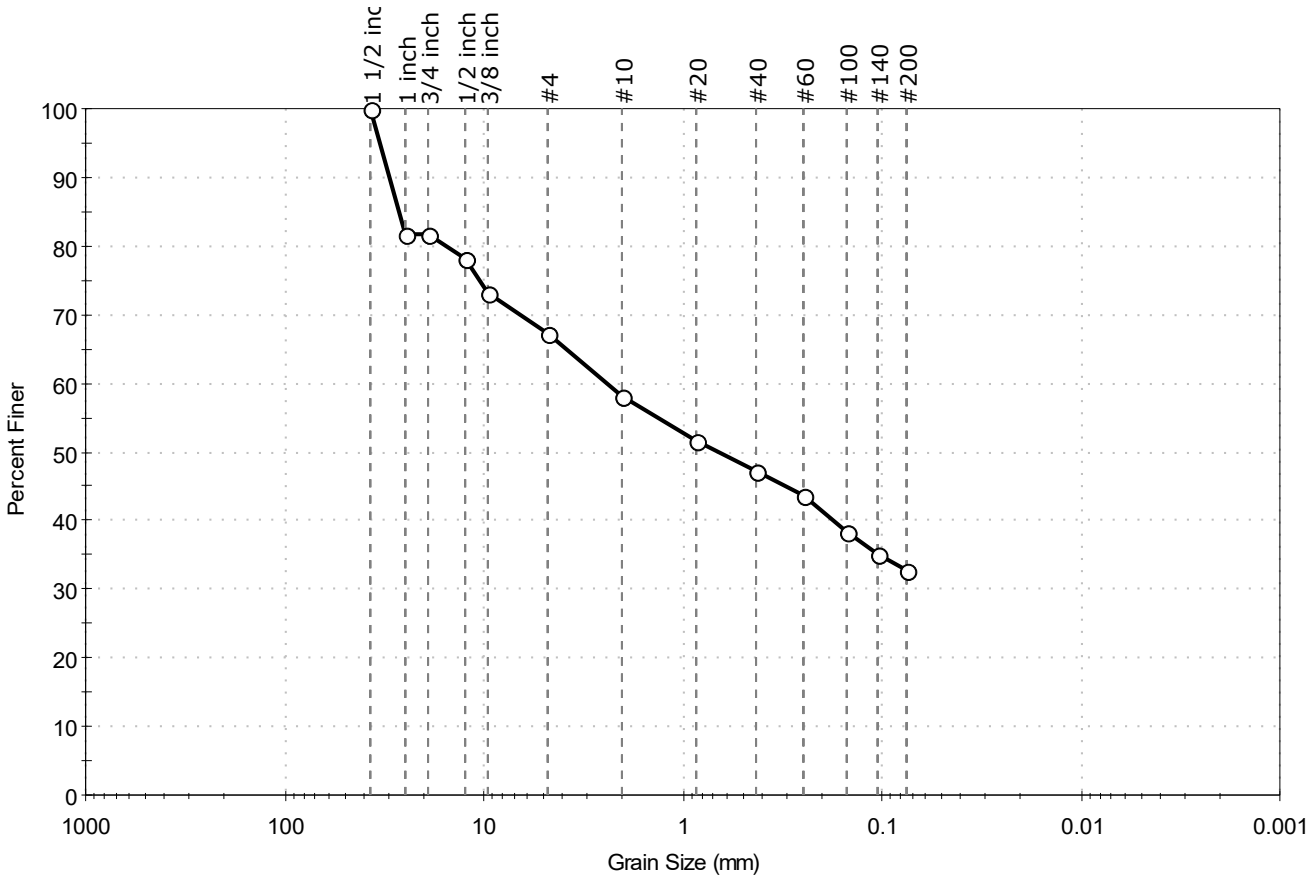
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	9D	Test Date:	05/20/22	Checked By:	bfs
Depth :	30-32 ft	Test Id:	669100		
Test Comment:	---				
Visual Description:	Moist, light olive brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	32.7	34.6	32.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	82		
3/4 inch	19.00	82		
1/2 inch	12.50	78		
3/8 inch	9.50	73		
#4	4.75	67		
#10	2.00	58		
#20	0.85	51		
#40	0.42	47		
#60	0.25	44		
#100	0.15	38		
#140	0.11	35		
#200	0.075	33		

Coefficients

D_{85} = 26.9110 mm D_{30} = N/A

D_{60} = 2.3952 mm D_{15} = N/A

D_{50} = 0.6662 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

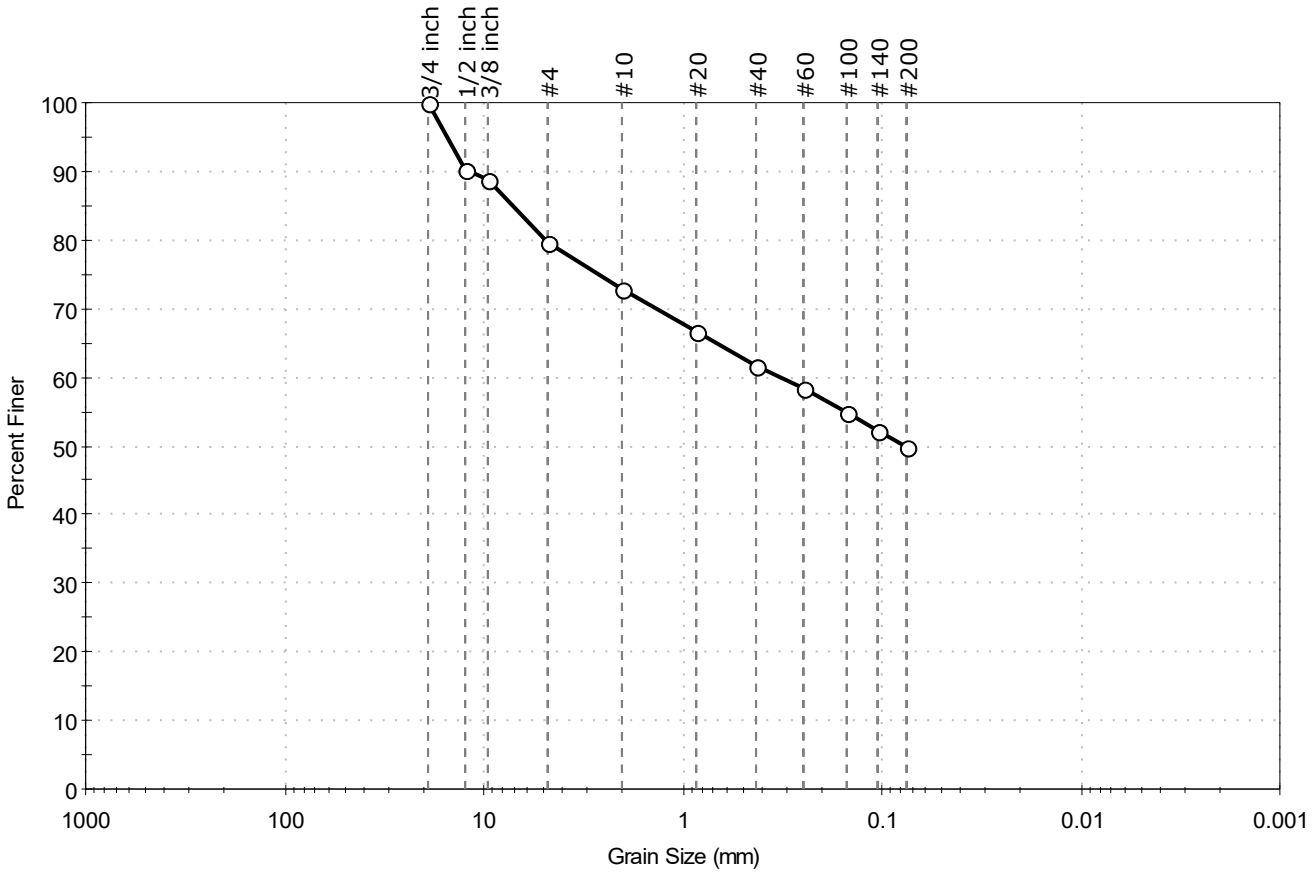
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	11D	Test Date:	05/22/22	Checked By:	bfs
Depth :	40-42 ft	Test Id:	669101		
Test Comment:	---				
Visual Description:	Moist, gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	20.4	29.8	49.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	90		
3/8 inch	9.50	89		
#4	4.75	80		
#10	2.00	73		
#20	0.85	67		
#40	0.42	62		
#60	0.25	58		
#100	0.15	55		
#140	0.11	52		
#200	0.075	50		

Coefficients

D_{85} = 7.1819 mm D_{30} = N/A

D_{60} = 0.3277 mm D_{15} = N/A

D_{50} = 0.0766 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

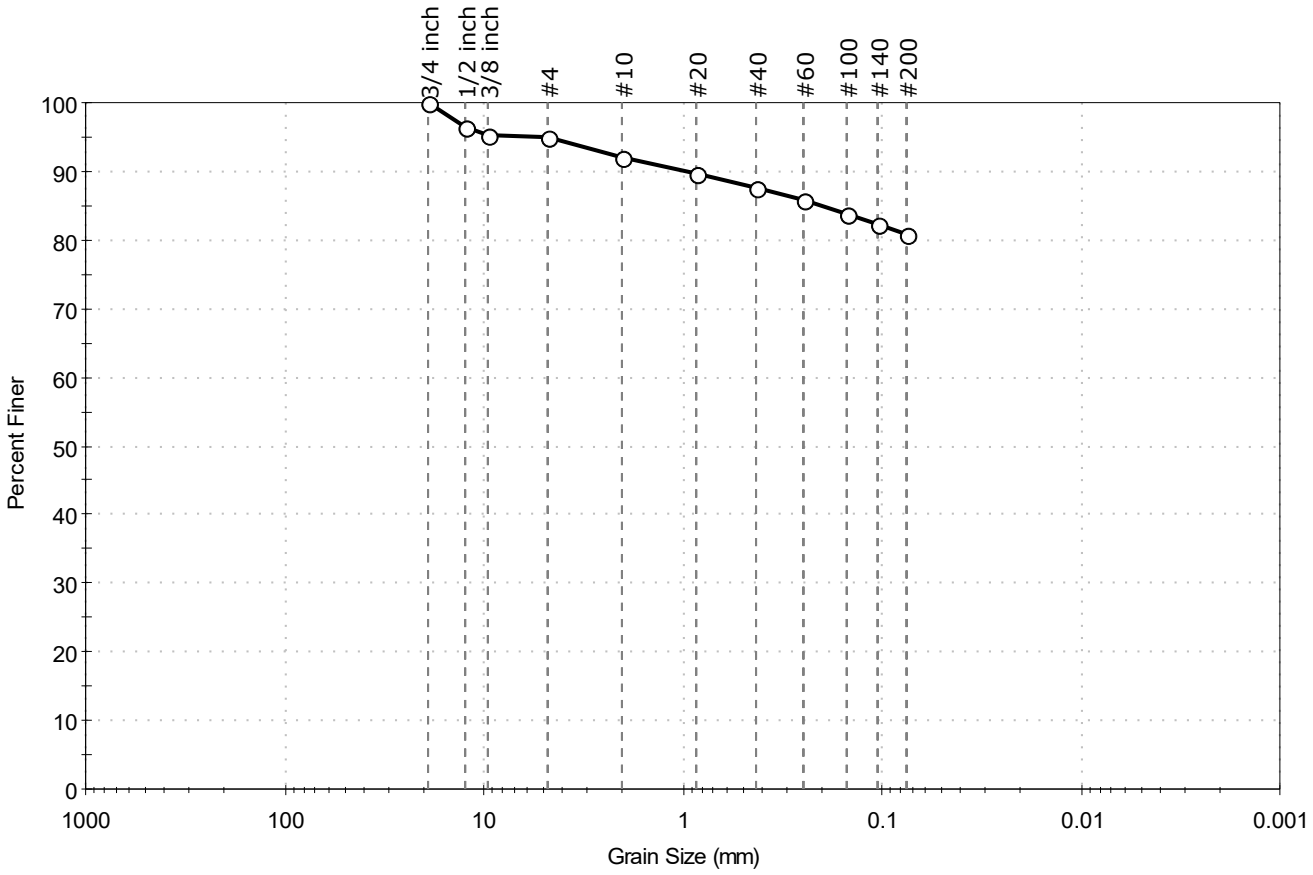
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/20/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669113		
Test Comment:	---				
Visual Description:	Moist, olive brown clay with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.1	14.2	80.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	96		
3/8 inch	9.50	95		
#4	4.75	95		
#10	2.00	92		
#20	0.85	90		
#40	0.42	88		
#60	0.25	86		
#100	0.15	84		
#140	0.11	82		
#200	0.075	81		

Coefficients

D_{85} = 0.2044 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY with Sand (CL)

AASHTO Clayey Soils (A-6 (11))

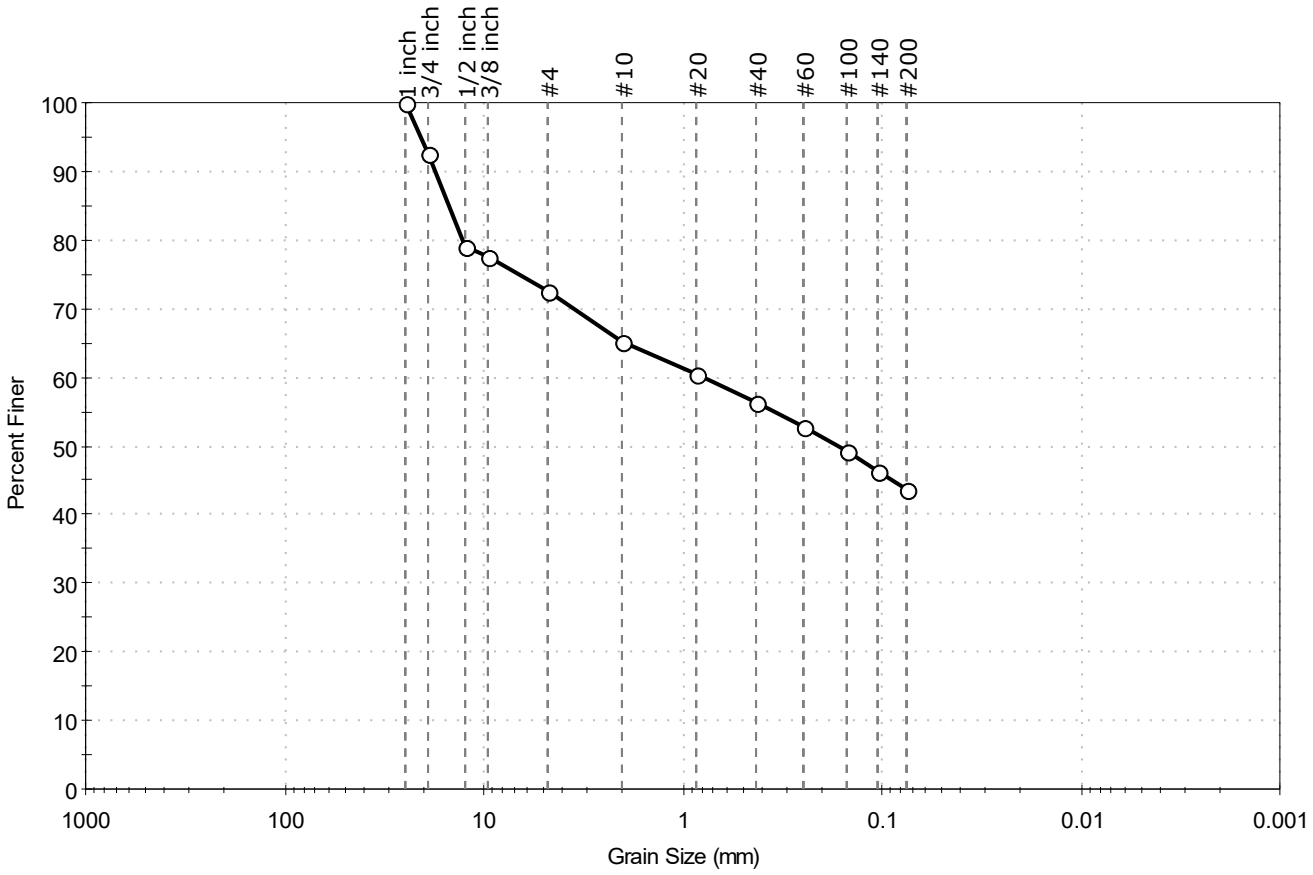
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	3D	Test Date:	05/20/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669114		
Test Comment:	---				
Visual Description:	Moist, olive silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	27.5	28.8	43.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	93		
1/2 inch	12.50	79		
3/8 inch	9.50	78		
#4	4.75	73		
#10	2.00	65		
#20	0.85	60		
#40	0.42	56		
#60	0.25	53		
#100	0.15	49		
#140	0.11	46		
#200	0.075	44		

Coefficients

D_{85} = 15.0150 mm D_{30} = N/A

D_{60} = 0.7955 mm D_{15} = N/A

D_{50} = 0.1685 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

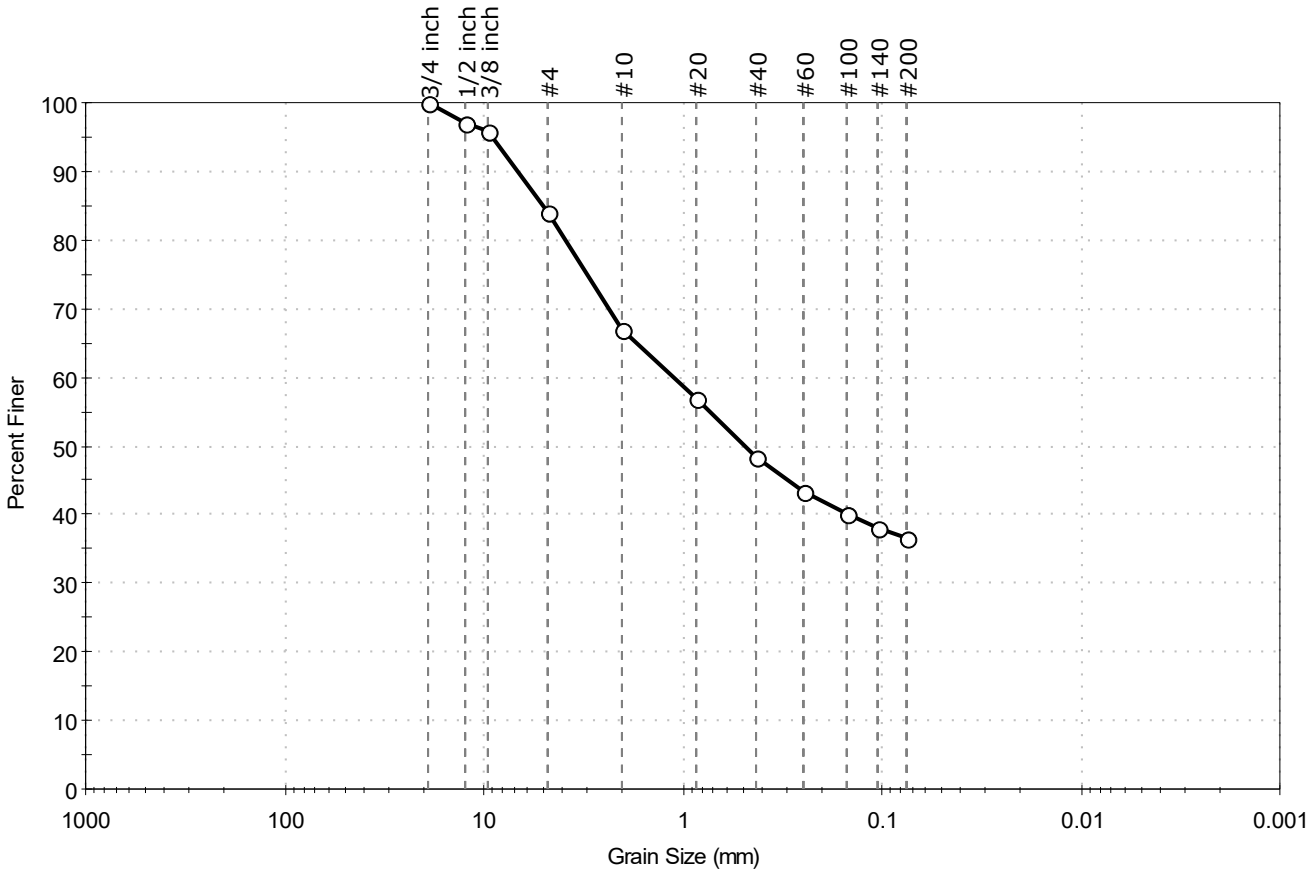
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/20/22	Checked By:	bfs
Depth :	17-19 ft	Test Id:	669115		
Test Comment:	---				
Visual Description:	Moist, olive silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	15.8	47.7	36.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	97		
3/8 inch	9.50	96		
#4	4.75	84		
#10	2.00	67		
#20	0.85	57		
#40	0.42	48		
#60	0.25	43		
#100	0.15	40		
#140	0.11	38		
#200	0.075	36		

Coefficients

D_{85} = 5.0016 mm D_{30} = N/A

D_{60} = 1.0981 mm D_{15} = N/A

D_{50} = 0.4816 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

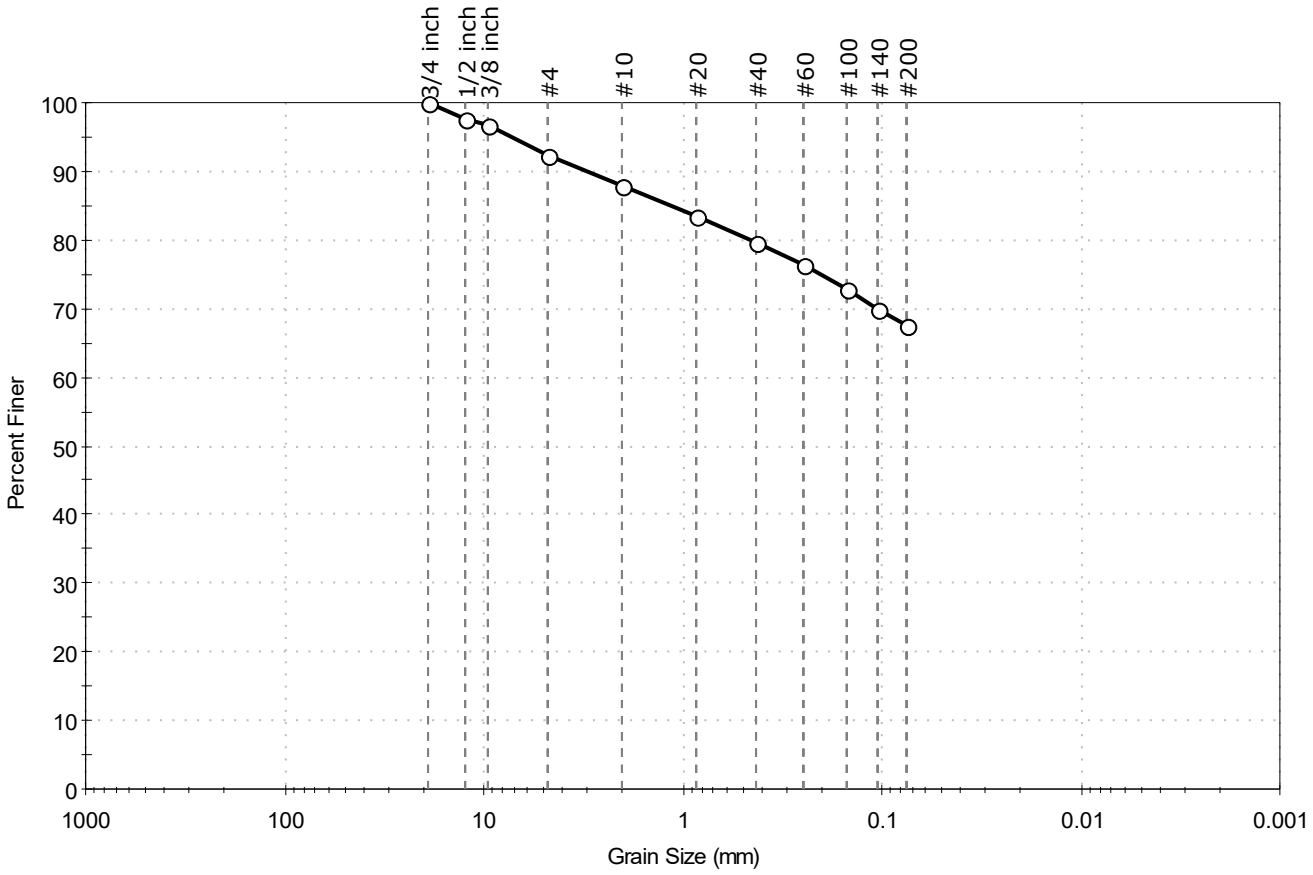
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/22/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669102		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	7.7	24.6	67.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	98		
3/8 inch	9.50	97		
#4	4.75	92		
#10	2.00	88		
#20	0.85	83		
#40	0.42	80		
#60	0.25	76		
#100	0.15	73		
#140	0.11	70		
#200	0.075	68		

Coefficients

D_{85} = 1.1395 mm D_{30} = N/A

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

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

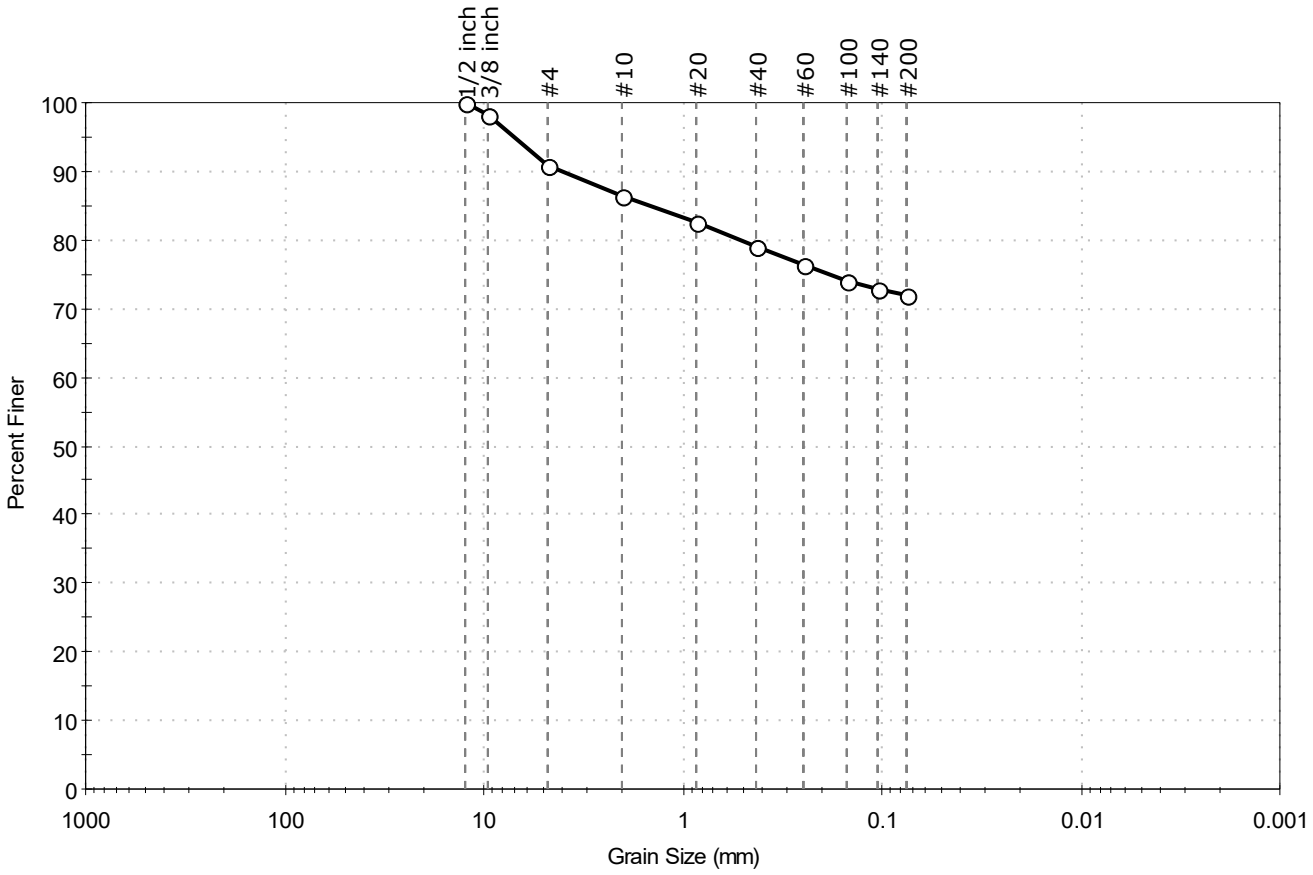
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	ckg
Sample ID:	5D	Test Date:	05/23/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669103		
Test Comment:	---				
Visual Description:	Moist, olive brown silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	9.3	18.9	71.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.50	100		
3/8 inch	9.50	98		
#4	4.75	91		
#10	2.00	86		
#20	0.85	82		
#40	0.42	79		
#60	0.25	76		
#100	0.15	74		
#140	0.11	73		
#200	0.075	72		

Coefficients

D_{85} = 1.4682 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

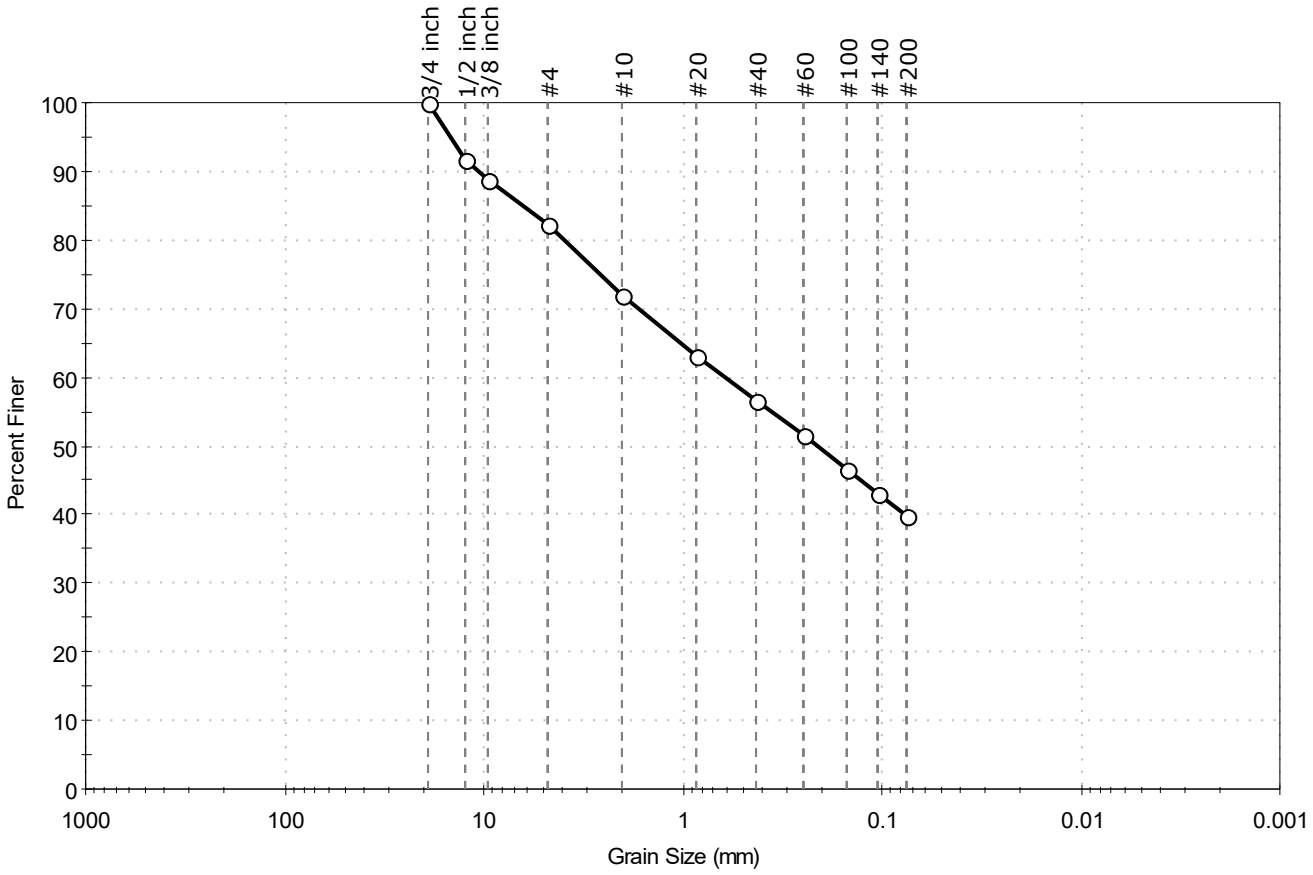
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	ckg
Sample ID:	7D	Test Date:	05/22/22	Checked By:	bfs
Depth :	30-32 ft	Test Id:	669104		
Test Comment:	---				
Visual Description:	Moist, gray silty, clayey sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.8	42.5	39.7

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	89		
#4	4.75	82		
#10	2.00	72		
#20	0.85	63		
#40	0.42	57		
#60	0.25	52		
#100	0.15	47		
#140	0.11	43		
#200	0.075	40		

Coefficients

D_{85} = 6.4158 mm D_{30} = N/A

D_{60} = 0.6080 mm D_{15} = N/A

D_{50} = 0.2102 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM: Silty, Clayey SAND with Gravel (SC-SM)

AASHTO: Silty Soils (A-4 (0))

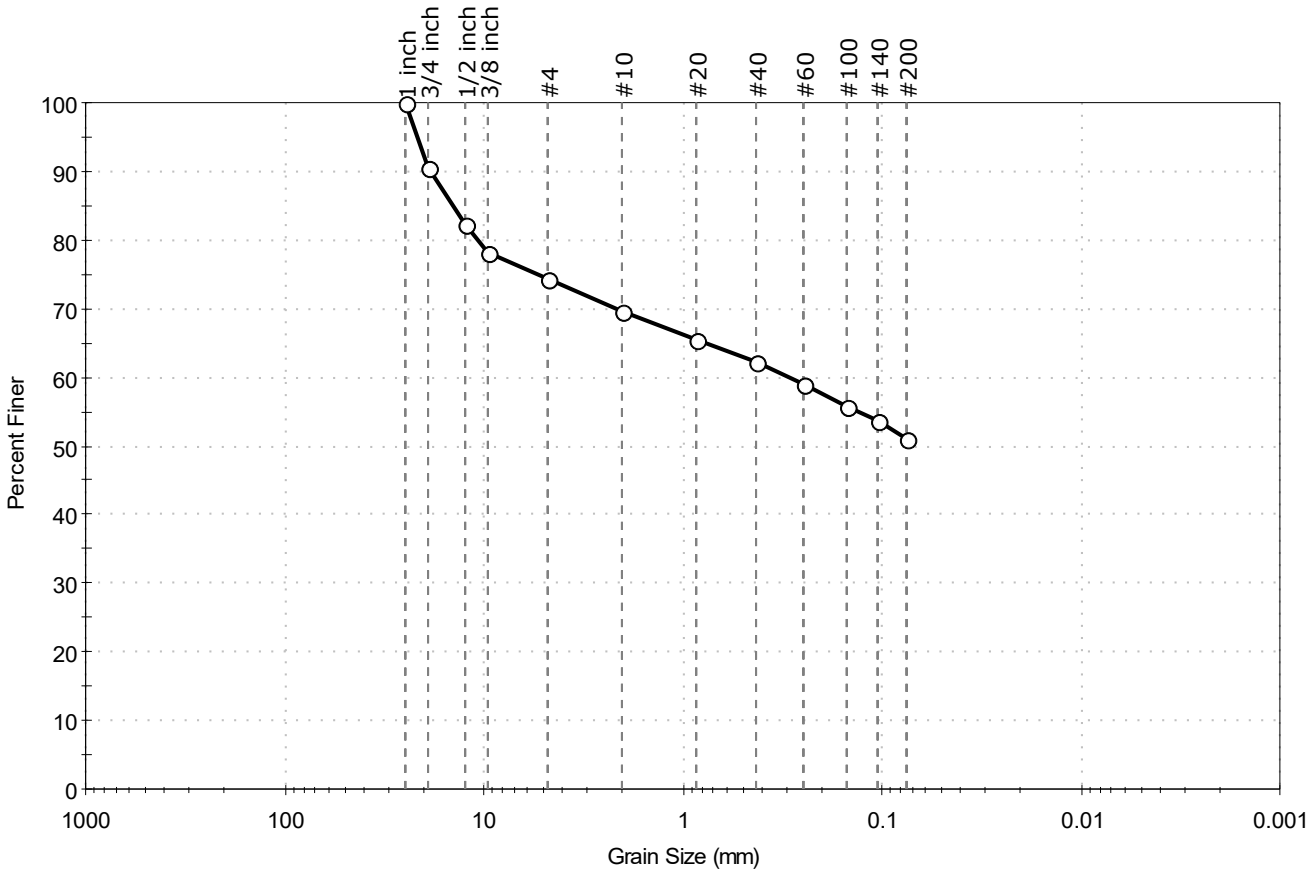
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	ckg
Sample ID:	10D	Test Date:	05/23/22	Checked By:	bfs
Depth :	45-47 ft	Test Id:	669105		
Test Comment:	---				
Visual Description:	Moist, light gray gravelly silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.6	23.3	51.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	91		
1/2 inch	12.50	82		
3/8 inch	9.50	78		
#4	4.75	74		
#10	2.00	70		
#20	0.85	66		
#40	0.42	62		
#60	0.25	59		
#100	0.15	56		
#140	0.11	54		
#200	0.075	51		

Coefficients

D_{85} = 14.2566 mm D_{30} = N/A

D_{60} = 0.2925 mm D_{15} = N/A

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

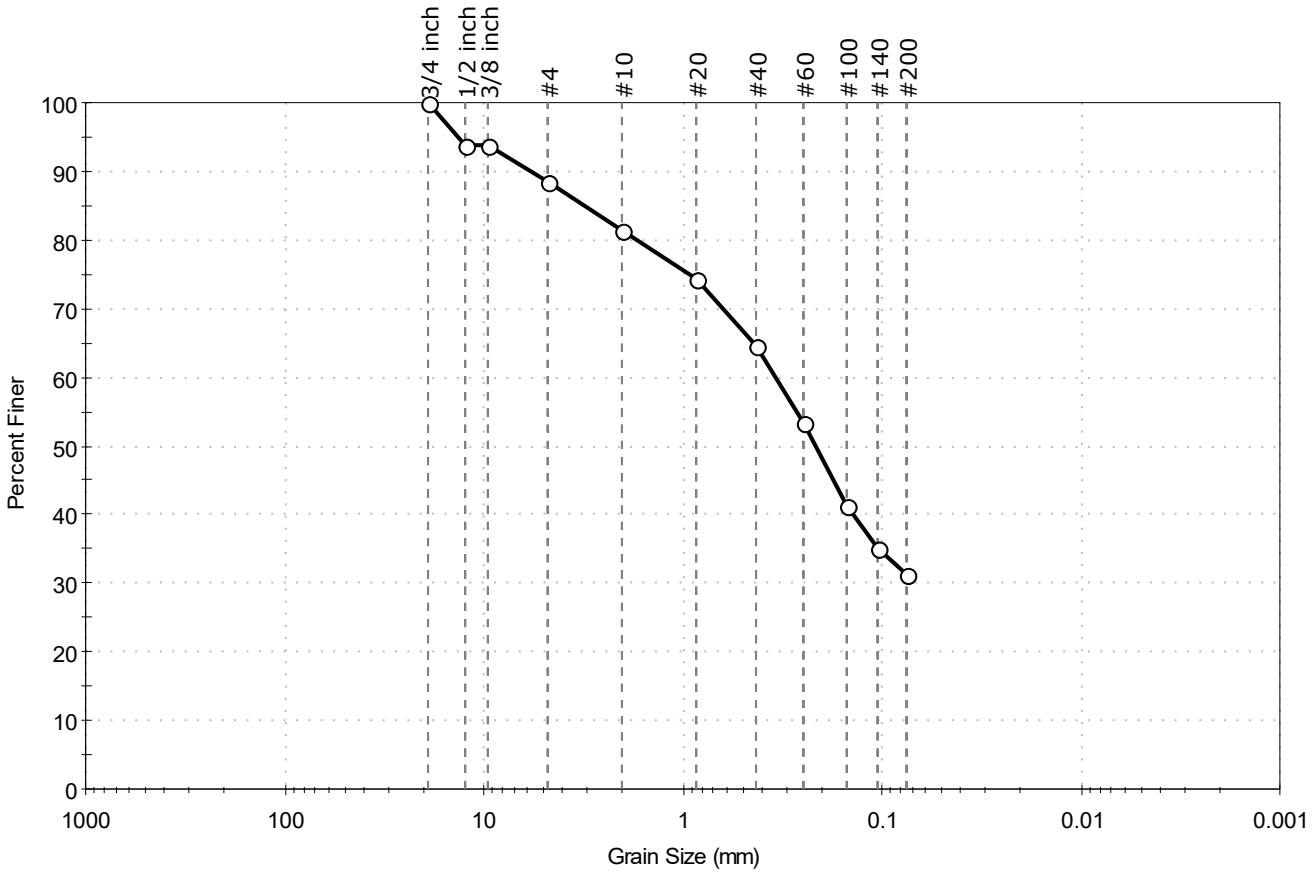
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	ckg
Sample ID:	12D	Test Date:	05/20/22	Checked By:	bfs
Depth :	55-57 ft	Test Id:	669106		
Test Comment:	---				
Visual Description:	Moist, brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	11.6	57.0	31.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	94		
3/8 inch	9.50	94		
#4	4.75	88		
#10	2.00	82		
#20	0.85	74		
#40	0.42	65		
#60	0.25	53		
#100	0.15	41		
#140	0.11	35		
#200	0.075	31		

Coefficients

D_{85} = 3.0981 mm D_{30} = N/A

D_{60} = 0.3417 mm D_{15} = N/A

D_{50} = 0.2161 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

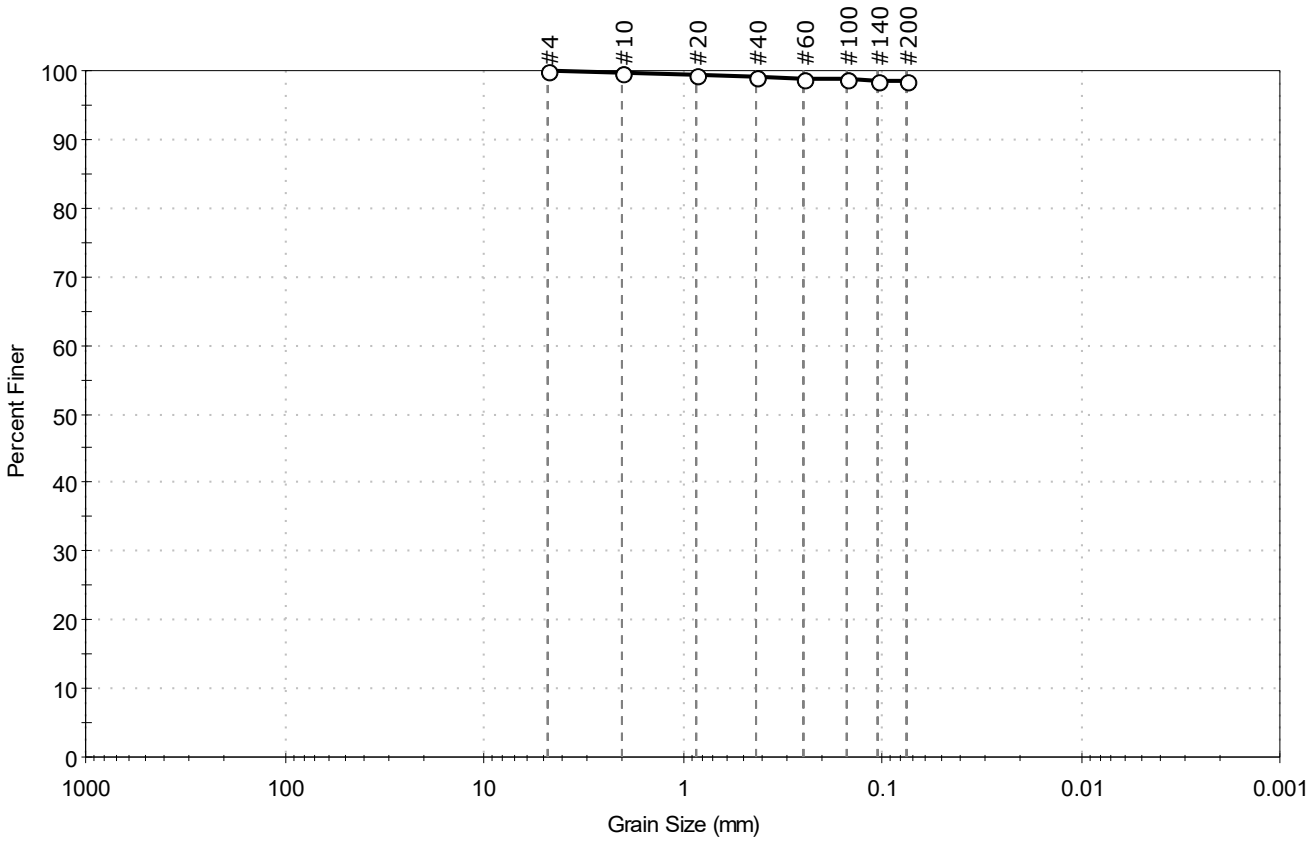
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/22/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669090		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	1.6	98.4

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	99		
#60	0.25	99		
#100	0.15	99		
#140	0.11	99		
#200	0.075	98		

Coefficients

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

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (16))

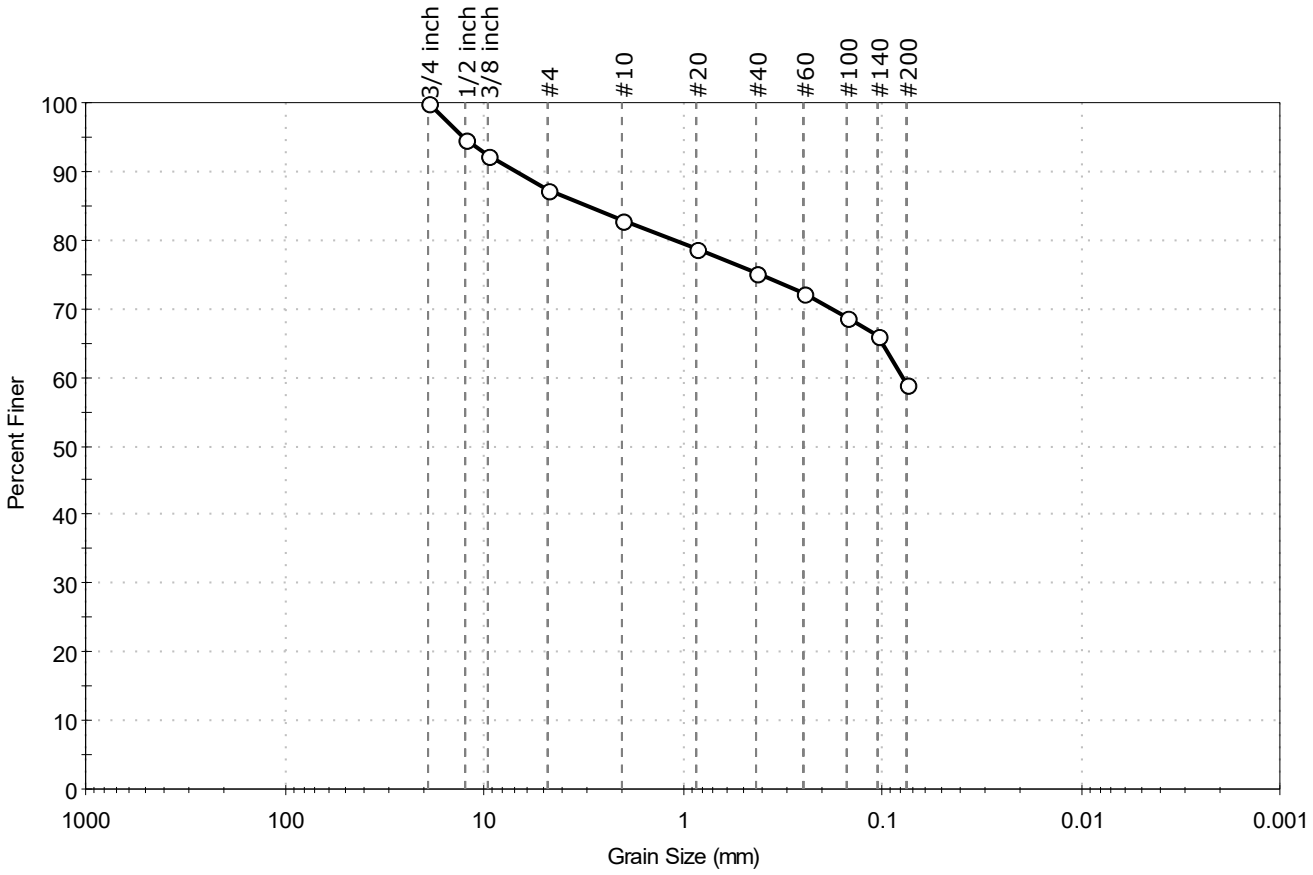
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	7D	Test Date:	05/20/22	Checked By:	bfs
Depth :	30-32 ft	Test Id:	669091		
Test Comment:	---				
Visual Description:	Moist, light gray sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.8	28.3	58.9

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	92		
#4	4.75	87		
#10	2.00	83		
#20	0.85	79		
#40	0.42	75		
#60	0.25	72		
#100	0.15	69		
#140	0.11	66		
#200	0.075	59		

Coefficients

D_{85} = 3.0588 mm D_{30} = N/A

D_{60} = 0.0789 mm D_{15} = N/A

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

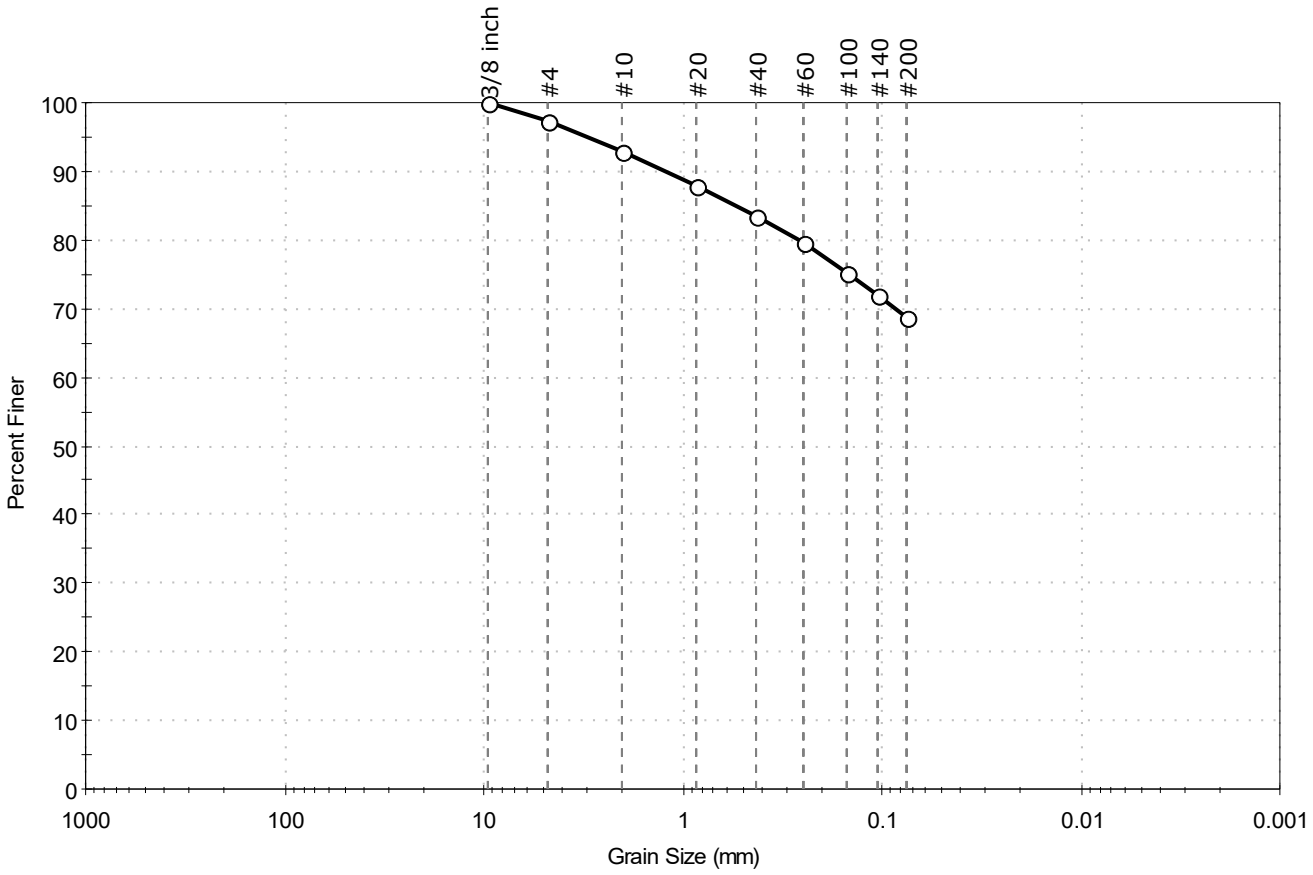
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	9D	Test Date:	05/22/22	Checked By:	bfs
Depth :	40-42 ft	Test Id:	669092		
Test Comment:	---				
Visual Description:	Moist, gray sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	2.6	28.7	68.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	97		
#10	2.00	93		
#20	0.85	88		
#40	0.42	84		
#60	0.25	80		
#100	0.15	75		
#140	0.11	72		
#200	0.075	69		

Coefficients

D_{85} = 0.5359 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

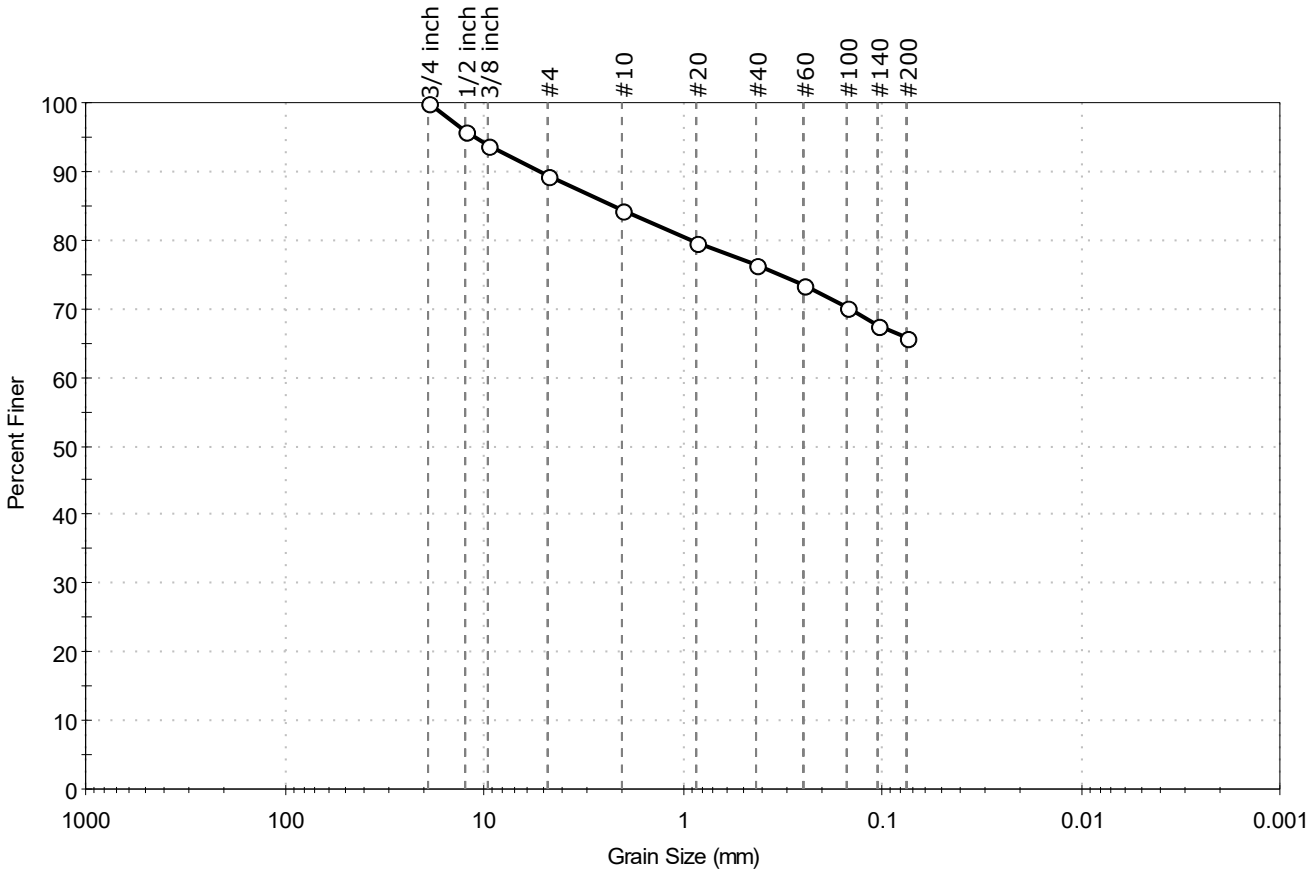
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	11D	Test Date:	05/20/22	Checked By:	bfs
Depth :	50-52 ft	Test Id:	669093		
Test Comment:	---				
Visual Description:	Moist, gray sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	10.7	23.6	65.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	96		
3/8 inch	9.50	94		
#4	4.75	89		
#10	2.00	84		
#20	0.85	80		
#40	0.42	76		
#60	0.25	74		
#100	0.15	70		
#140	0.11	68		
#200	0.075	66		

Coefficients

D_{85} = 2.2482 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

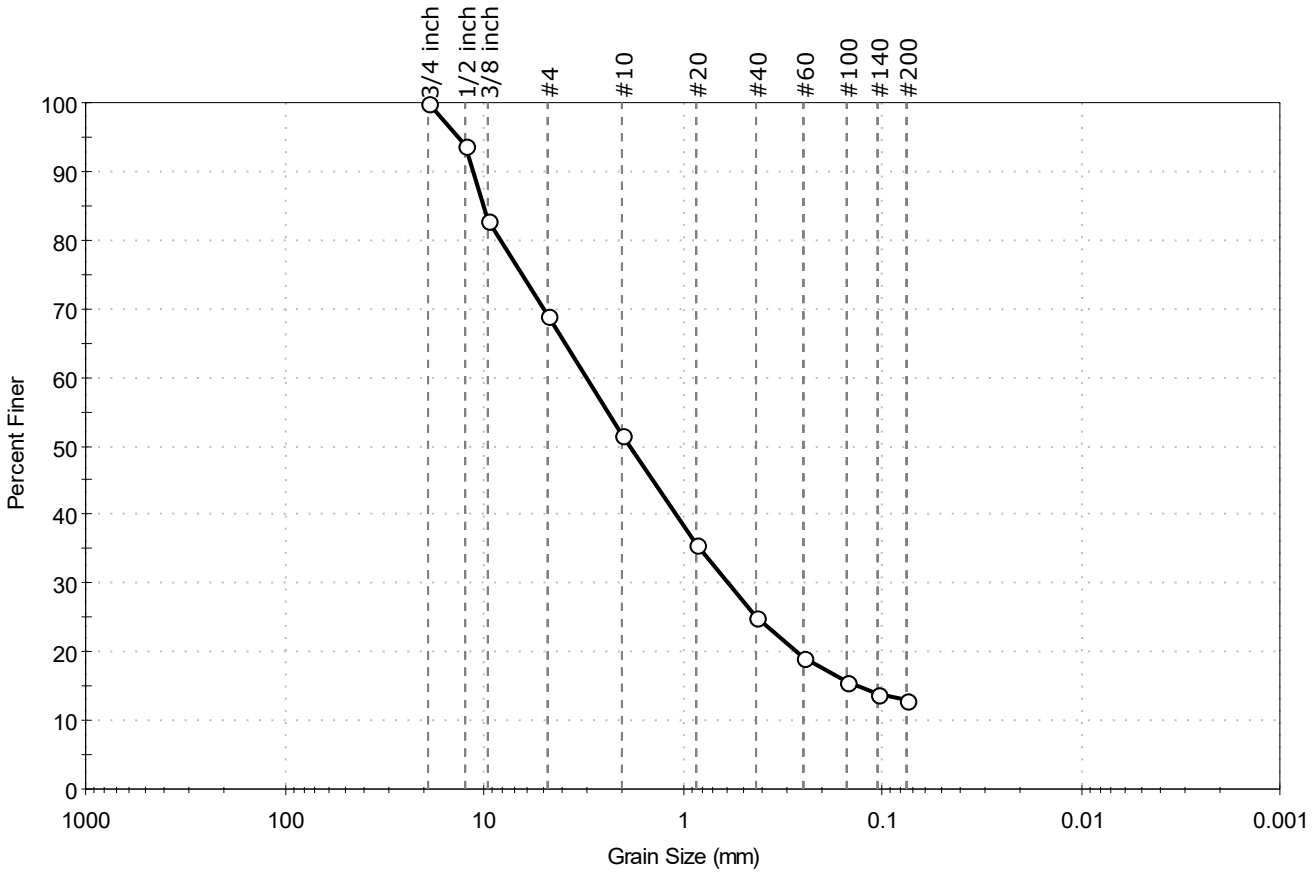
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	1D	Test Date:	05/20/22	Checked By:	bfs
Depth :	1-3 ft	Test Id:	669107		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	30.9	56.2	12.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	94		
3/8 inch	9.50	83		
#4	4.75	69		
#10	2.00	52		
#20	0.85	36		
#40	0.42	25		
#60	0.25	19		
#100	0.15	16		
#140	0.11	14		
#200	0.075	13		

Coefficients

D_{85} = 10.0324 mm
D_{30} = 0.5897 mm
D_{60} = 3.0288 mm
D_{15} = 0.1348 mm
D_{50} = 1.8326 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM N/A

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

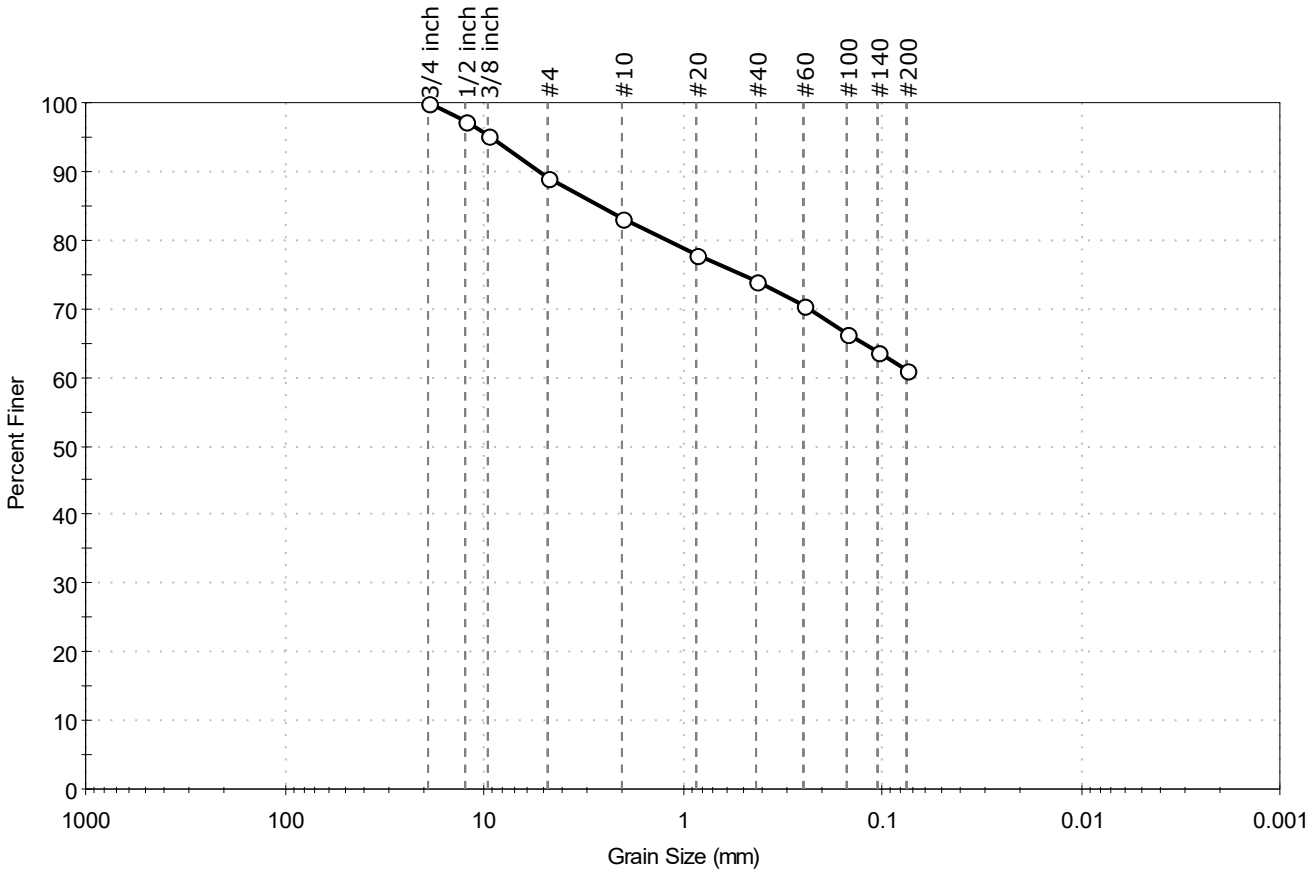
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/26/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669108		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	10.8	28.2	61.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	97		
3/8 inch	9.50	95		
#4	4.75	89		
#10	2.00	83		
#20	0.85	78		
#40	0.42	74		
#60	0.25	70		
#100	0.15	66		
#140	0.11	64		
#200	0.075	61		

Coefficients

D_{85} = 2.6121 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Sandy Lean CLAY (CL)

AASHTO Silty Soils (A-4 (2))

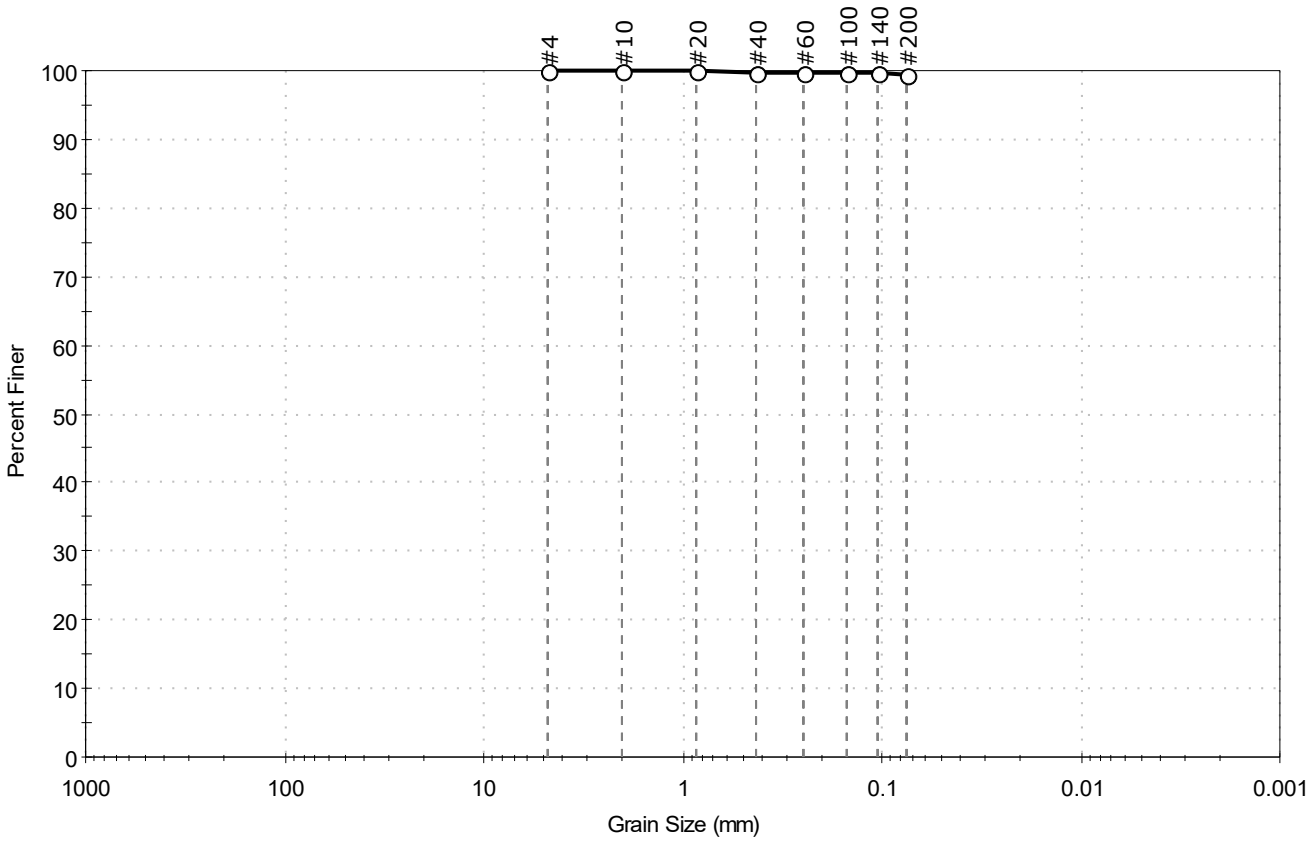
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-116	Sample Type:	jar	Tested By:	ckg
Sample ID:	2D	Test Date:	05/22/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669094		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.1	0.5	99.4

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

Coefficients

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

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

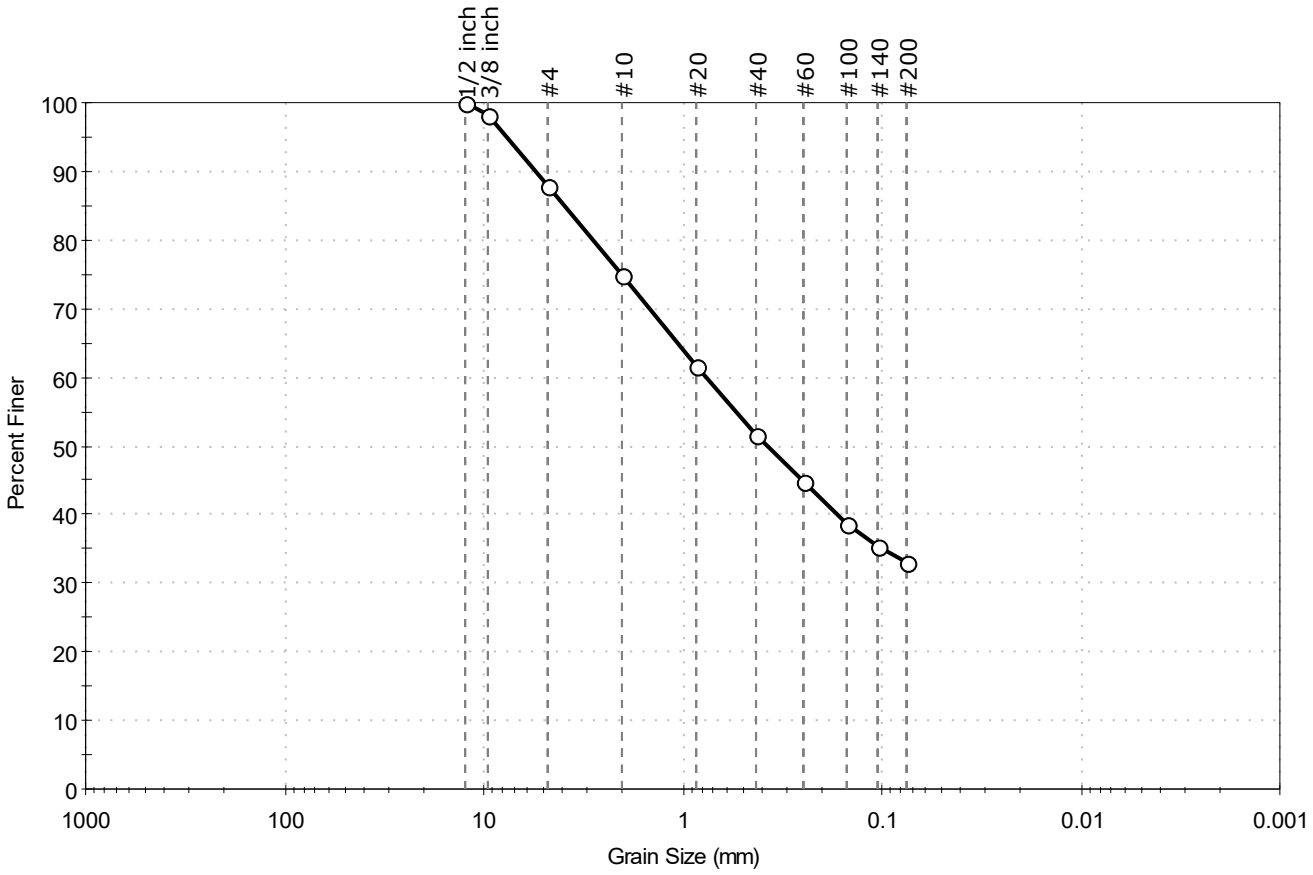
Sand/Gravel Hardness : ---

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GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-116	Sample Type:	jar	Tested By:	ckg
Sample ID:	4D	Test Date:	05/20/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669095		
Test Comment:	---				
Visual Description:	Moist, light olive brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	12.0	54.9	33.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.50	100		
3/8 inch	9.50	98		
#4	4.75	88		
#10	2.00	75		
#20	0.85	62		
#40	0.42	51		
#60	0.25	45		
#100	0.15	39		
#140	0.11	35		
#200	0.075	33		

Coefficients

D_{85} = 3.8920 mm D_{30} = N/A
D_{60} = 0.7633 mm D_{15} = N/A
D_{50} = 0.3784 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

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

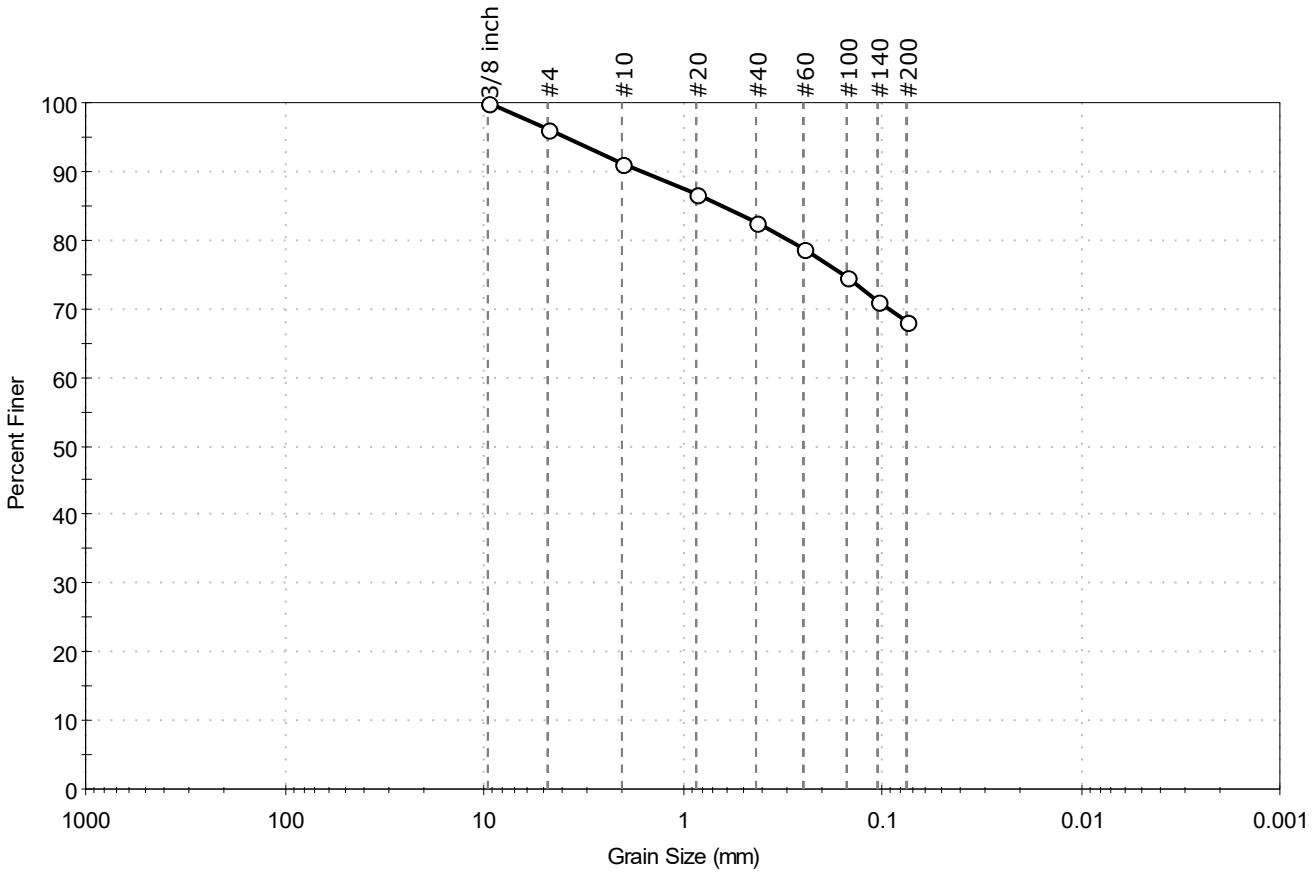
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-116	Sample Type:	jar	Tested By:	ckg
Sample ID:	7D	Test Date:	05/20/22	Checked By:	bfs
Depth :	30-32 ft	Test Id:	669096		
Test Comment:	---				
Visual Description:	Moist, gray sandy clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.8	28.2	68.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	96		
#10	2.00	91		
#20	0.85	87		
#40	0.42	83		
#60	0.25	79		
#100	0.15	75		
#140	0.11	71		
#200	0.075	68		

Coefficients

D_{85} = 0.6397 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

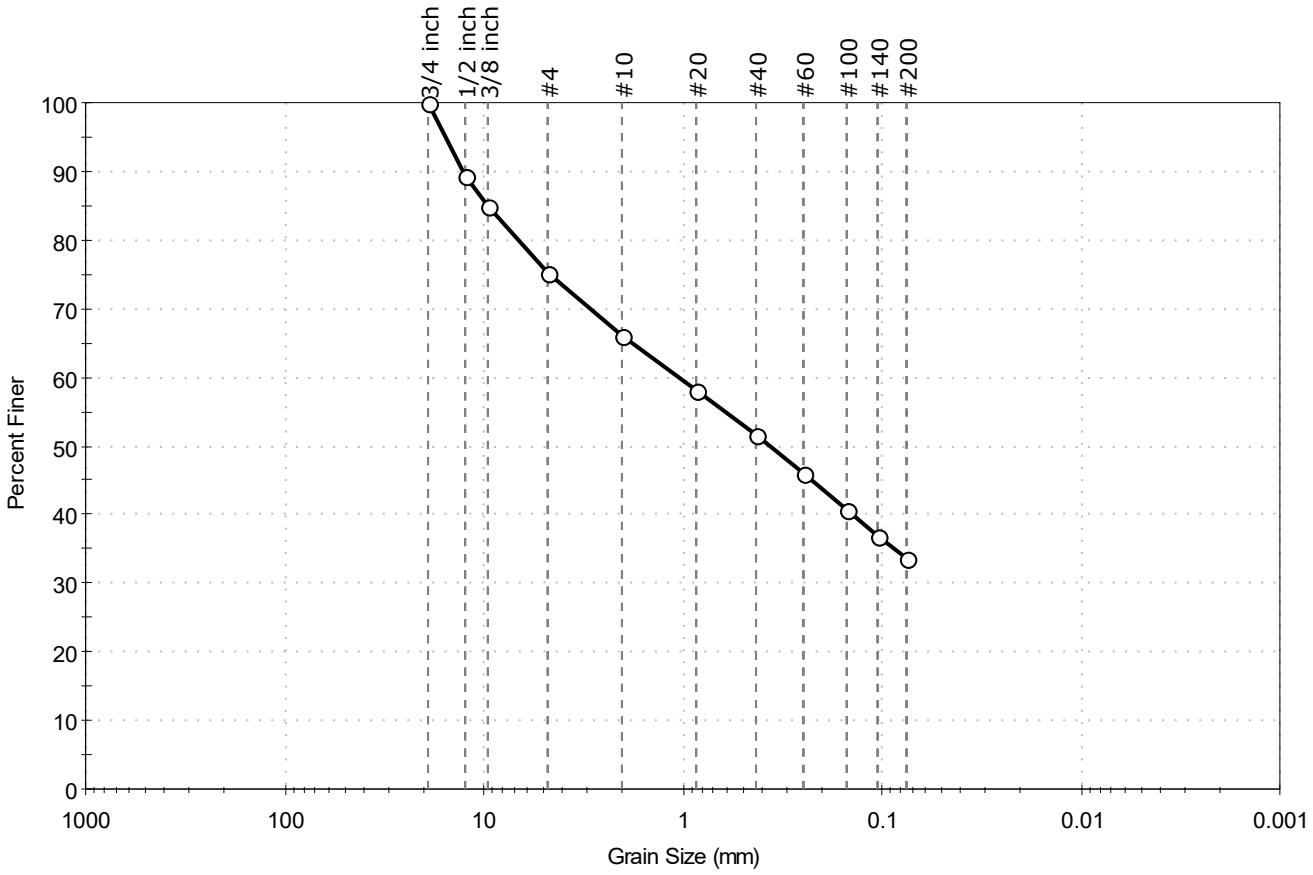
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-116	Sample Type:	jar	Tested By:	ckg
Sample ID:	11D	Test Date:	05/22/22	Checked By:	bfs
Depth :	50-52 ft	Test Id:	669097		
Test Comment:	---				
Visual Description:	Moist, light olive brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	24.8	41.6	33.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	89		
3/8 inch	9.50	85		
#4	4.75	75		
#10	2.00	66		
#20	0.85	58		
#40	0.42	52		
#60	0.25	46		
#100	0.15	41		
#140	0.11	37		
#200	0.075	34		

Coefficients

D_{85} = 9.5309 mm D_{30} = N/A

D_{60} = 1.0392 mm D_{15} = N/A

D_{50} = 0.3658 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

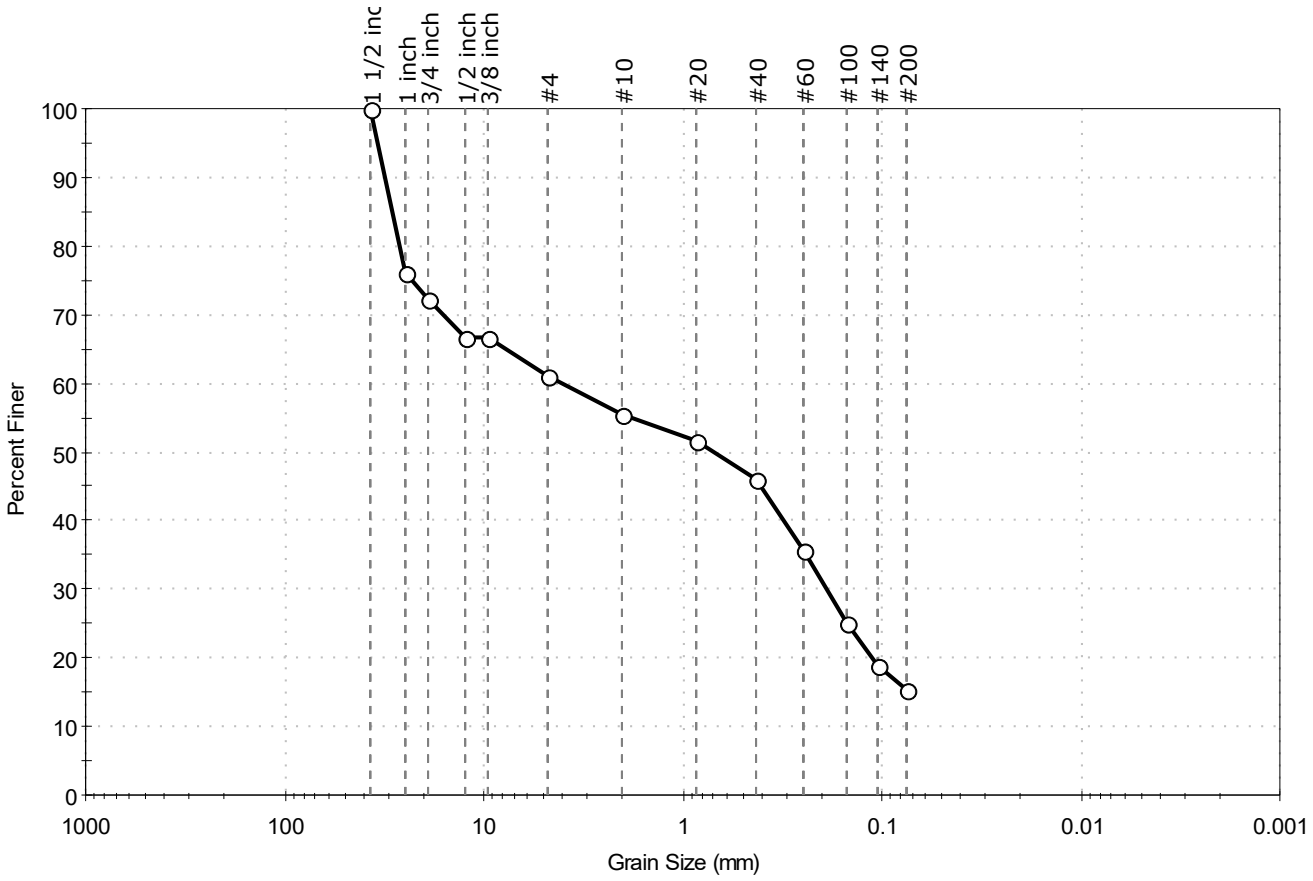
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-116	Sample Type:	jar	Tested By:	ckg
Sample ID:	14D	Test Date:	05/23/22	Checked By:	bfs
Depth :	72-74 ft	Test Id:	669098		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	39.0	45.6	15.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	76		
3/4 inch	19.00	72		
1/2 inch	12.50	67		
3/8 inch	9.50	67		
#4	4.75	61		
#10	2.00	55		
#20	0.85	51		
#40	0.42	46		
#60	0.25	36		
#100	0.15	25		
#140	0.11	19		
#200	0.075	15		

Coefficients

D_{85} = 29.1116 mm	D_{30} = 0.1894 mm
D_{60} = 4.0549 mm	D_{15} = N/A
D_{50} = 0.7006 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

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

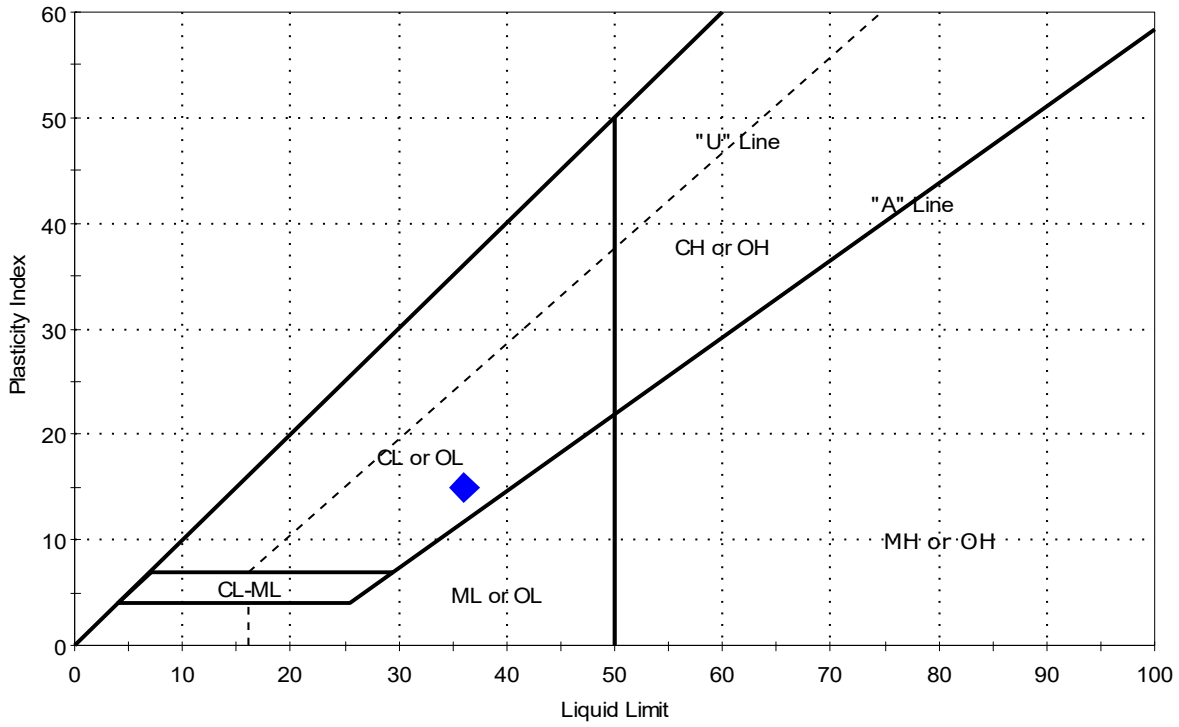
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-101	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	06/02/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669826		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-BHR-10	10-12 ft	28	36	21	15	0.5	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-101	Sample Type:	jar	Tested By:	cam
Sample ID:	6D	Test Date:	05/23/22	Checked By:	bfs
Depth :	25-27 ft	Test Id:	669031		
Test Comment:	---				
Visual Description:	Moist, olive silt with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6D	B-BHR-10	25-27 ft	9	n/a	n/a	n/a	n/a	

Dry Strength: LOW

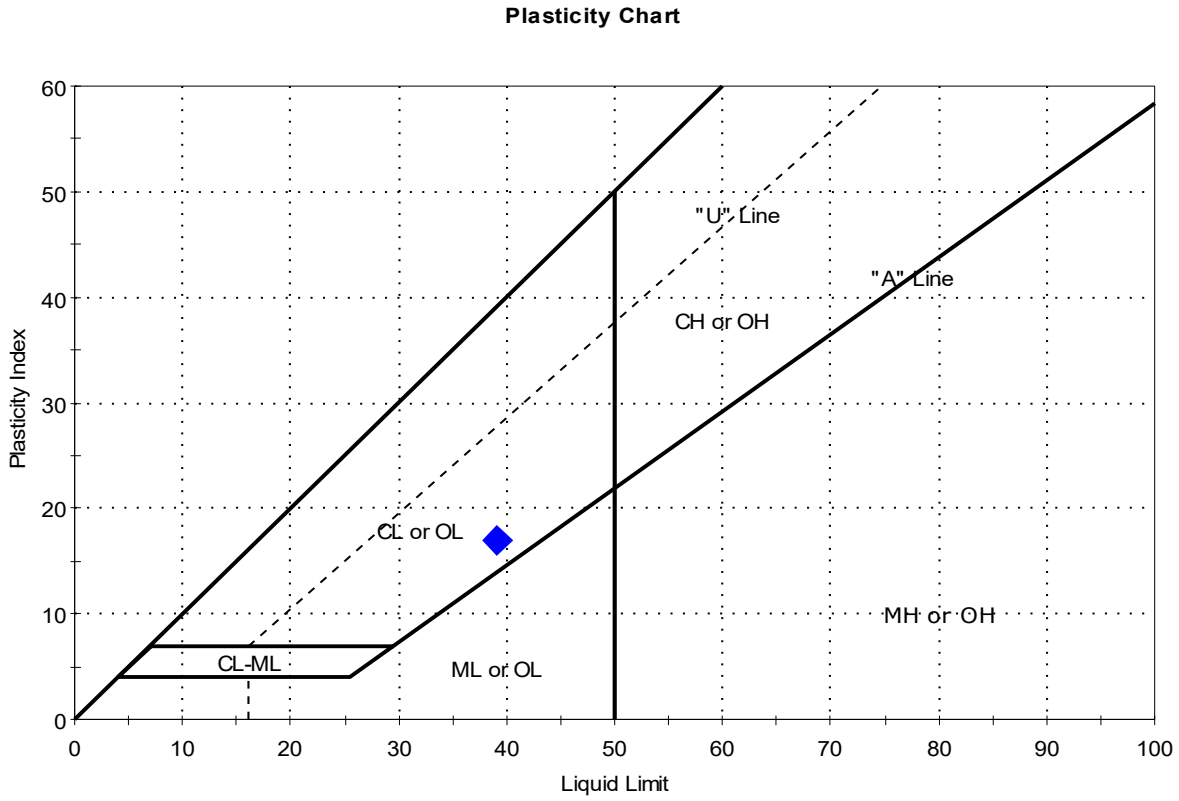
Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	06/02/22	Checked By:	bfs
Depth :	10.5-12.5 ft	Test Id:	669827		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

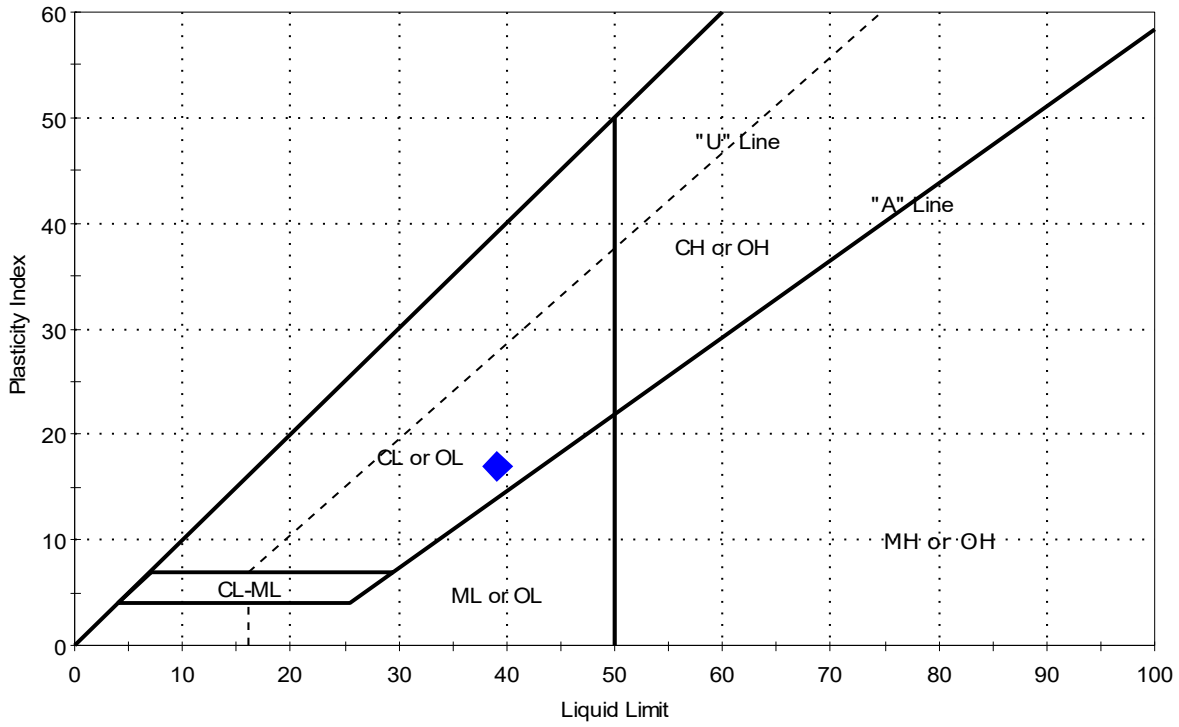


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-BHR-10	10.5-12.5 ft	25	39	22	17	0.2	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/24/22	Checked By:	bfs
Depth :	12-14 ft	Test Id:	669032		
Test Comment:	---				
Visual Description:	Moist, pale olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

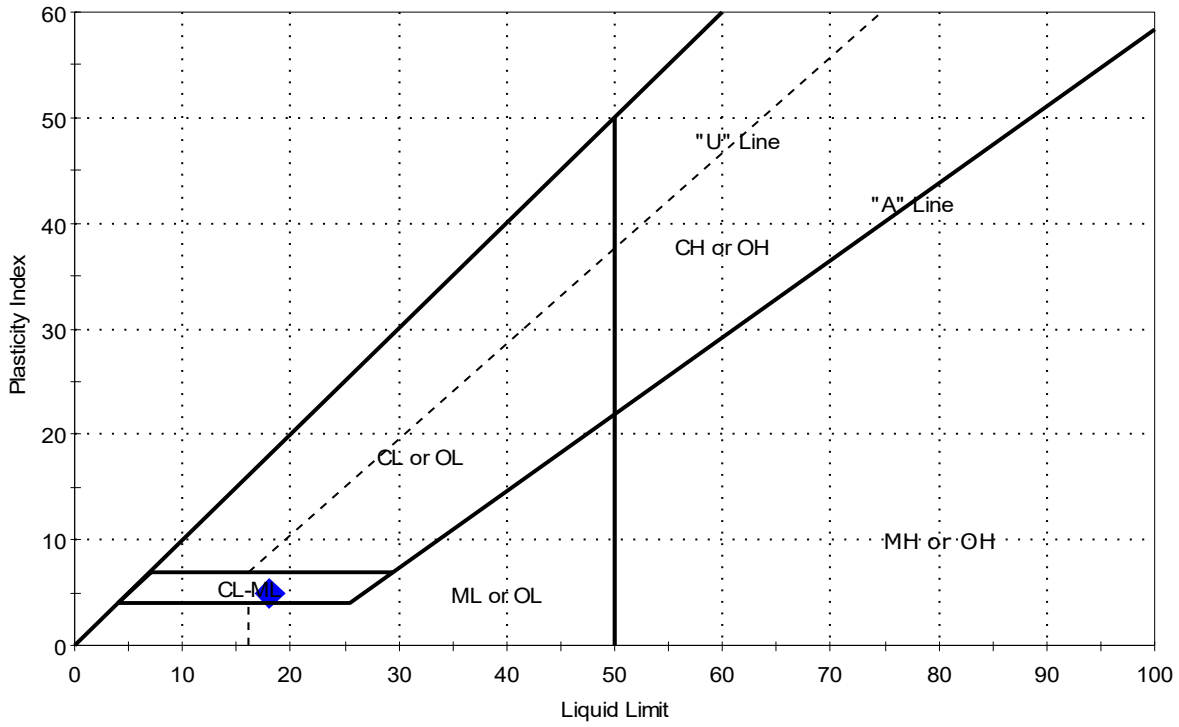


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	12-14 ft	28	39	22	17	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	8D	Test Date:	05/27/22	Checked By:	bfs
Depth :	35-37 ft	Test Id:	669033		
Test Comment:	---				
Visual Description:	Moist, olive gray silty, clayey sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

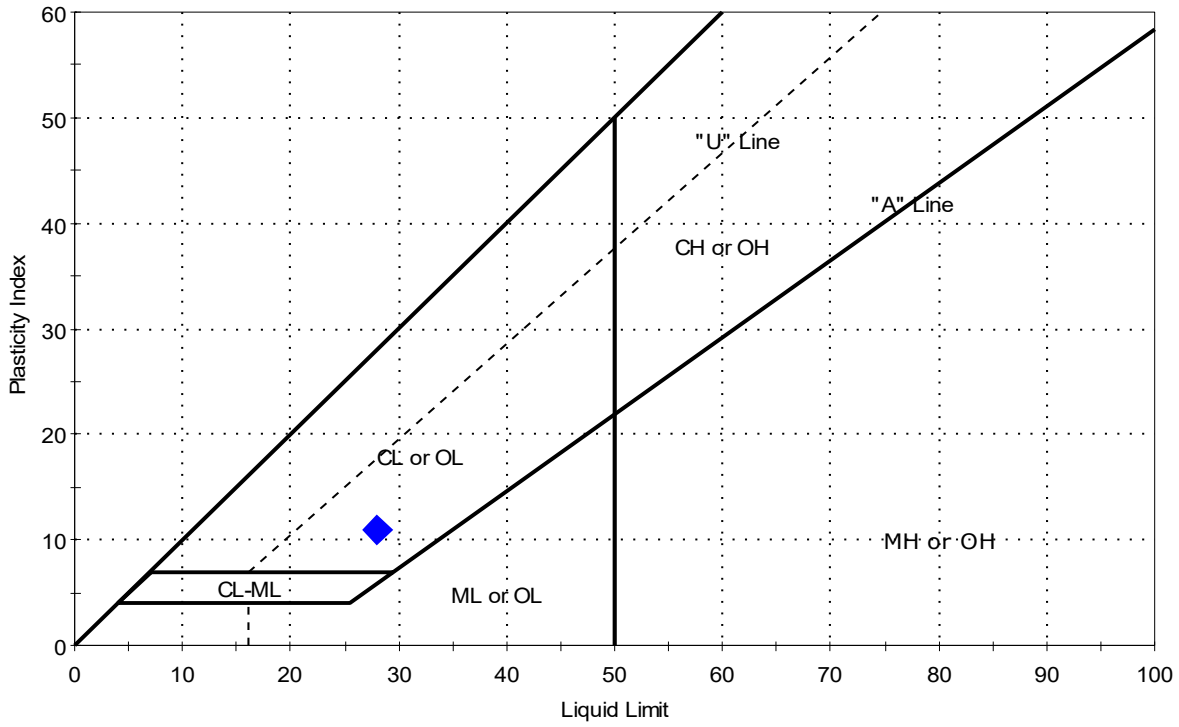


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	8D	B-BHR-10	35-37 ft	9	18	13	5	-0.7	Silty, Clayey SAND (SC-SM)

Sample Prepared using the WET method
37% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-103	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/19/22	Checked By:	bfs
Depth :	10-11.4 ft	Test Id:	669034		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

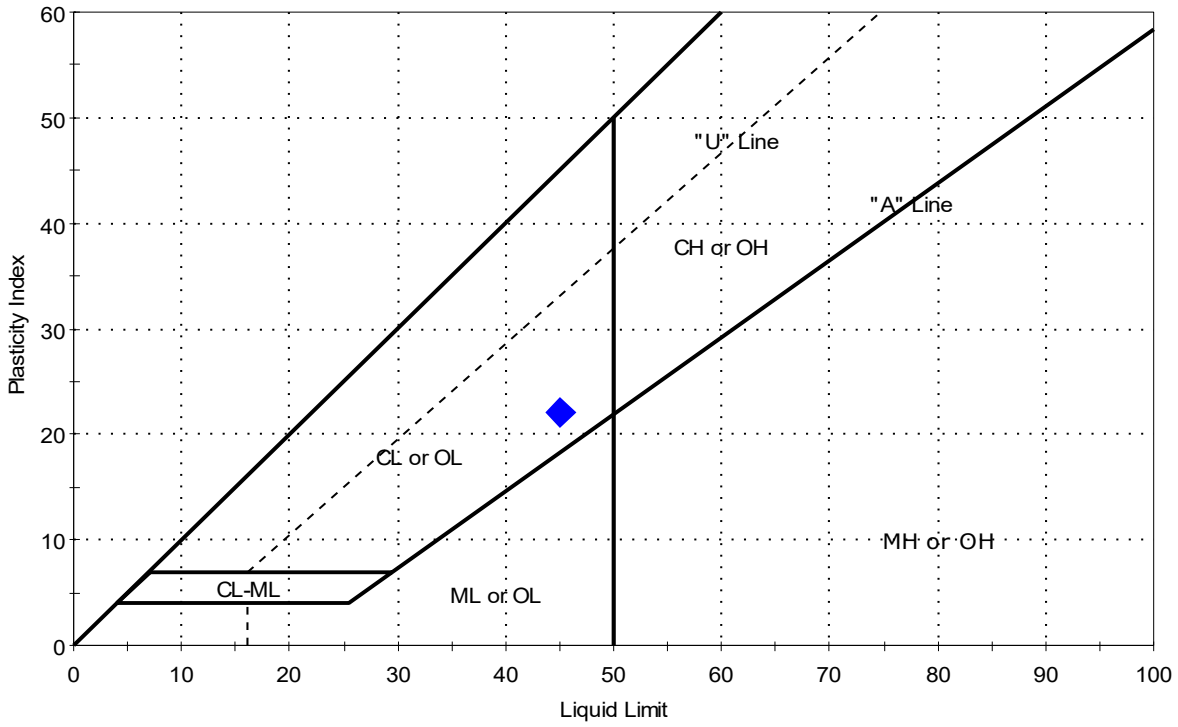


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	10-11.4 ft	24	28	17	11	0.6	Lean CLAY (CL)

Sample Prepared using the WET method
2% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-104	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/20/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669035		
Test Comment:	---				
Visual Description:	Moist, olive clay with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-10	5-7 ft	26	45	23	22	0.1	

Sample Prepared using the WET method

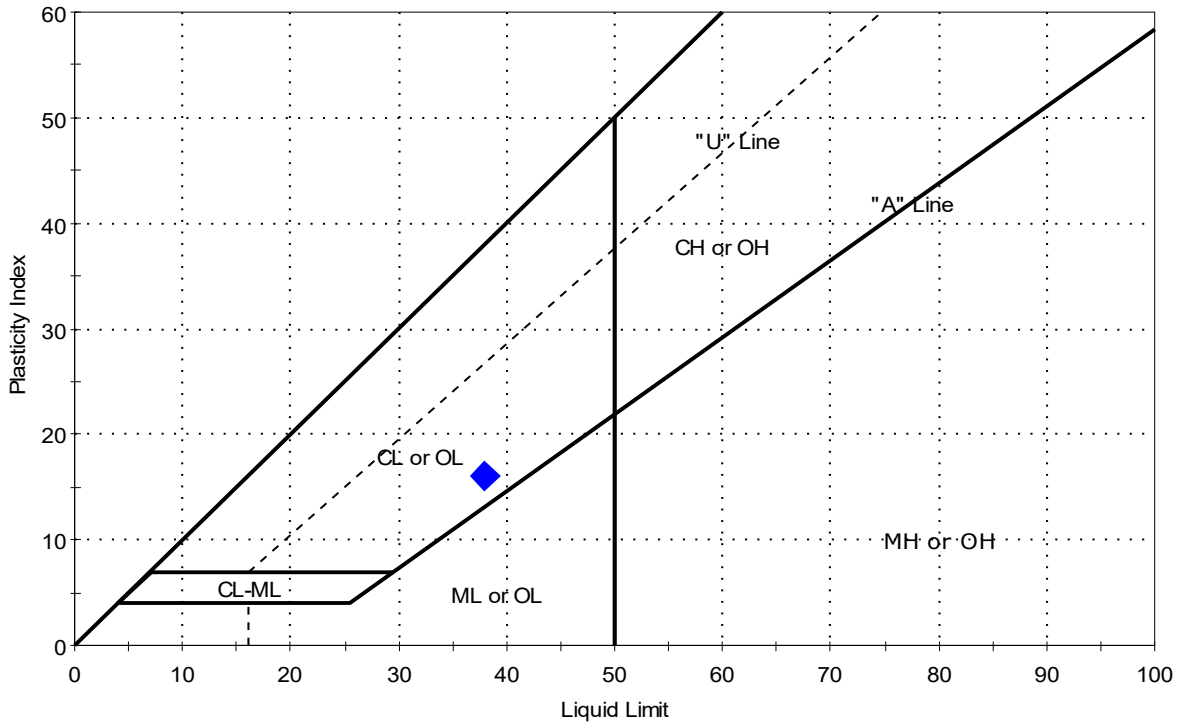
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/24/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669036		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-10	5-7 ft	25	38	22	16	0.2	

Sample Prepared using the WET method

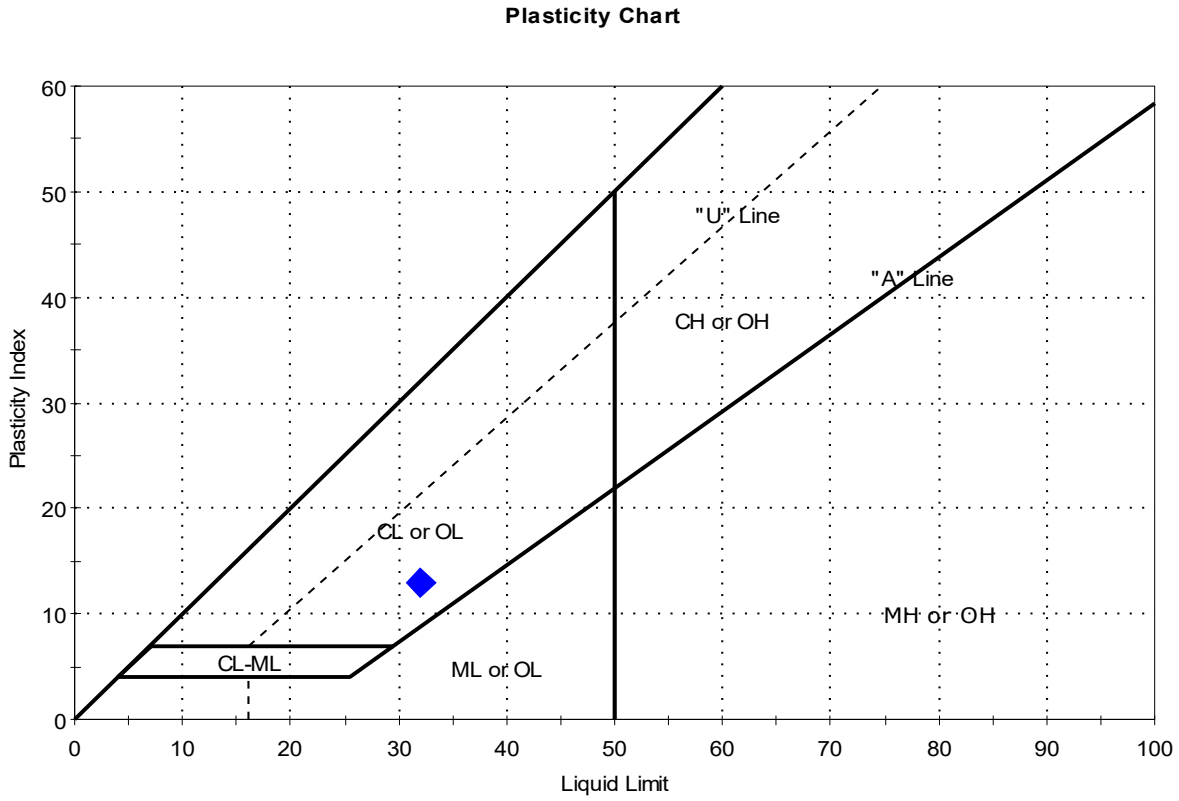
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	05/19/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669037		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

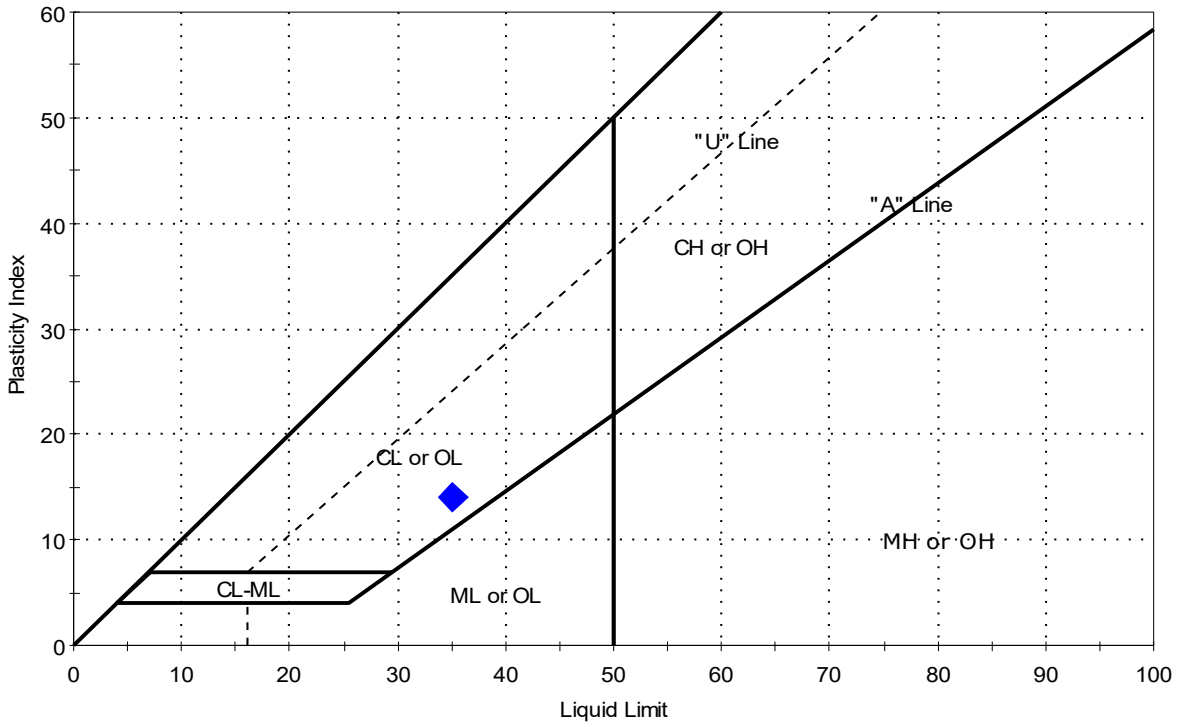


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-BHR-10	15-17 ft	30	32	19	13	0.8	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-106	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/20/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669026		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	10-12 ft	26	35	21	14	0.3	

Sample Prepared using the WET method

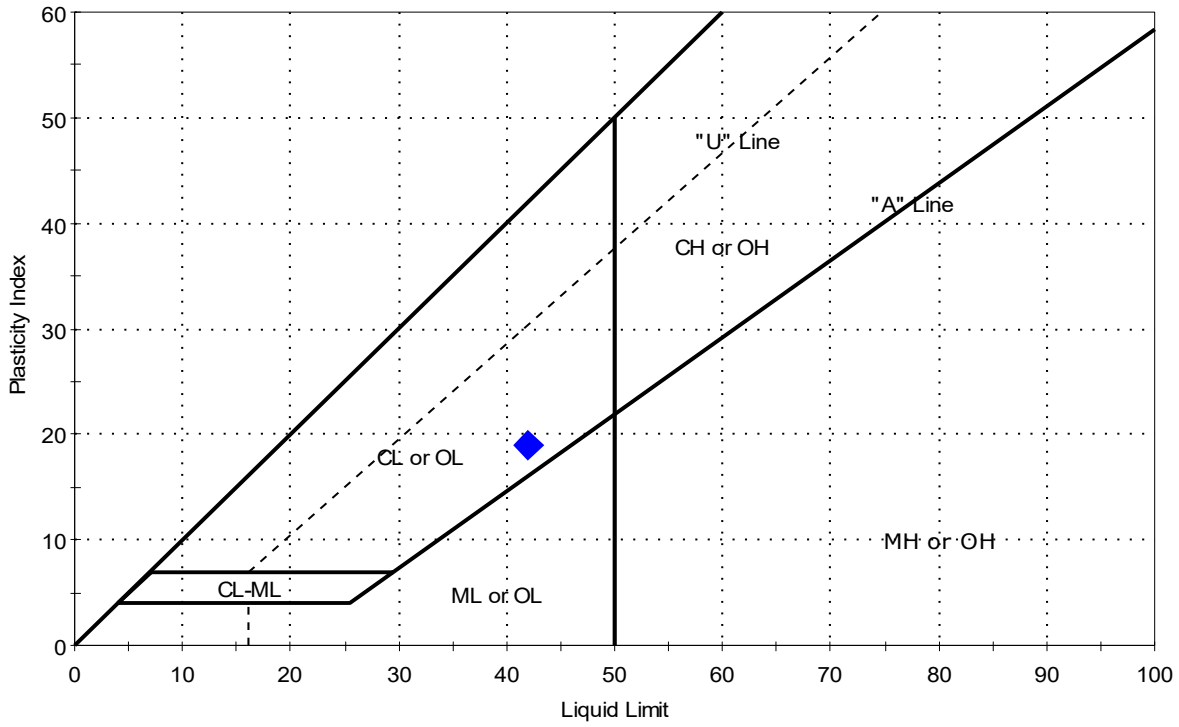
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-107	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/24/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669027		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	10-12 ft	30	42	23	19	0.4	

Sample Prepared using the WET method

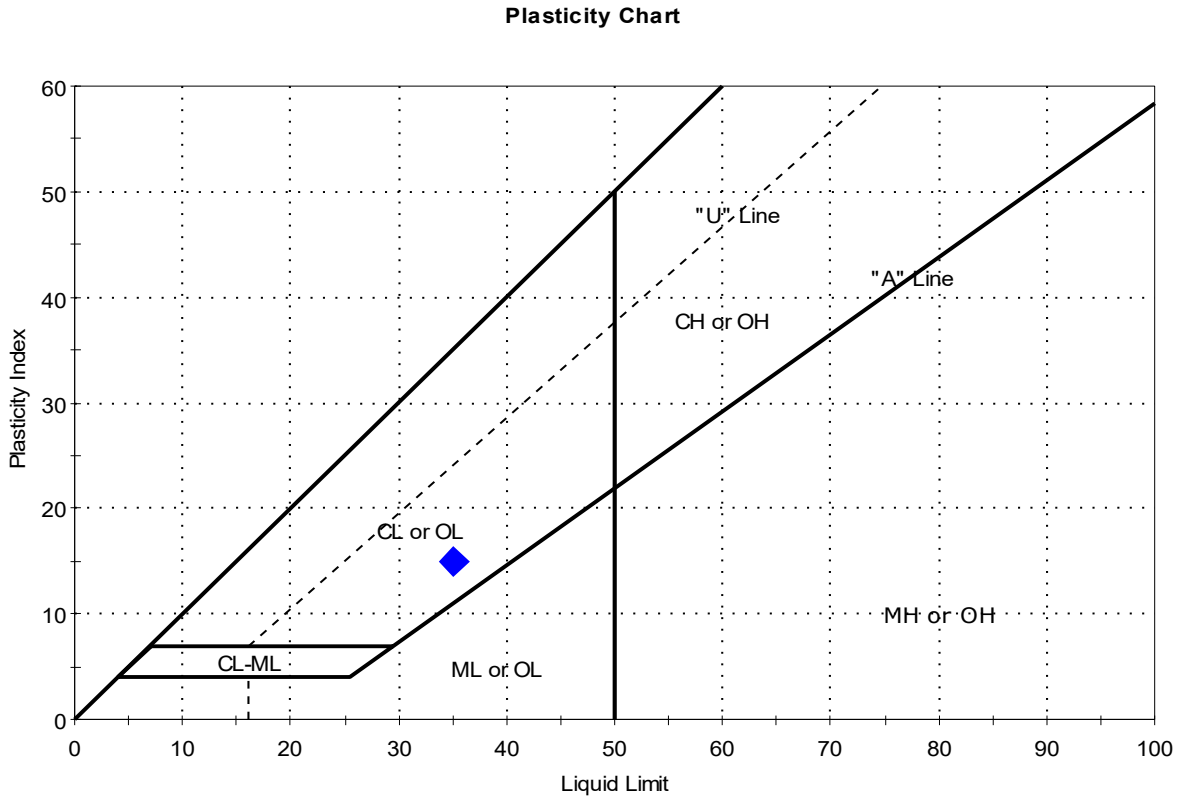
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-108	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	06/02/22	Checked By:	bfs
Depth :	8-10 ft	Test Id:	669828		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

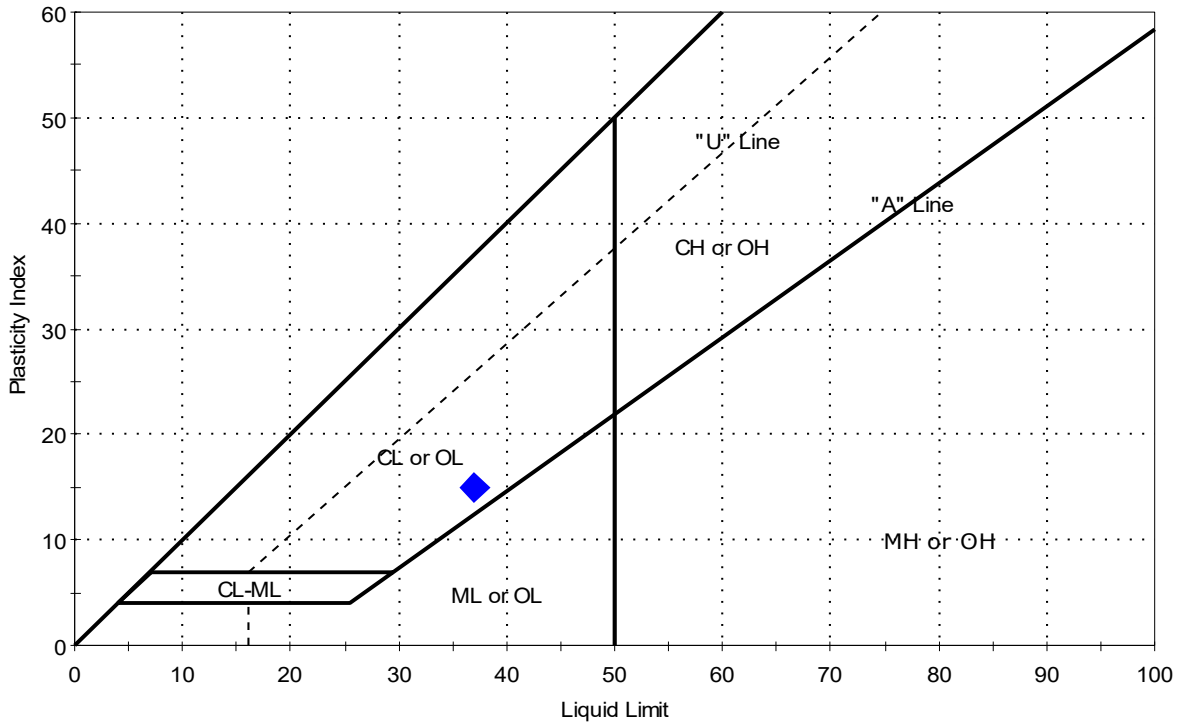


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-BHR-10	8-10 ft	28	35	20	15	0.6	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-108	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/24/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669028		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-10	5-7 ft	25	37	22	15	0.2	

Sample Prepared using the WET method

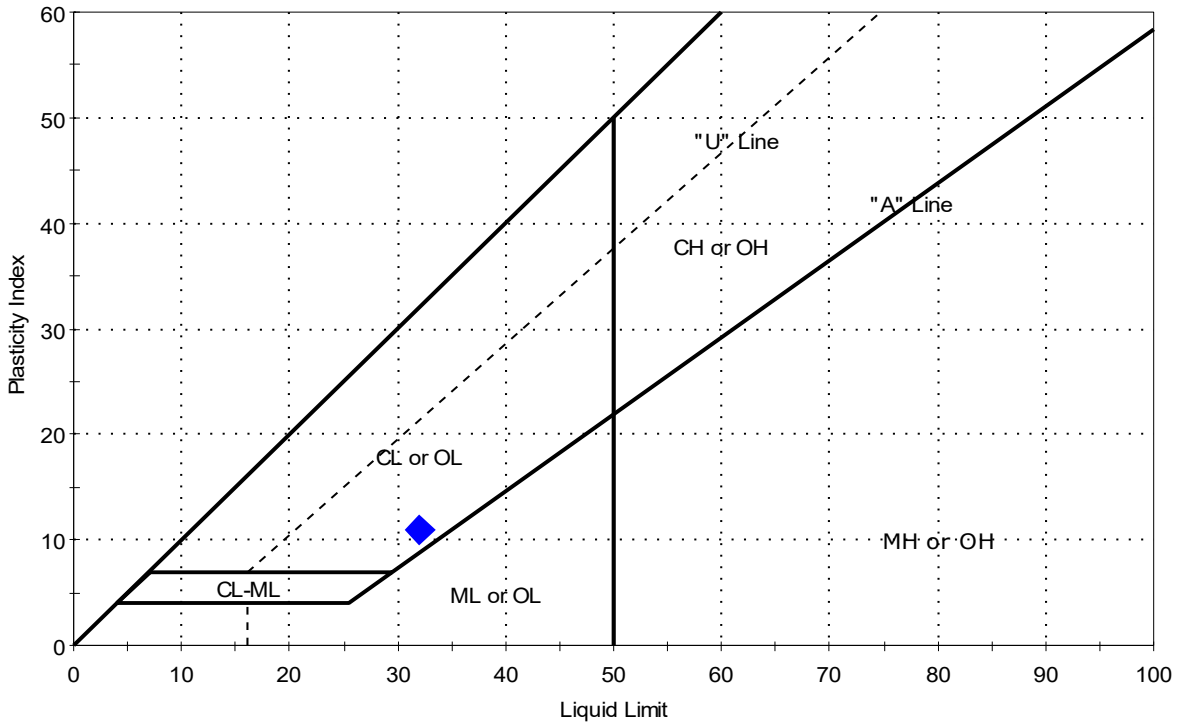
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-108	Sample Type:	jar	Tested By:	cam
Sample ID:	3D-A	Test Date:	05/19/22	Checked By:	bfs
Depth :	10-11.6 ft	Test Id:	669029		
Test Comment:	---				
Visual Description:	Moist, olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D-A	B-BHR-10	10-11.6 ft	26	32	21	11	0.5	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

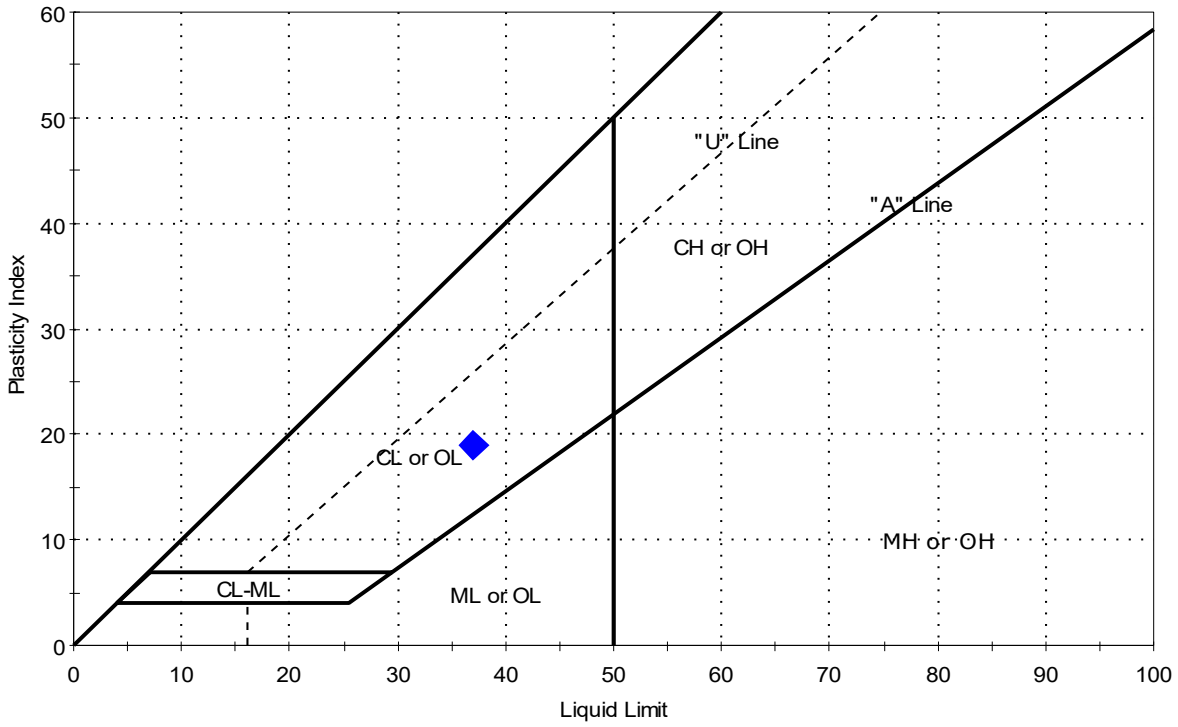
Toughness: LOW

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GeoTesting EXPRESS

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-109	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/23/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669030		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

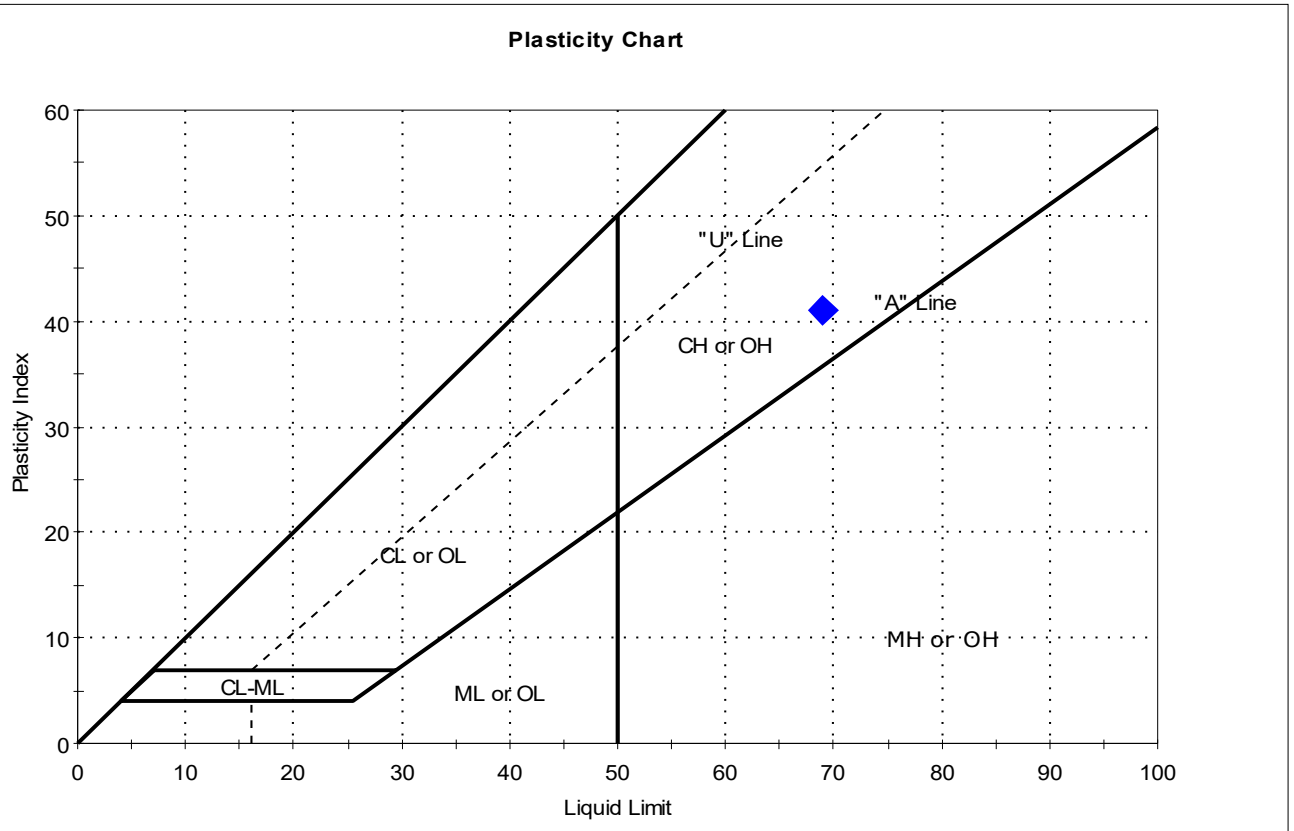


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-10	5-7 ft	27	37	18	19	0.5	Lean CLAY (CL)

Sample Prepared using the WET method
1% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-110	Sample Type:	jar	Tested By:	cam
Sample ID:	2DA	Test Date:	05/19/22	Checked By:	bfs
Depth :	5-6.9 ft	Test Id:	669022		
Test Comment:	---				
Visual Description:	Moist, dark gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2DA	B-BHR-11	5-6.9 ft	45	69	28	41	0.4	

Sample Prepared using the WET method

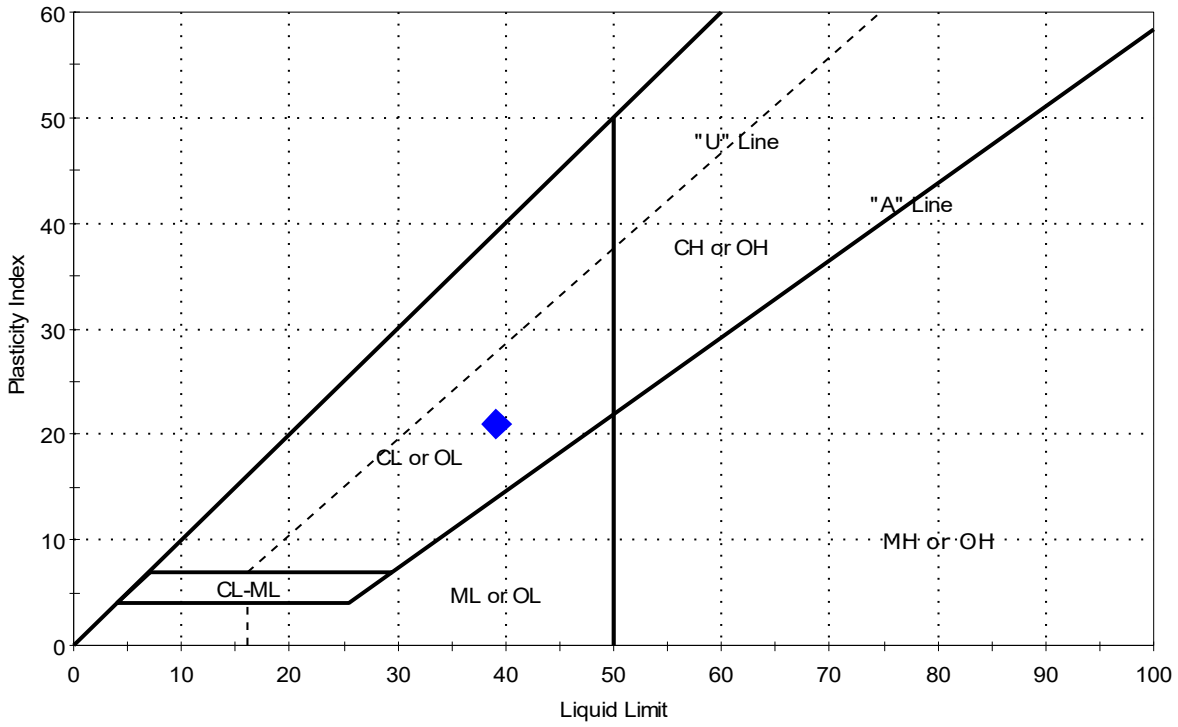
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-111	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/24/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669023		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-11	5-7 ft	25	39	18	21	0.3	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

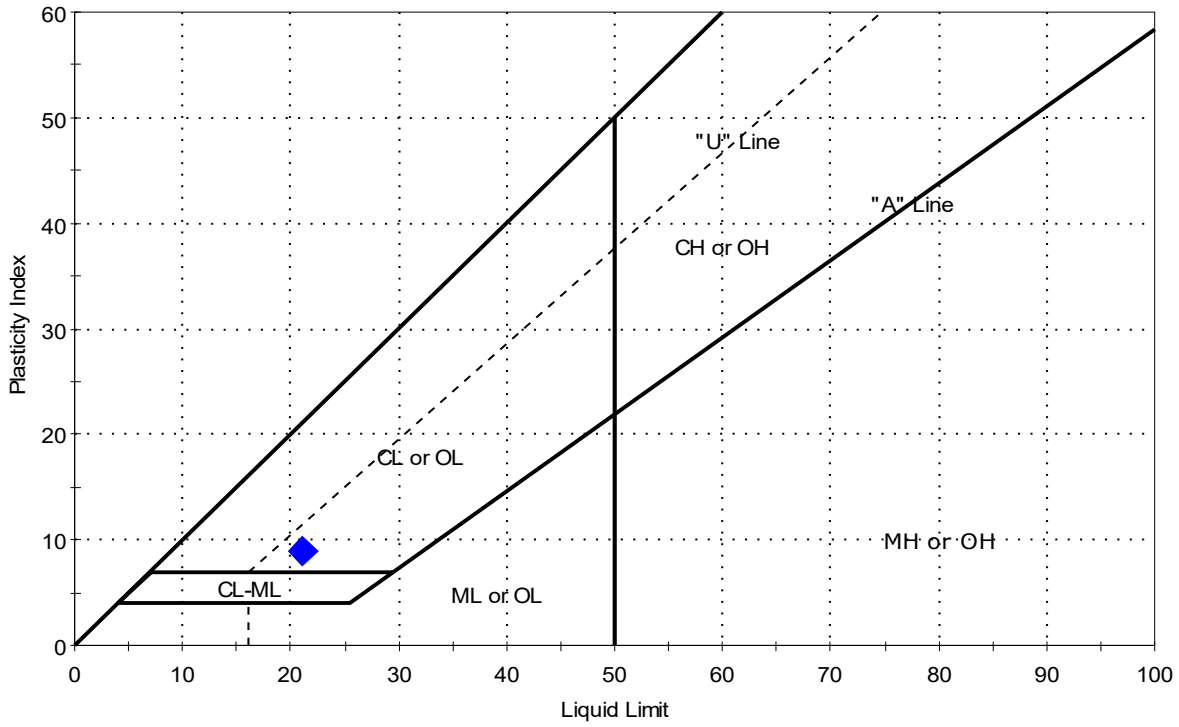
Dilatancy: SLOW

Toughness: LOW

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-112	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/20/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669024		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-11	5-7 ft	16	21	12	9	0.4	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

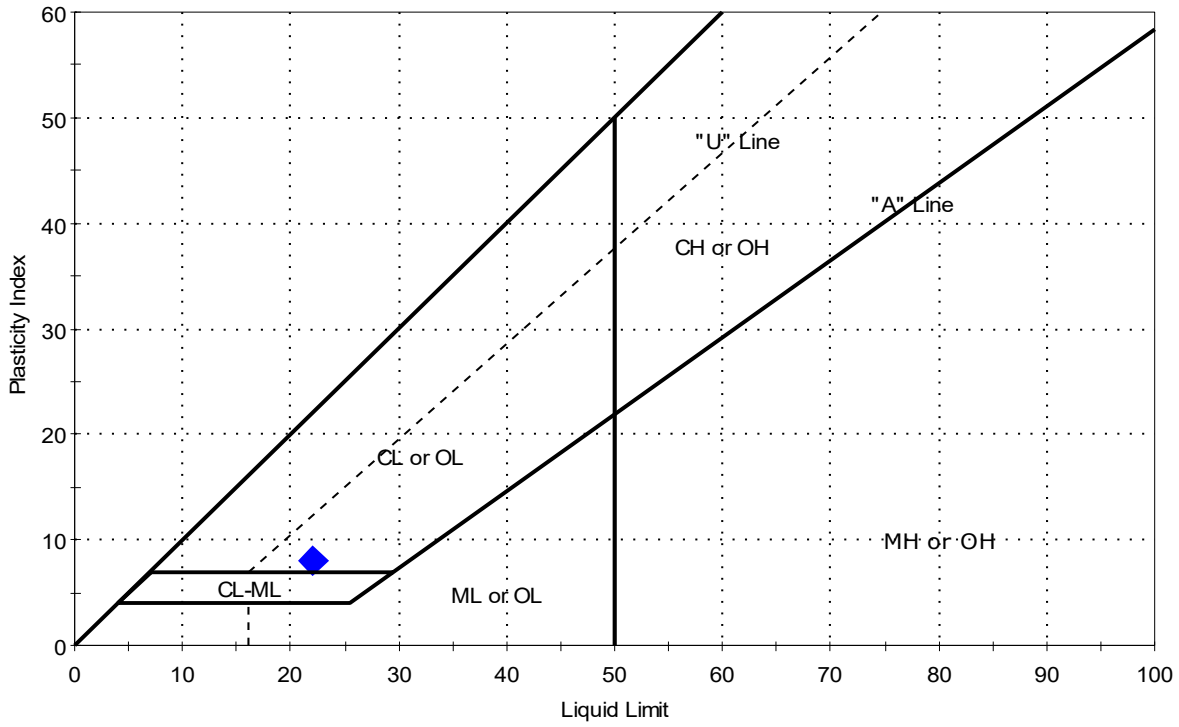
Dilatancy: SLOW

Toughness: LOW

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	cam
Sample ID:	6D	Test Date:	05/25/22	Checked By:	bfs
Depth :	20-22 ft	Test Id:	669018		
Test Comment:	---				
Visual Description:	Moist, light gray clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6D	B-BHR-11	20-22 ft	18	22	14	8	0.5	Lean CLAY with Sand (CL)

Sample Prepared using the WET method
13% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	cam
Sample ID:	8D	Test Date:	05/24/22	Checked By:	bfs
Depth :	27-29 ft	Test Id:	669019		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Sample Determined to be non-plastic

Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	8D	B-BHR-11	27-29 ft	14	n/a	n/a	n/a	n/a	

Dry Strength: LOW

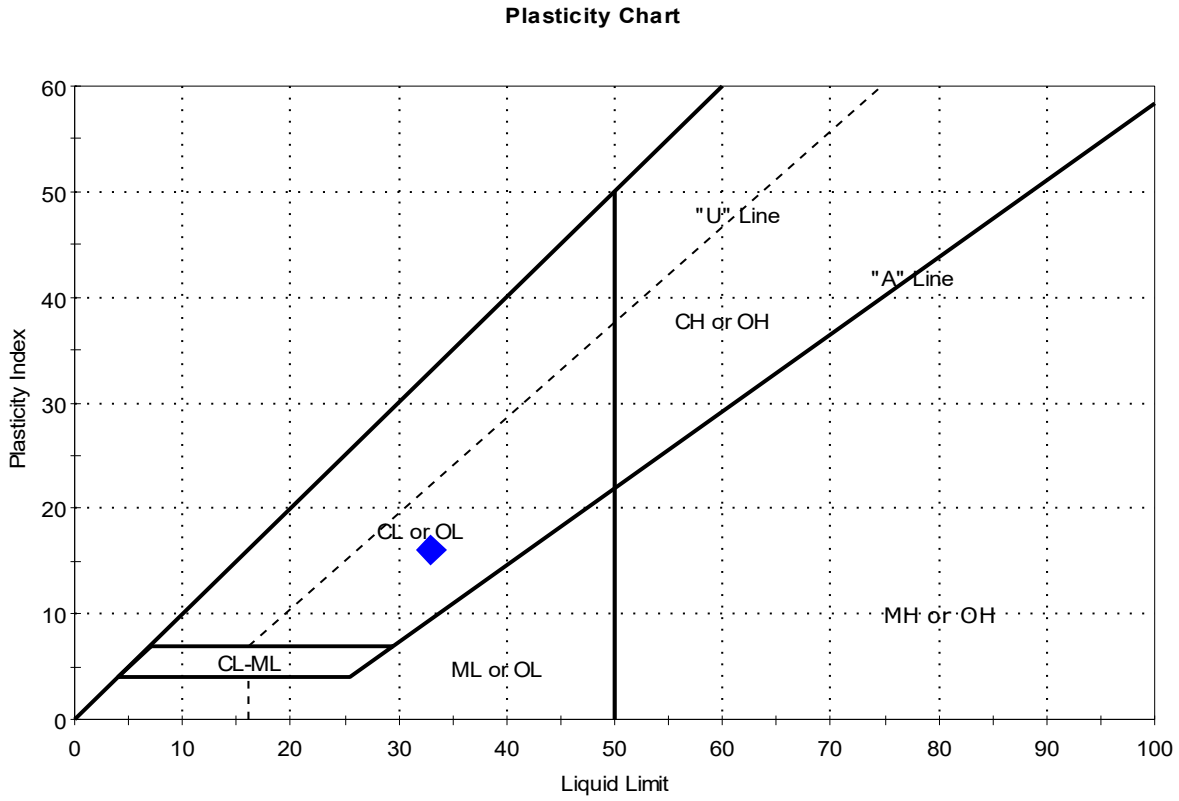
Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-113	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/24/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669025		
Test Comment:	---				
Visual Description:	Moist, olive brown clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



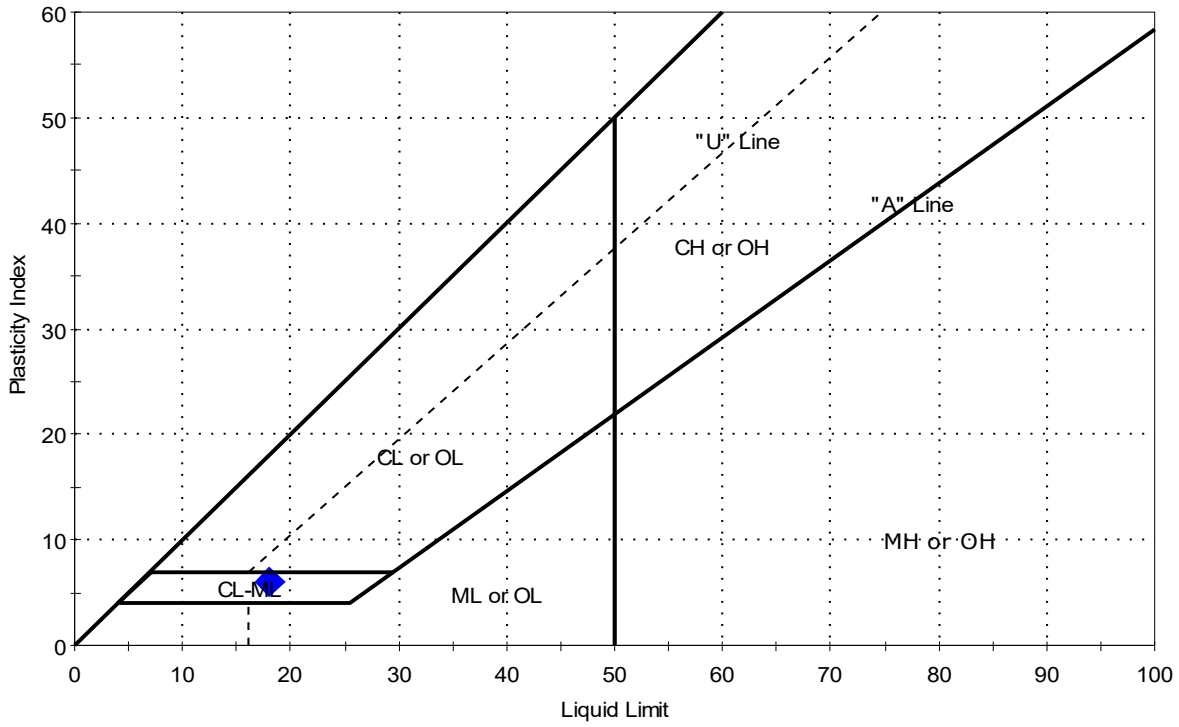
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-11	5-7 ft	16	33	17	16	-0.1	Lean CLAY with Sand (CL)

Sample Prepared using the WET method
12% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

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Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-114	Sample Type:	jar	Tested By:	cam
Sample ID:	7D	Test Date:	05/31/22	Checked By:	bfs
Depth :	30-32 ft	Test Id:	669020		
Test Comment:	---				
Visual Description:	Moist, gray silty, clayey sand with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

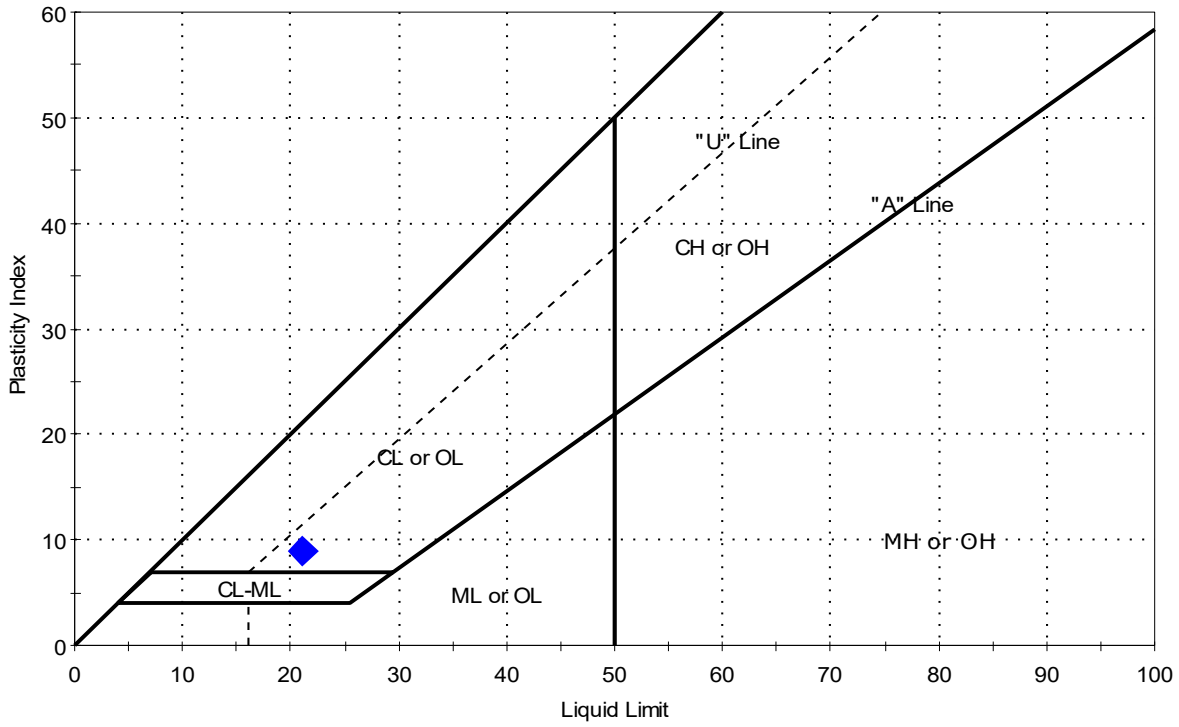


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	7D	B-BHR-11	30-32 ft	9	18	12	6	-0.5	Silty, Clayey SAND with Gravel (SC-SM)

Sample Prepared using the WET method
43% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/27/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669021		
Test Comment:	---				
Visual Description:	Moist, light olive brown sandy clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

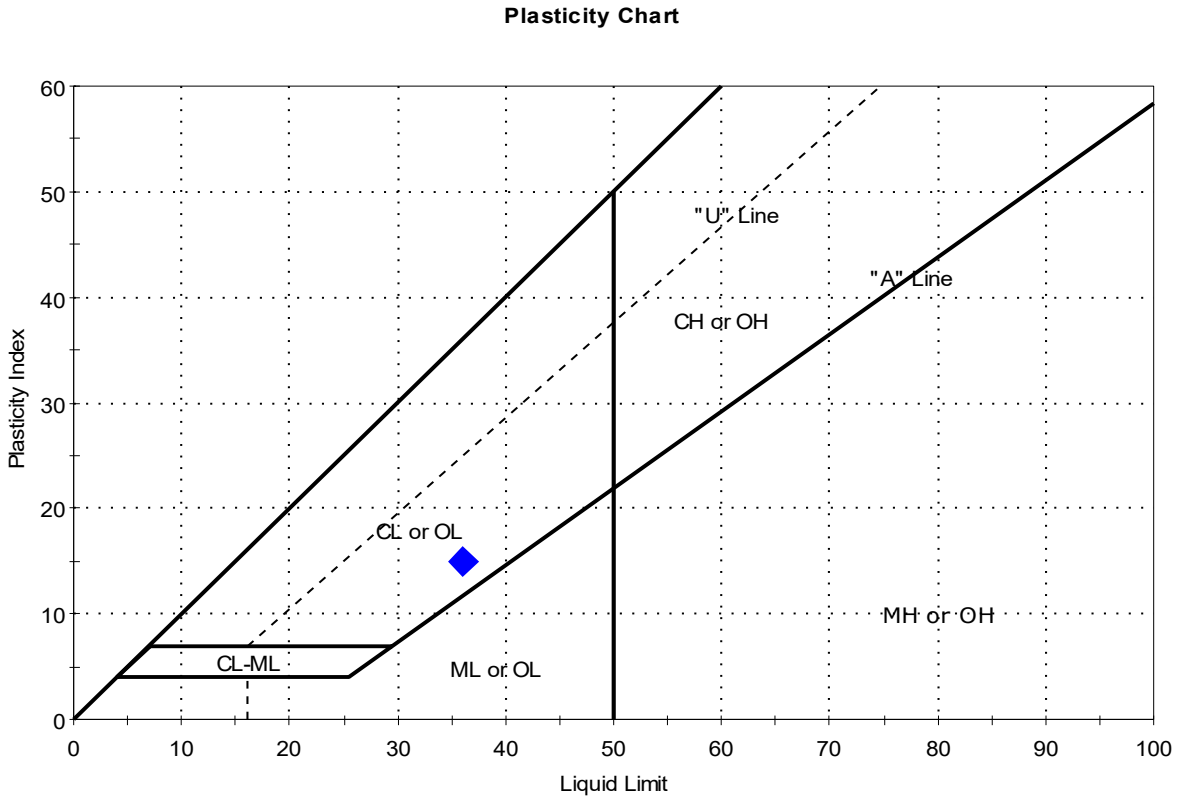


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-11	5-7 ft	13	21	12	9	0.1	Sandy Lean CLAY (CL)

Sample Prepared using the WET method
26% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-115	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	05/25/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669017		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

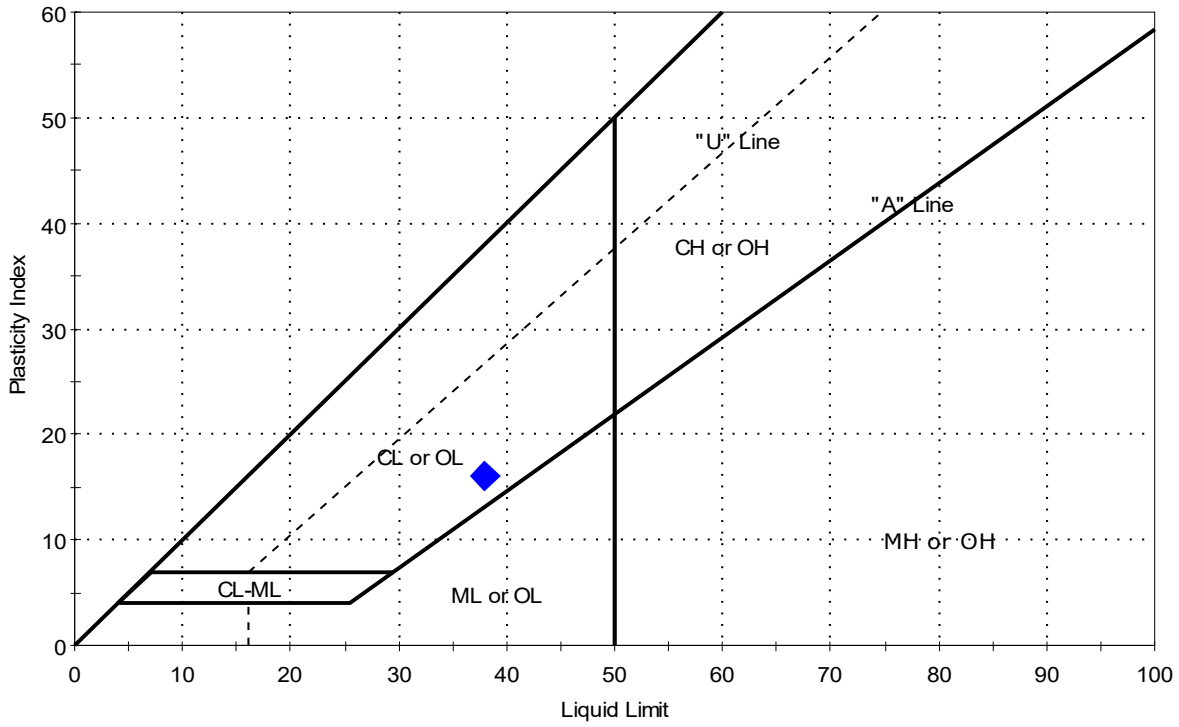


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-BHR-11	15-17 ft	26	36	21	15	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
1% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	cam
Sample ID:	2D	Test Date:	05/24/22	Checked By:	bfs
Depth :	5-7 ft	Test Id:	669036		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2D	B-BHR-10	5-7 ft	25	38	22	16	0.2	

Sample Prepared using the WET method

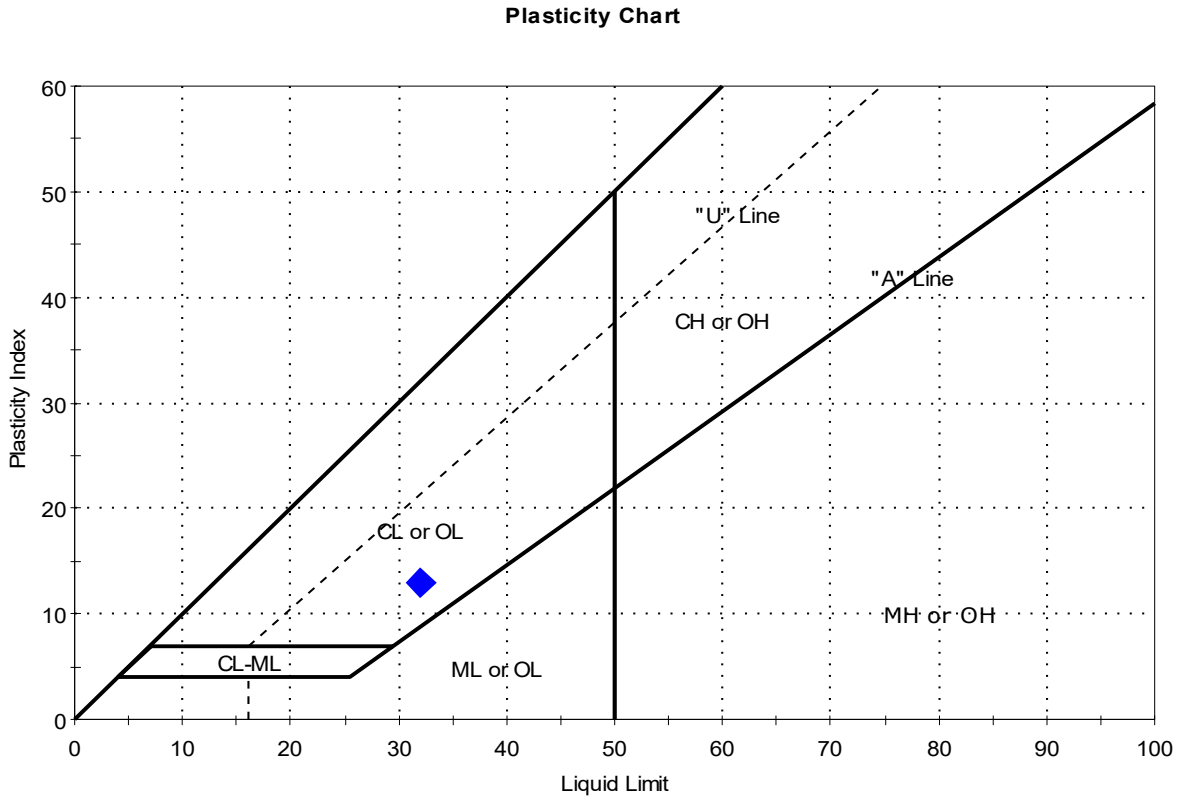
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-105	Sample Type:	jar	Tested By:	cam
Sample ID:	4D	Test Date:	05/19/22	Checked By:	bfs
Depth :	15-17 ft	Test Id:	669037		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

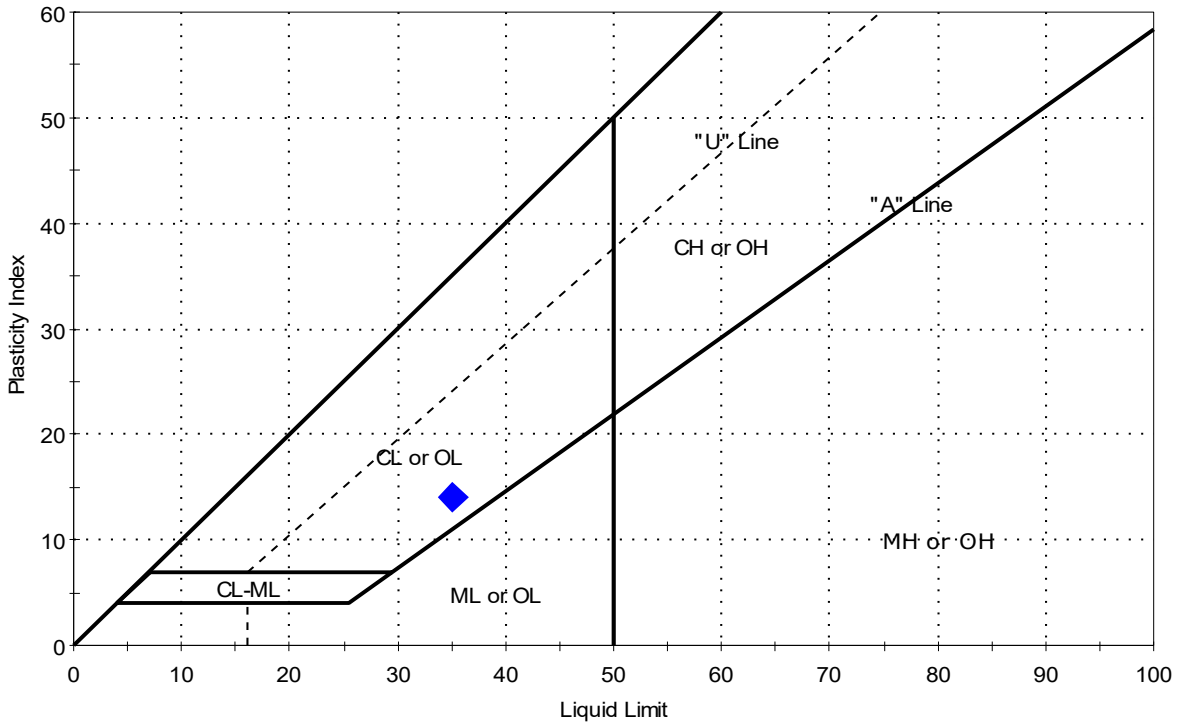


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	B-BHR-10	15-17 ft	30	32	19	13	0.8	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-106	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/20/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669026		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	10-12 ft	26	35	21	14	0.3	

Sample Prepared using the WET method

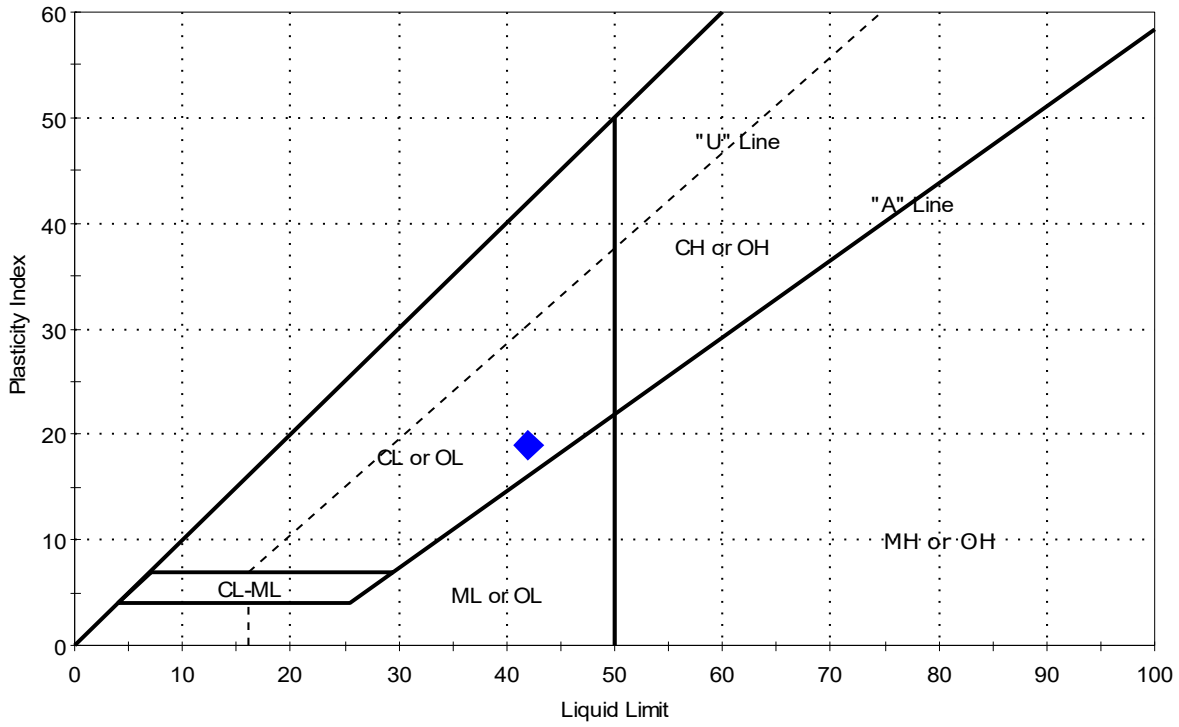
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Highway				
Location:	Bangor, ME			Project No:	GTX-315483
Boring ID:	HB-BHR-107	Sample Type:	jar	Tested By:	cam
Sample ID:	3D	Test Date:	05/24/22	Checked By:	bfs
Depth :	10-12 ft	Test Id:	669027		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3D	B-BHR-10	10-12 ft	30	42	23	19	0.4	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils by ASTM D6528

Client:	Golder Associates, USA, Inc.	GTX#:	315483
Project Name:	Hogan Road Highway	Test Date:	5/25/22
Project Location:	Bangor, ME		

Boring ID:	HB-BHR-101
Sample ID:	1U
Depth, ft:	10-12
Elevation, ft:	---

Visual Description: Moist, olive clay

Test Equipment: Top and bottom box (circular) = 2.50 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch. Stacked rings used. Set up included porous stones with pins.

Test Condition: Inundated prior to consolidating to 925 psf.

Sample Type and Preparation: density: Extruded from tube, cut, trimmed and placed into apparatus at as-received and moisture content.

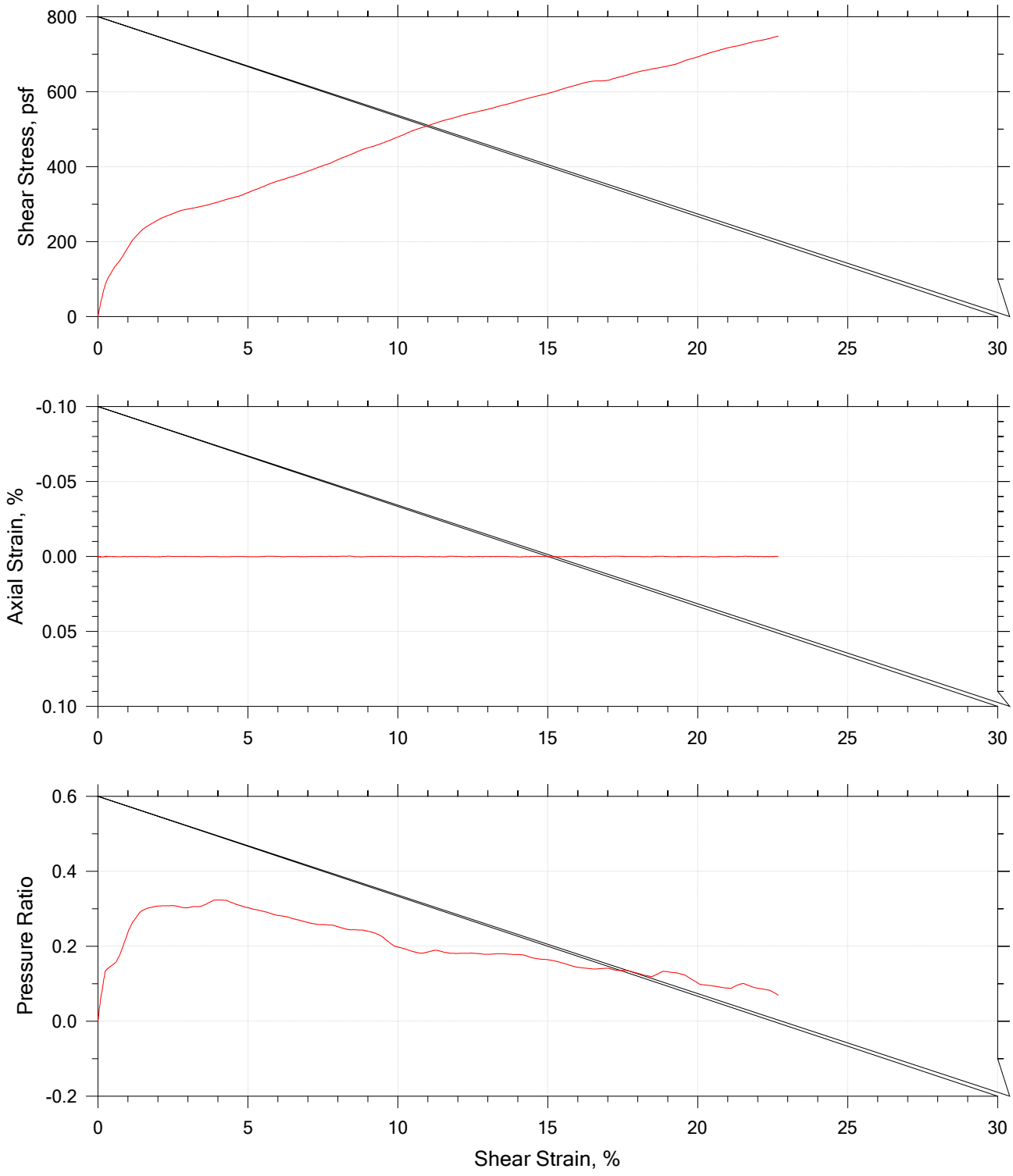
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-3				
Initial Moisture Content, %	26.9				
Initial Dry Density, pcf	95.4				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	925				
Vertical Consolidation Stress at time of shear, psf	925				
Final Moisture Content, %	34.0				
Measured Peak Shear Stress, psf	693				
Shear Strain at Peak Shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Peak Shear Stress, psf	644				
S_u / σ'_{vc}	0.70				

Comments:

Tested By: md Checked By: njh

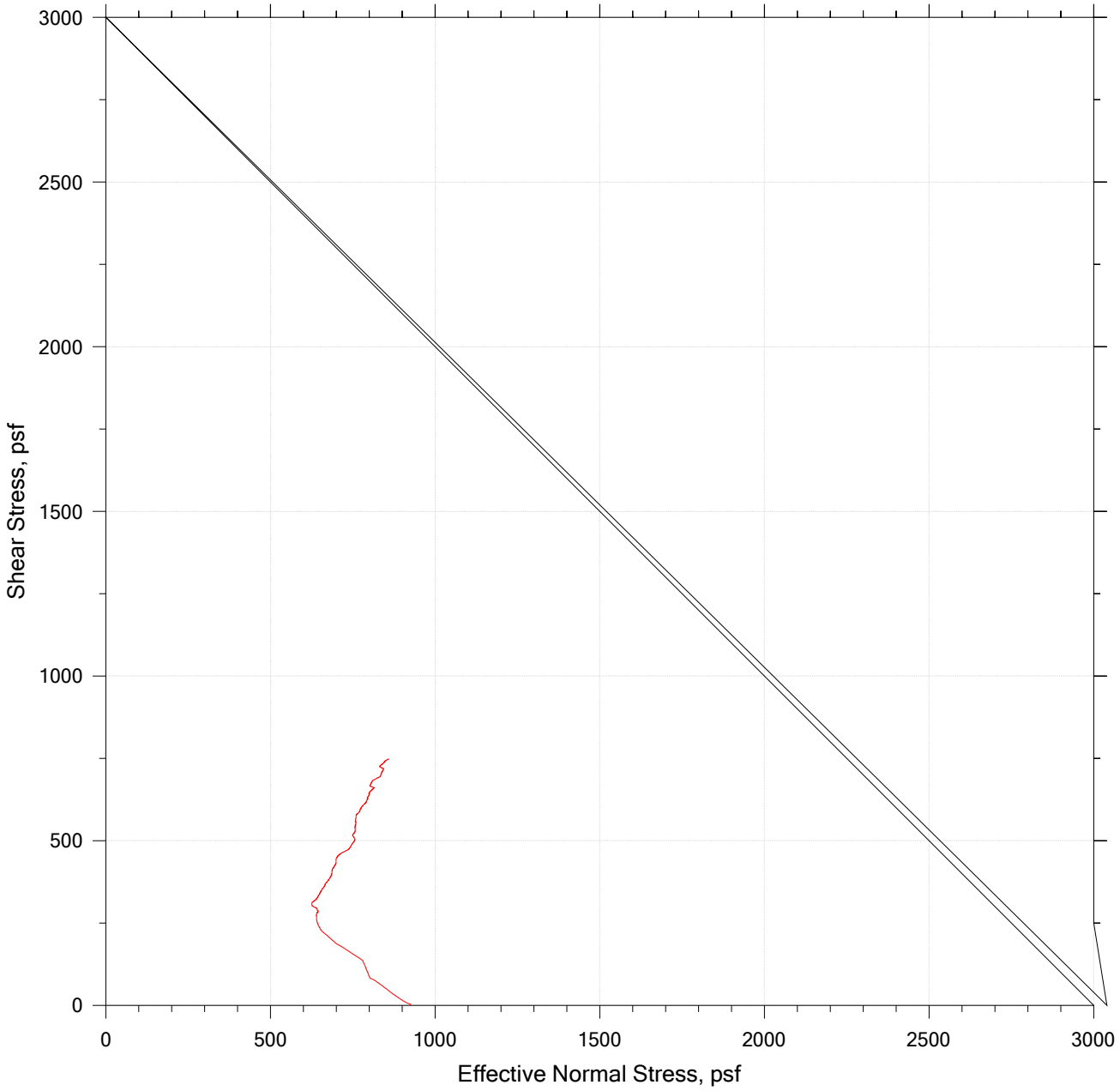
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

Direct Simple Shear Test by ASTM D6528



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: DSS-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks:		

Direct Simple Shear Test by ASTM D6528



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: DSS-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks:		

Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils by ASTM D6528

Client:	Golder Associates, USA, Inc.	GTX#:	315483
Project Name:	Hogan Road Highway	Test Date:	5/24/22
Project Location:	Bangor, ME		

Boring ID:	HB-BHR-102
Sample ID:	1U
Depth, ft:	10.5-12.5
Elevation, ft:	---

Visual Description: Moist, olive clay

Test Equipment: Top and bottom box (circular) = 2.50 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch. Stacked rings used. Set up included porous stones with pins.

Test Condition: Inundated prior to consolidating to 1,050 psf.

Sample Type and Preparation: density: Extruded from tube, cut, trimmed and placed into apparatus at as-received and moisture content.

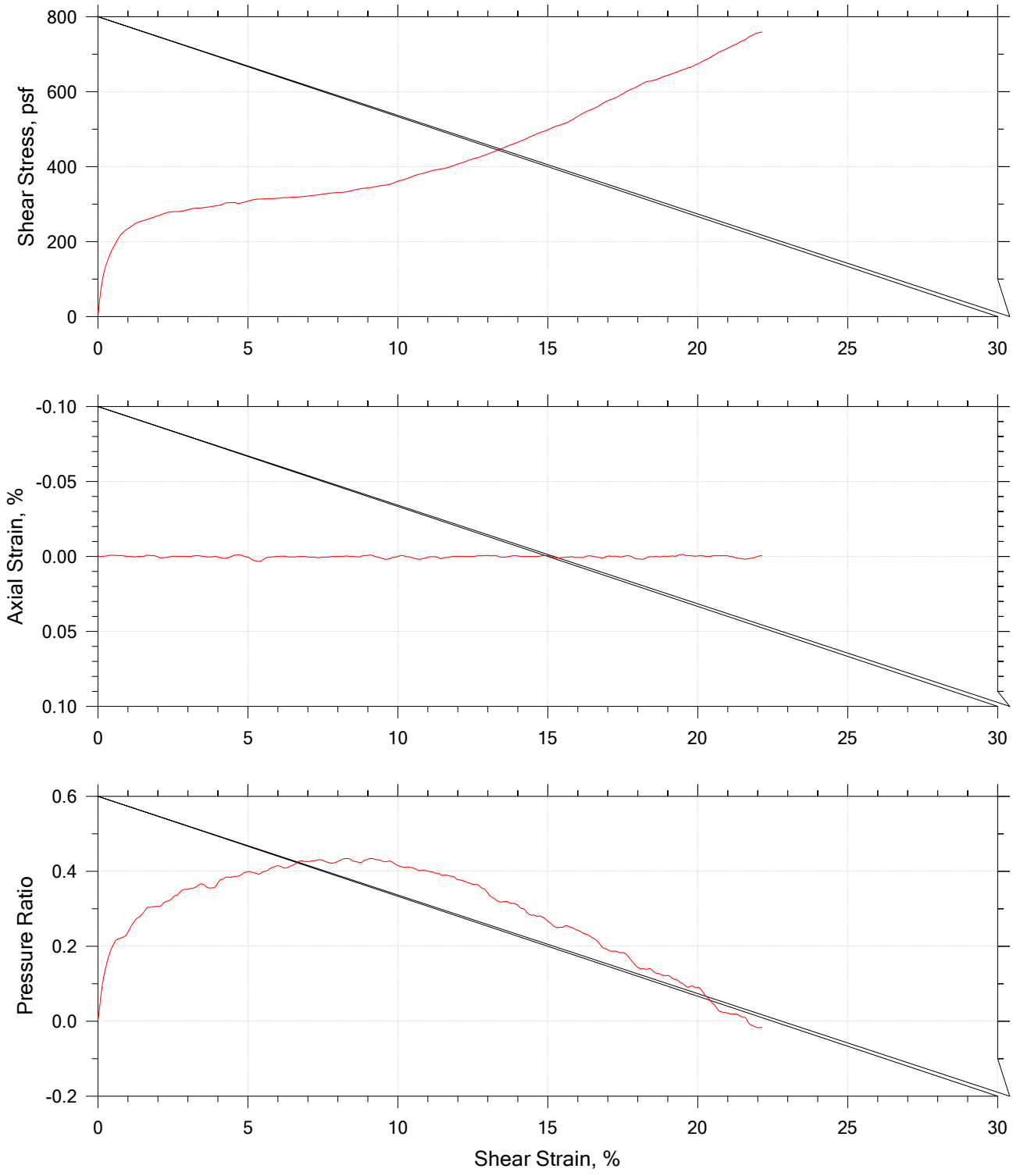
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-2				
Initial Moisture Content, %	25.3				
Initial Dry Density, pcf	98.9				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	1,050				
Vertical Consolidation Stress at time of shear, psf	1,050				
Final Moisture Content, %	32.2				
Measured Peak Shear Stress, psf	675				
Shear Strain at Peak Shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Peak Shear Stress, psf	626				
S_u / σ'_{vc}	0.60				

Comments:

Tested By: md Checked By: njh

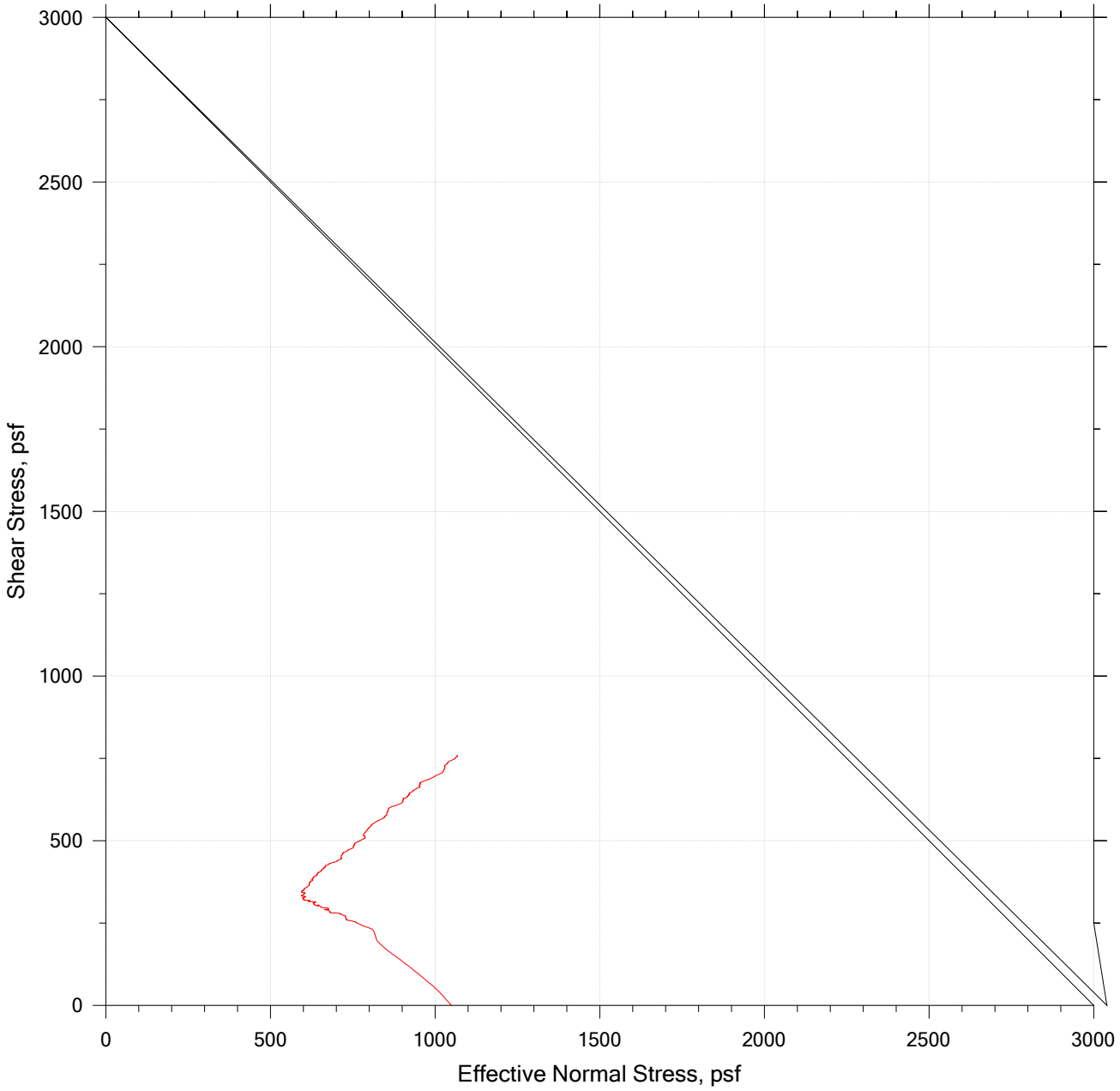
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

Direct Simple Shear Test by ASTM D6528



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: DSS-2	Preparation: Tube	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System HH		

Direct Simple Shear Test by ASTM D6528



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: DSS-2	Preparation: Tube	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System HH		

Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils by ASTM D6528

Client:	Golder Associates, USA, Inc.	GTX#:	315483
Project Name:	Hogan Road Highway	Test Date:	5/24/22
Project Location:	Bangor, ME		

Boring ID: HB-BHR-108
Sample ID: 1U
Depth, ft: 8-10
Elevation, ft: ---

Visual Description: Moist, olive clay

Test Equipment: Top and bottom box (circular) = 2.50 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch. Stacked rings used. Set up included porous stones with pins.

Test Condition: Inundated prior to consolidating to 900 psf.

Sample Type and Preparation: density: Extruded from tube, cut, trimmed and placed into apparatus at as-received and moisture content.

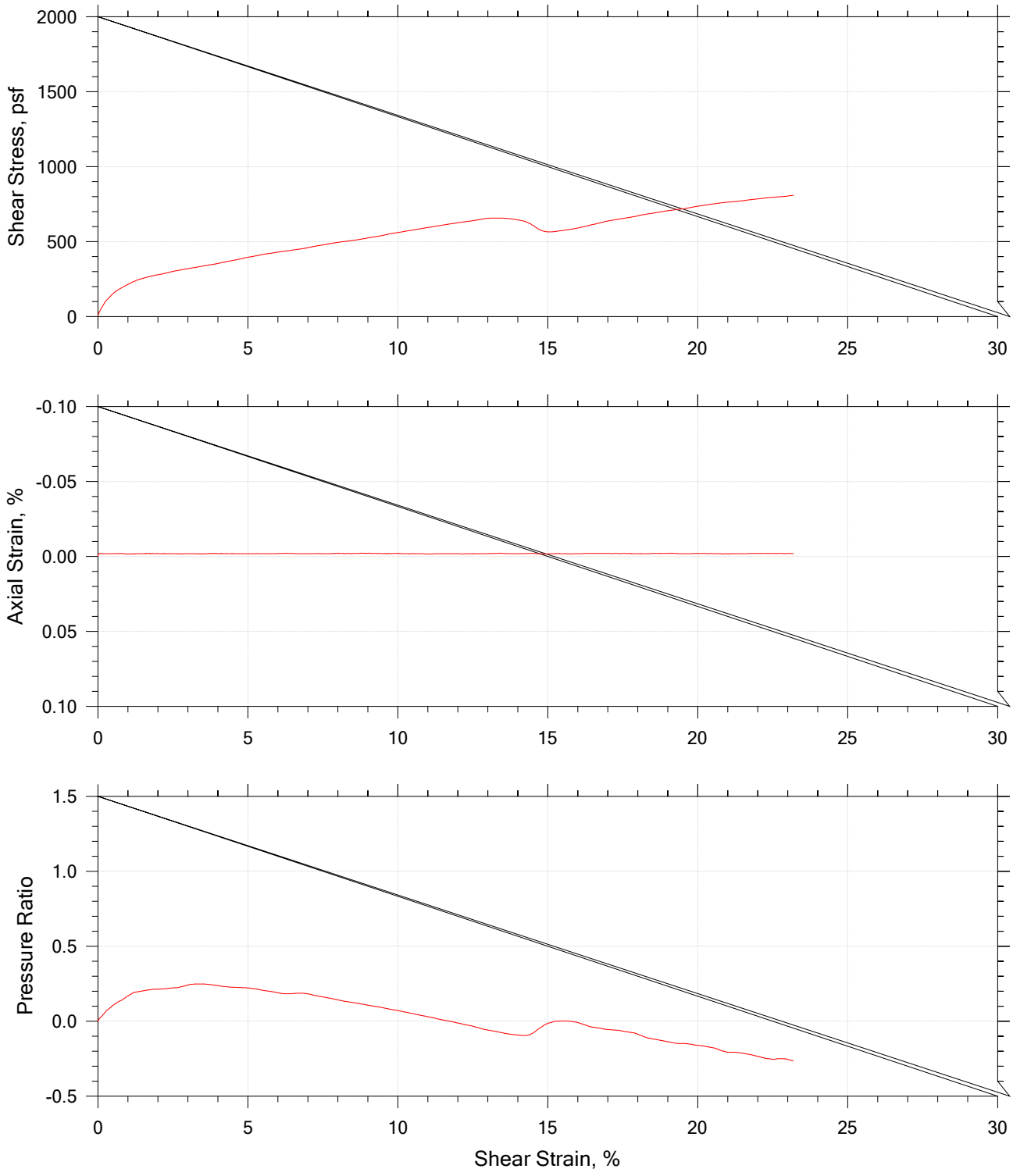
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	27.8				
Initial Dry Density, pcf	95.4				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	900				
Vertical Consolidation Stress at time of shear, psf	900				
Final Moisture Content, %	34.1				
Measured Peak Shear Stress, psf	737				
Shear Strain at Peak Shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Peak Shear Stress, psf	688				
S_u / σ'_{vc}	0.76				

Comments:

Tested By: md Checked By: njh

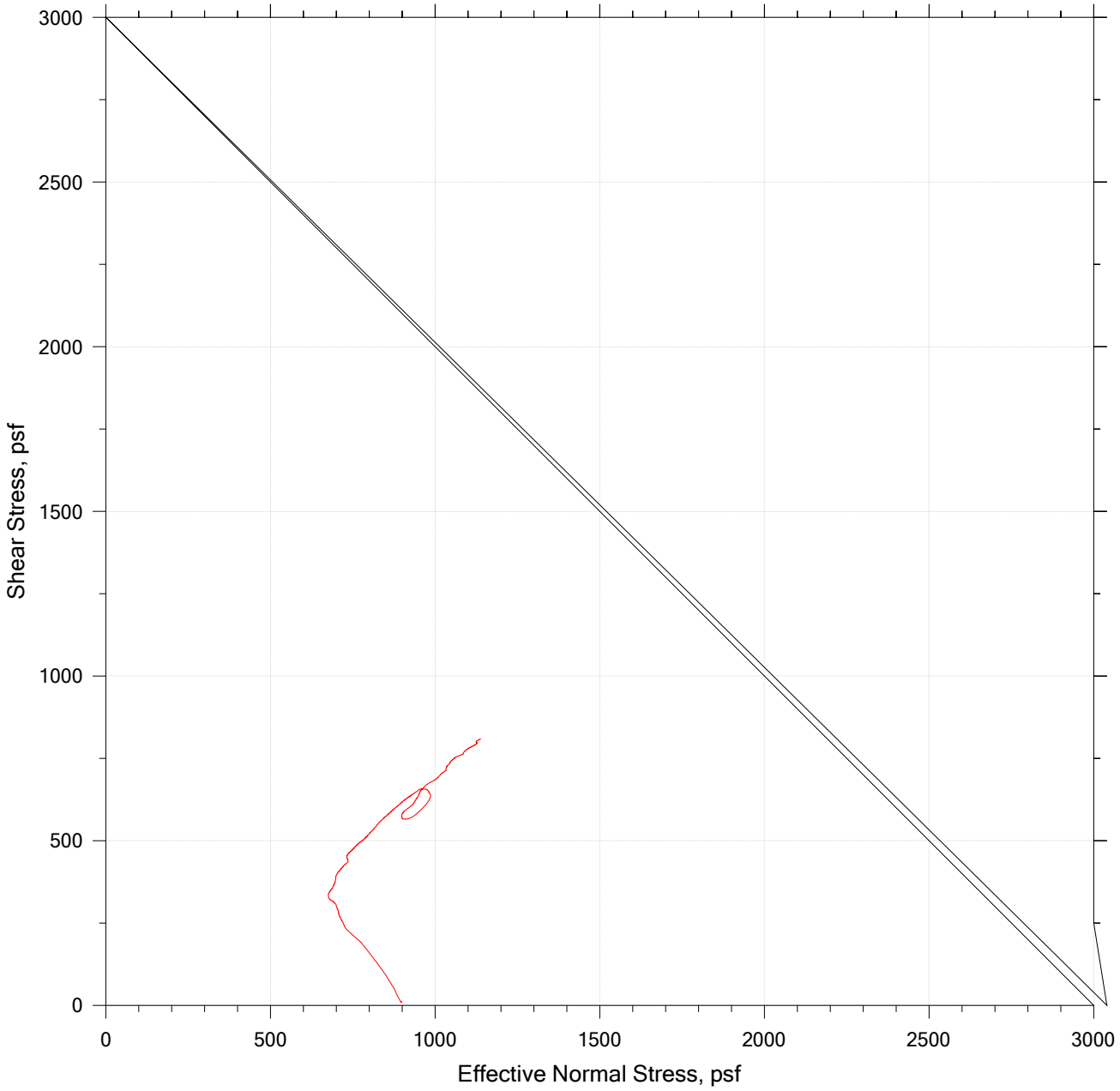
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

Direct Simple Shear Test by ASTM D6528



Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
Boring Number: HB-BHR-108	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
Test Number: DSS-1	Preparation: intact	Elevation: ---
Description: Moist, olive clay		
Remarks:		

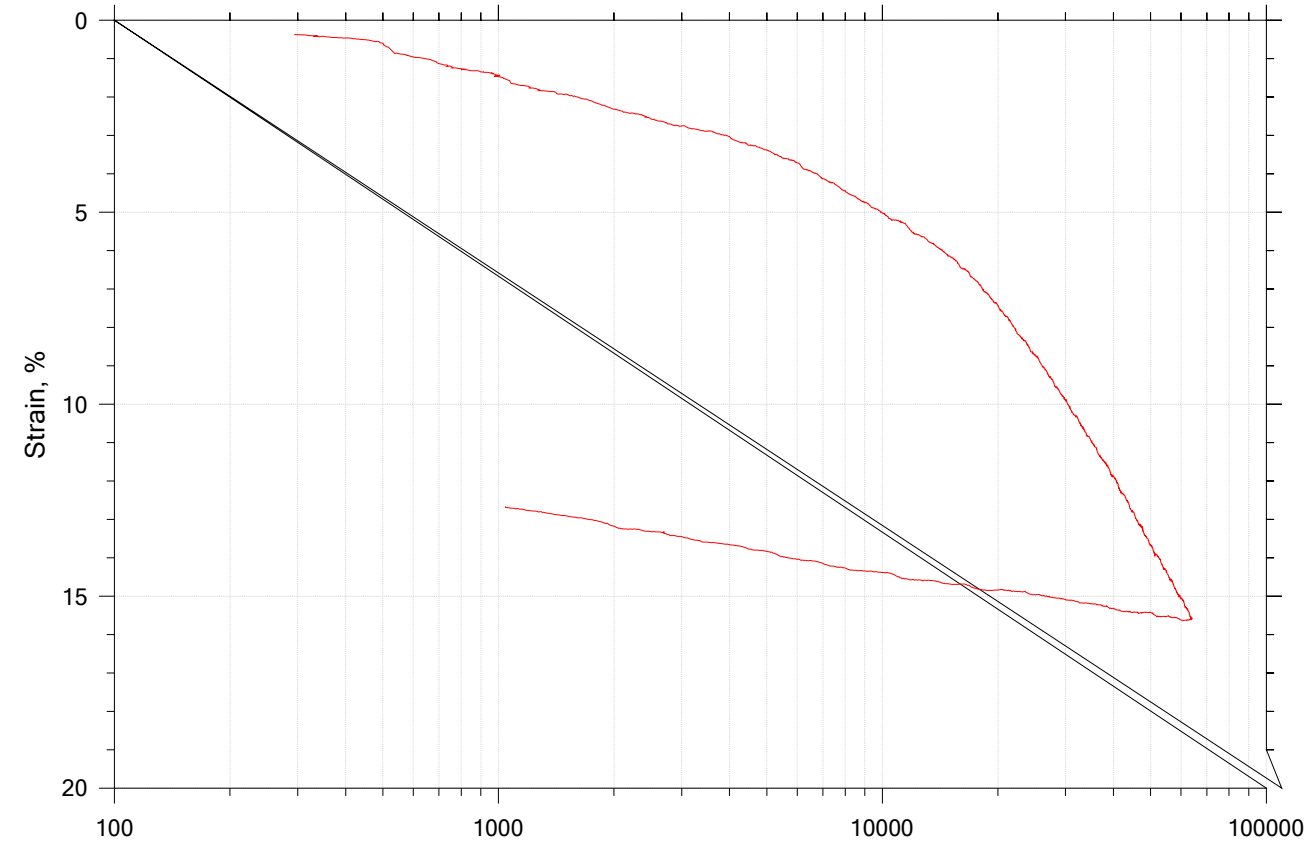
Direct Simple Shear Test by ASTM D6528

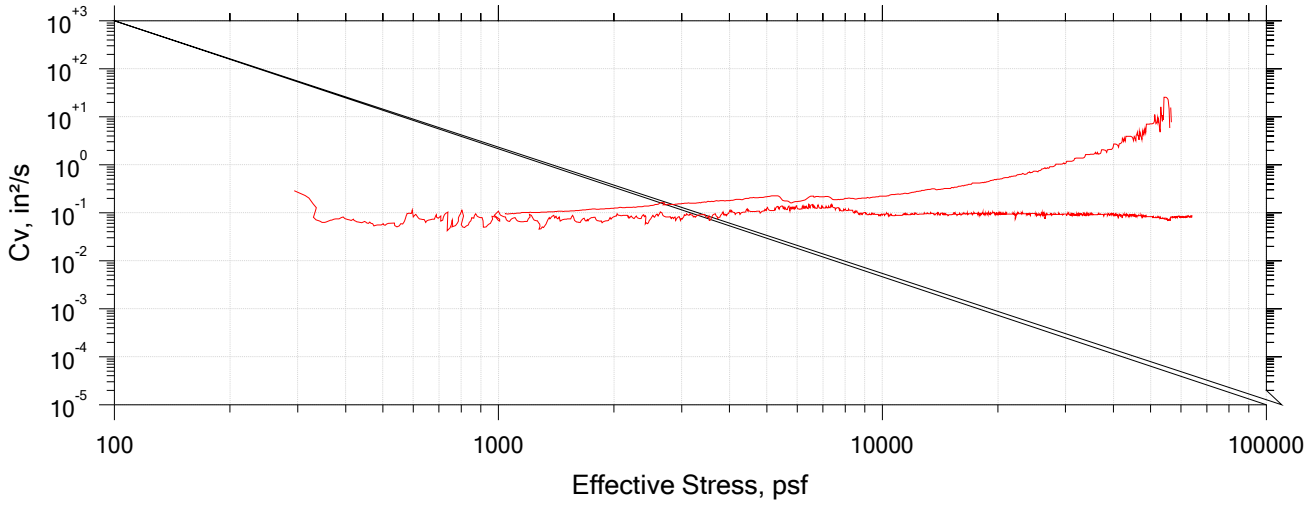


GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: DSS-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks:		

CRC Test

Summary

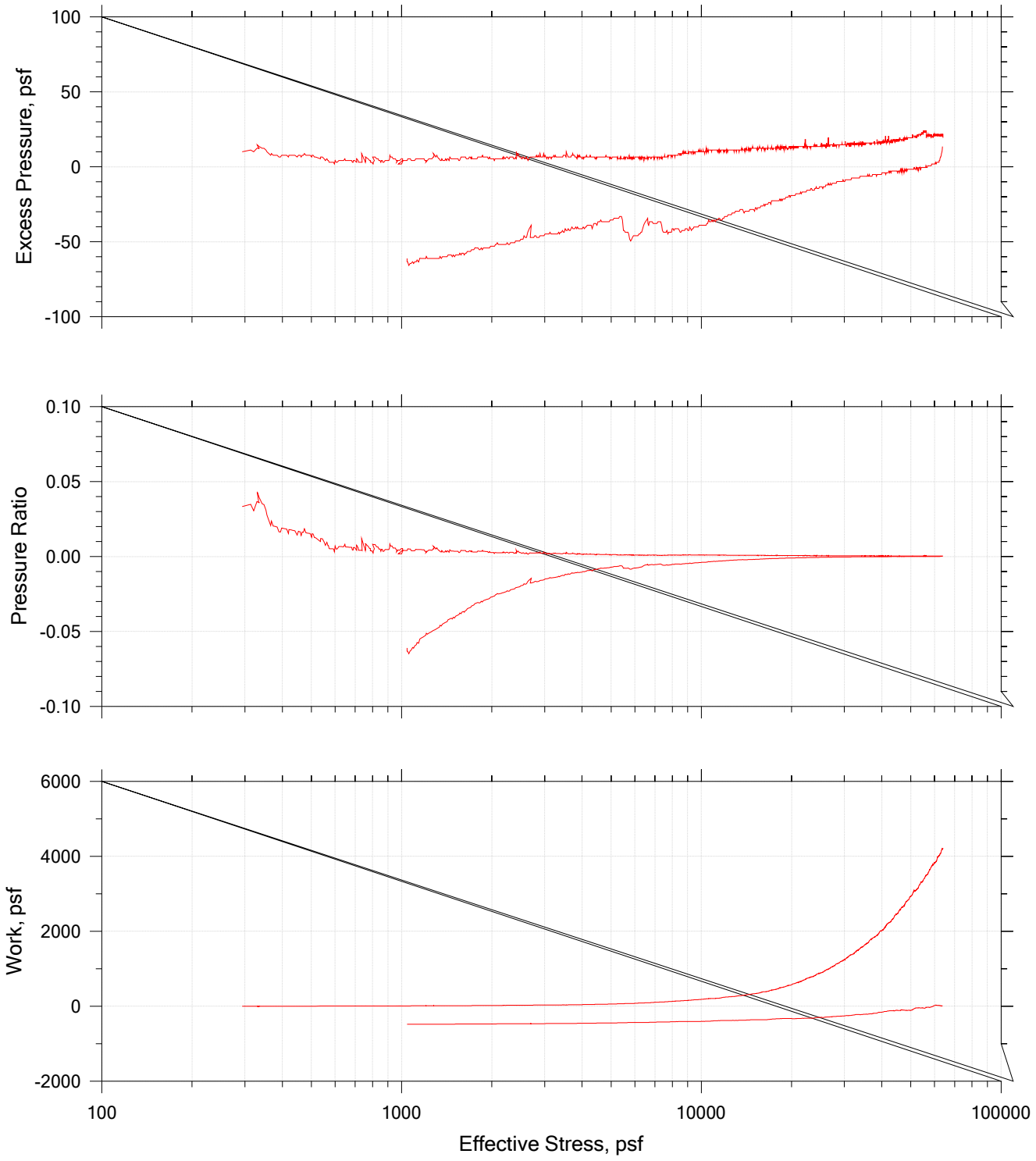




GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: ---		

CRC Test

Pressure Curves



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: ---		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.77 (Estimated)	Liquid Limit: 36
Specimen Height, in: 1.00	Initial Void Ratio: 0.871	Plastic Limit: 21
Final Height, in: 0.87	Final Void Ratio: 0.628	Plasticity Index: 15

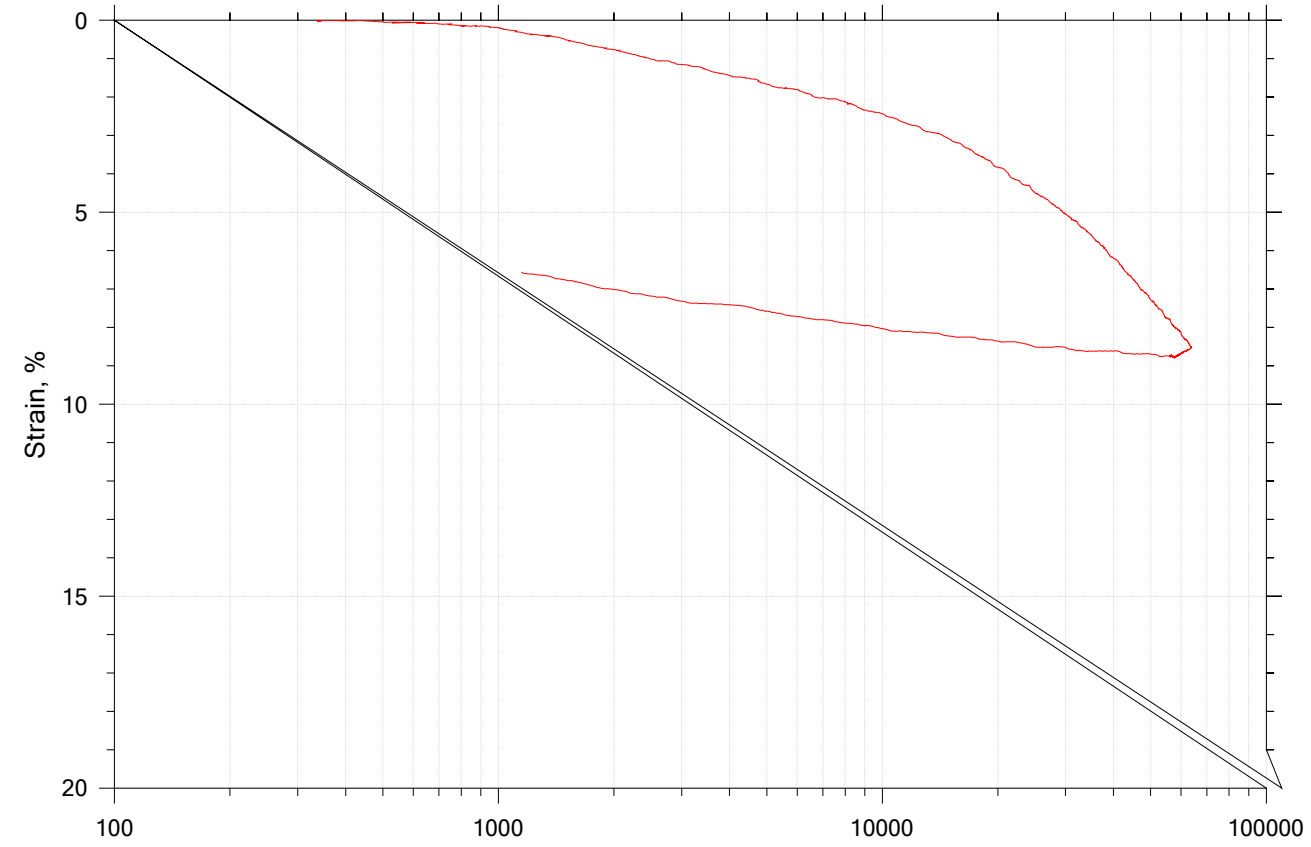
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-1859	---		E4886
Mass Container, gm	8.61	108.88	108.88	8.05
Mass Container + Wet Soil, gm	74.09	265.01	255.14	157.65
Mass Container + Dry Soil, gm	58.93	228.16	228.16	130.05
Mass Dry Soil, gm	50.32	119.28	119.28	122
Water Content, %	30.13	30.90	22.62	22.62
Void Ratio	---	0.87	0.63	---
Degree of Saturation, %	---	98.43	100.00	---
Dry Unit Weight, pcf	---	92.568	106.4	---

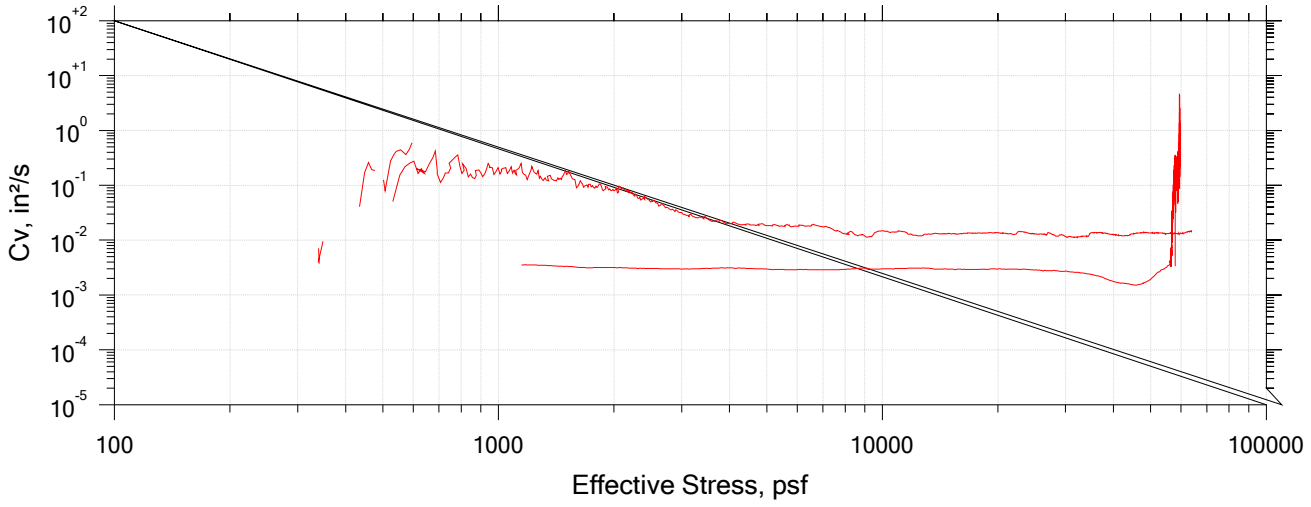
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: ---		

CRC Test

Summary

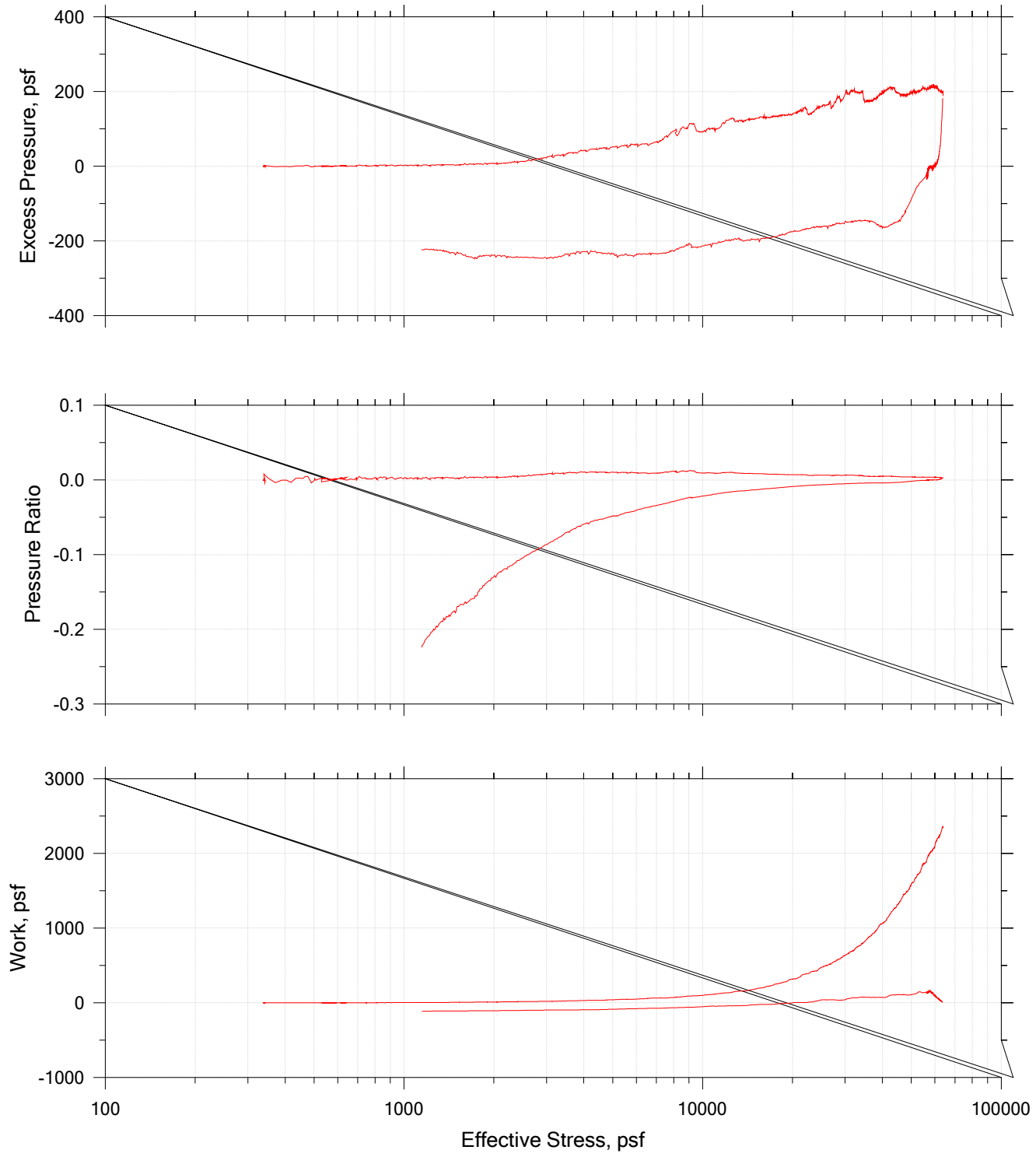




GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: TX-017		

CRC Test

Pressure Curves



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: TX-017		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.75 (Estimated)	Liquid Limit: 39
Specimen Height, in: 1.00	Initial Void Ratio: 0.724	Plastic Limit: 22
Final Height, in: 0.92	Final Void Ratio: 0.586	Plasticity Index: 17

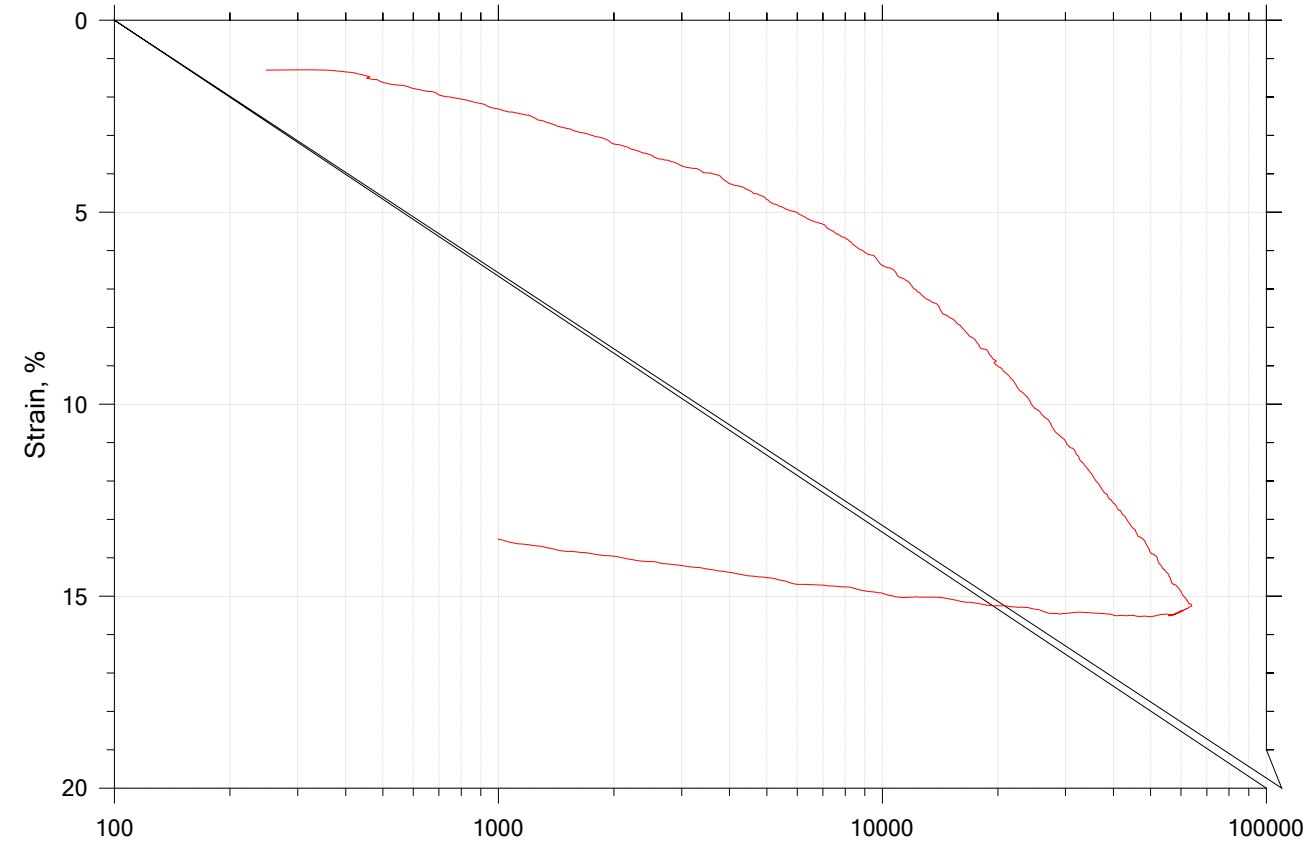
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-2107	---		E1885
Mass Container, gm	8.7	110.07	110.07	8.5
Mass Container + Wet Soil, gm	163.2	271.82	265.69	167.49
Mass Container + Dry Soil, gm	132.66	238.35	238.35	139.56
Mass Dry Soil, gm	123.96	128.28	128.28	131.06
Water Content, %	24.64	26.09	21.31	21.31
Void Ratio	---	0.72	0.59	---
Degree of Saturation, %	---	99.10	100.00	---
Dry Unit Weight, pcf	---	99.557	108.21	---

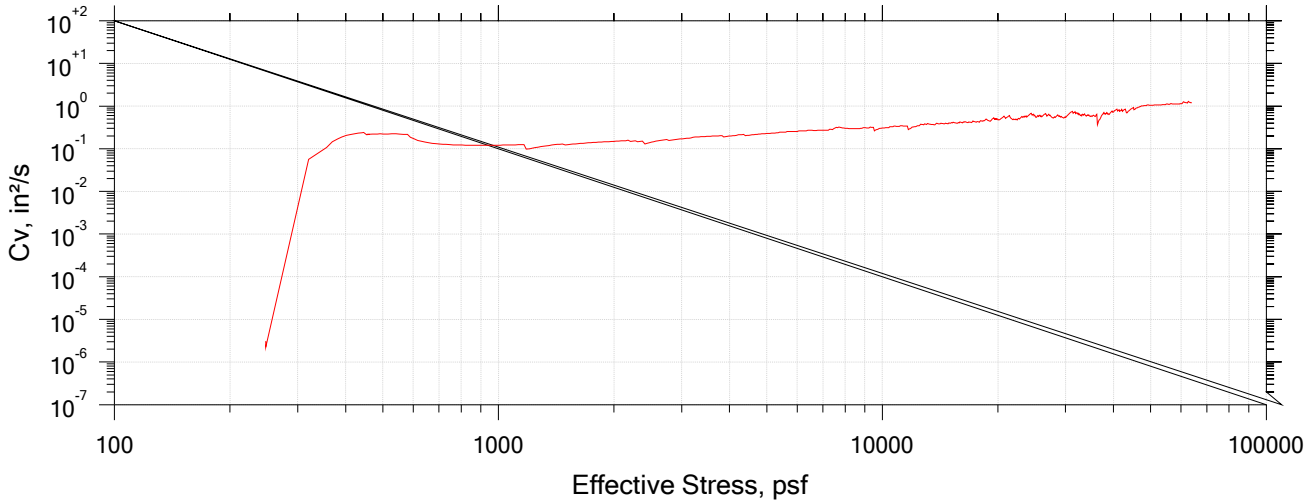
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: TX-017		

CRC Test

Summary

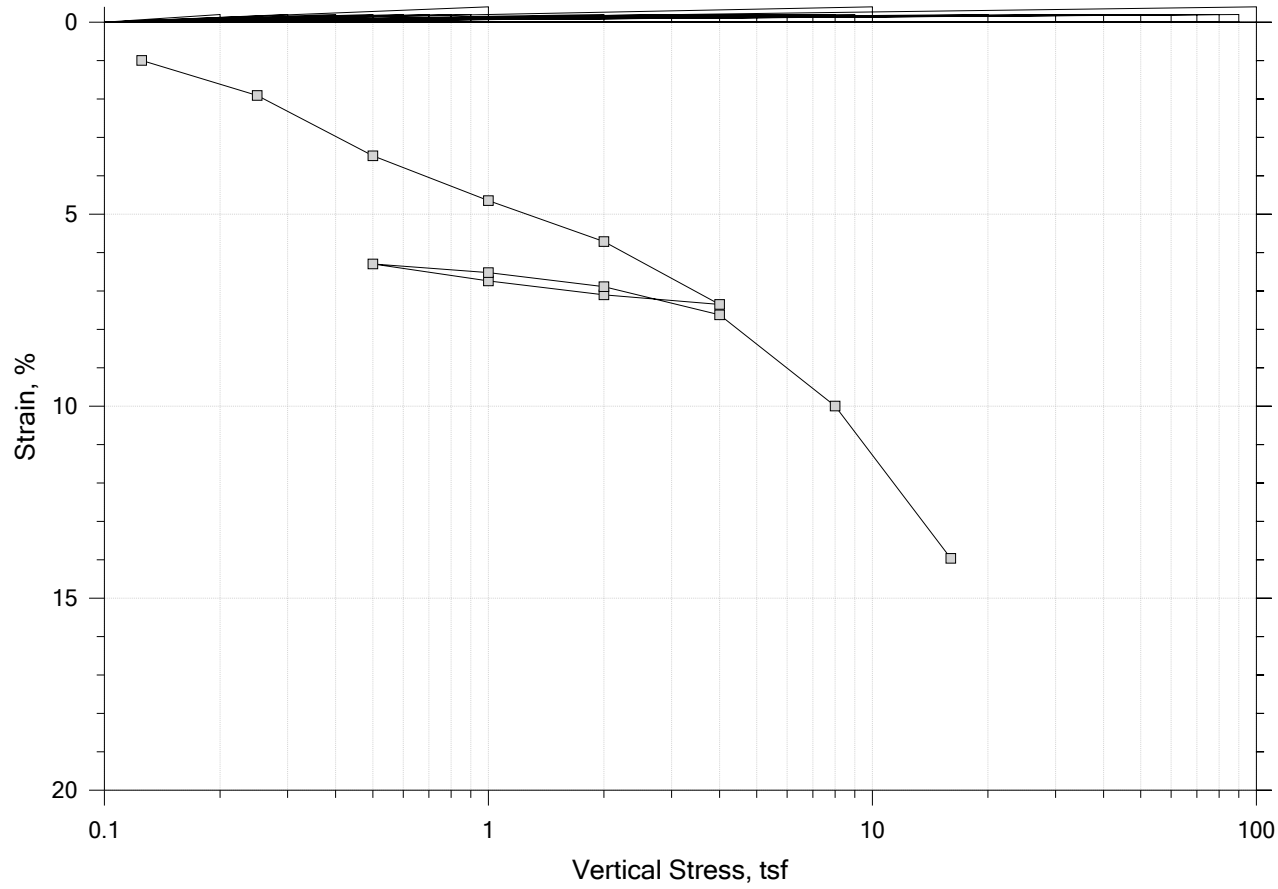


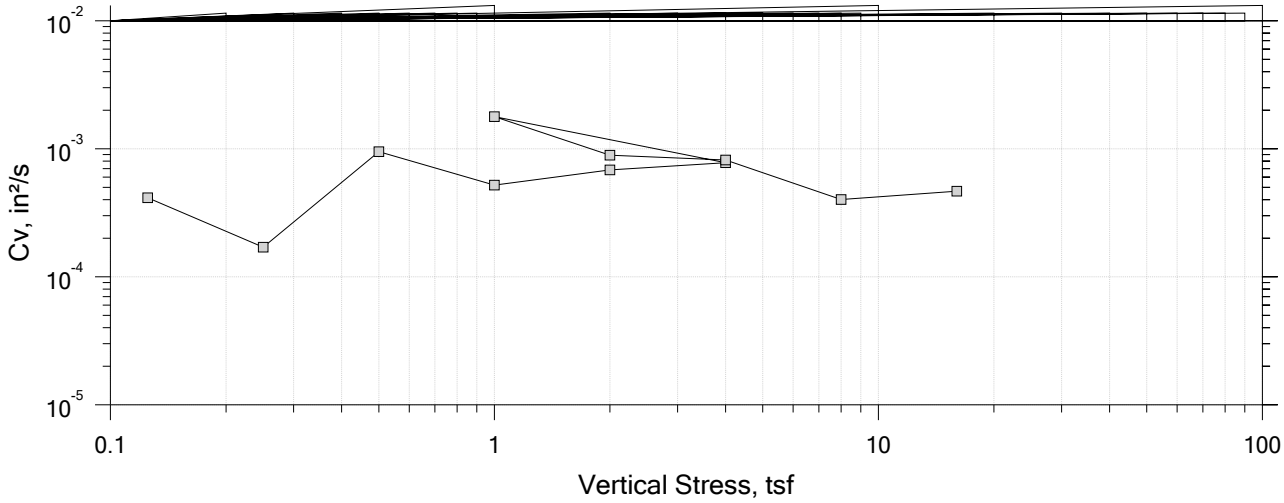


GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System JJ		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

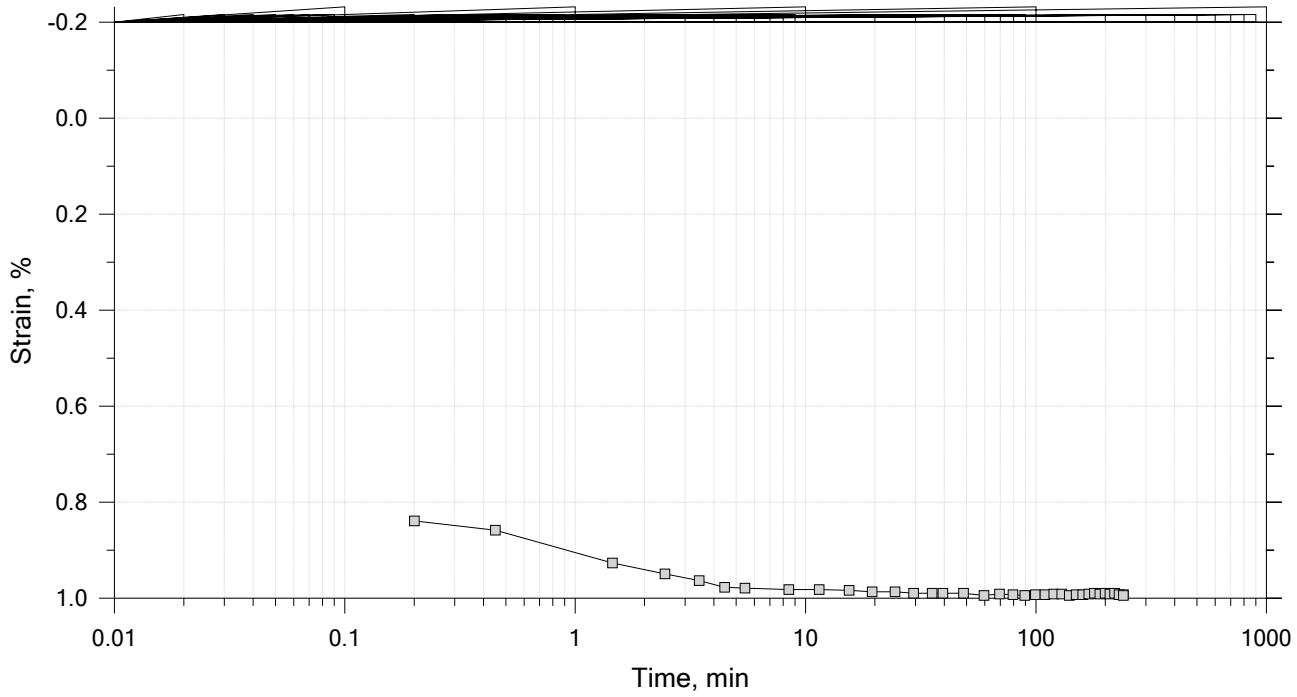


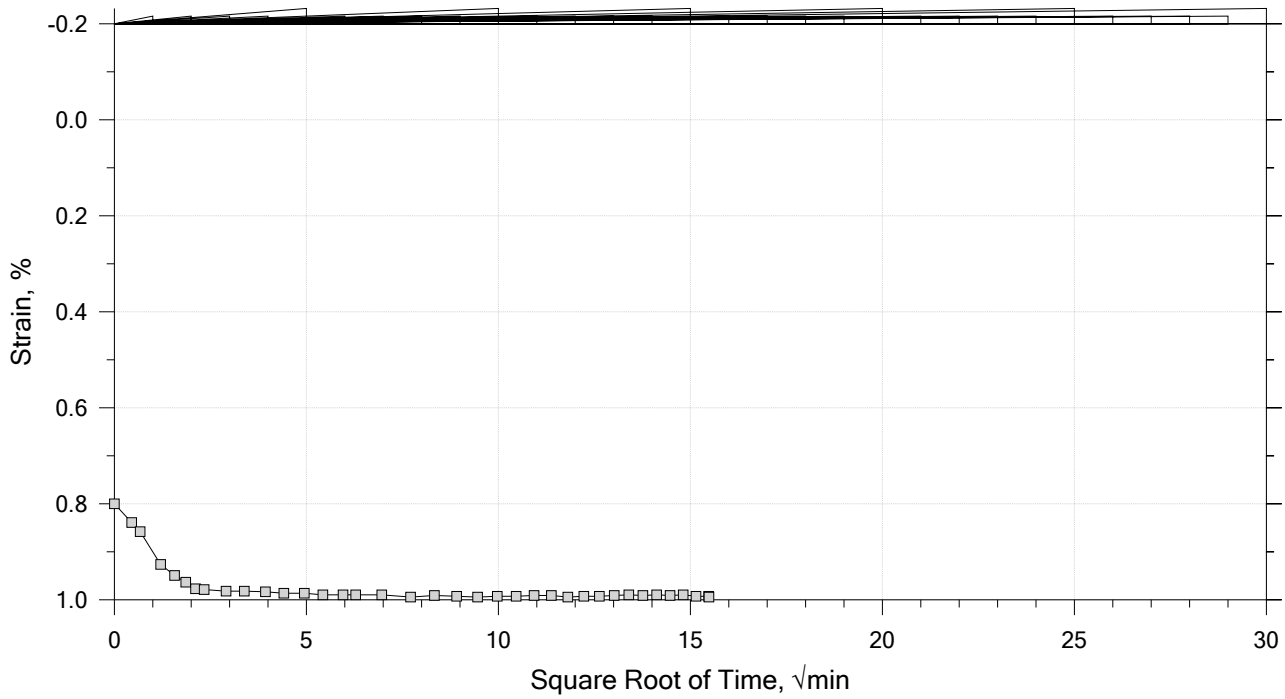


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14
Constant Load Step
Stress: 0.125 tsf

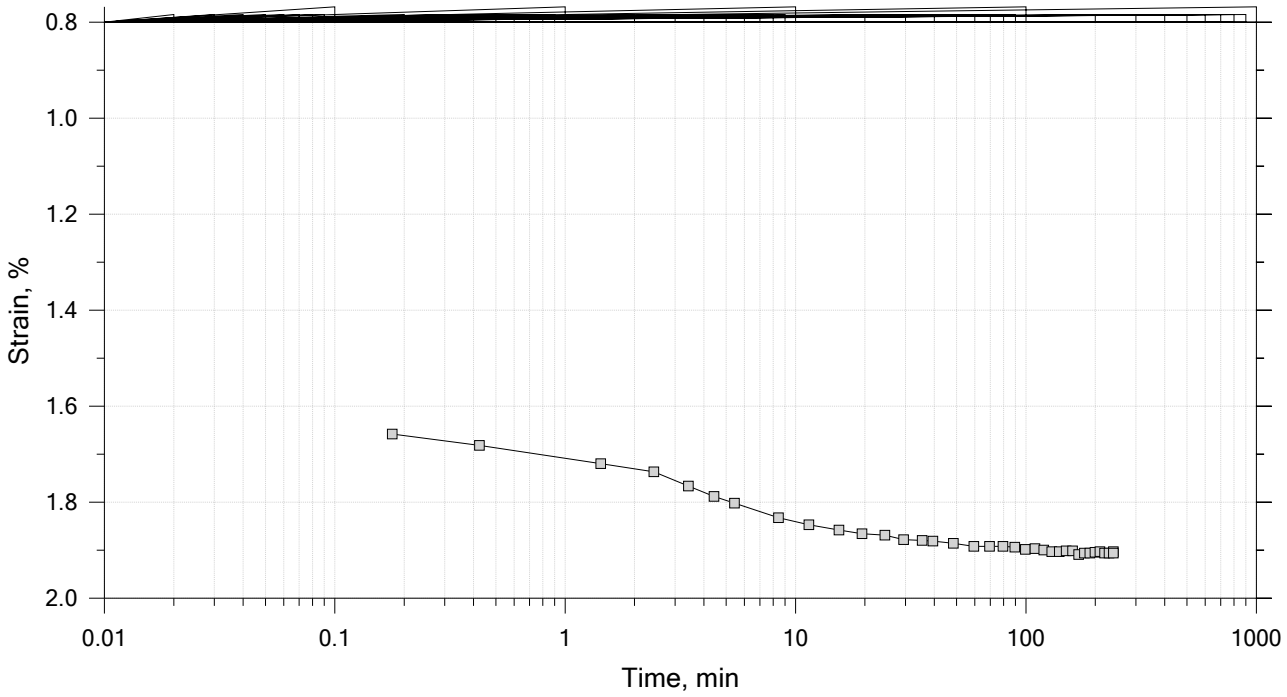


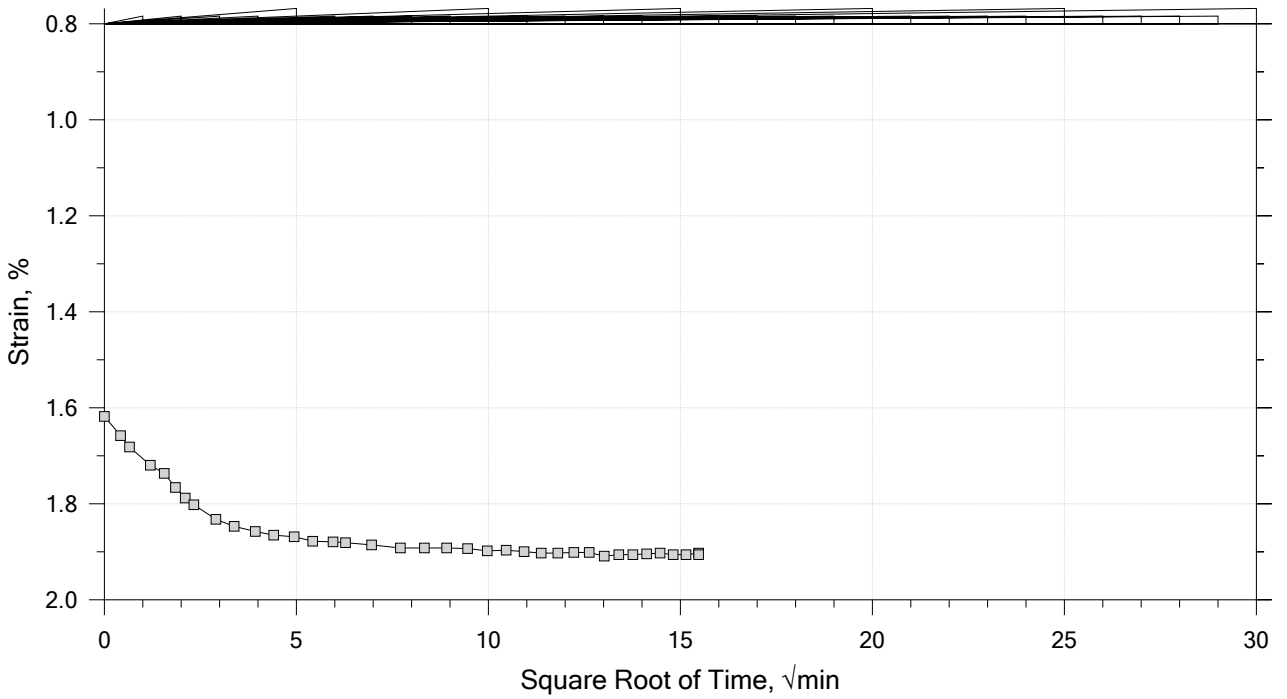


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf

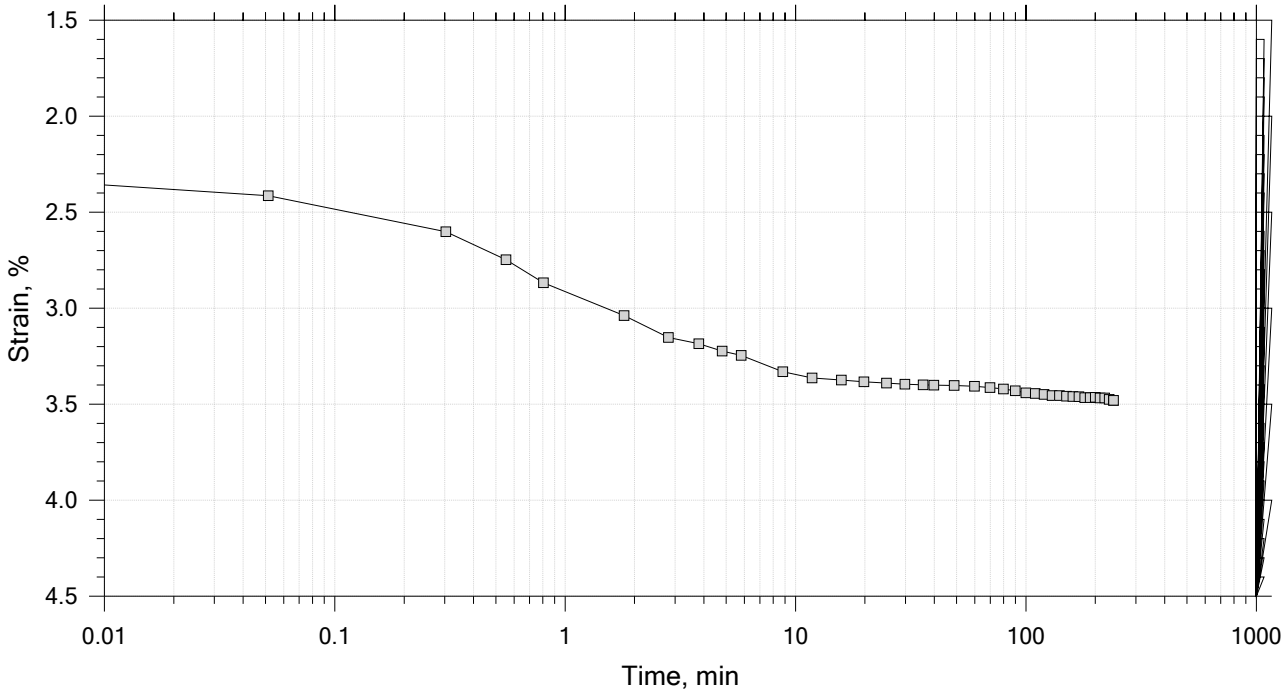


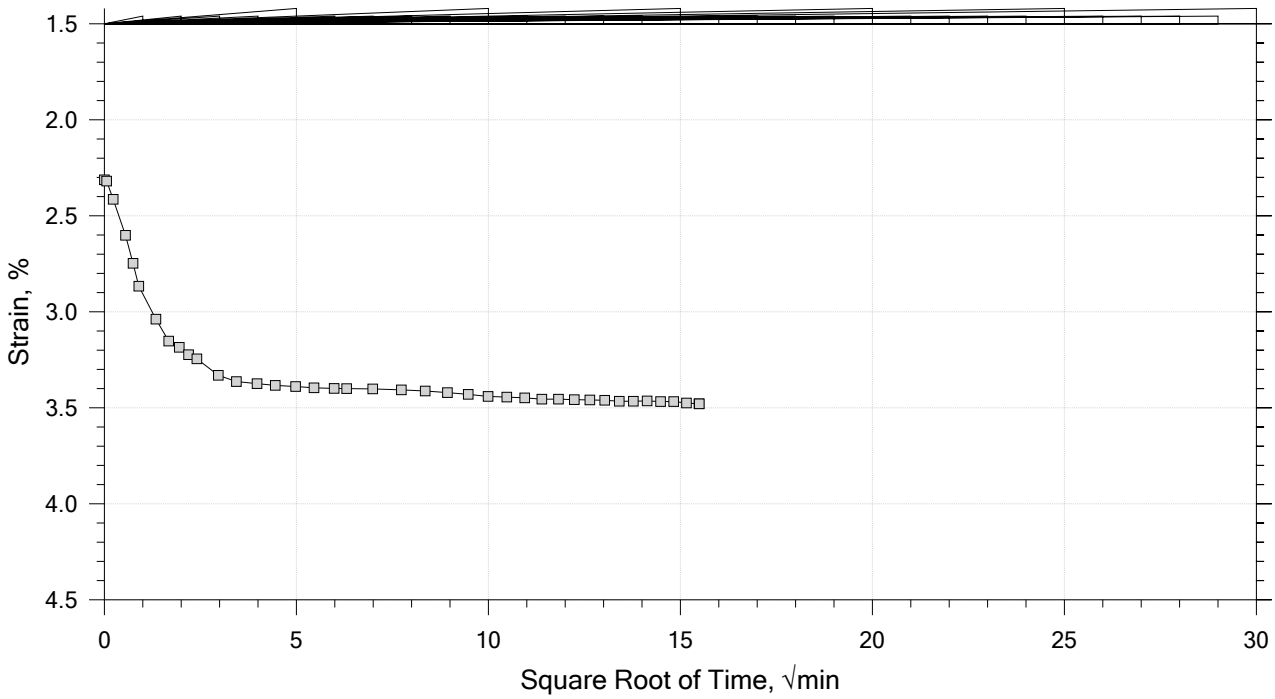


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf





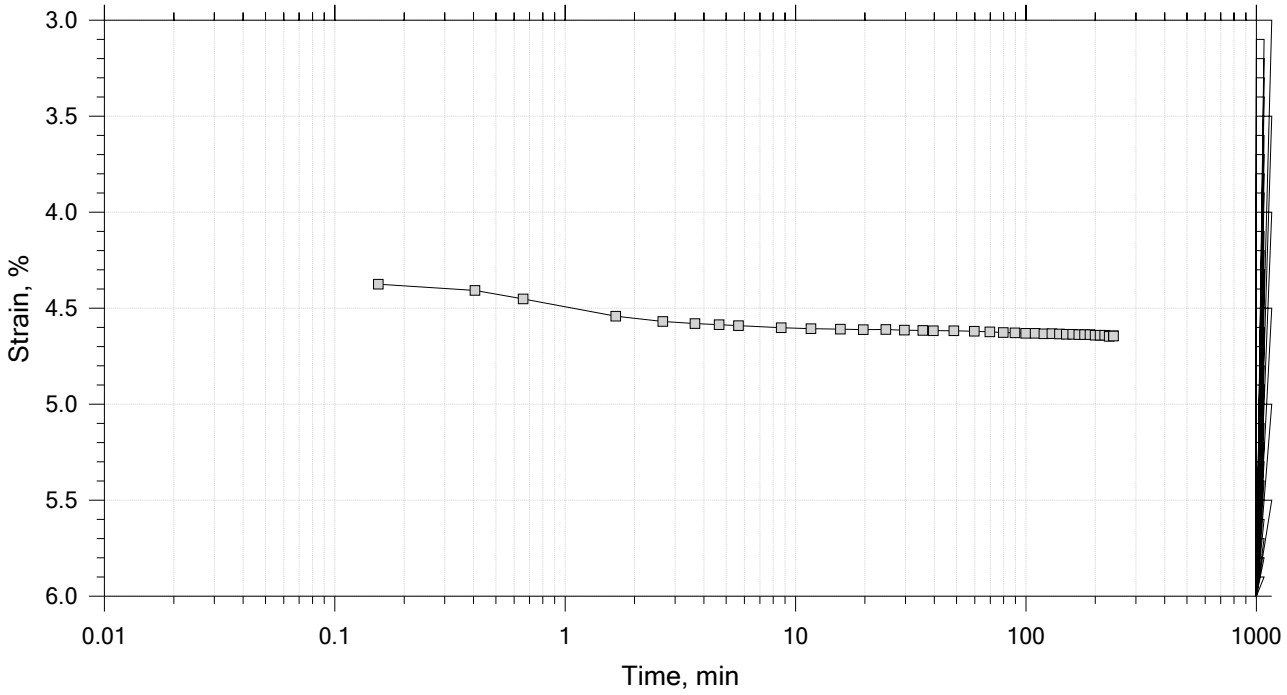
Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-G		

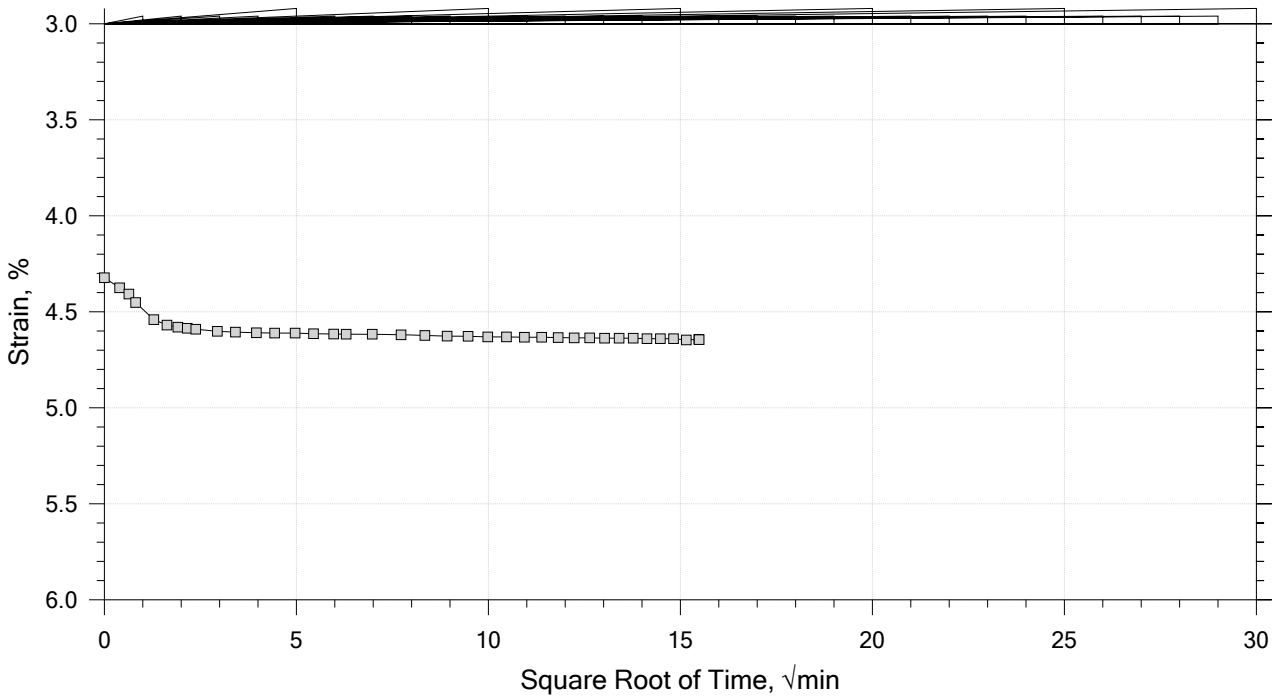
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 1 tsf





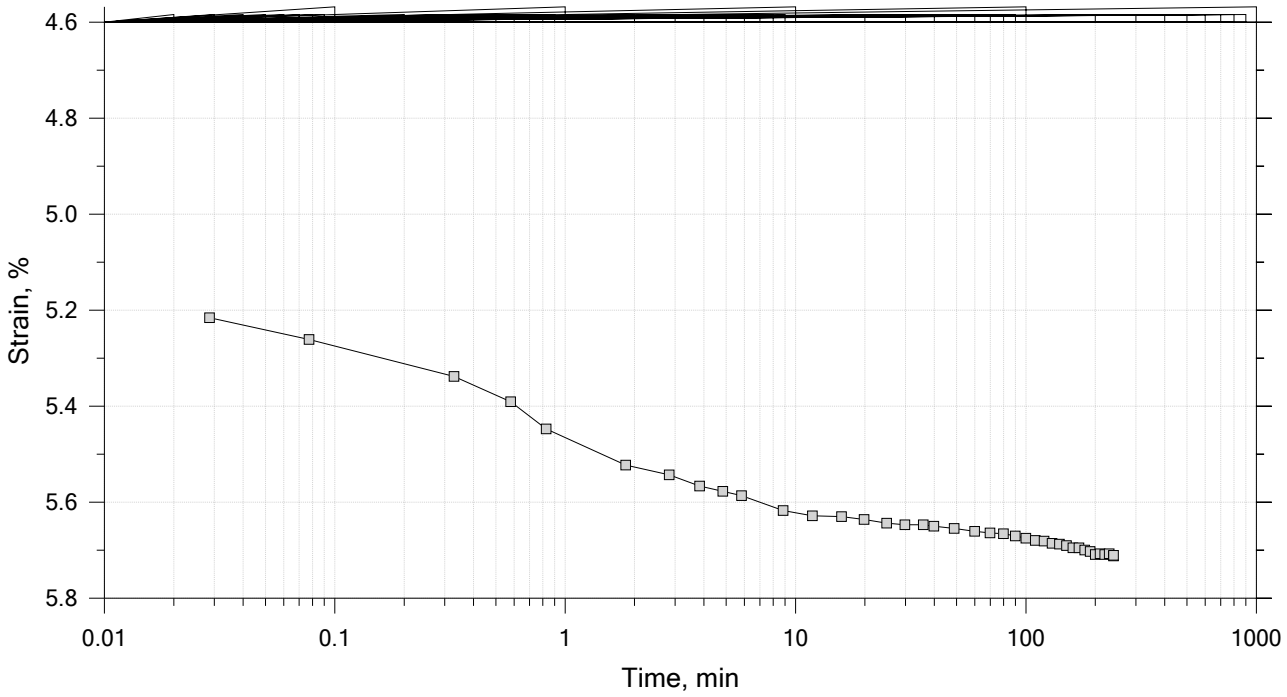
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

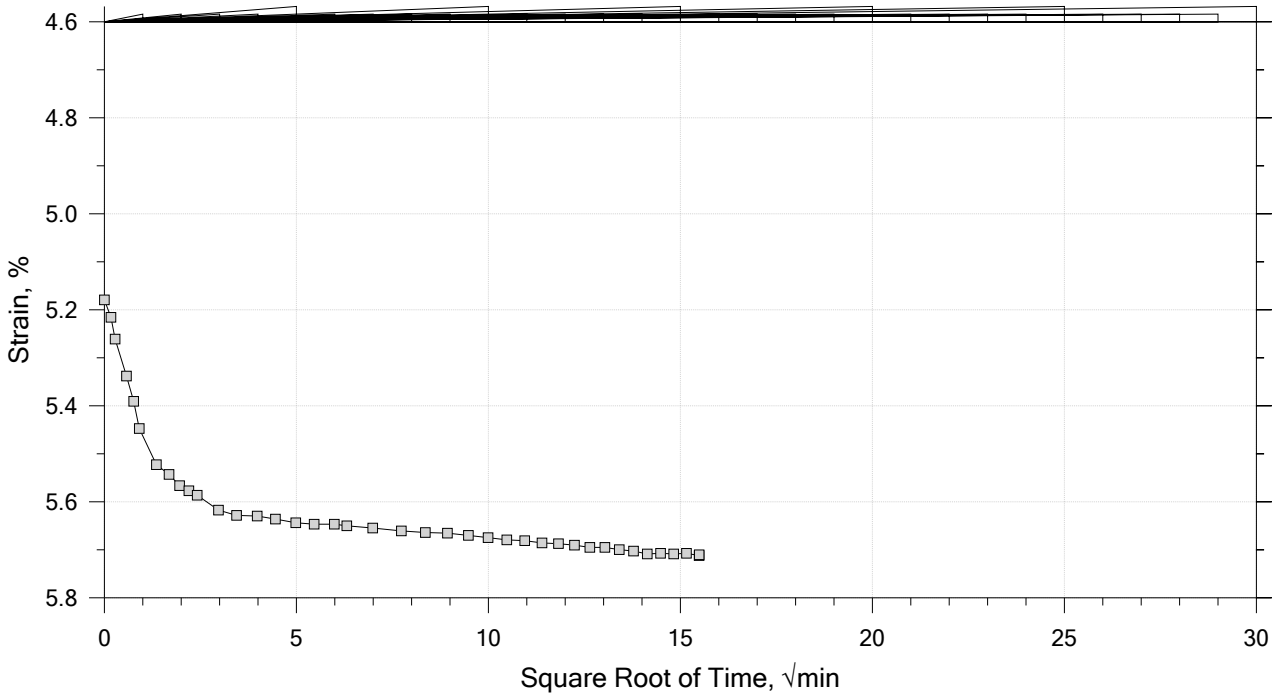
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14

Constant Load Step

Stress: 2 tsf





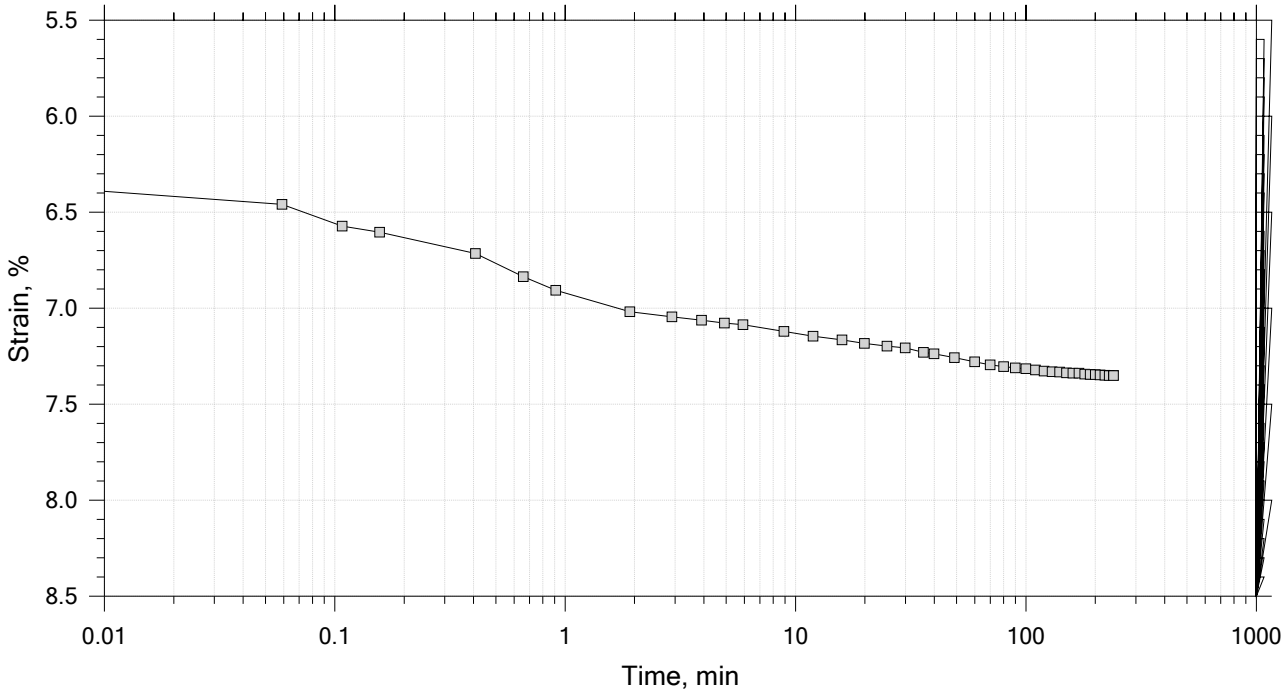
Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-G		

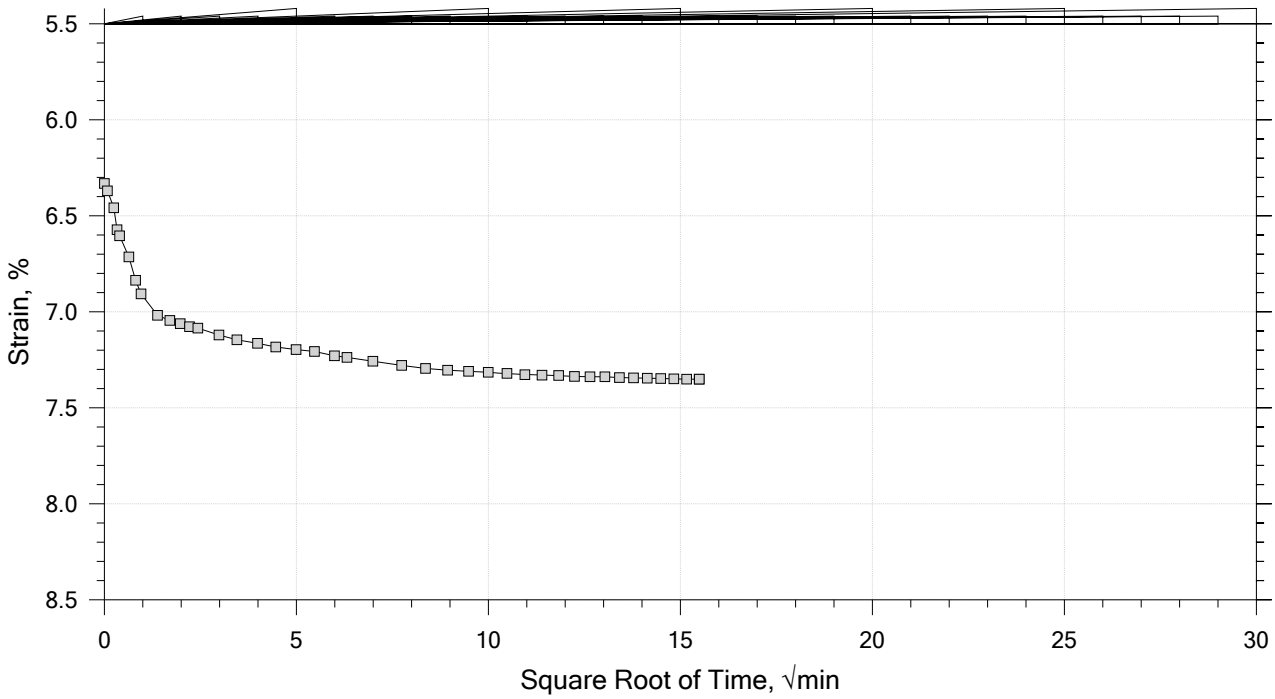
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 4 tsf

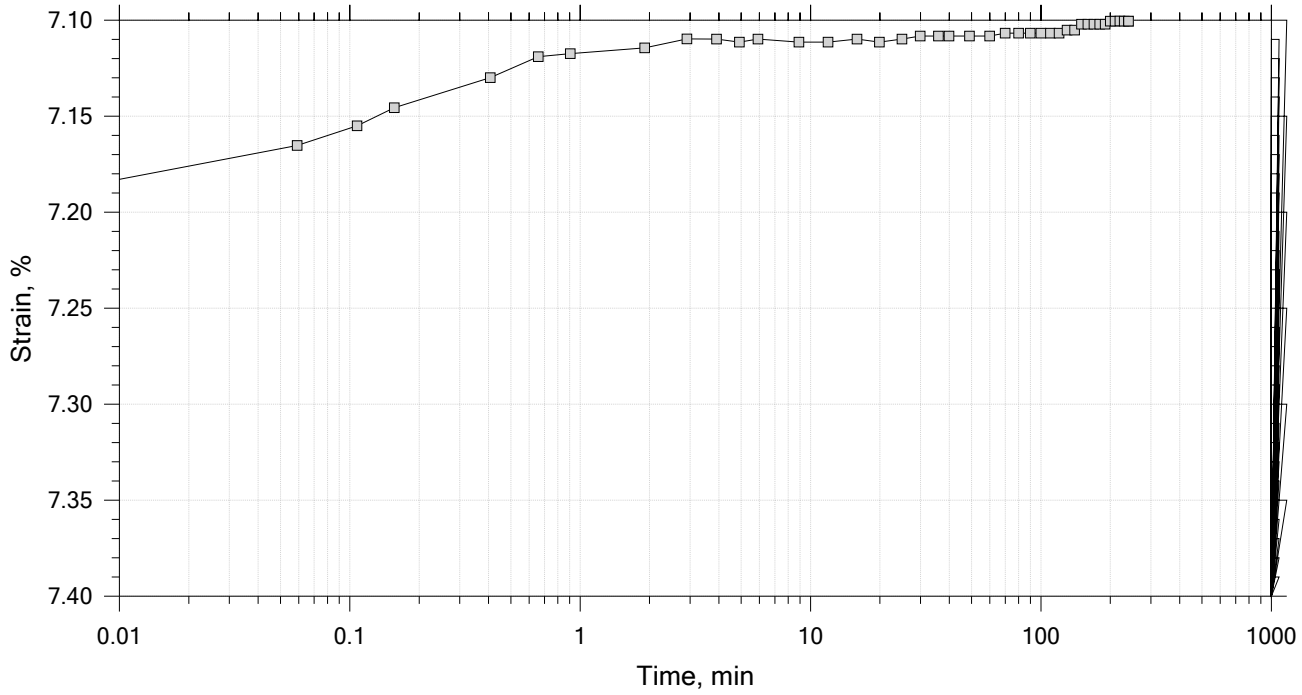


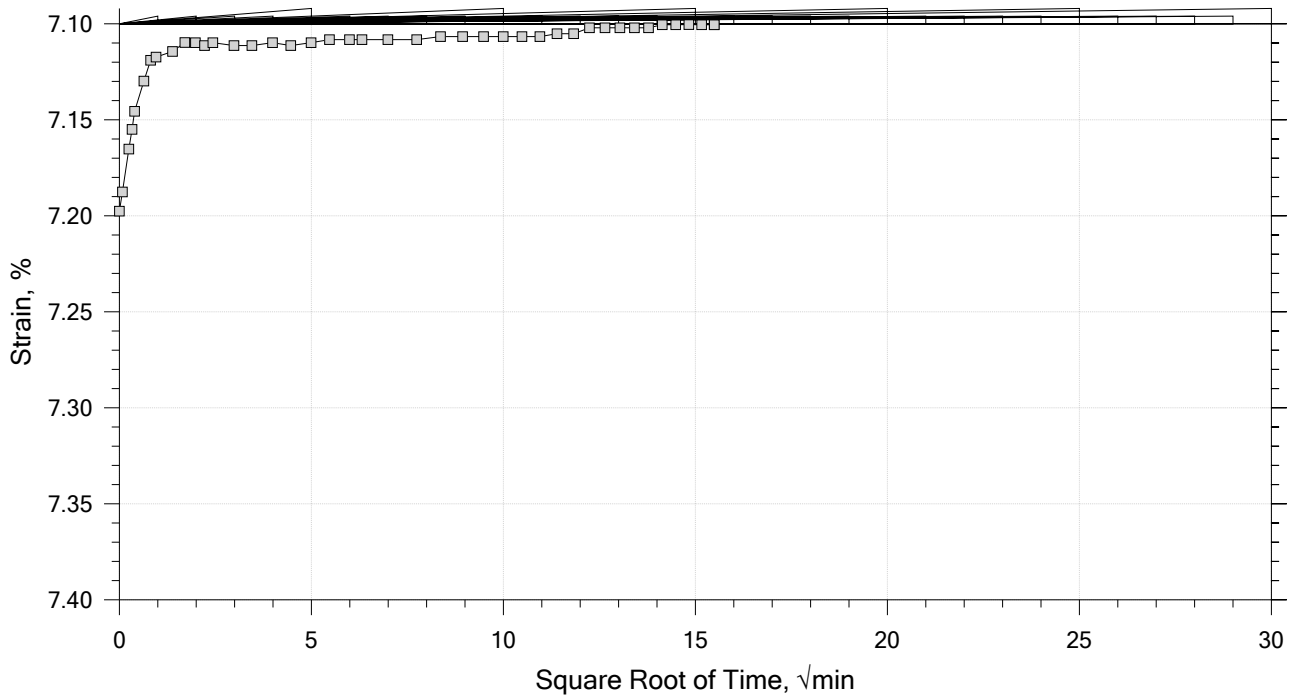


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14
Constant Load Step
Stress: 2 tsf

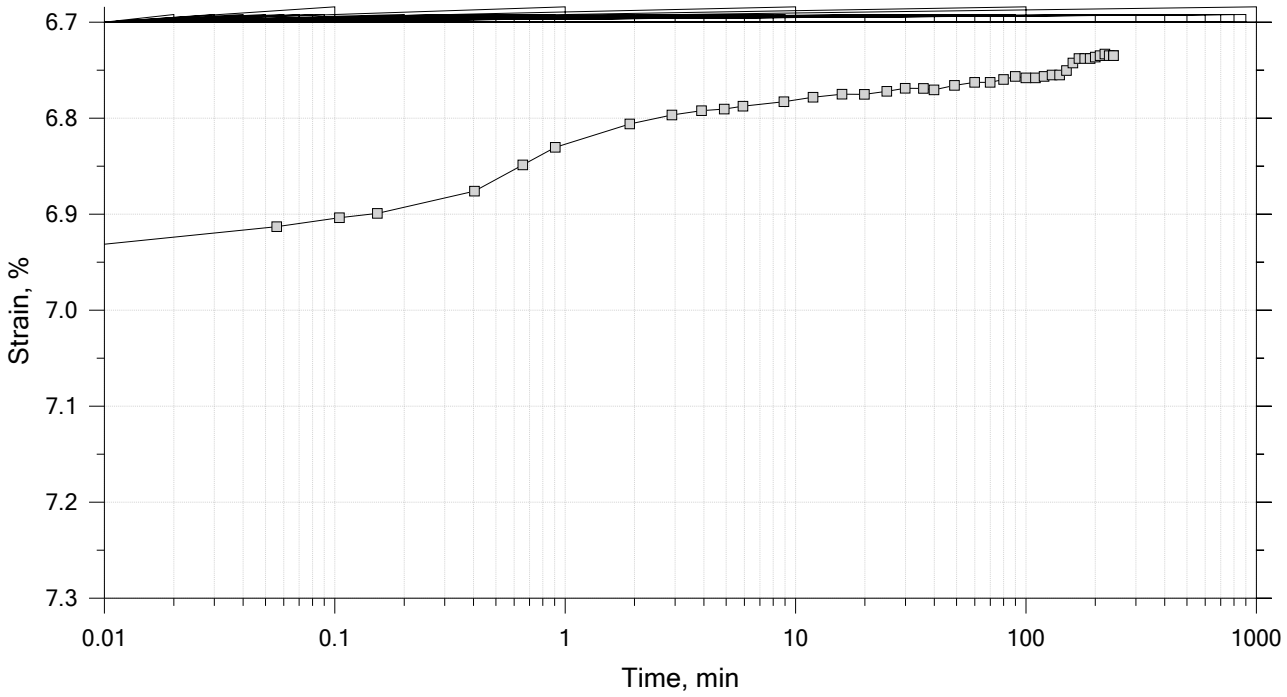


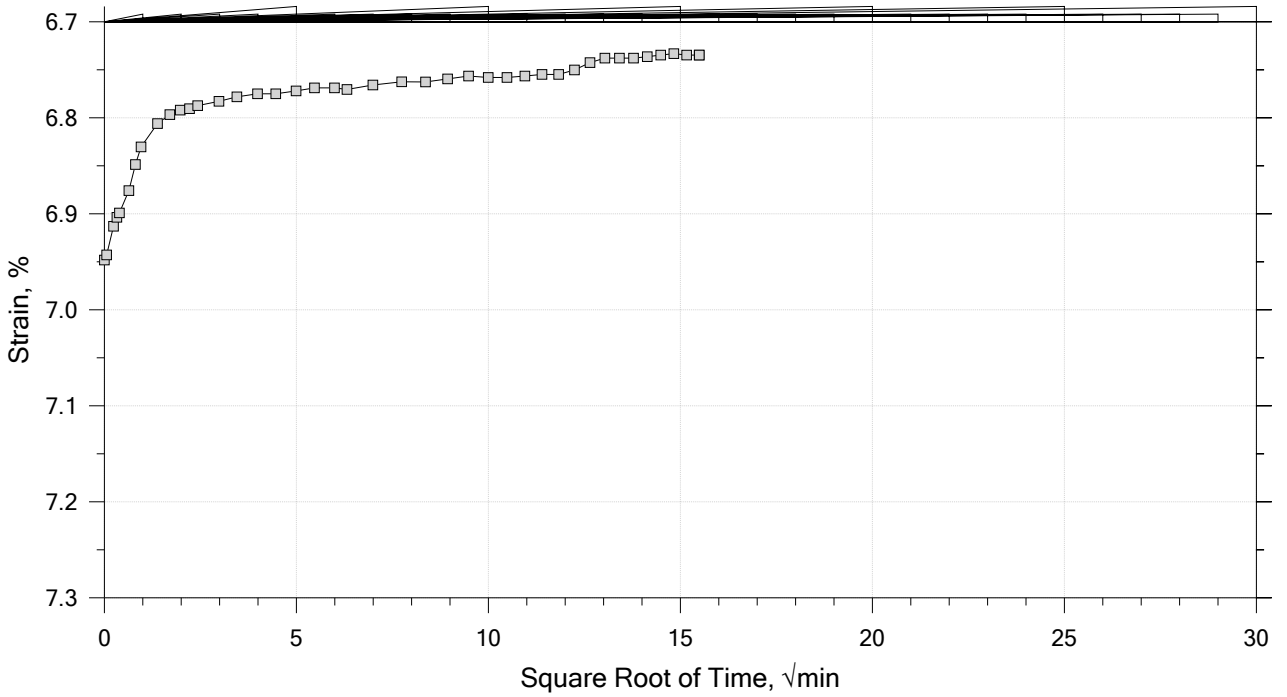


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14
Constant Load Step
Stress: 1 tsf

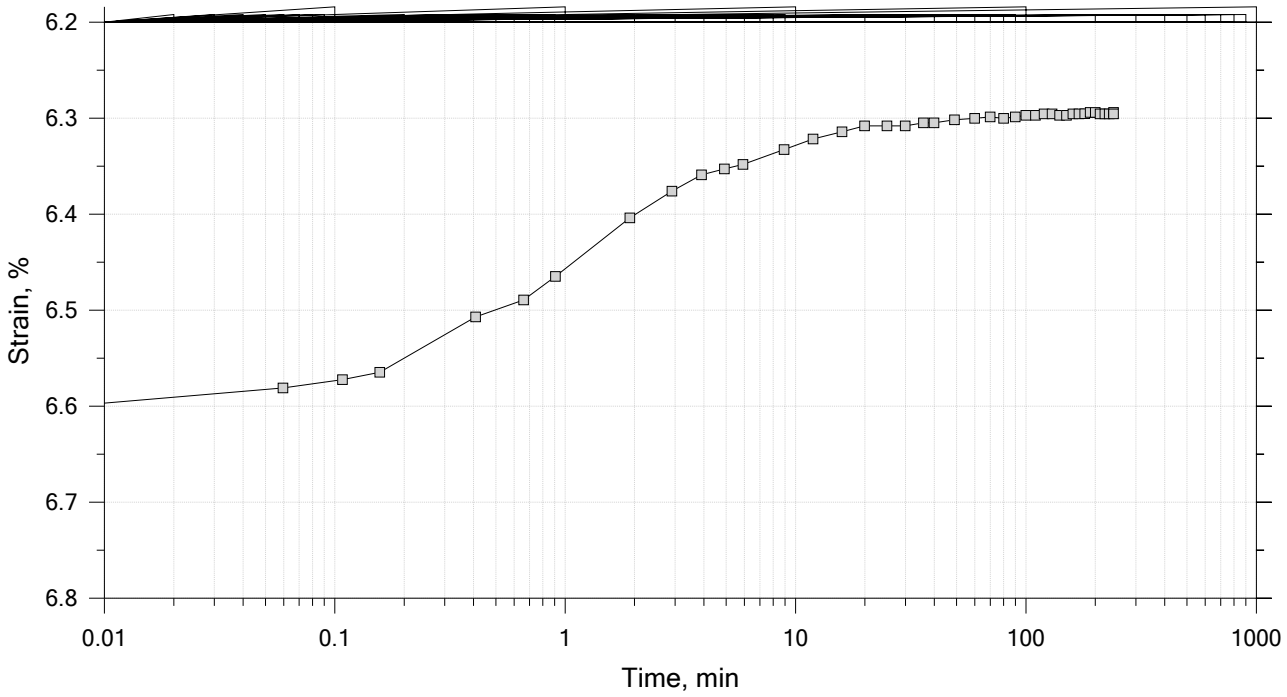


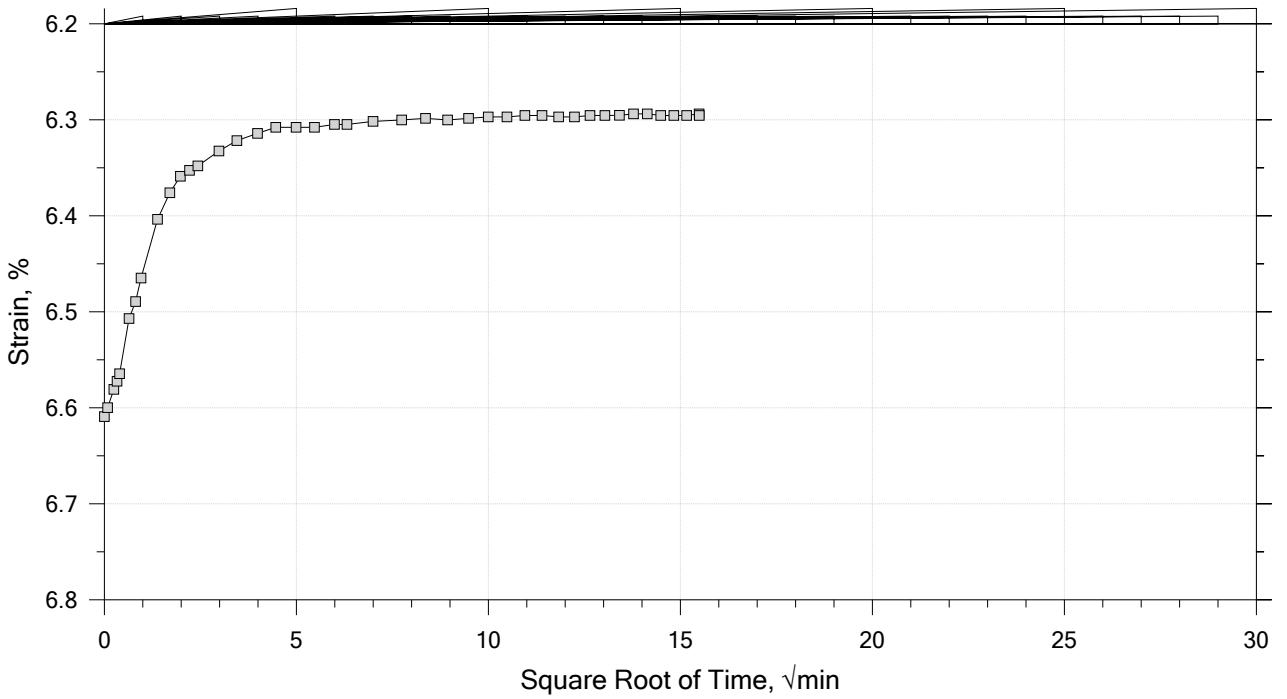


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

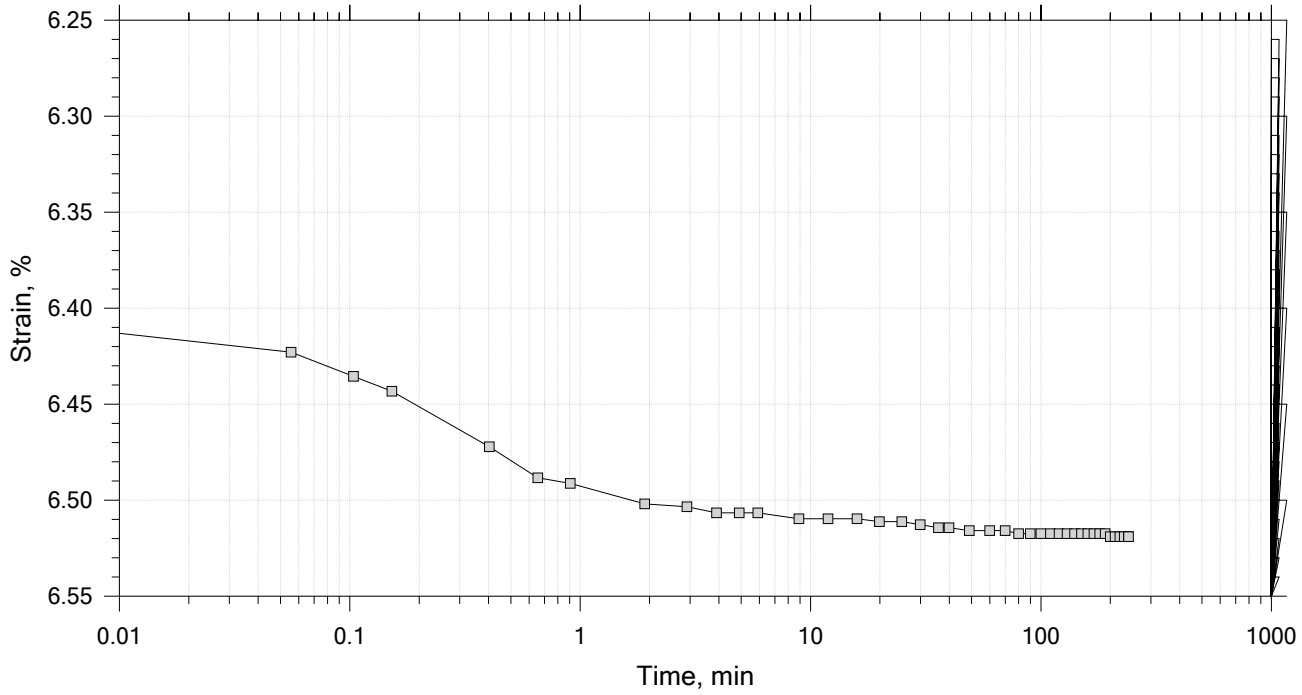


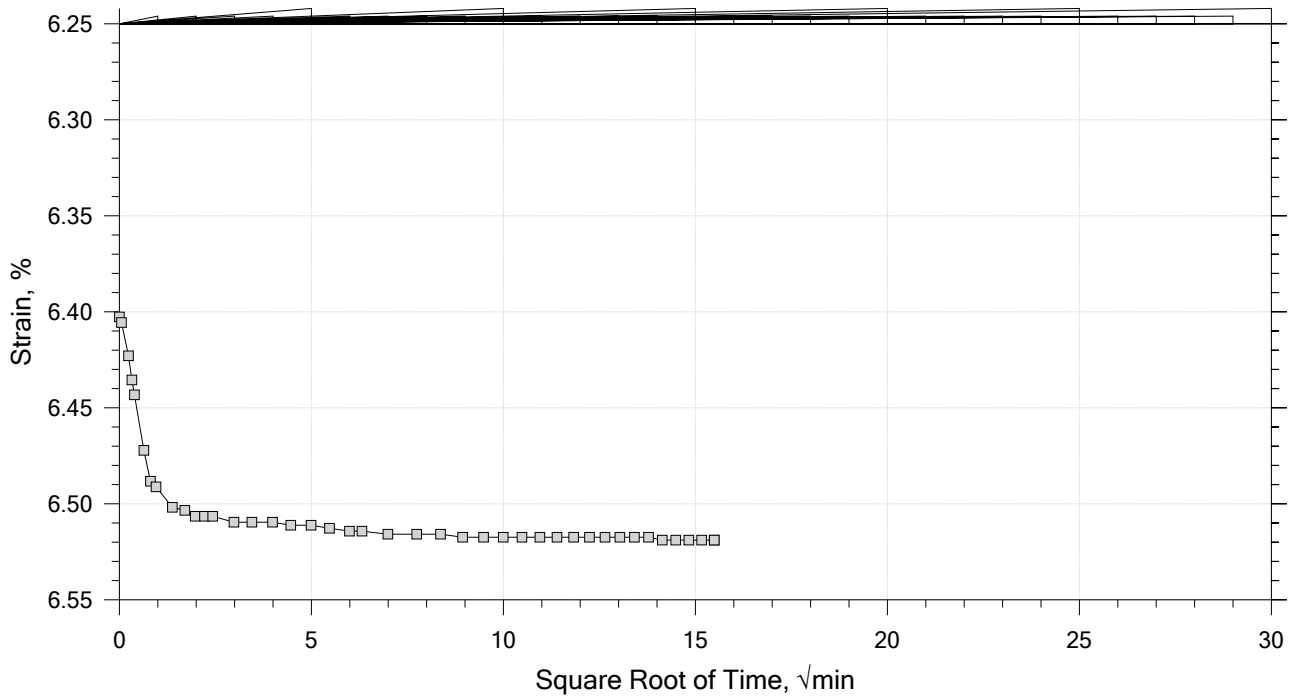


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

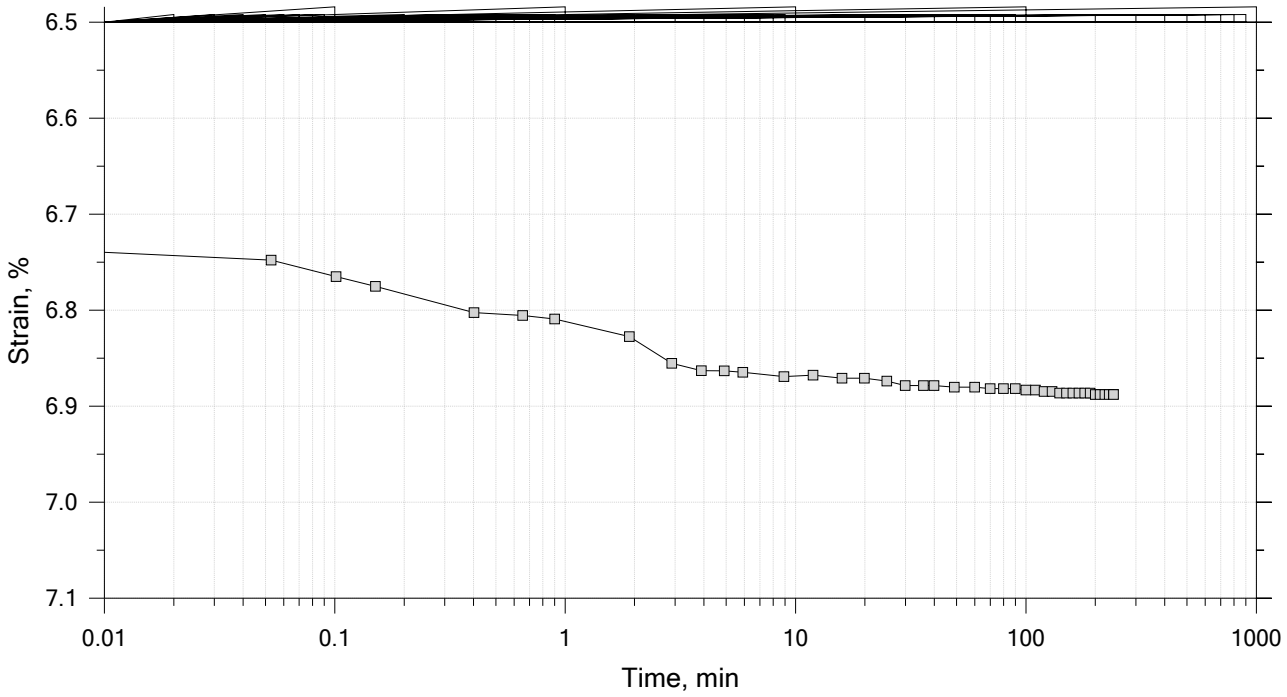


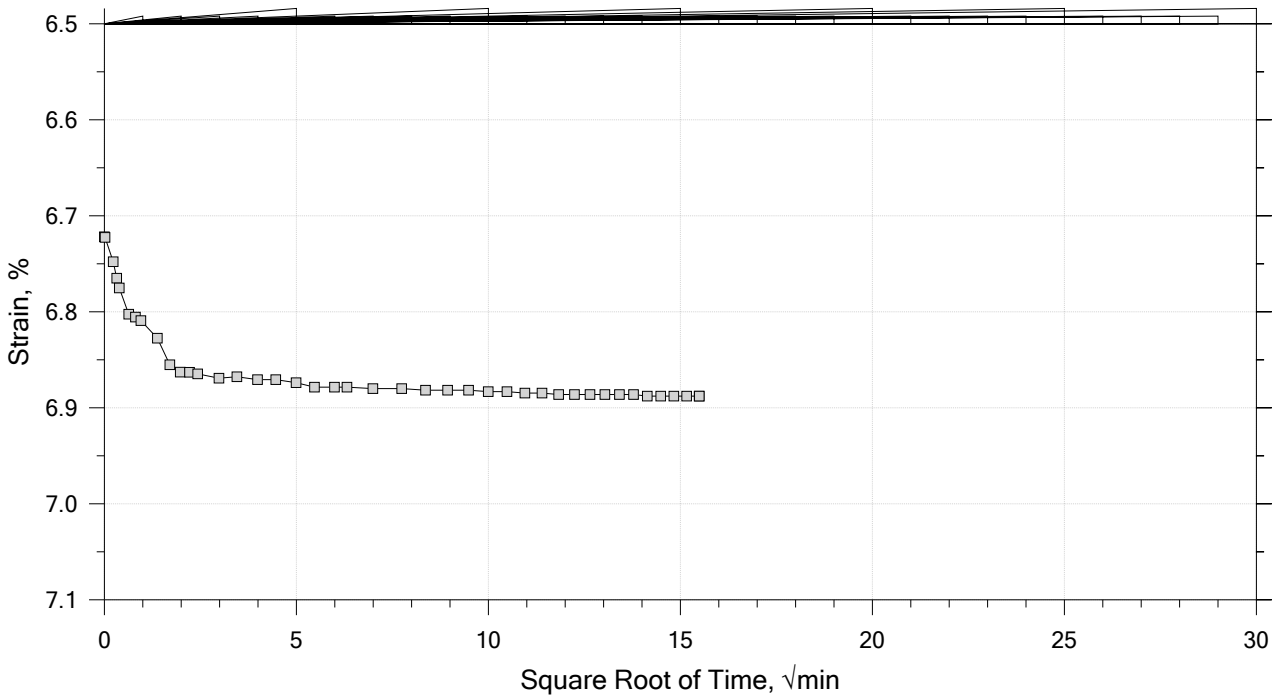


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

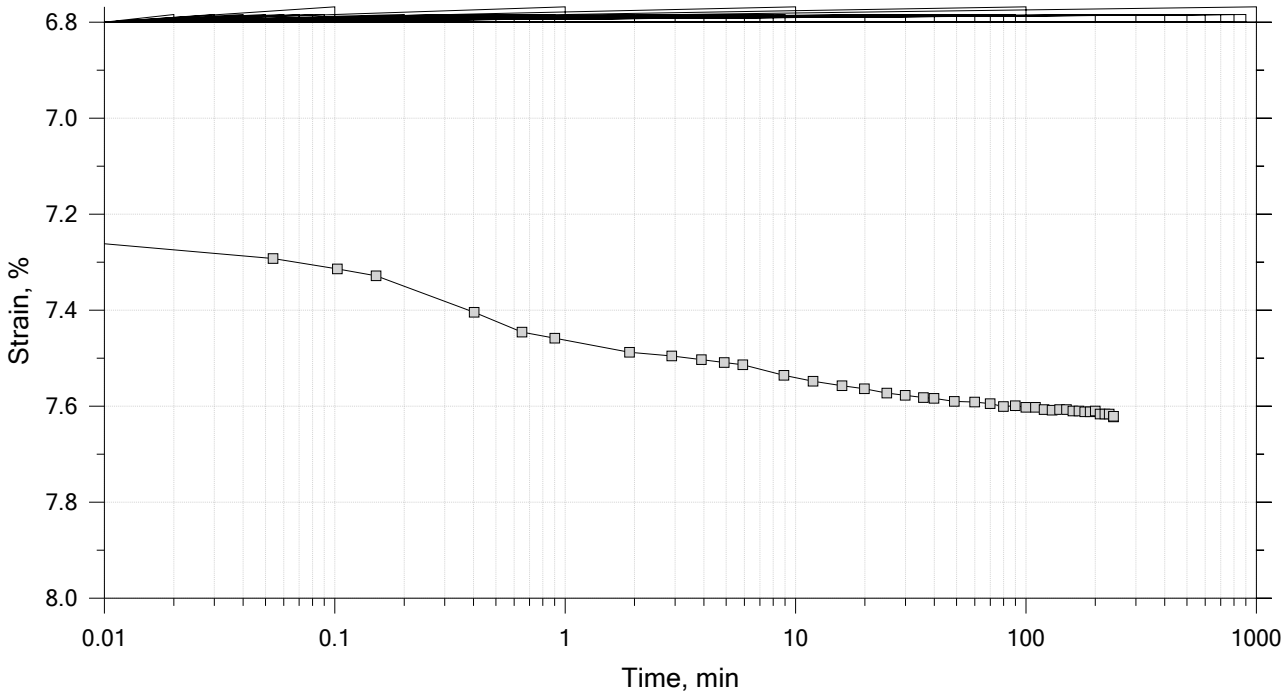


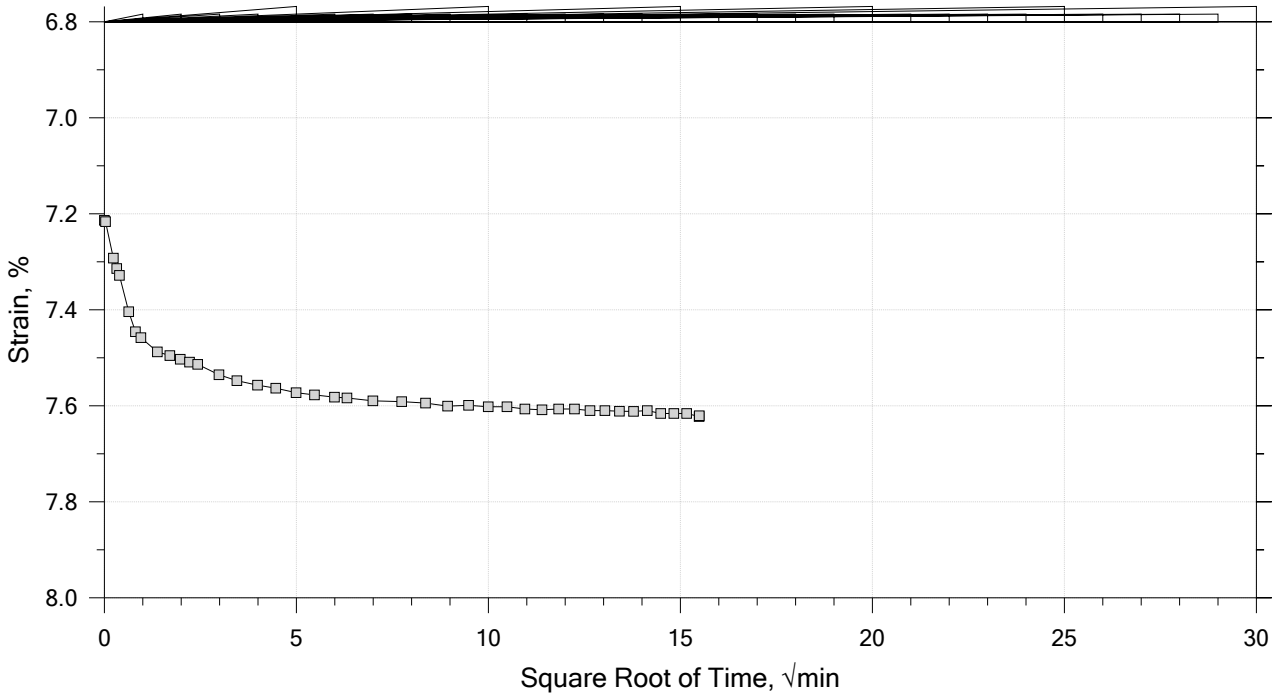


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf

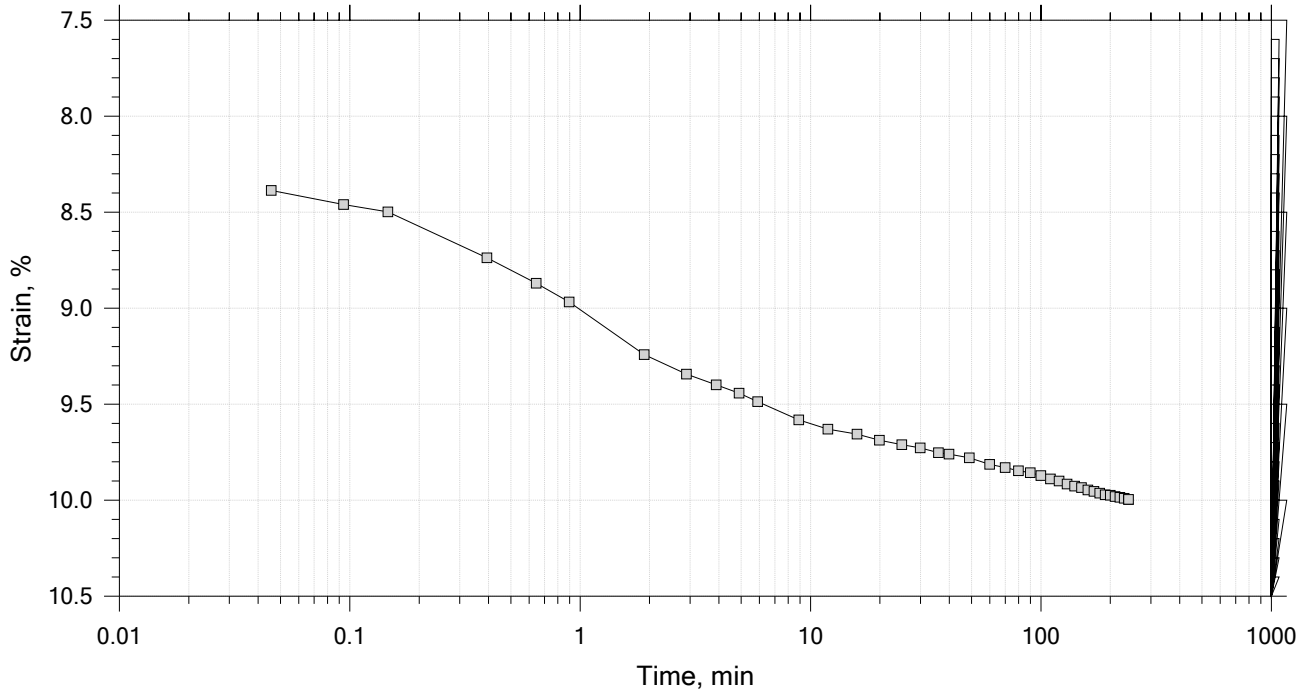


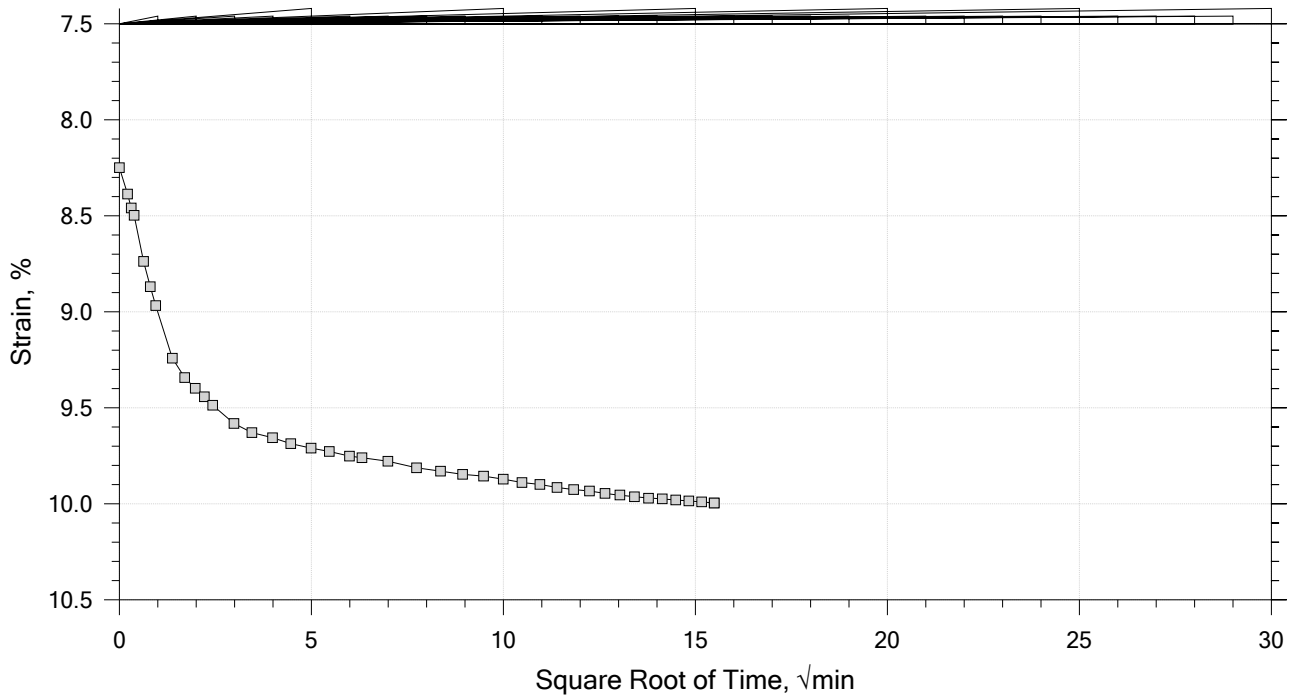


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14
Constant Load Step
Stress: 8 tsf

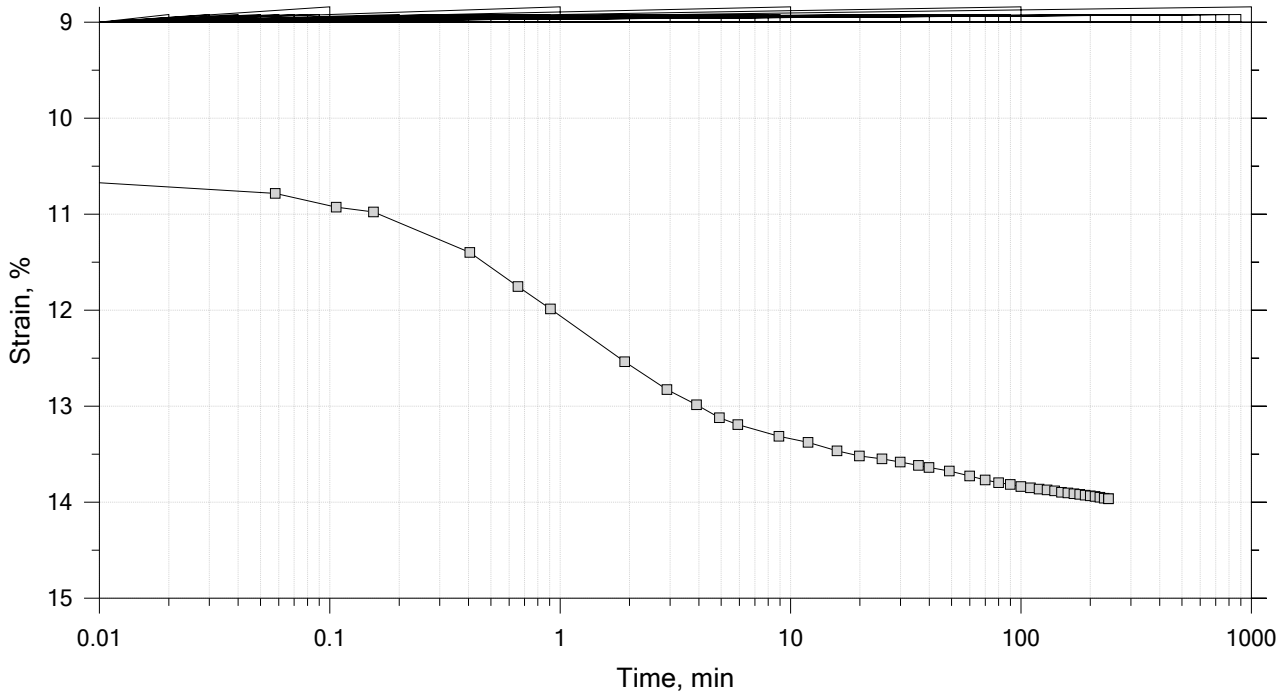


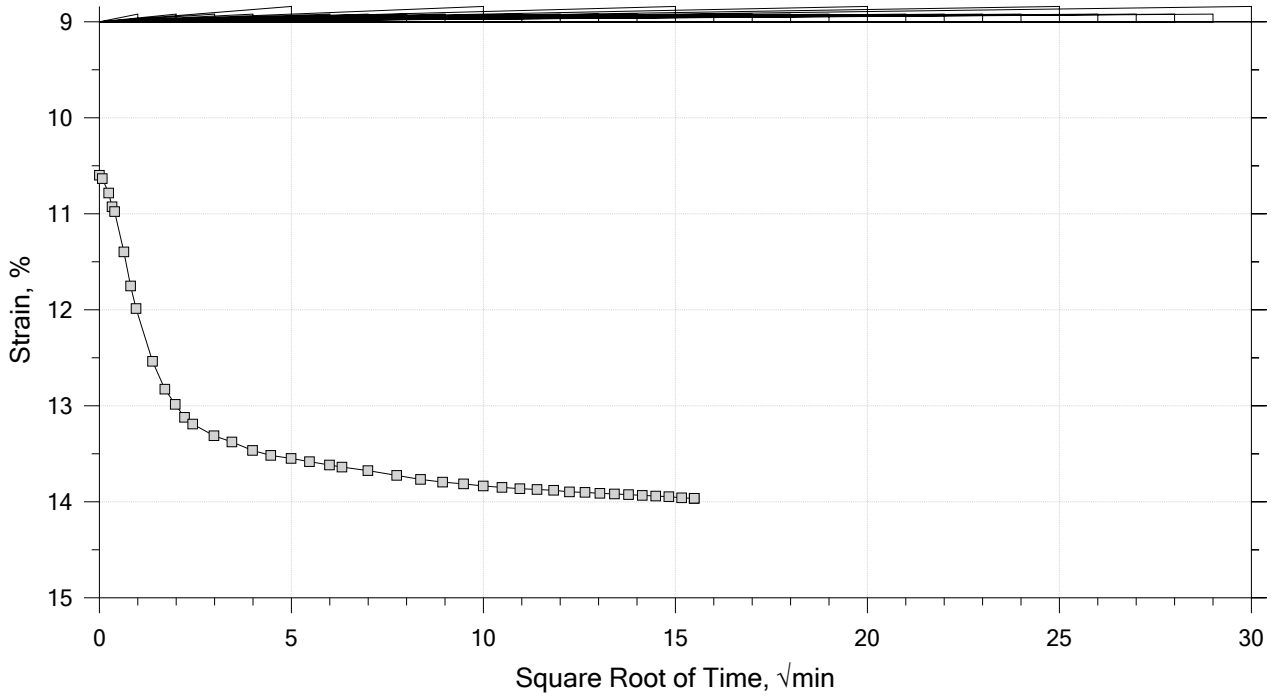


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 14
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.74	Liquid Limit: 36
Initial Height: 1.00 in	Initial Void Ratio: 0.855	Plastic Limit: 21
Final Height: 0.90 in	Final Void Ratio: 0.669	Plasticity Index: 15

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-1967	RING		E2661
Mass Container, gm	8.37	111.08	111.08	8.44
Mass Container + Wet Soil, gm	135.09	266.27	259	159.85
Mass Container + Dry Soil, gm	107.27	229.97	229.97	130.14
Mass Dry Soil, gm	98.9	118.89	118.89	121.7
Water Content, %	28.13	30.53	24.41	24.41
Void Ratio	---	0.85	0.67	---
Degree of Saturation, %	---	97.91	100.00	---
Dry Unit Weight, pcf	---	92.272	102.52	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

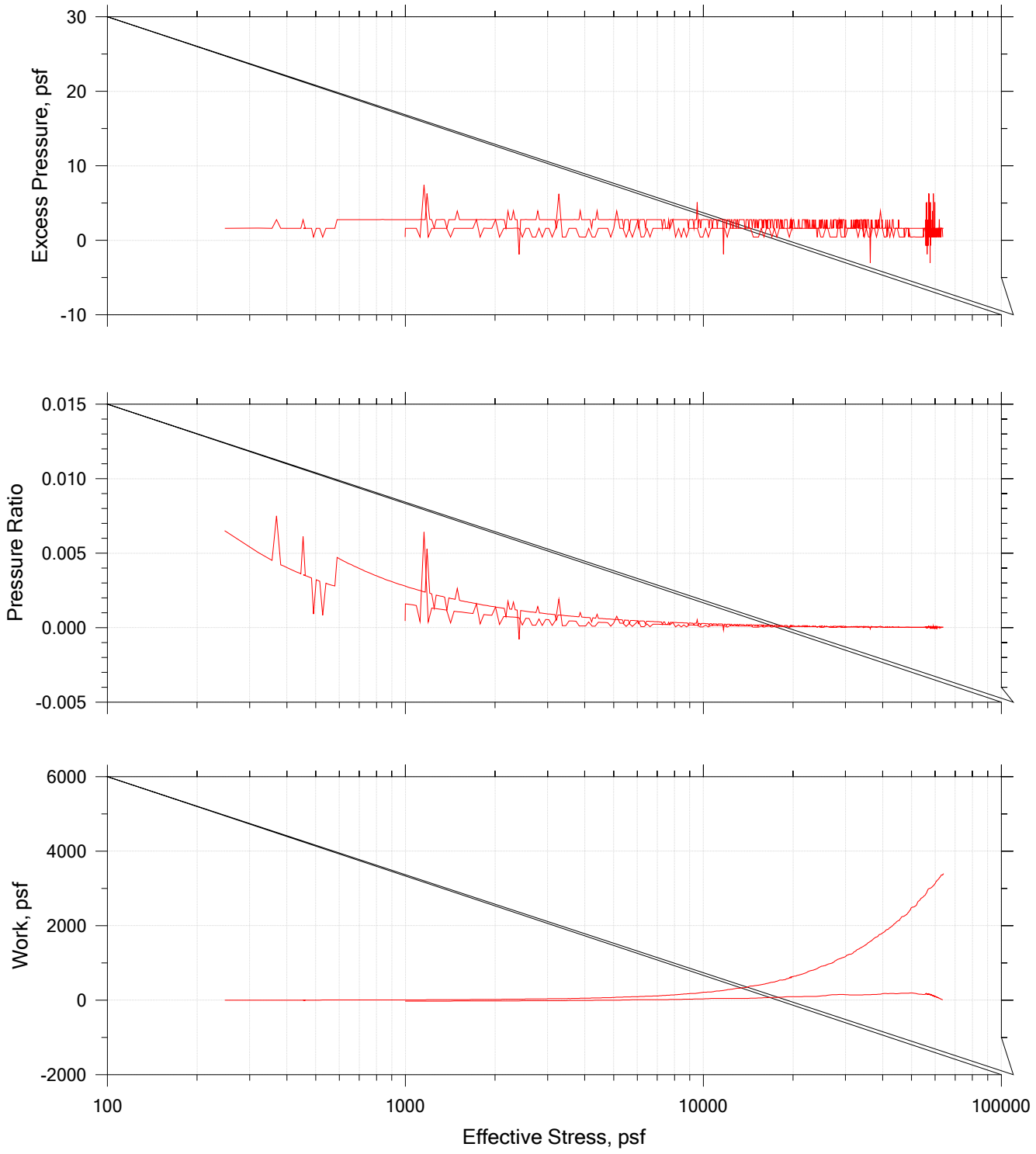
Square Root of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.125	0.009944	0.836	0.994	8.451	4.14e-04	7.95e-02	6.17e-04
2	0.250	0.01906	0.819	1.91	20.187	1.70e-04	7.29e-02	2.32e-04
3	0.500	0.03480	0.790	3.48	3.527	9.49e-04	6.30e-02	1.12e-03
4	1.00	0.04645	0.769	4.65	6.256	5.20e-04	2.33e-02	2.27e-04
5	2.00	0.05711	0.749	5.71	4.645	6.84e-04	1.07e-02	1.36e-04
6	4.00	0.07351	0.718	7.35	3.966	7.78e-04	8.20e-03	1.20e-04
7	2.00	0.07100	0.723	7.10	1.538	1.98e-03	1.25e-03	4.63e-05
8	1.00	0.06735	0.730	6.73	3.215	9.52e-04	3.66e-03	6.52e-05
9	0.500	0.06295	0.738	6.30	5.494	5.62e-04	8.78e-03	9.25e-05
10	1.00	0.06519	0.734	6.52	1.738	1.78e-03	4.47e-03	1.49e-04
11	2.00	0.06888	0.727	6.89	3.451	8.91e-04	3.69e-03	6.16e-05
12	4.00	0.07621	0.713	7.62	3.718	8.17e-04	3.67e-03	5.61e-05
13	8.00	0.09996	0.669	10.0	7.332	4.01e-04	5.94e-03	4.46e-05
14	16.0	0.1396	0.596	14.0	5.875	4.66e-04	4.96e-03	4.33e-05

GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		
	Displacement at End of Increment		

CRC Test

Pressure Curves



GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System JJ		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.76 (Estimated)	Liquid Limit: 35
Specimen Height, in: 1.00	Initial Void Ratio: 0.877	Plastic Limit: 20
Final Height, in: 0.86	Final Void Ratio: 0.615	Plasticity Index: 15

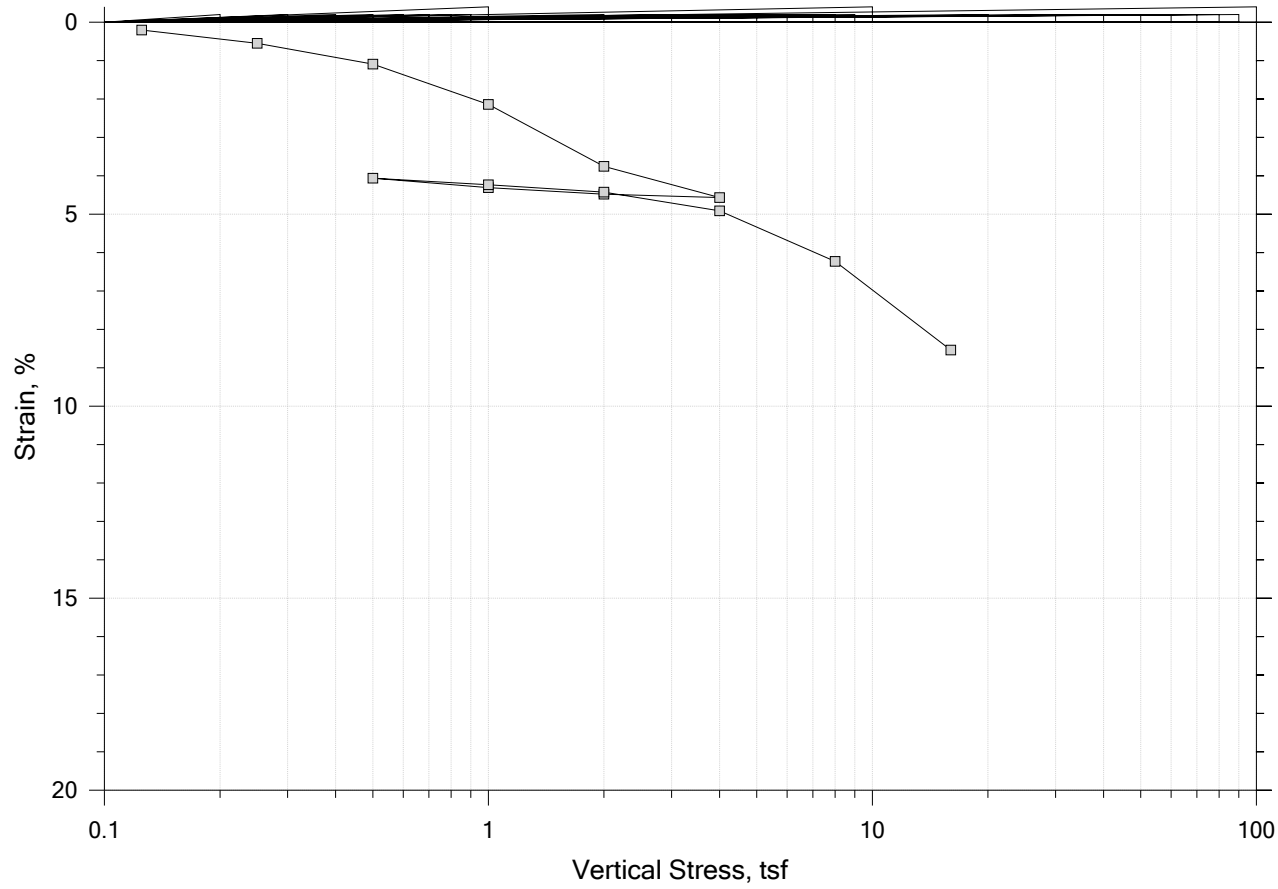
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-3892	---		E5211
Mass Container, gm	8.43	111.05	111.05	8.46
Mass Container + Wet Soil, gm	217.8	266.79	255.62	157.31
Mass Container + Dry Soil, gm	171.21	229.29	229.29	130.2
Mass Dry Soil, gm	162.78	118.24	118.24	121.74
Water Content, %	28.62	31.72	22.27	22.27
Void Ratio	---	0.88	0.61	---
Degree of Saturation, %	---	99.76	100.00	---
Dry Unit Weight, pcf	---	91.764	106.7	---

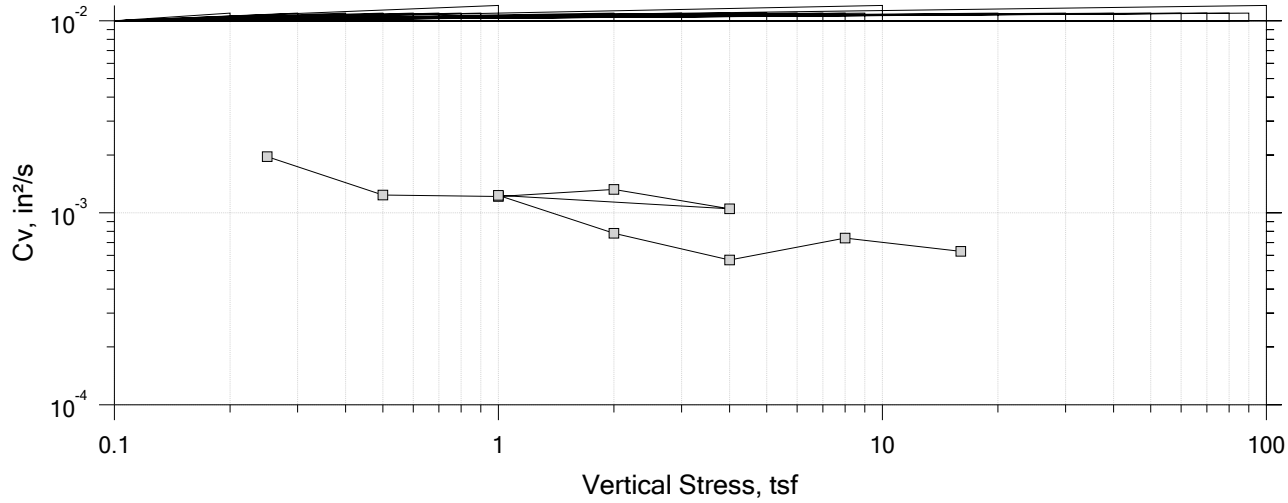
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System JJ		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

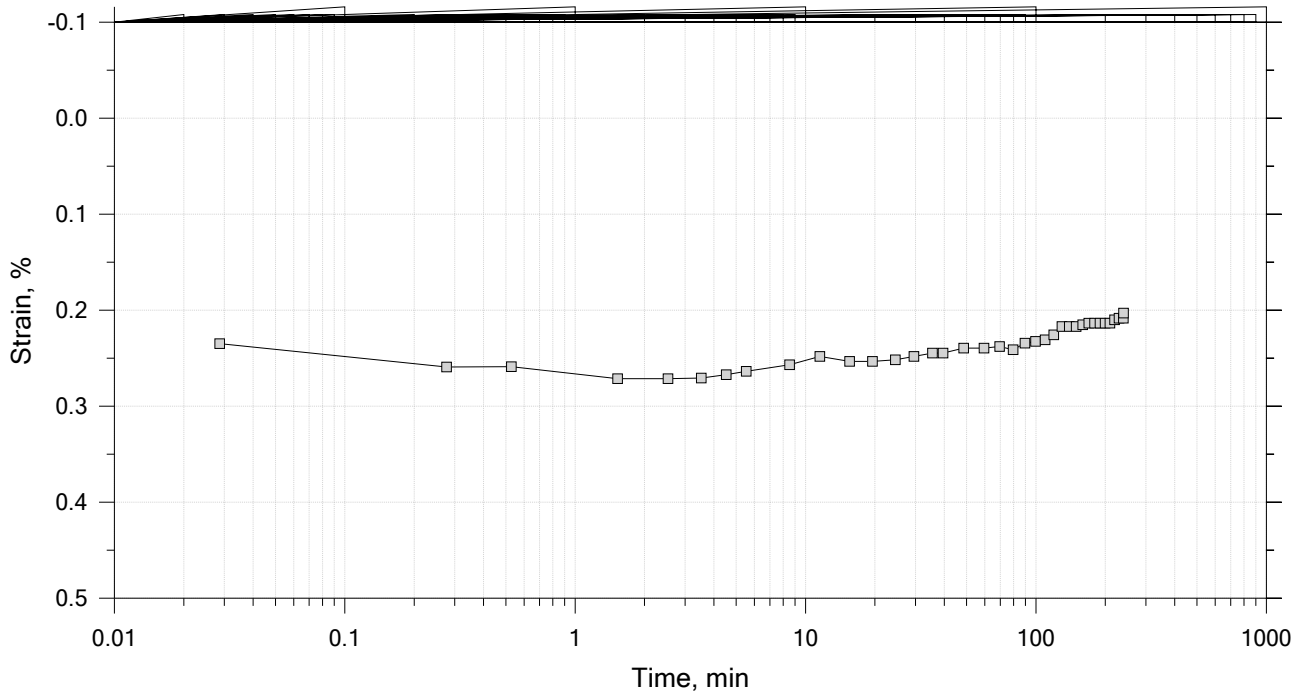


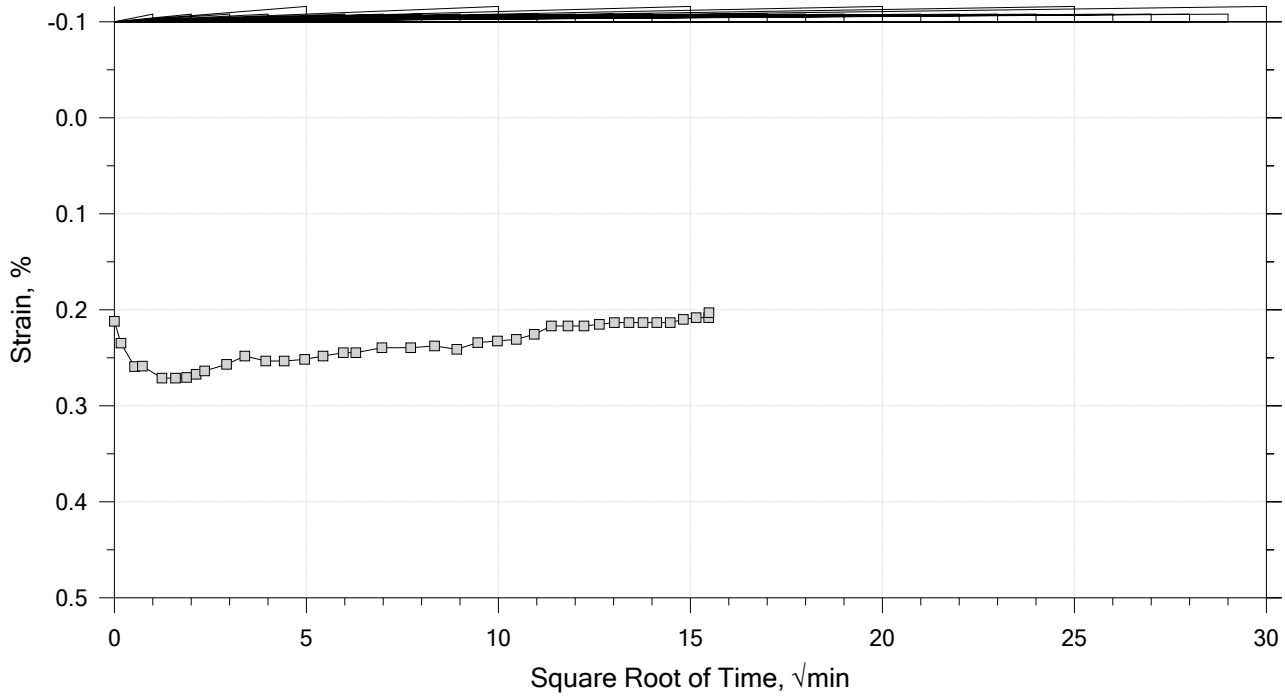


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14
Constant Load Step
Stress: 0.125 tsf

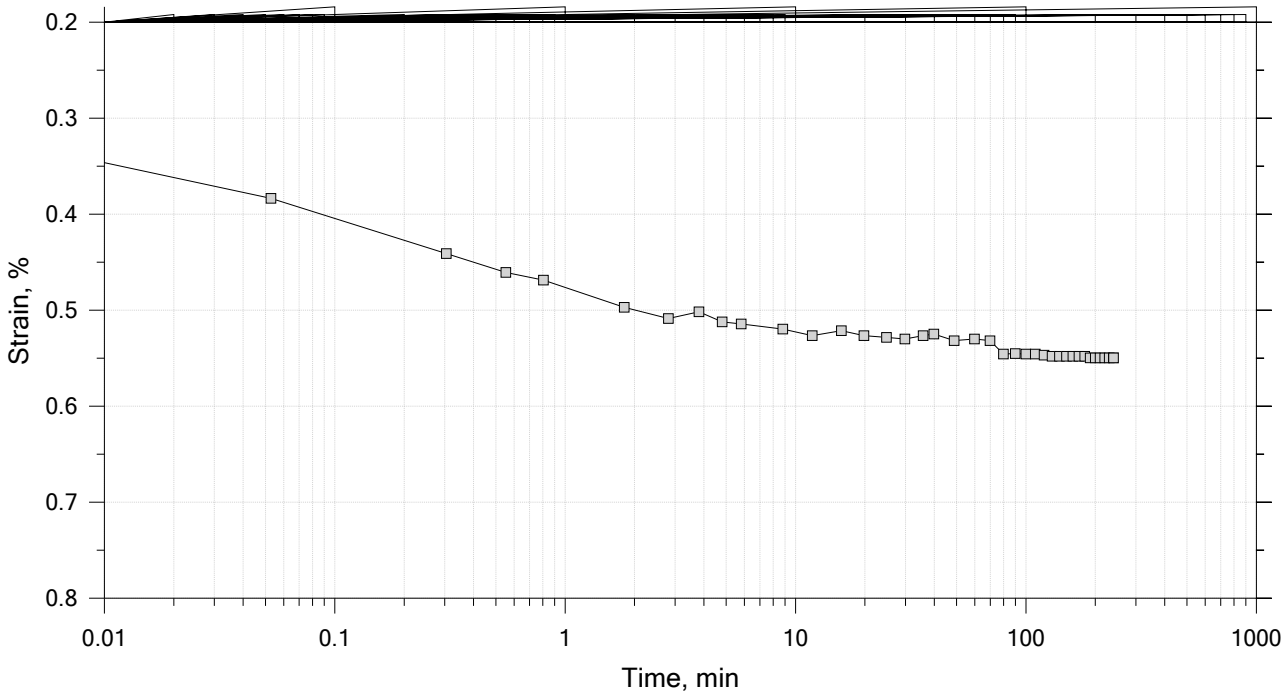


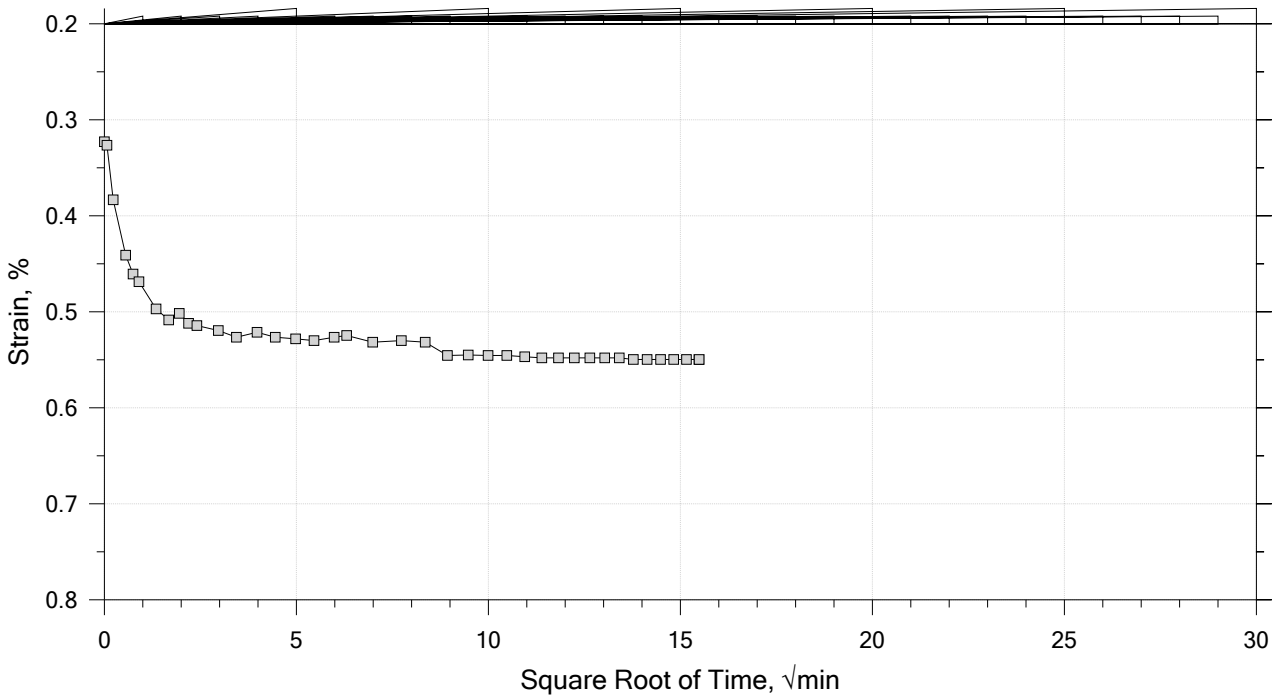


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf

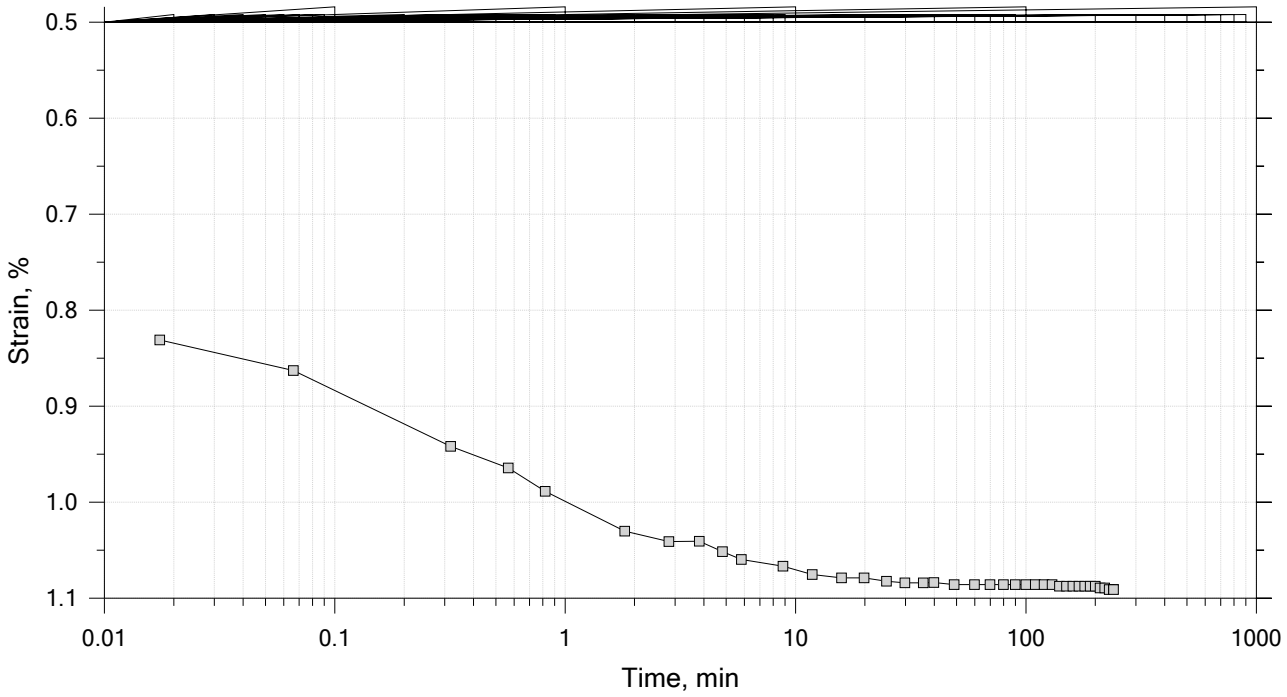


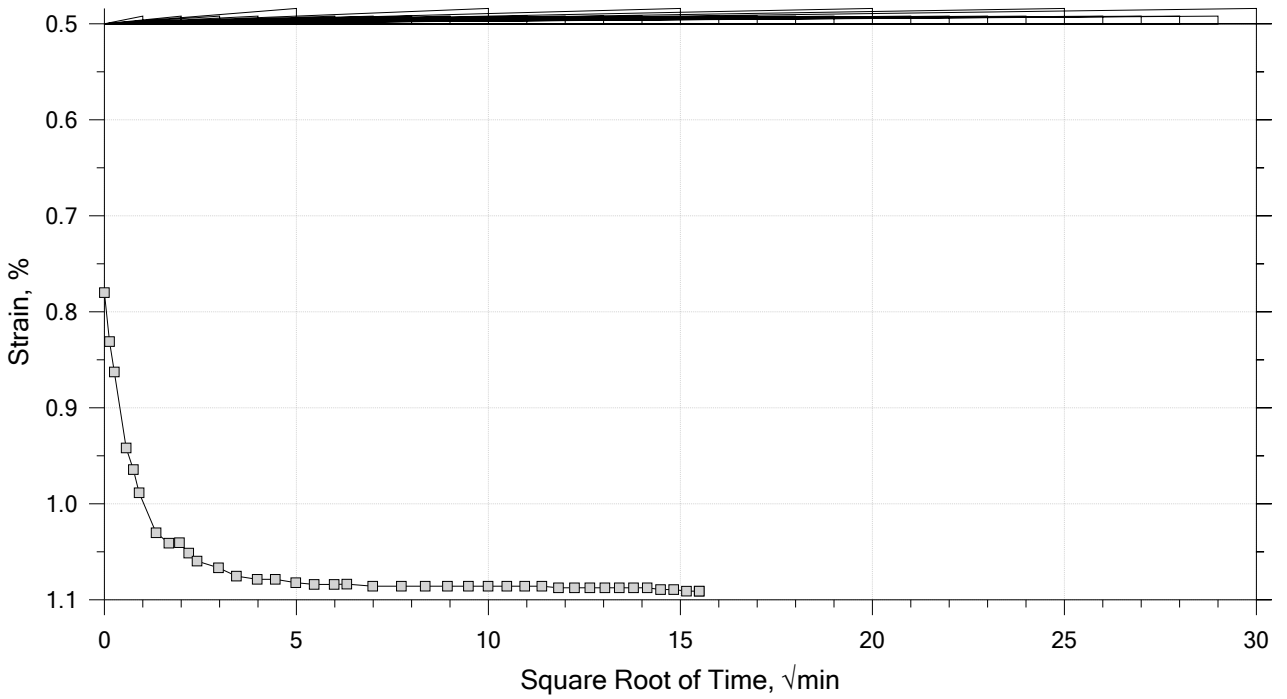


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf

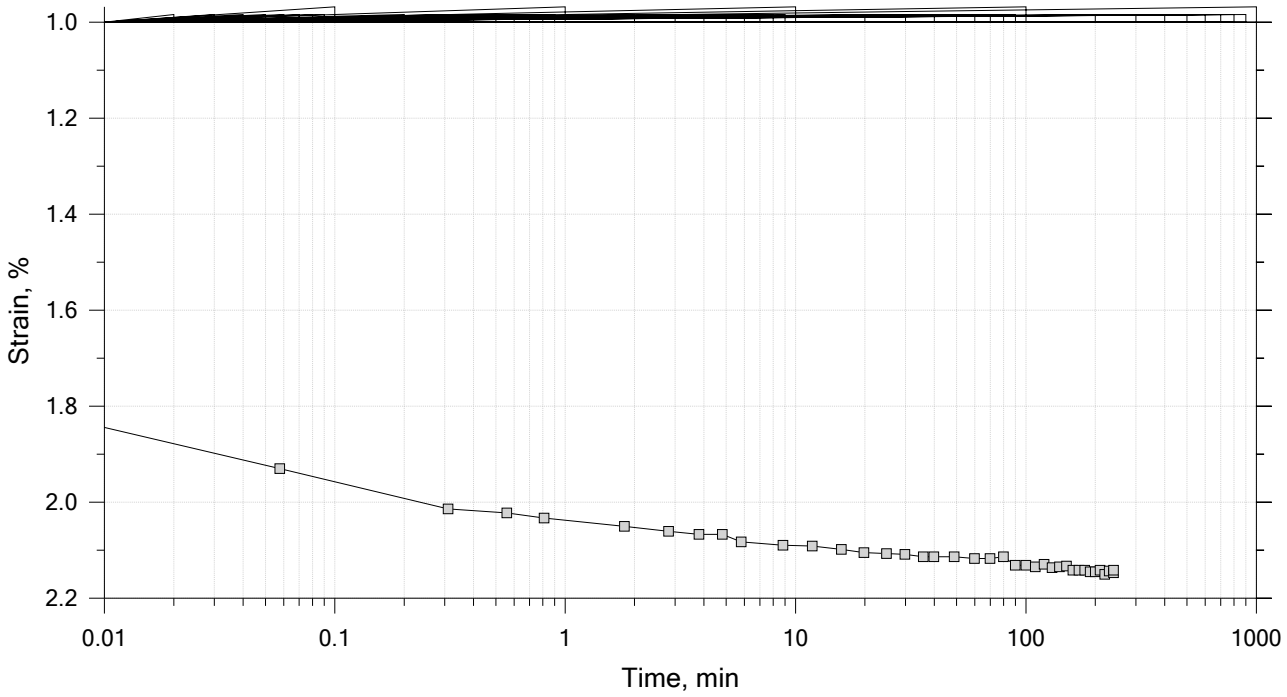


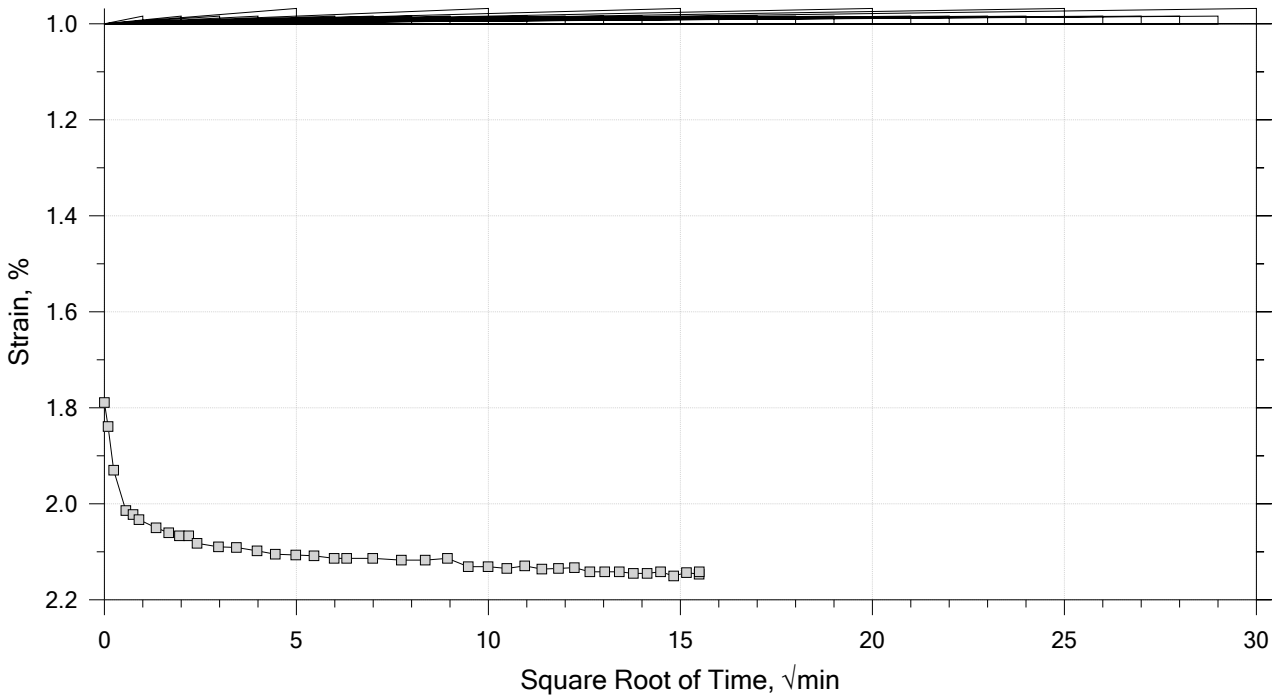


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14
Constant Load Step
Stress: 1 tsf





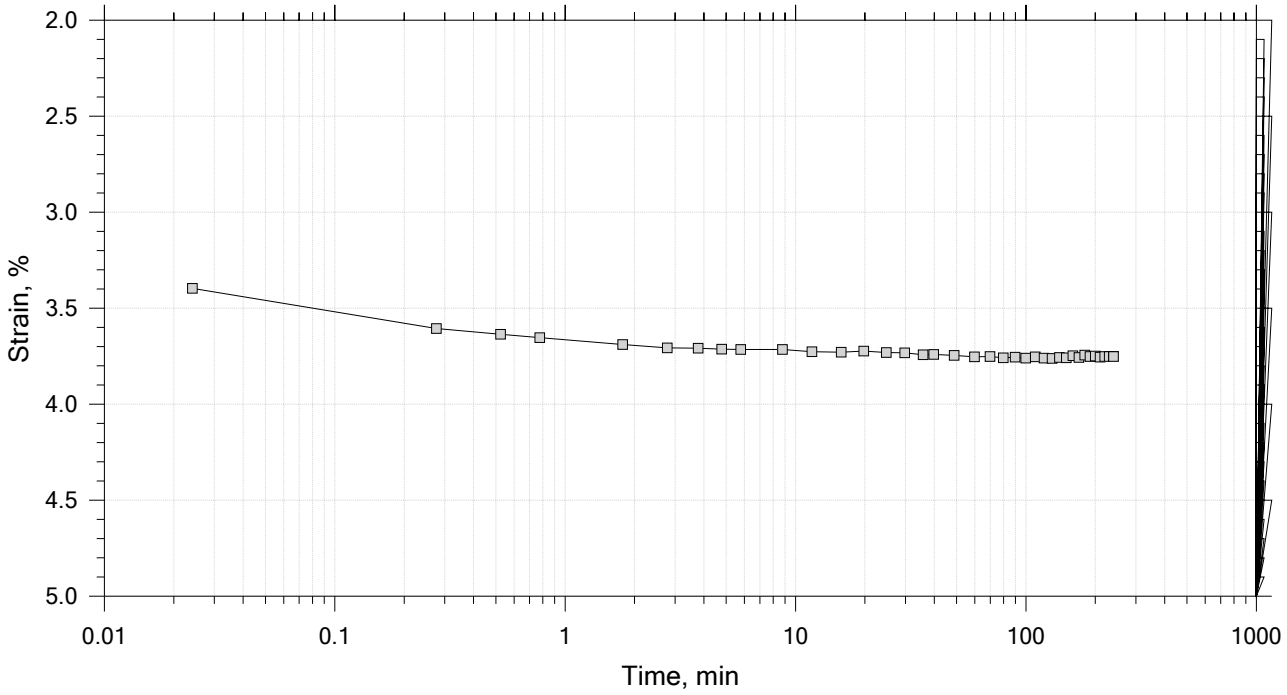
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

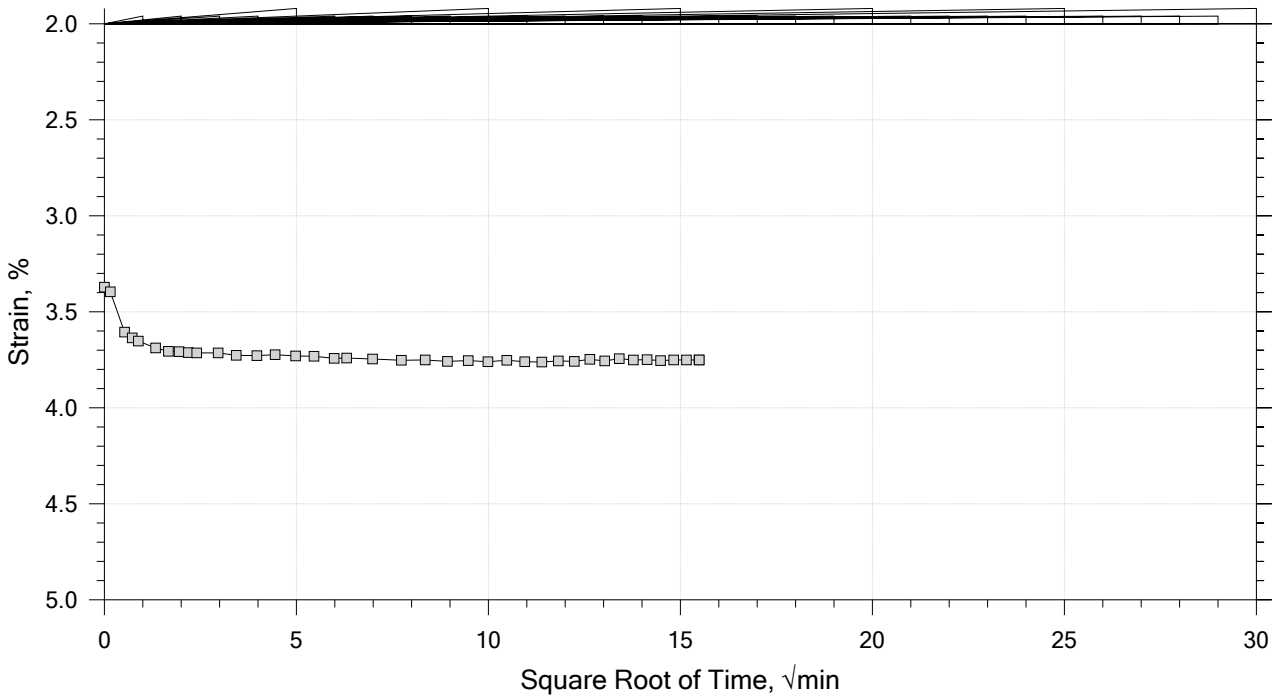
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14

Constant Load Step

Stress: 2 tsf





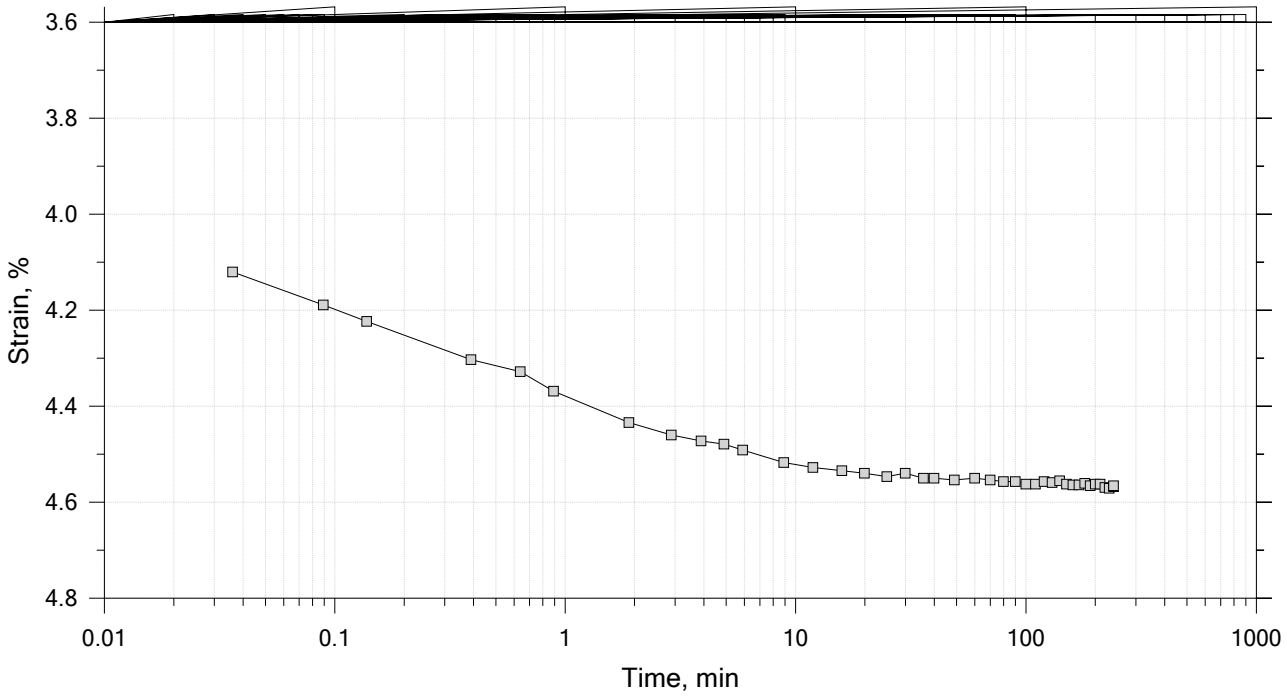
Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-F		

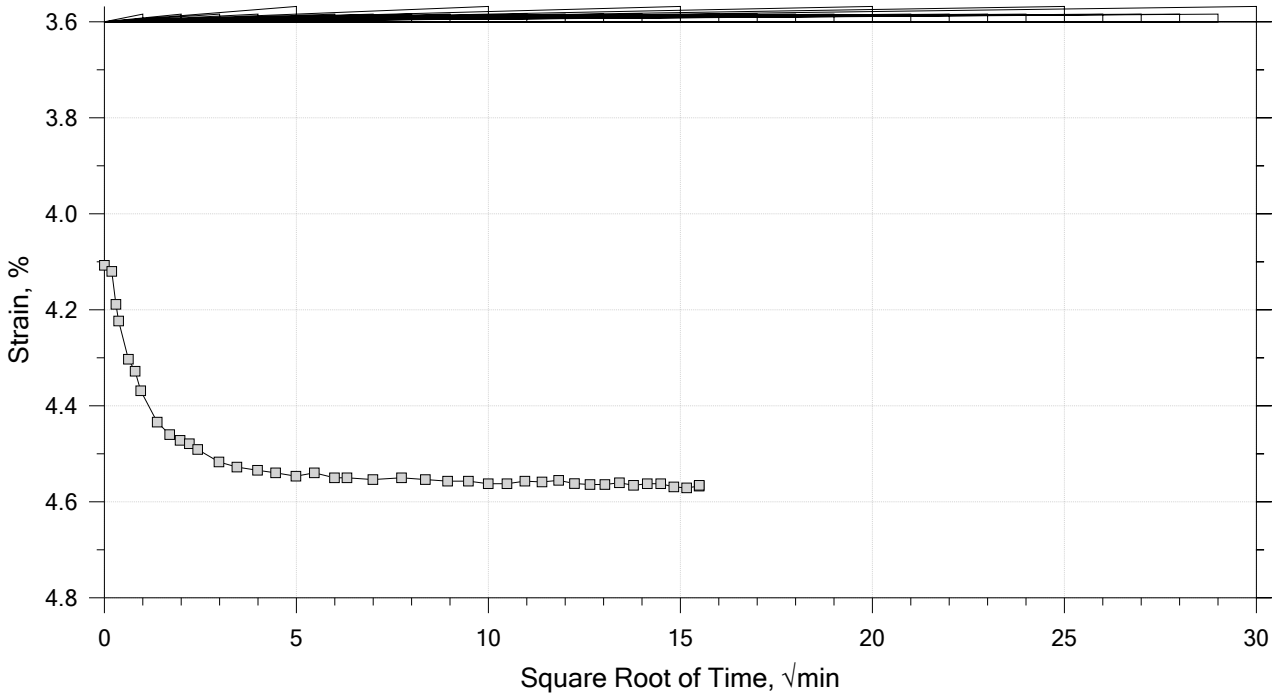
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 4 tsf





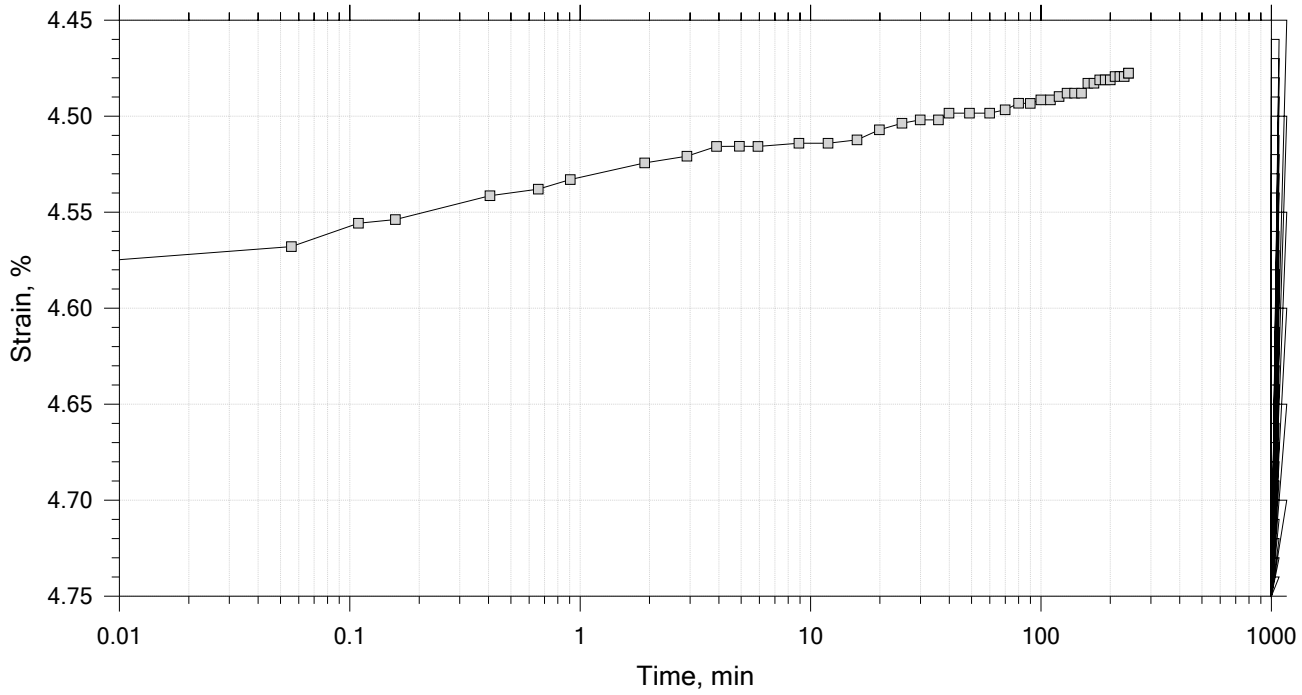
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

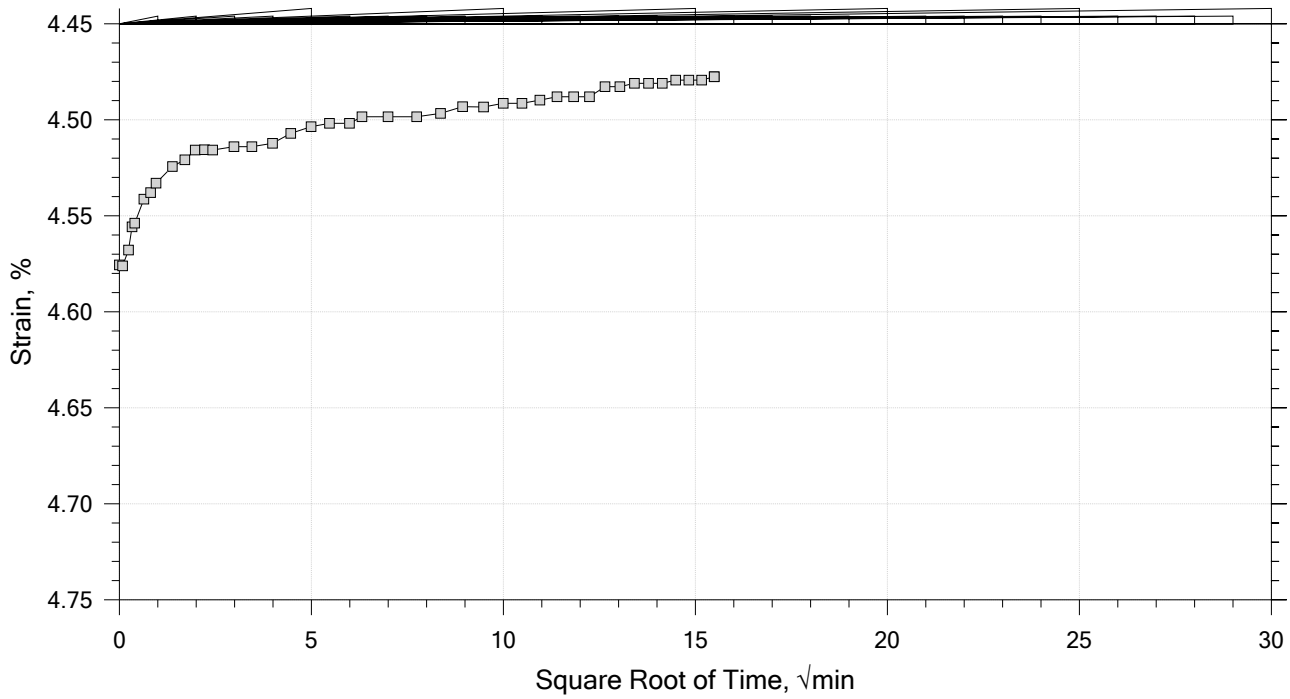
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14

Constant Load Step

Stress: 2 tsf





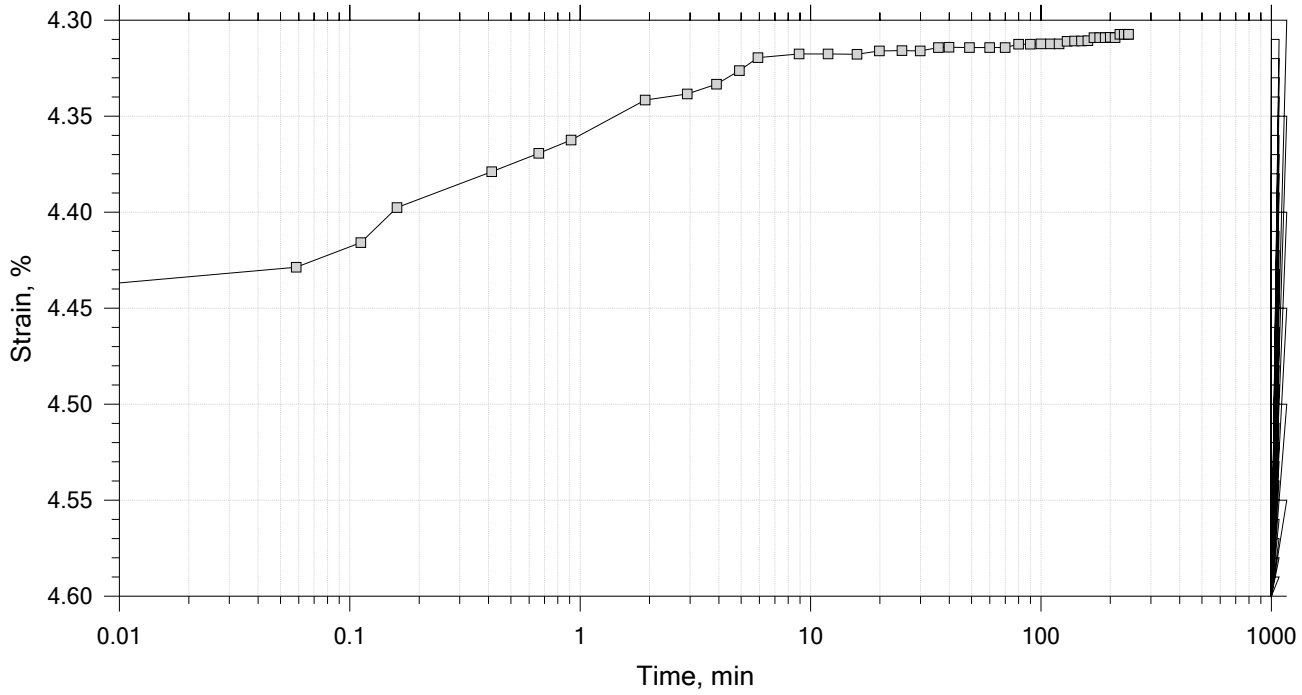
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

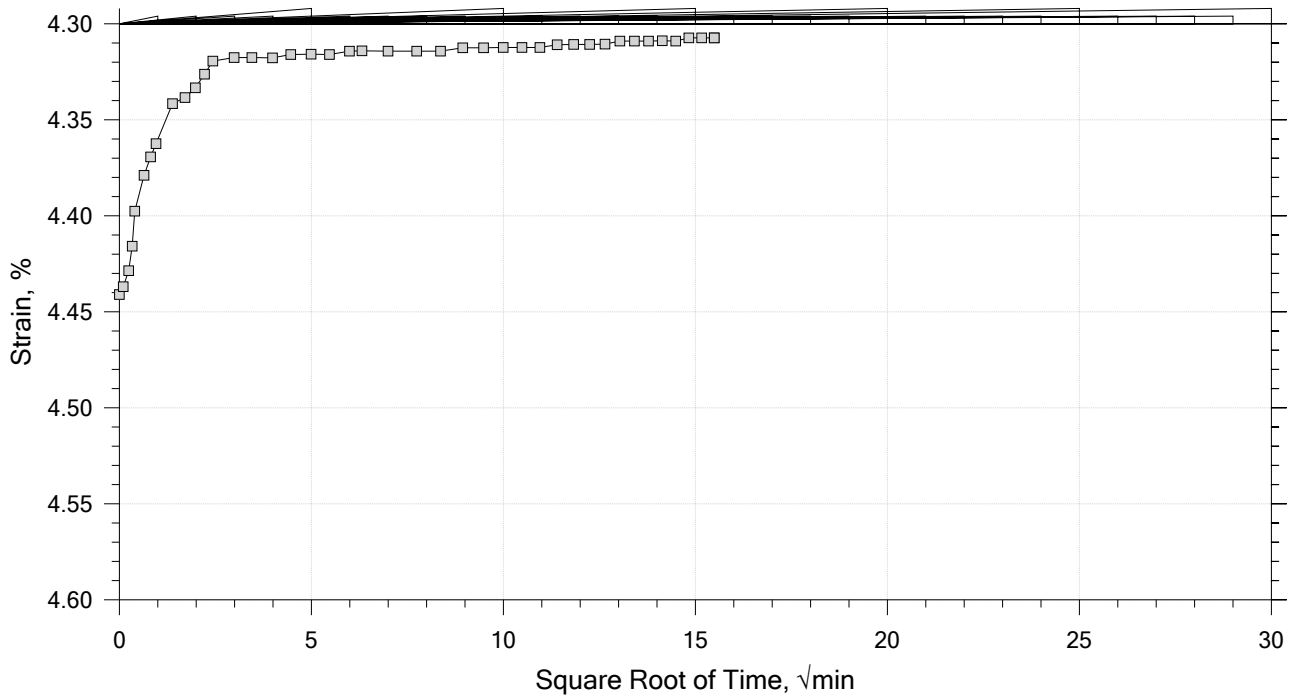
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf

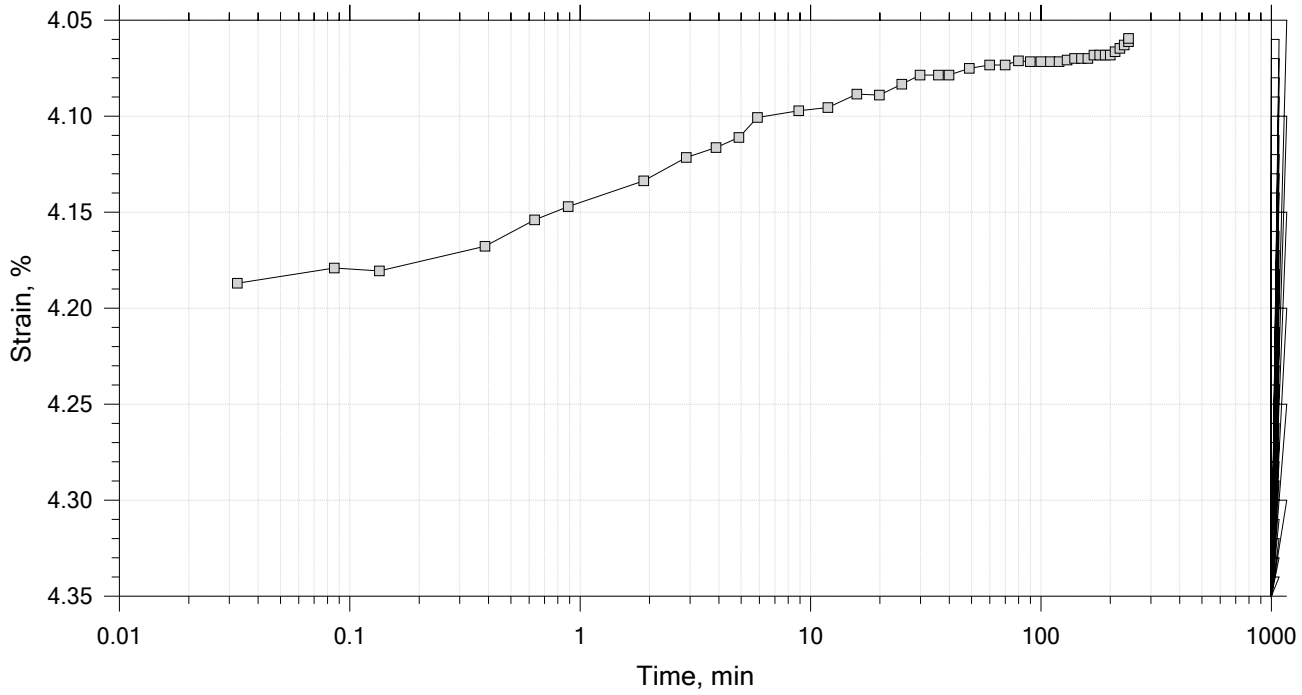


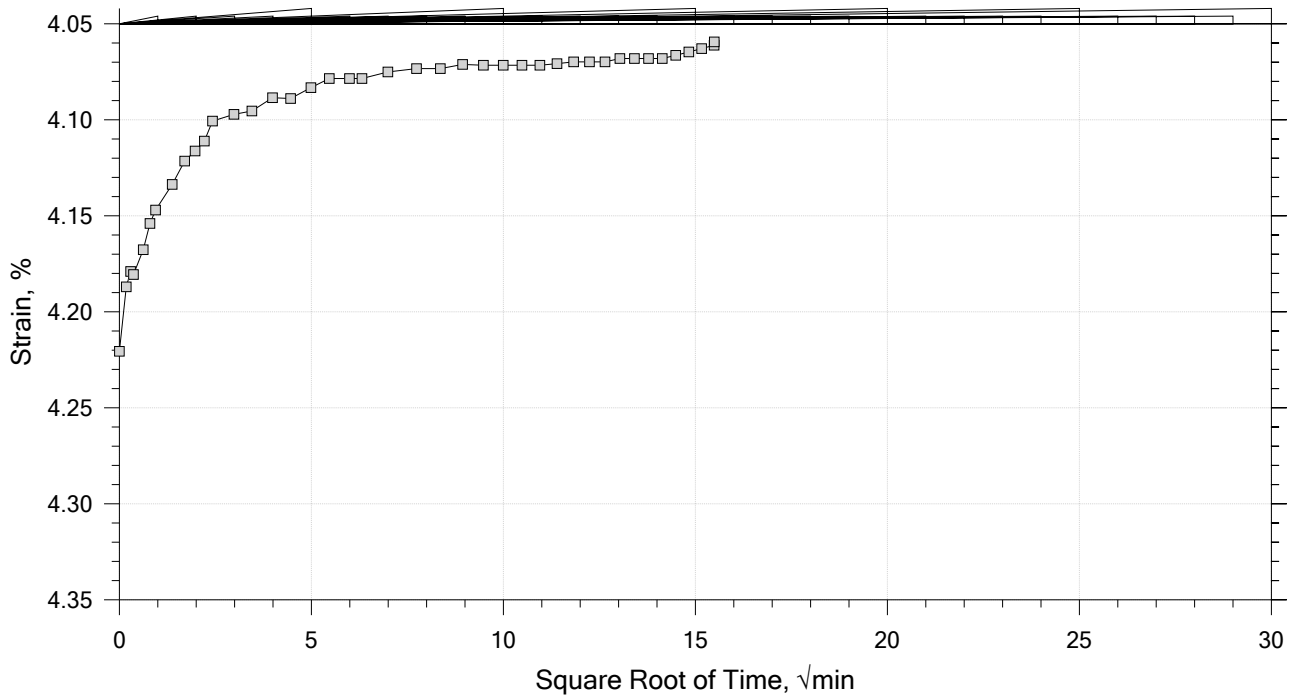


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

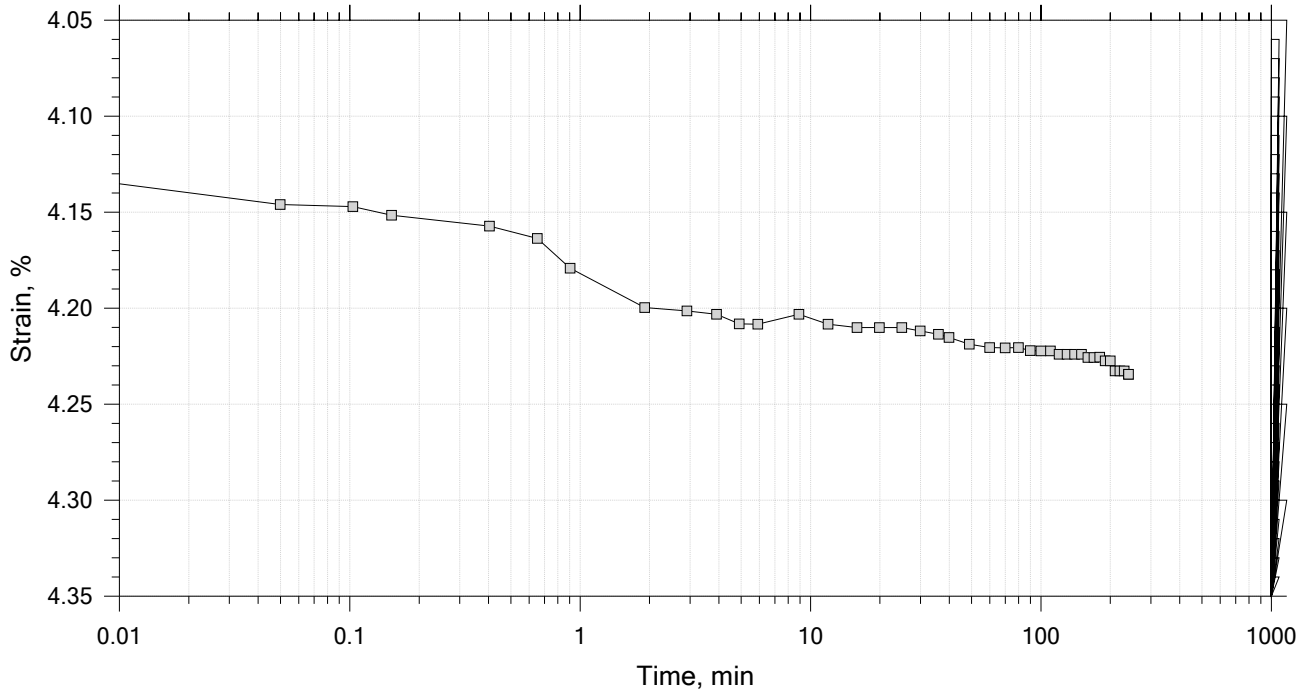


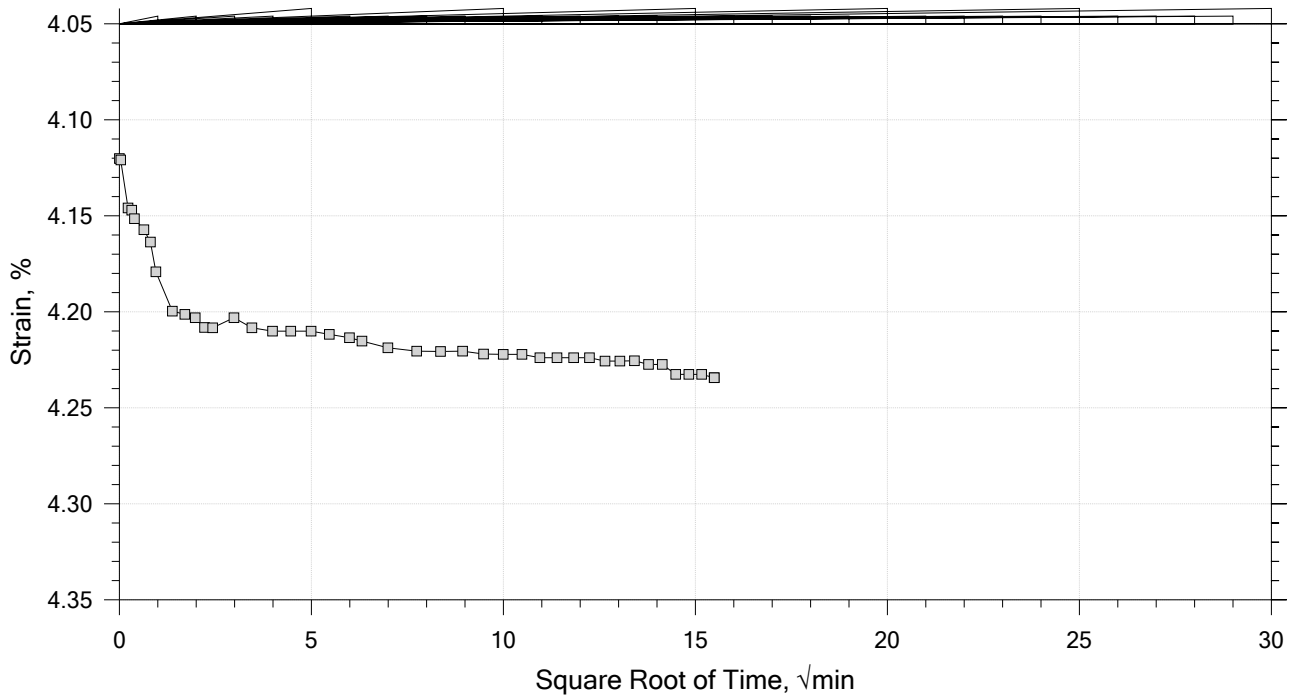


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

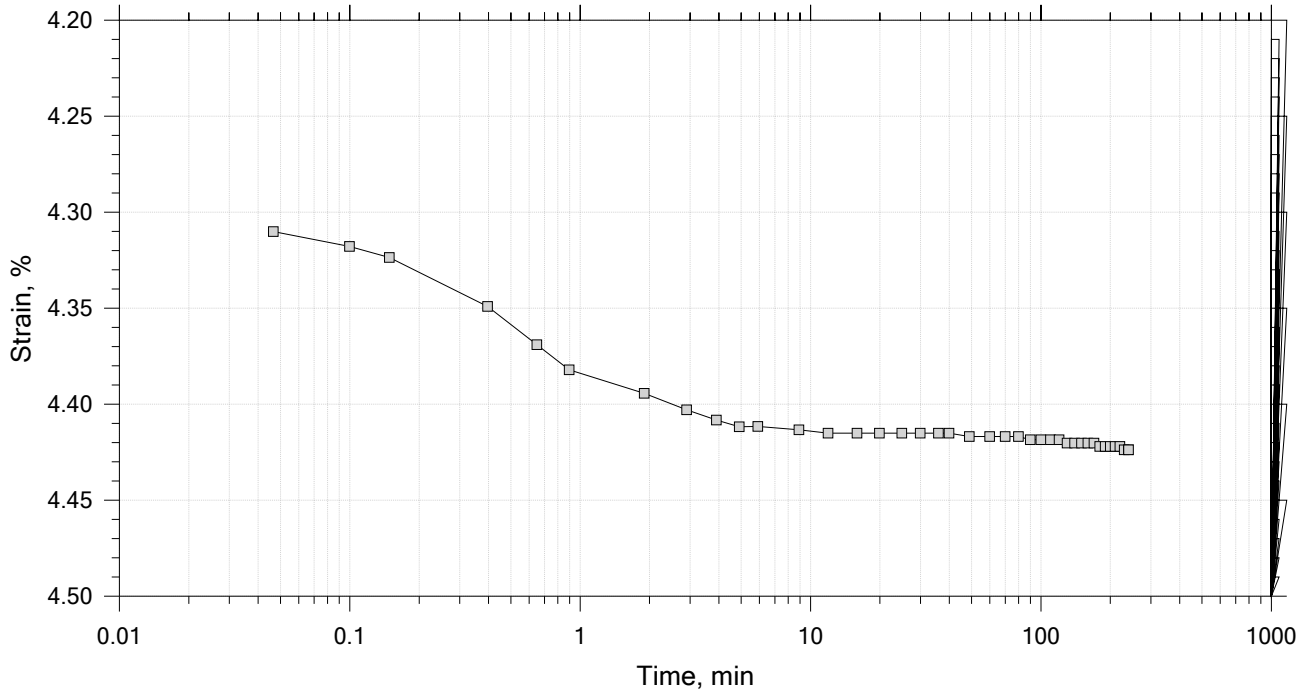


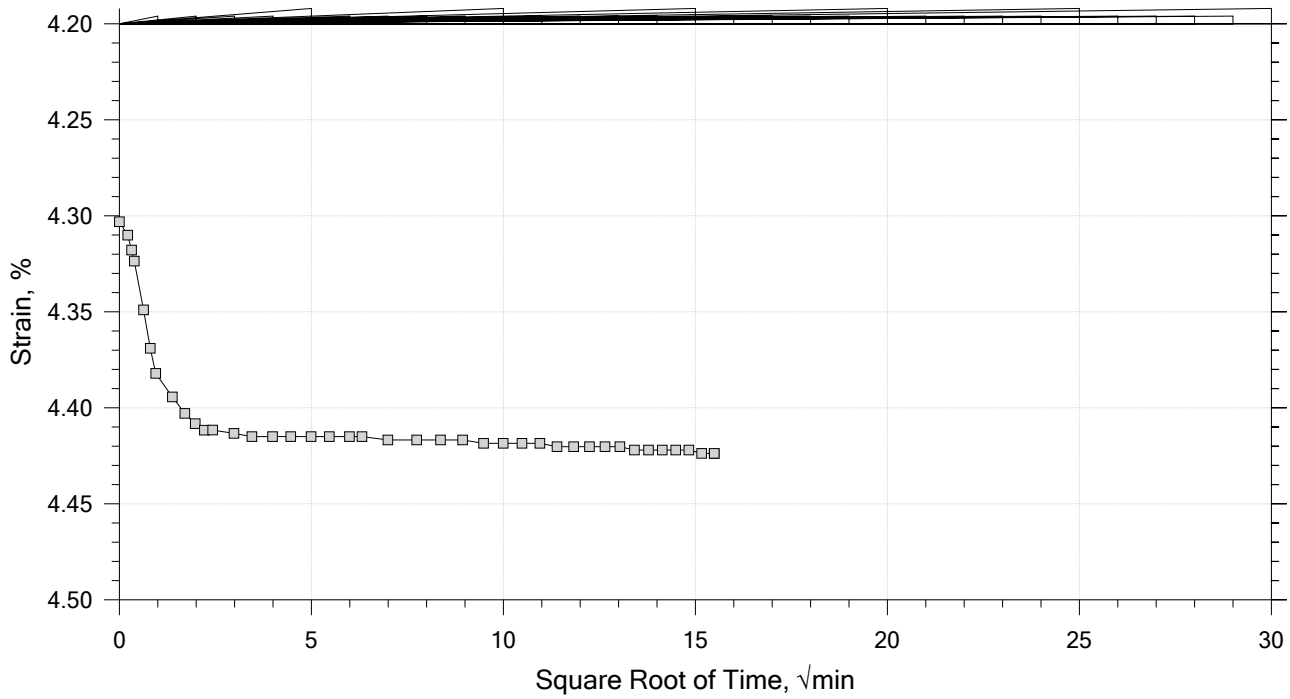


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

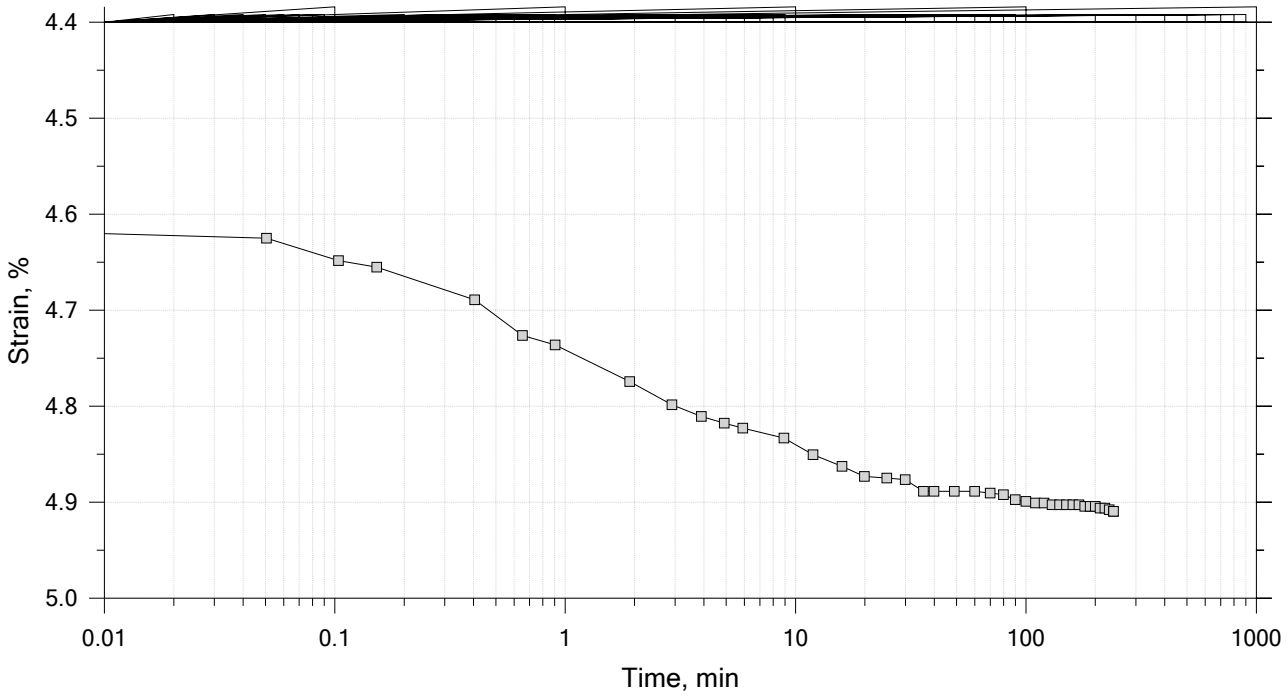


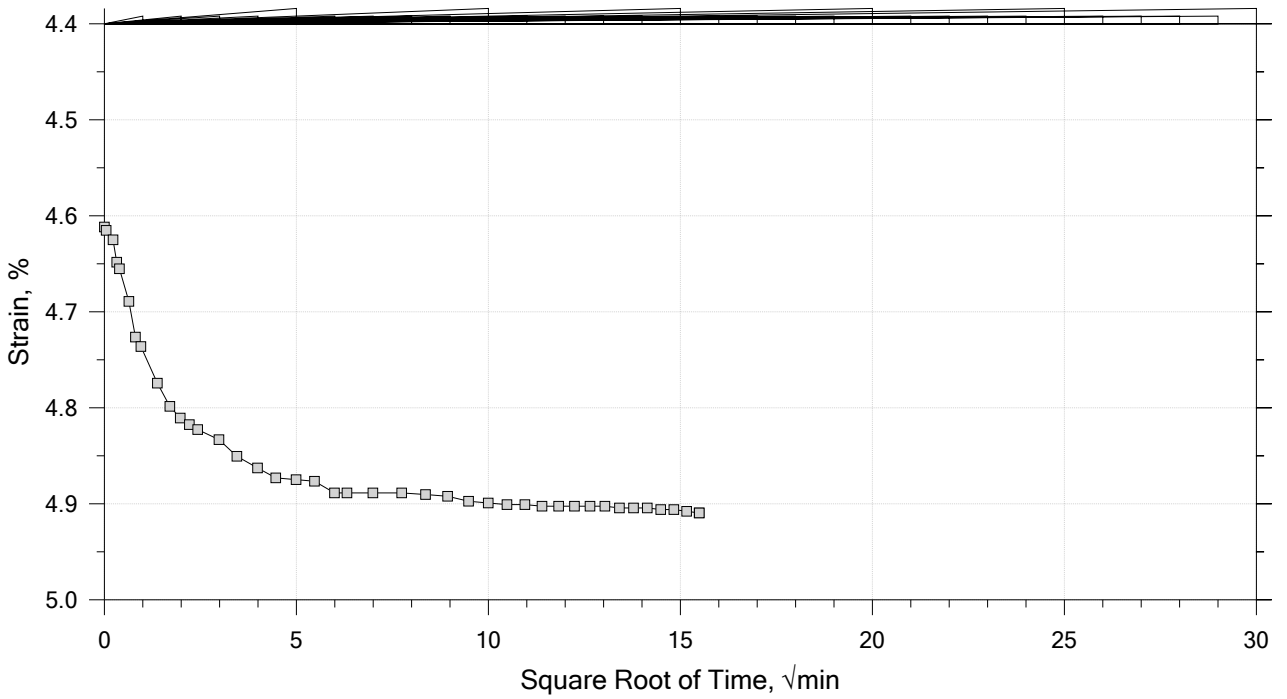


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf





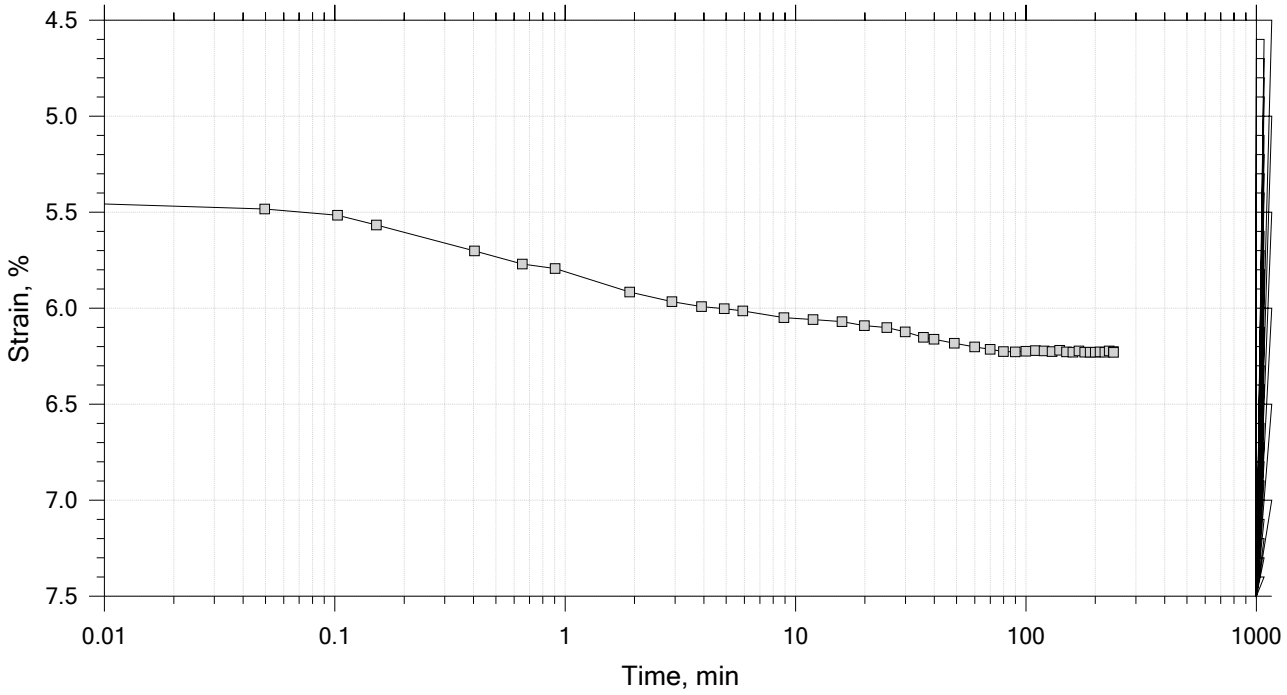
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

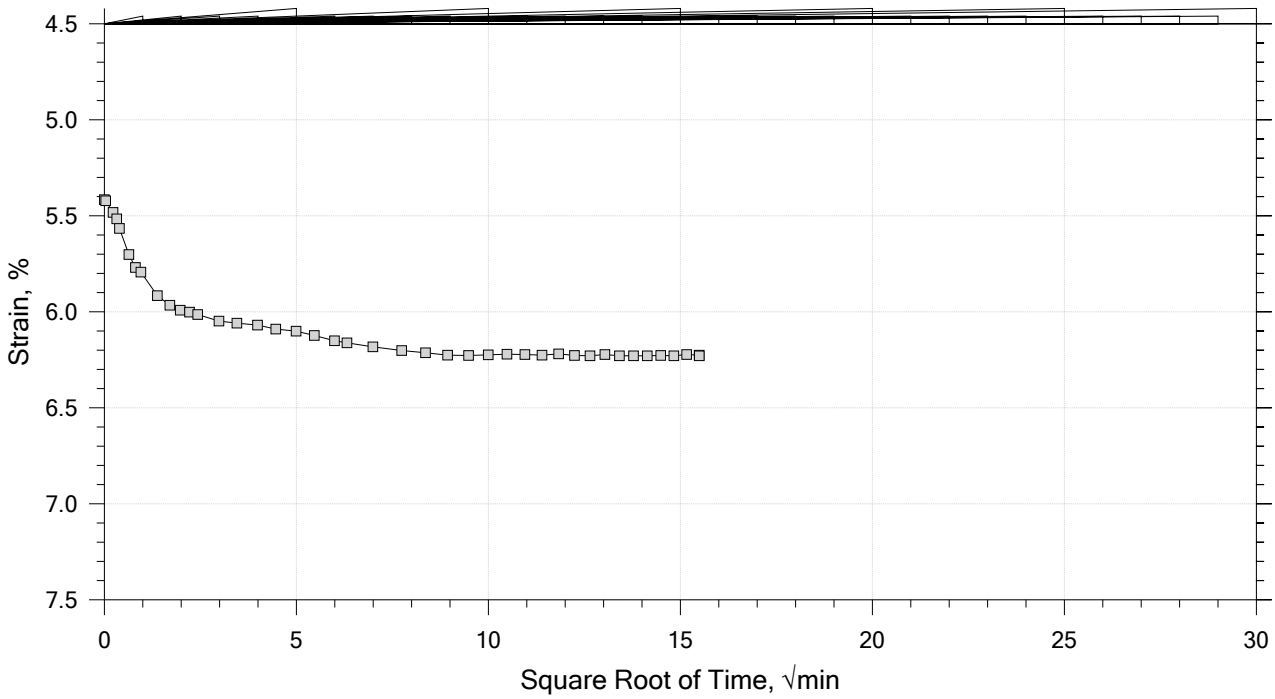
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 8 tsf

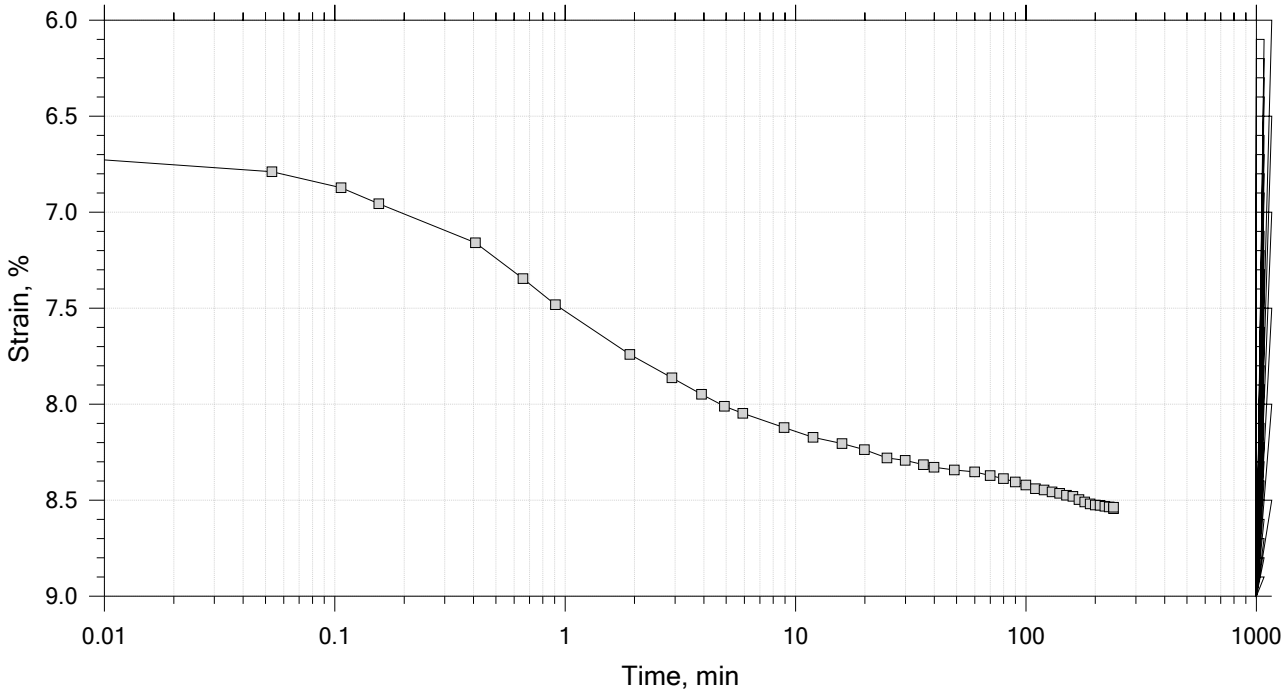


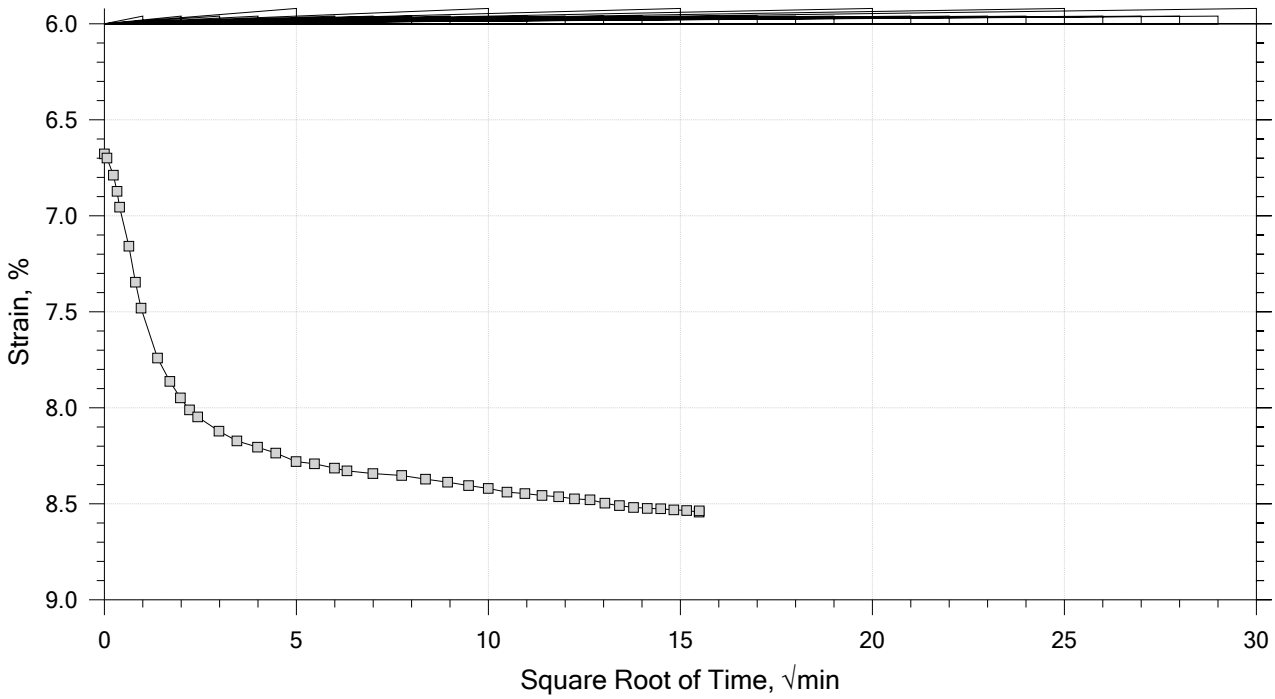


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 14
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.74	Liquid Limit: 39
Initial Height: 1.00 in	Initial Void Ratio: 0.762	Plastic Limit: 22
Final Height: 0.89 in	Final Void Ratio: 0.568	Plasticity Index: 17

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-0842	RING		E3190
Mass Container, gm	8.34	110.07	110.07	8.27
Mass Container + Wet Soil, gm	93.33	269.22	261	164.21
Mass Container + Dry Soil, gm	76.23	235.06	235.06	137.41
Mass Dry Soil, gm	67.89	124.99	124.99	129.14
Water Content, %	25.19	27.33	20.75	20.75
Void Ratio	---	0.76	0.57	---
Degree of Saturation, %	---	98.19	100.00	---
Dry Unit Weight, pcf	---	97.003	108.99	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

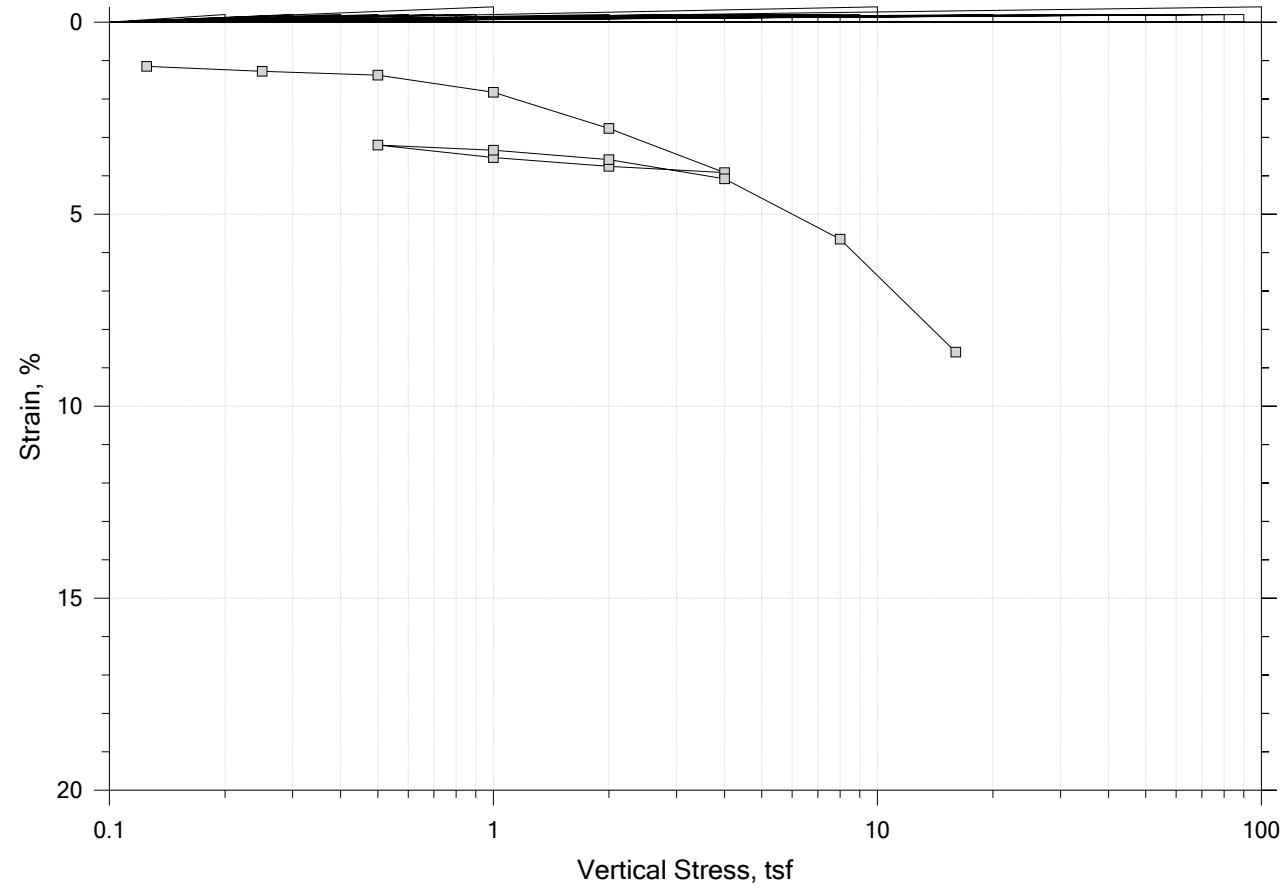
Square Root of Time Coefficients

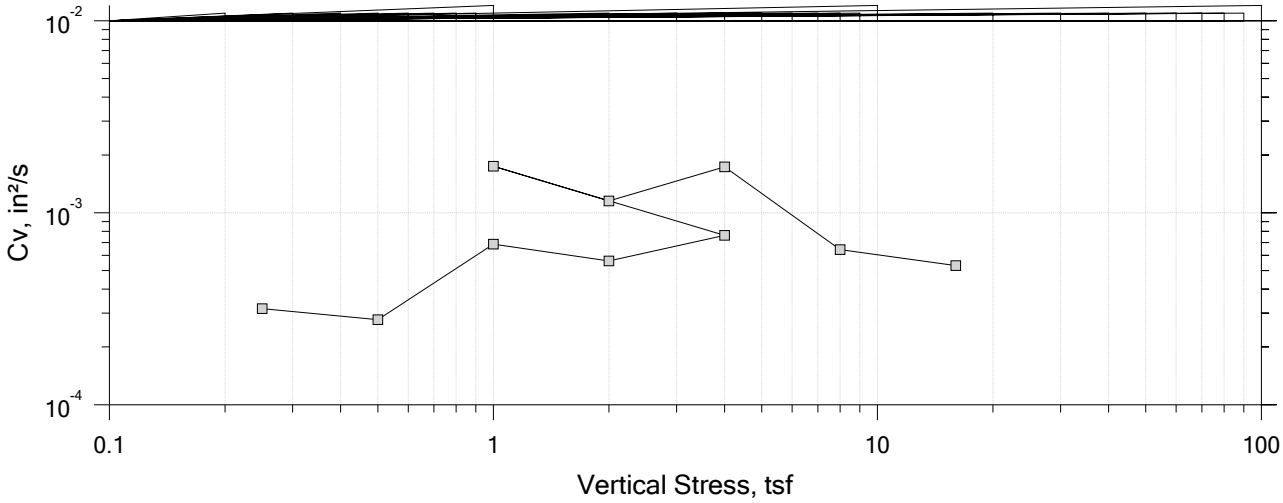
Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.125	0.002030	0.758	0.203	---	---	1.62e-02	---
2	0.250	0.005497	0.752	0.550	1.789	1.96e-03	2.77e-02	1.02e-03
3	0.500	0.01091	0.743	1.09	2.810	1.24e-03	2.17e-02	5.02e-04
4	1.00	0.02142	0.724	2.14	2.812	1.22e-03	2.10e-02	4.79e-04
5	2.00	0.03751	0.696	3.75	2.514	1.32e-03	1.61e-02	3.99e-04
6	4.00	0.04566	0.682	4.57	3.096	1.05e-03	4.07e-03	8.00e-05
7	2.00	0.04478	0.683	4.48	25.976	1.24e-04	4.41e-04	1.02e-06
8	1.00	0.04307	0.686	4.31	2.946	1.10e-03	1.70e-03	3.50e-05
9	0.500	0.04059	0.690	4.06	8.264	3.93e-04	4.96e-03	3.64e-05
10	1.00	0.04234	0.687	4.23	2.637	1.23e-03	3.50e-03	8.07e-05
11	2.00	0.04424	0.684	4.42	4.134	7.82e-04	1.89e-03	2.77e-05
12	4.00	0.04910	0.675	4.91	5.657	5.68e-04	2.43e-03	2.58e-05
13	8.00	0.06229	0.652	6.23	4.269	7.38e-04	3.30e-03	4.56e-05
14	16.0	0.08537	0.612	8.54	4.814	6.30e-04	2.88e-03	3.40e-05

GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

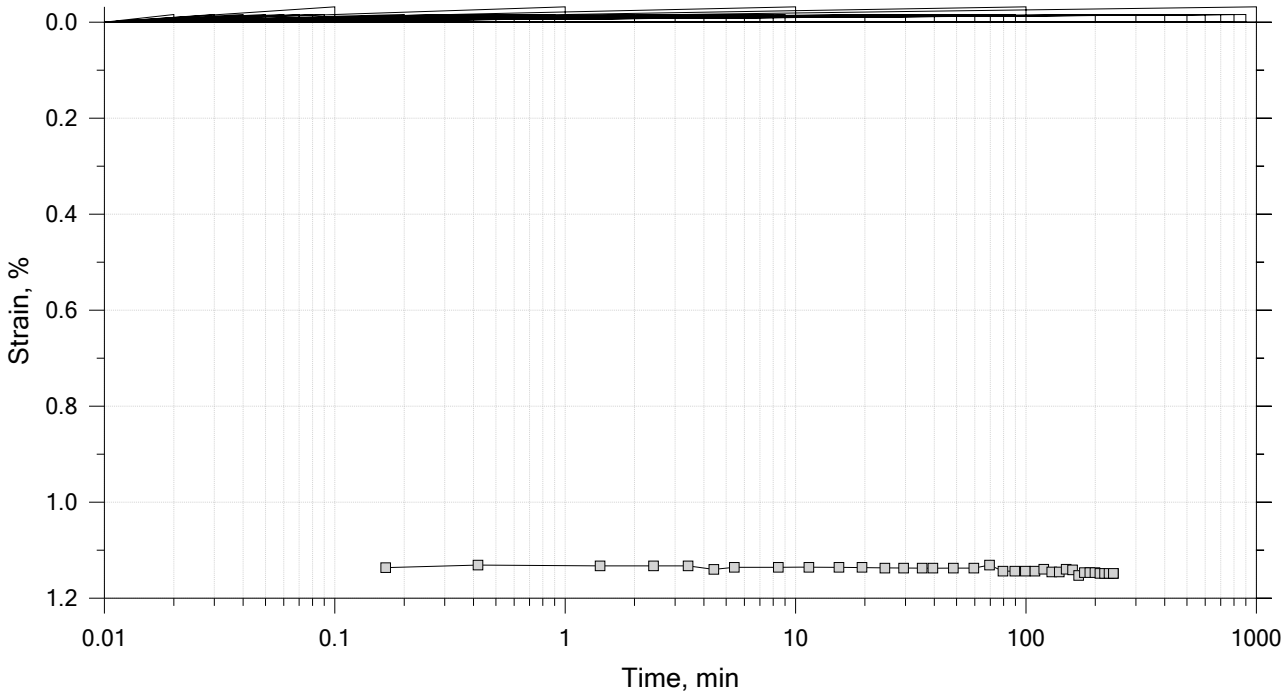


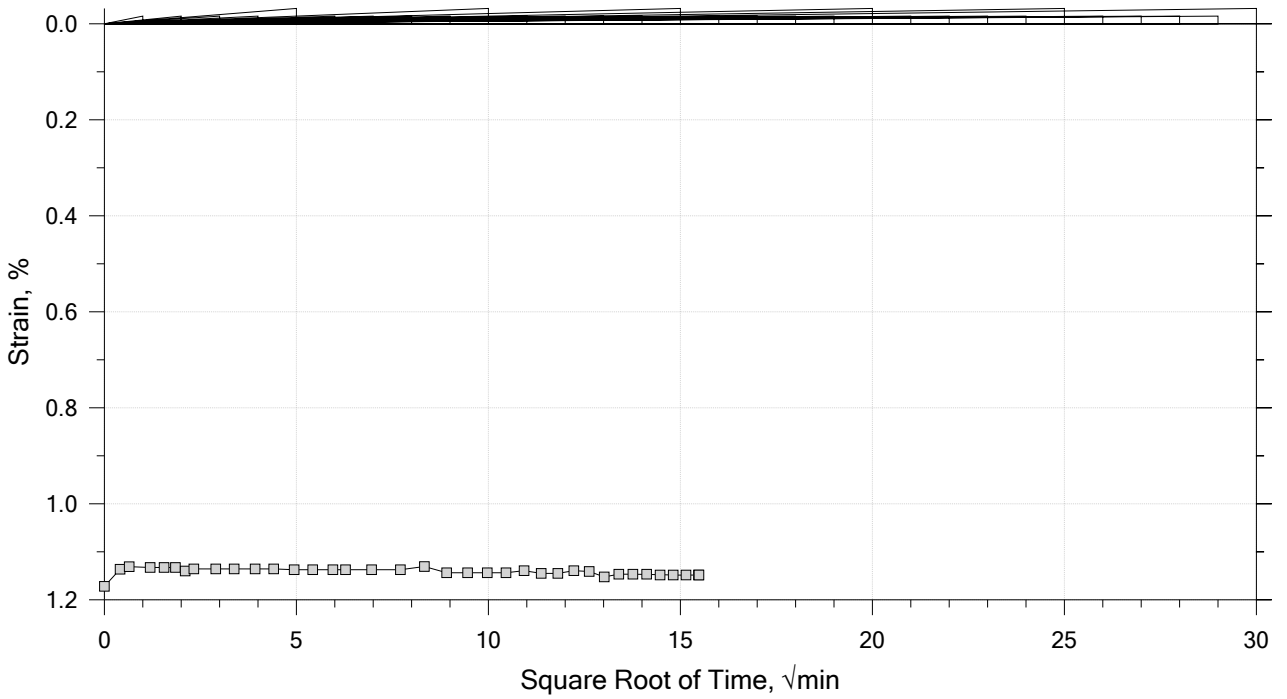


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14
Constant Load Step
Stress: 0.125 tsf

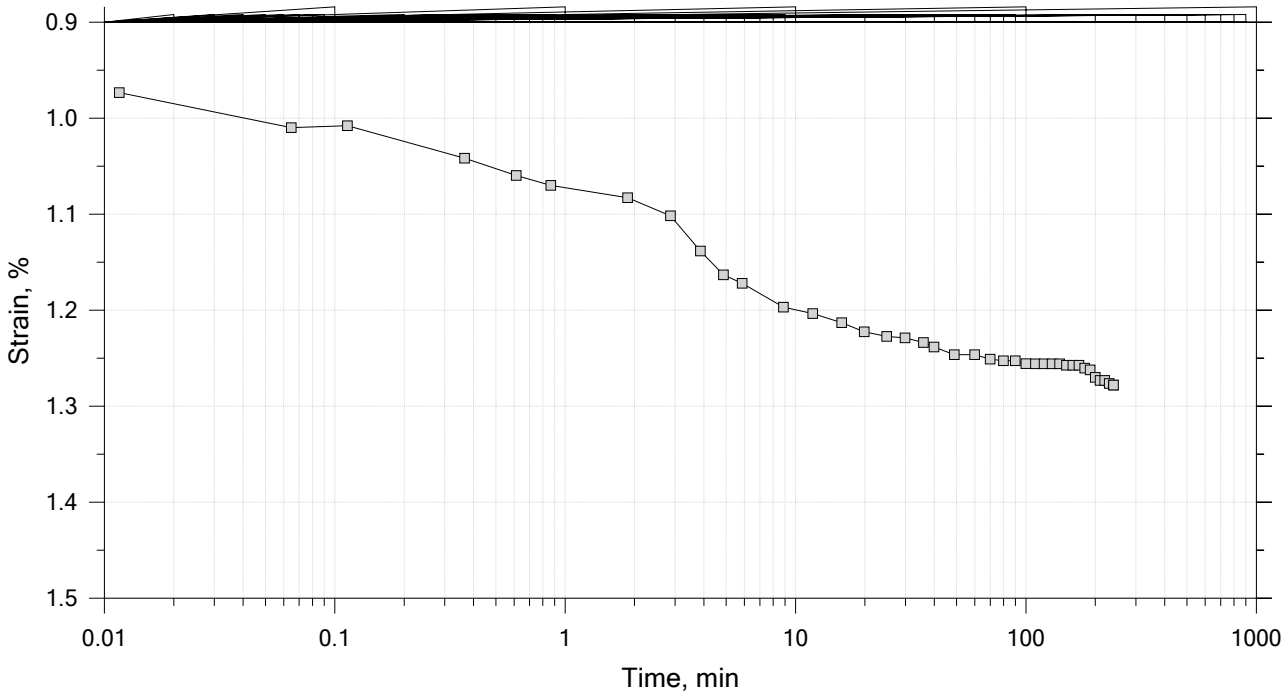


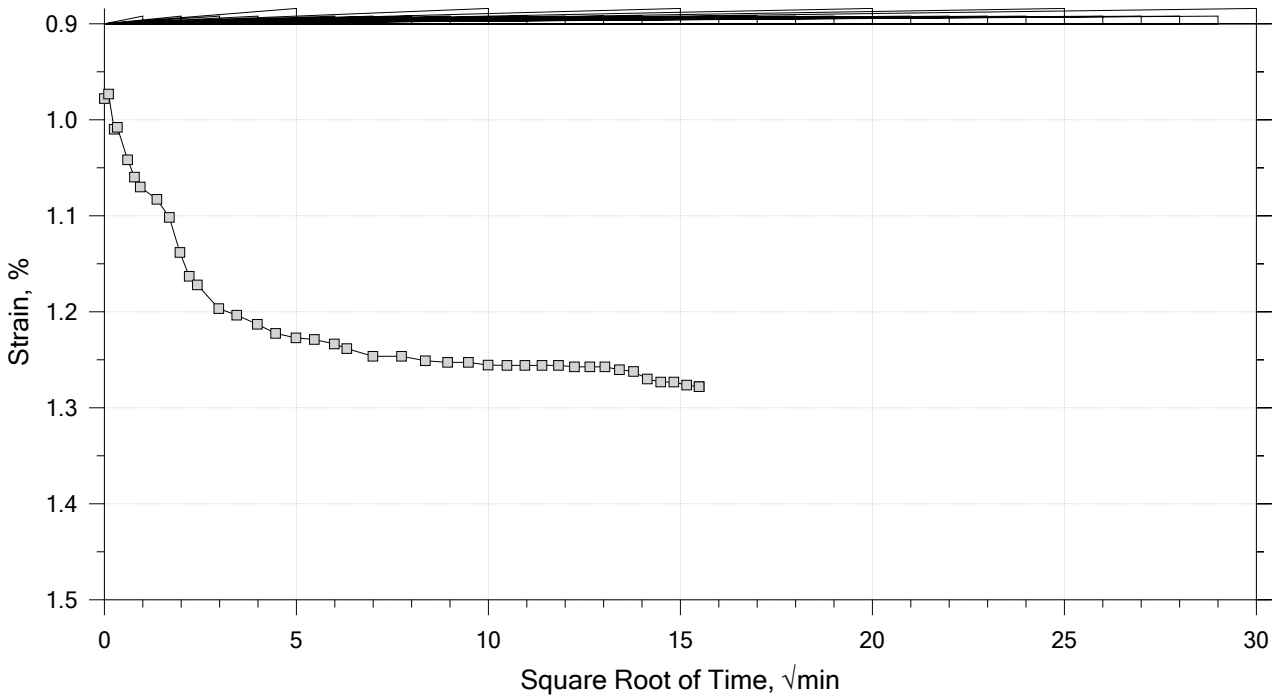


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf

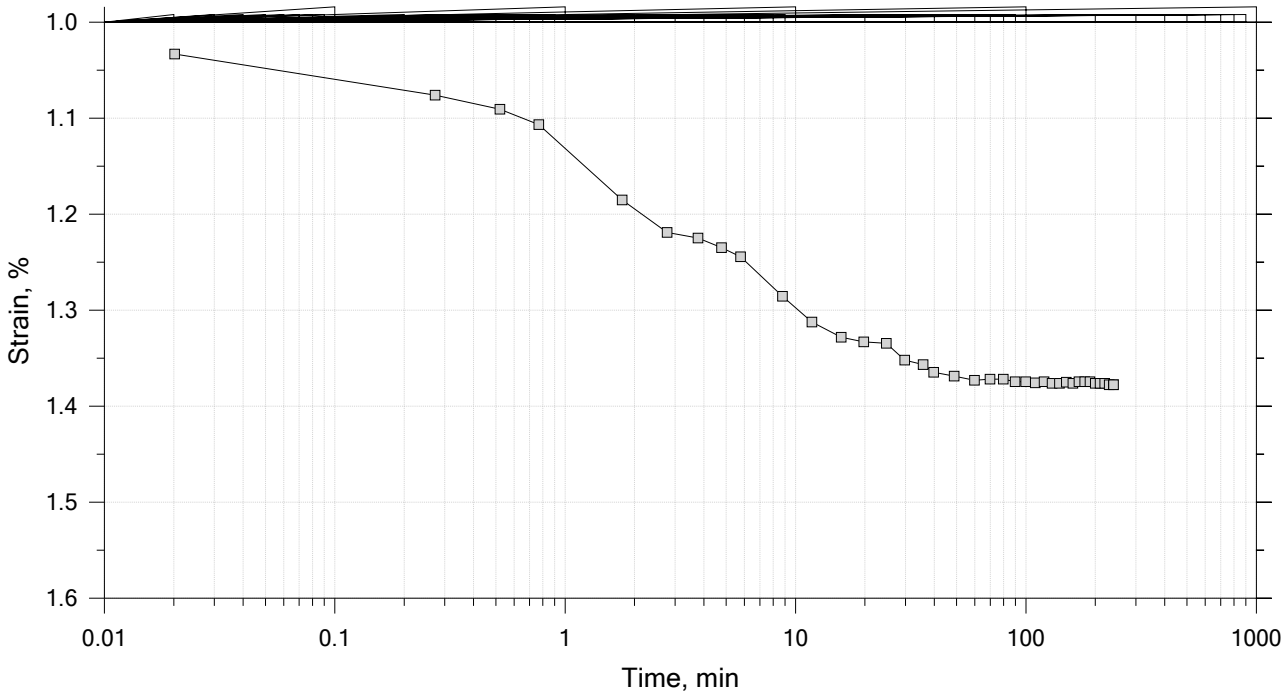


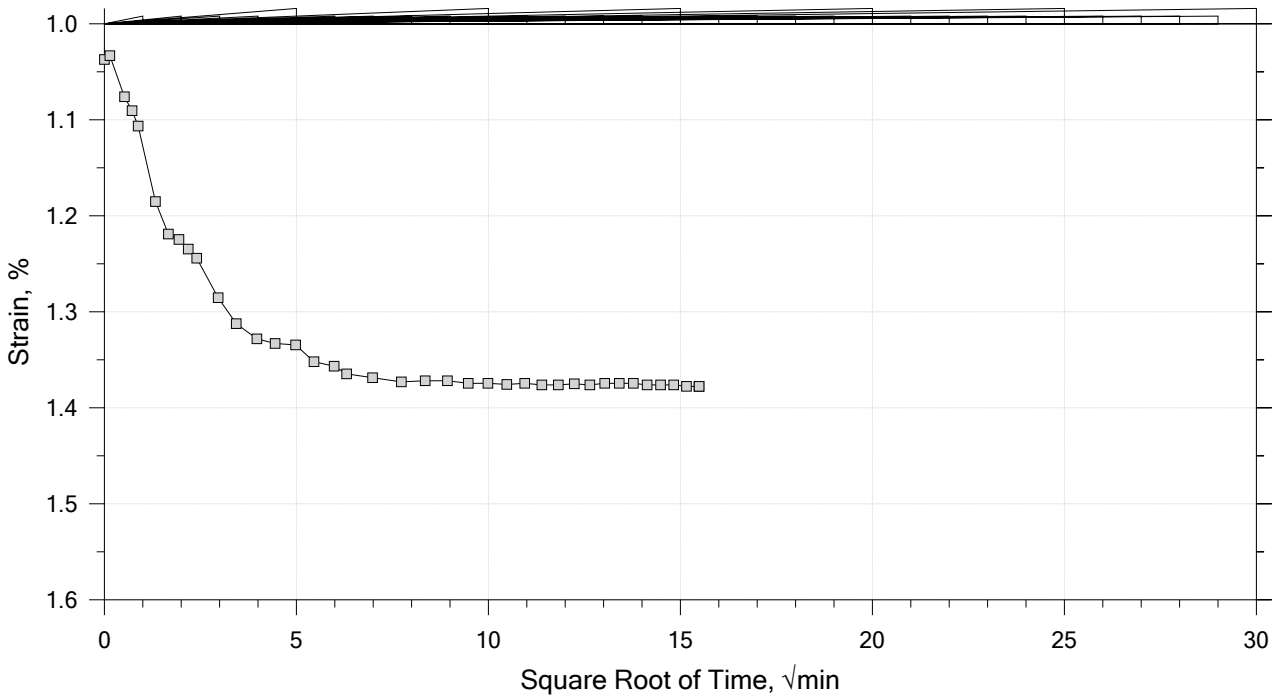


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf

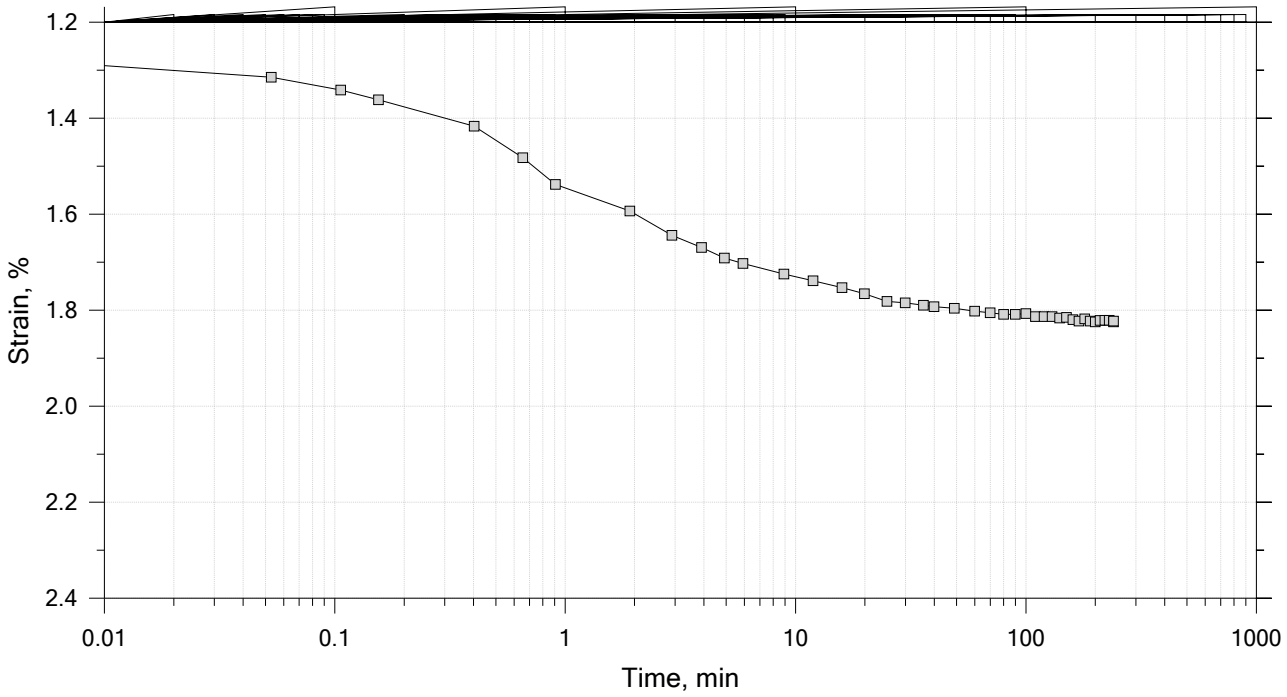


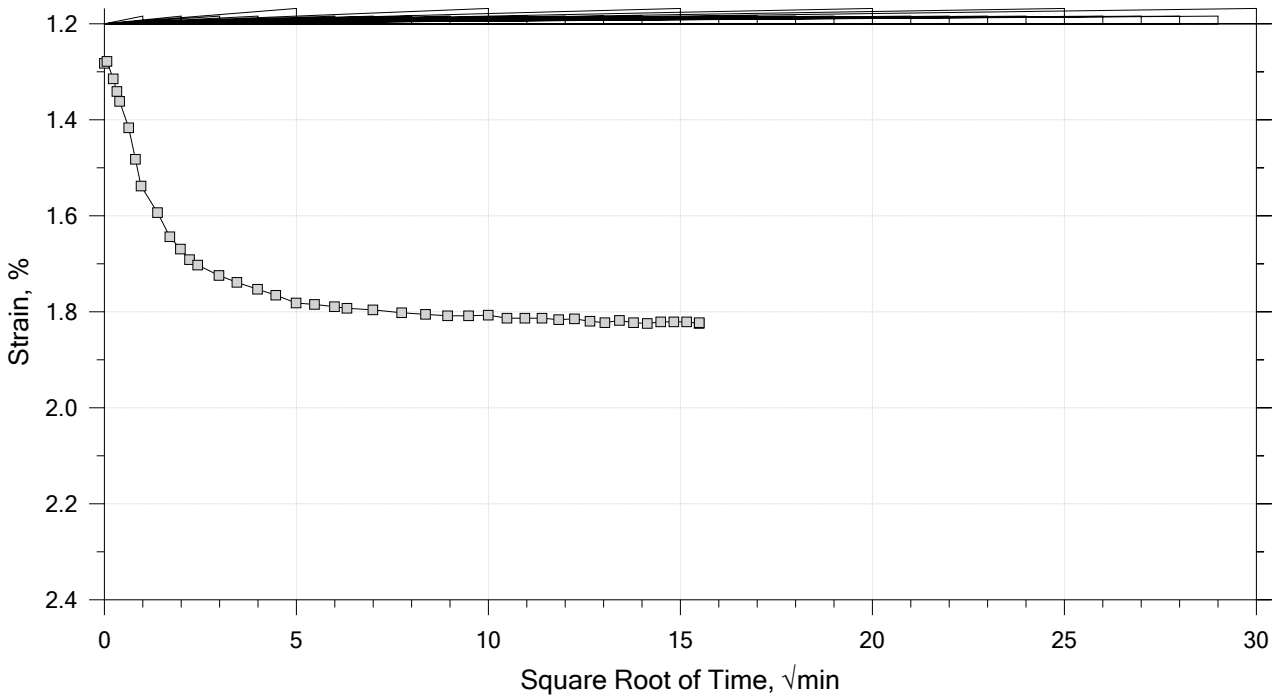


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14
Constant Load Step
Stress: 1 tsf

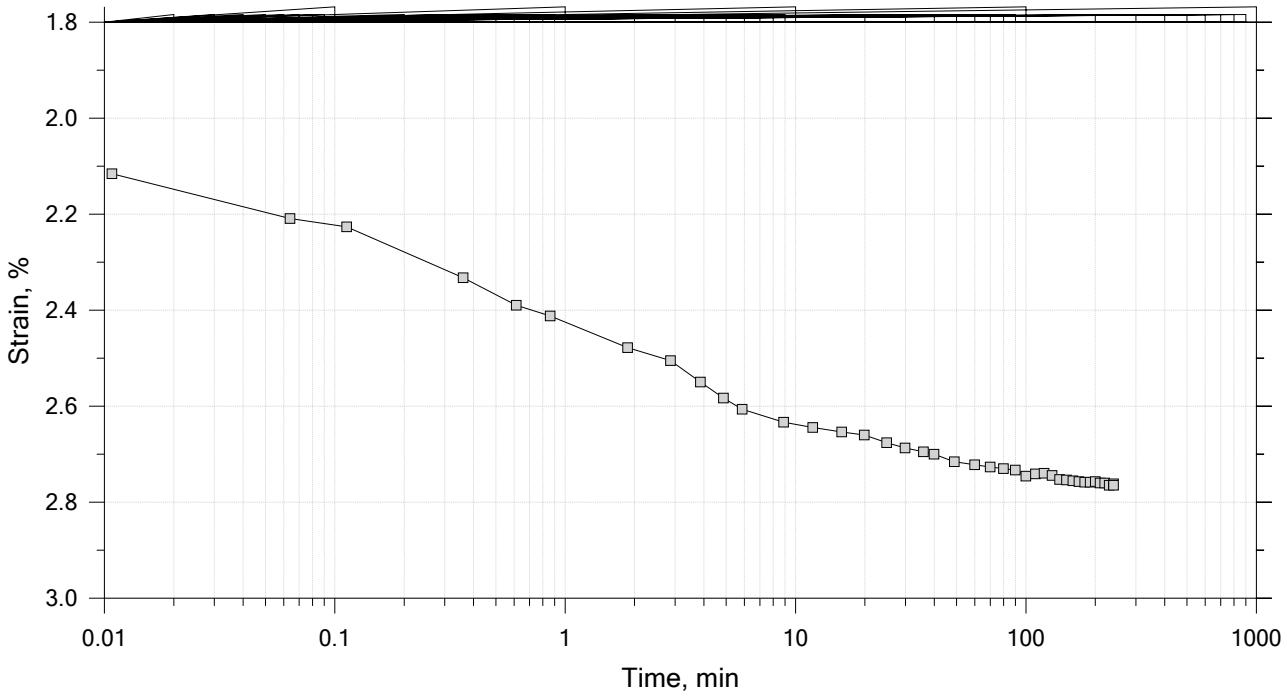


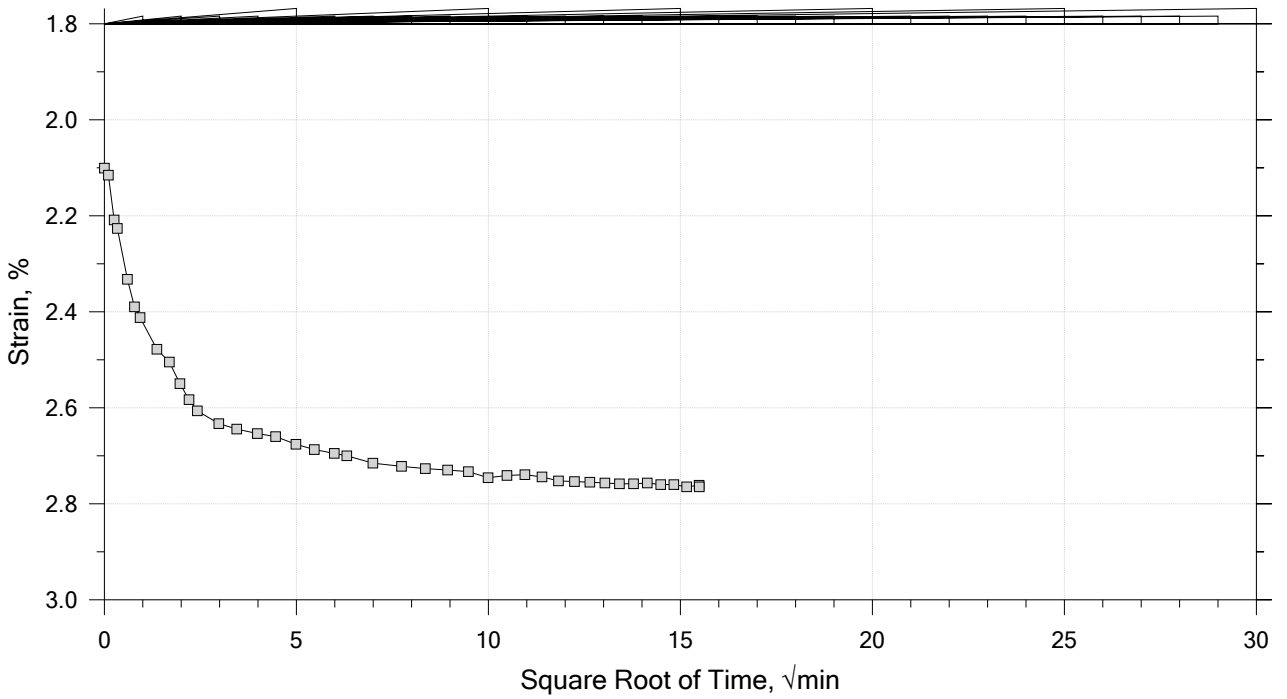


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14
Constant Load Step
Stress: 2 tsf

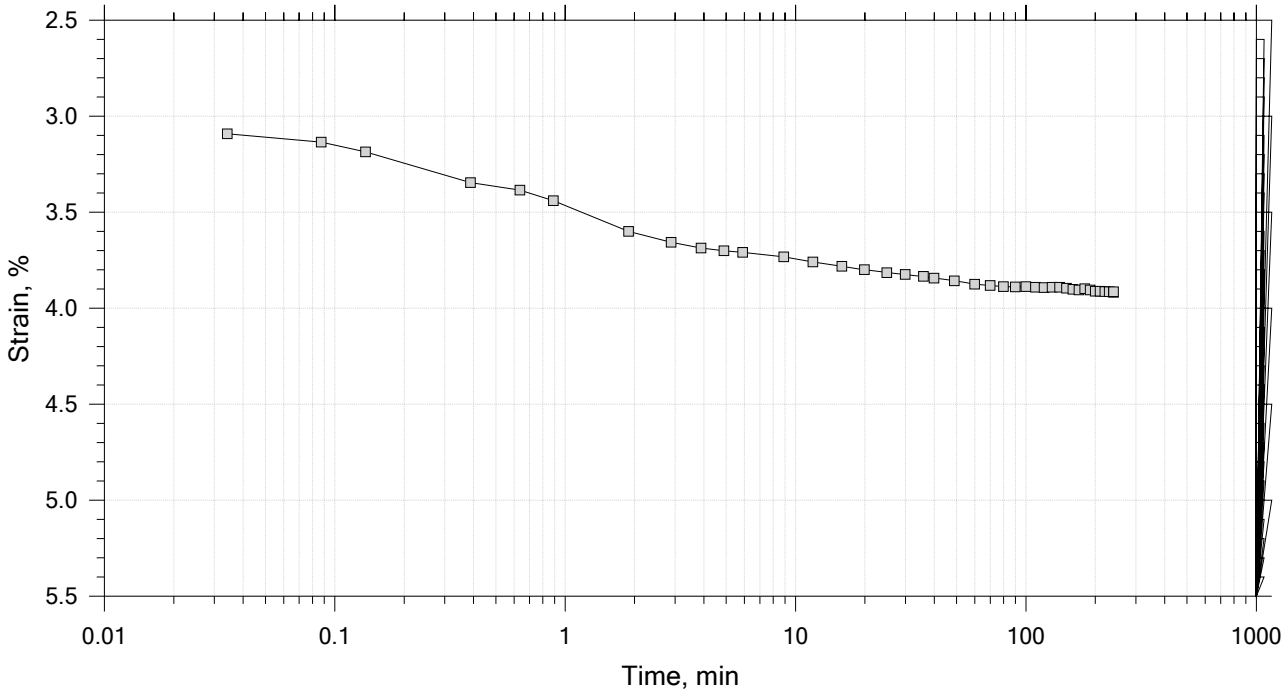


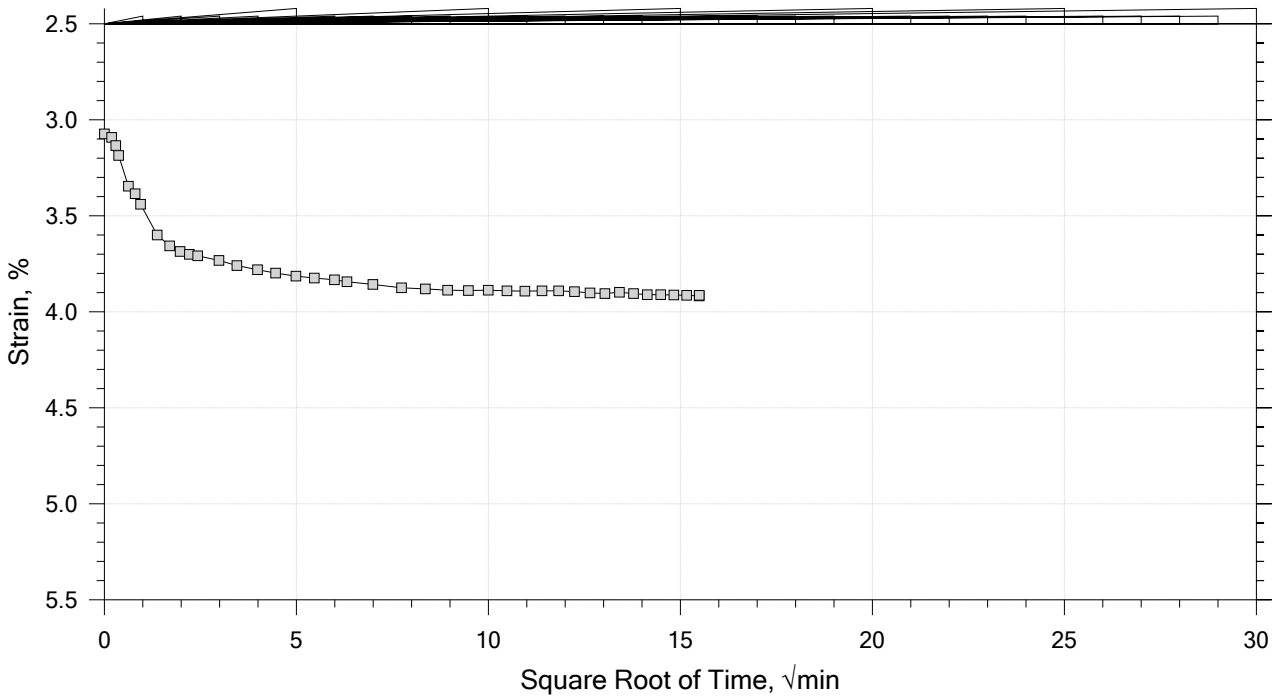


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14
Constant Load Step
Stress: 4 tsf

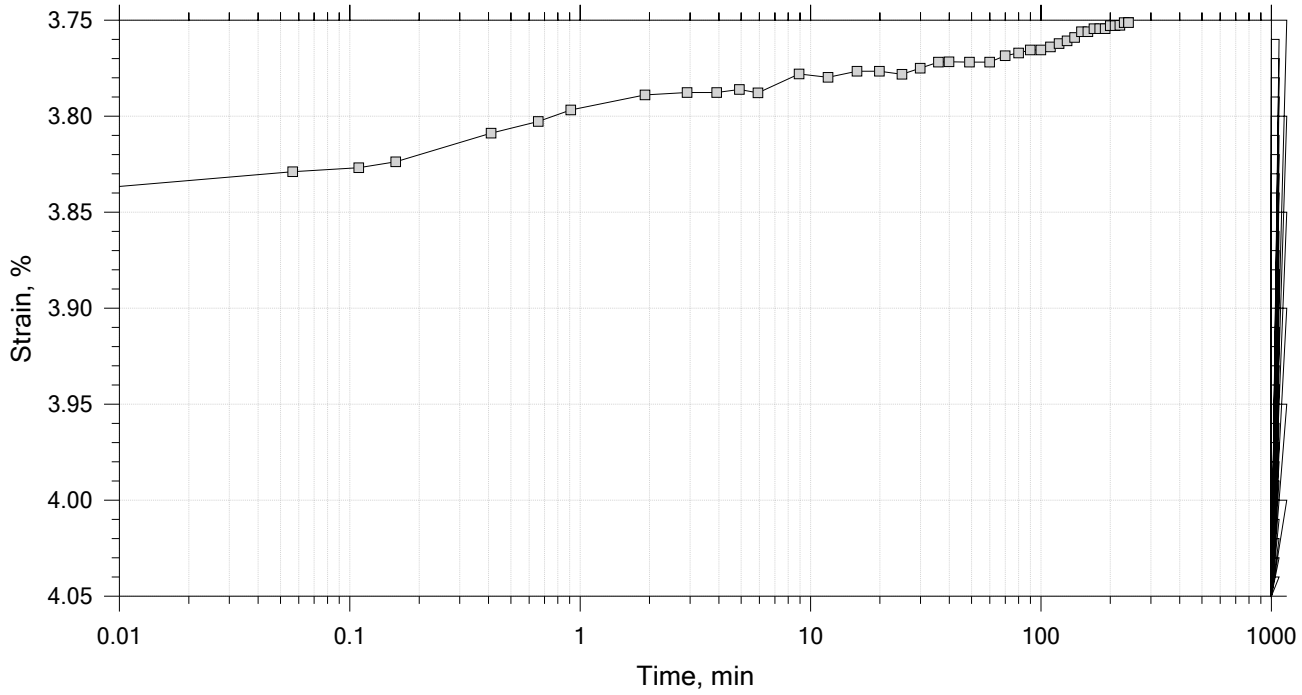


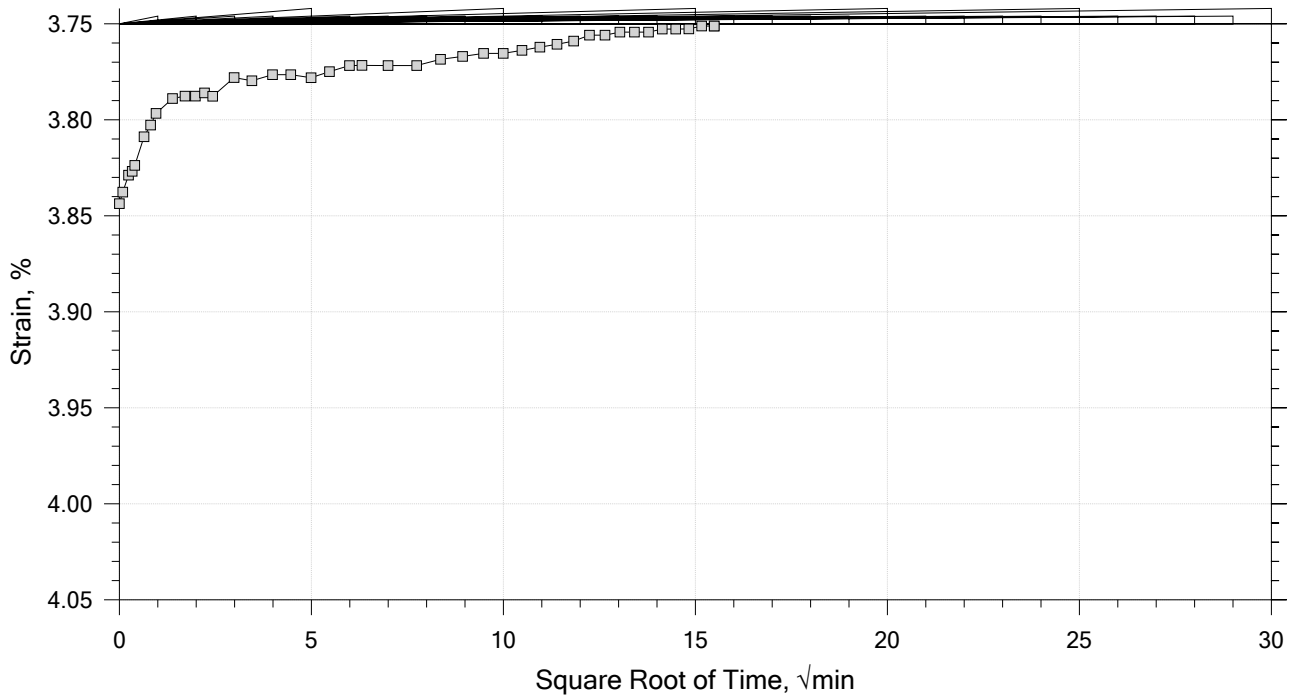


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14
Constant Load Step
Stress: 2 tsf





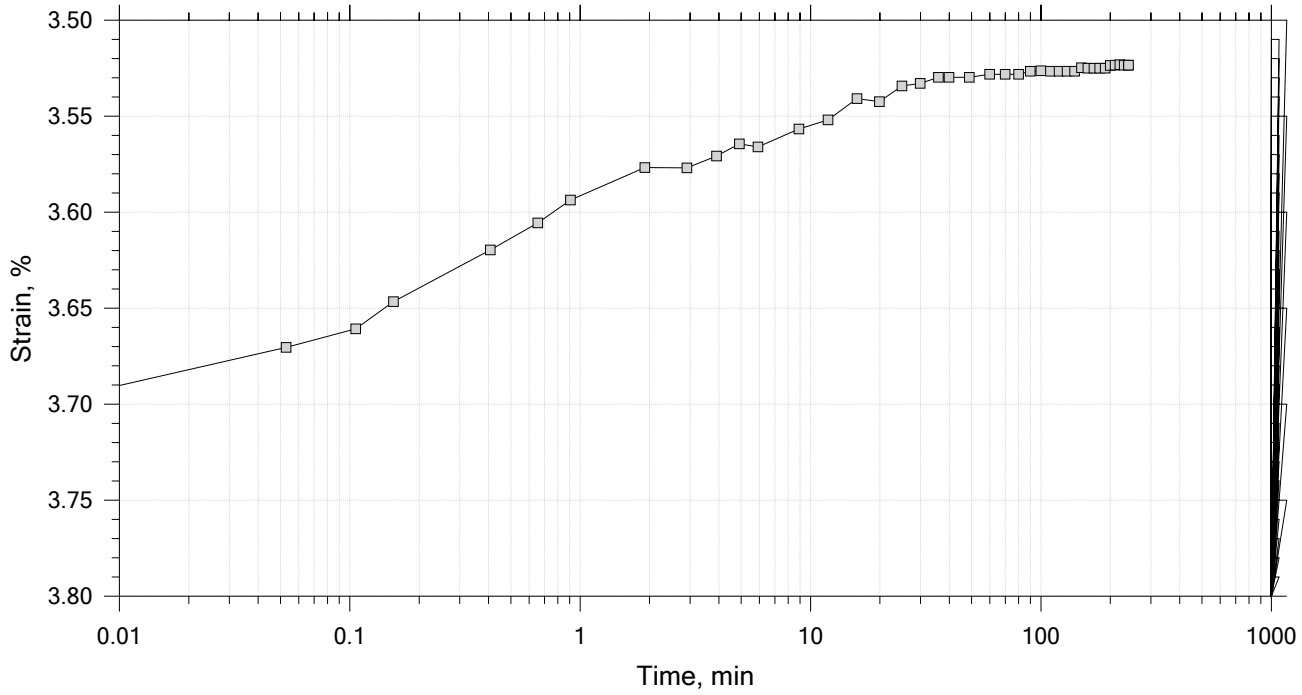
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

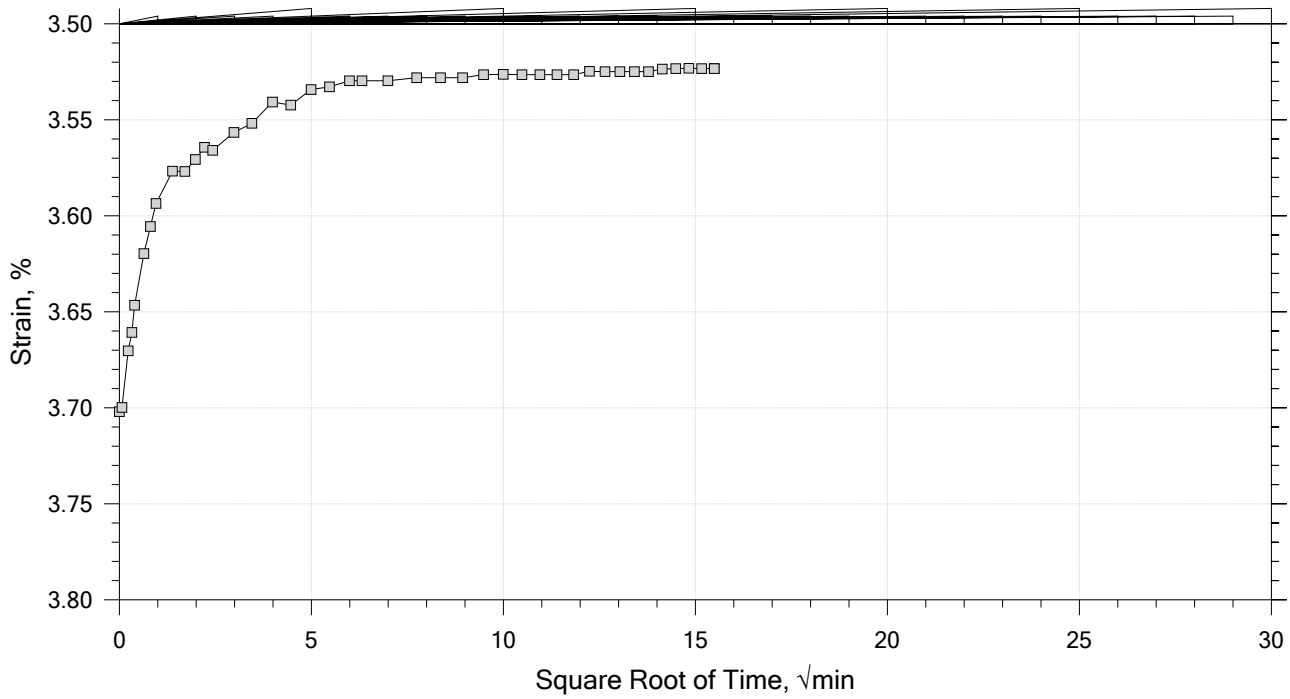
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf

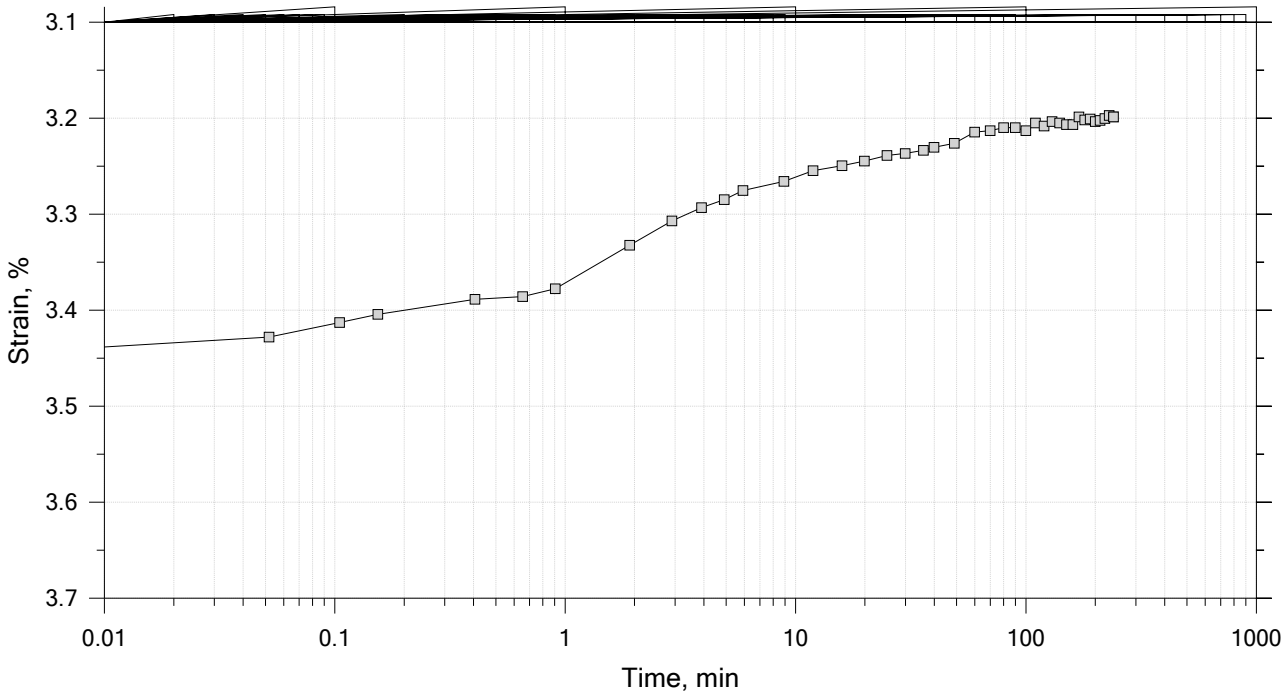


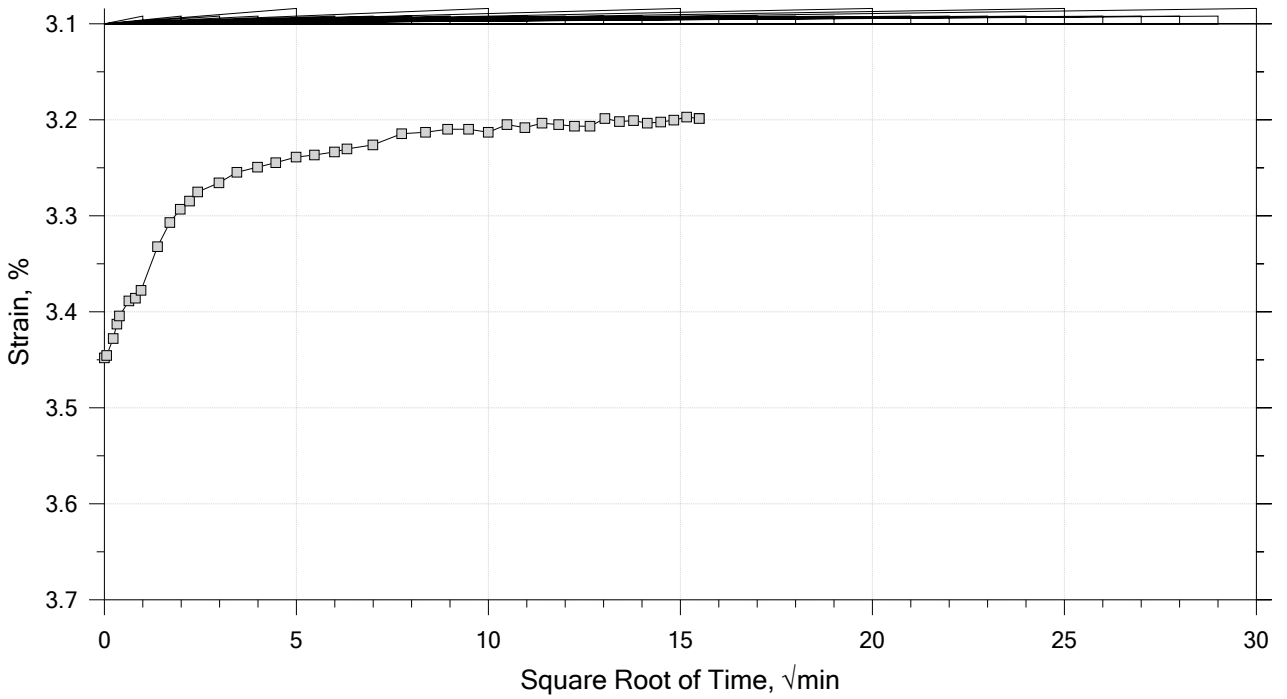


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

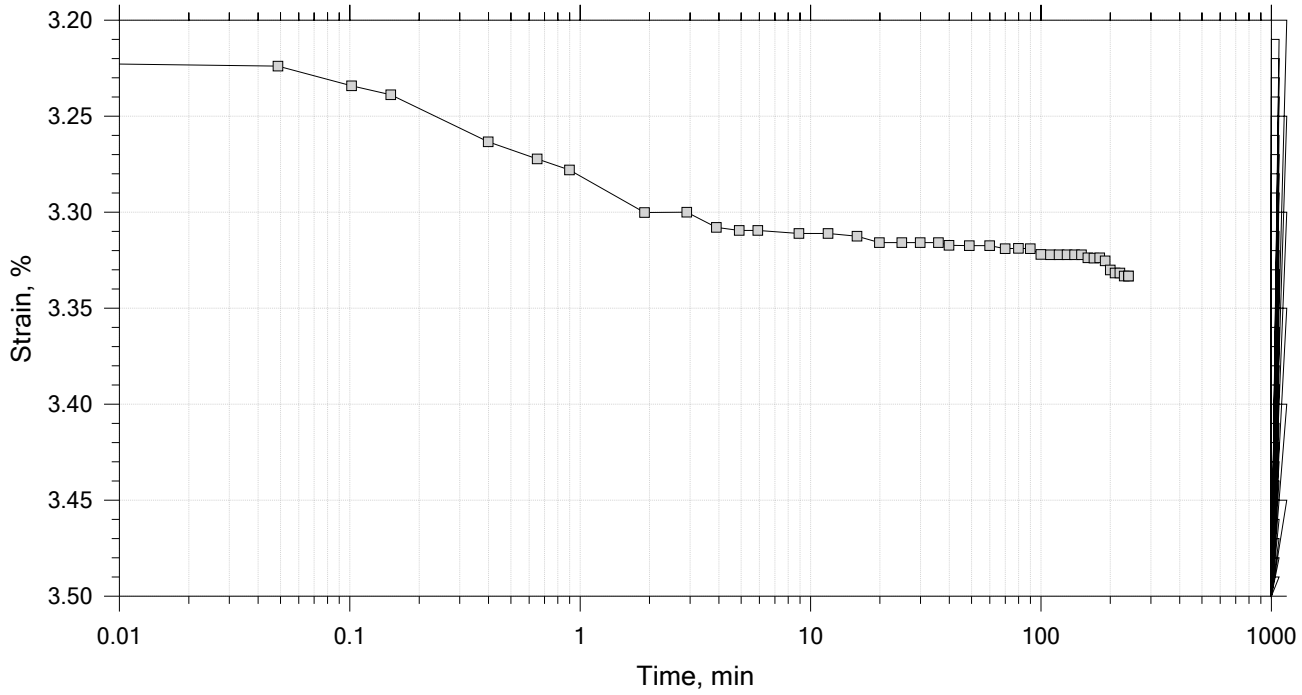


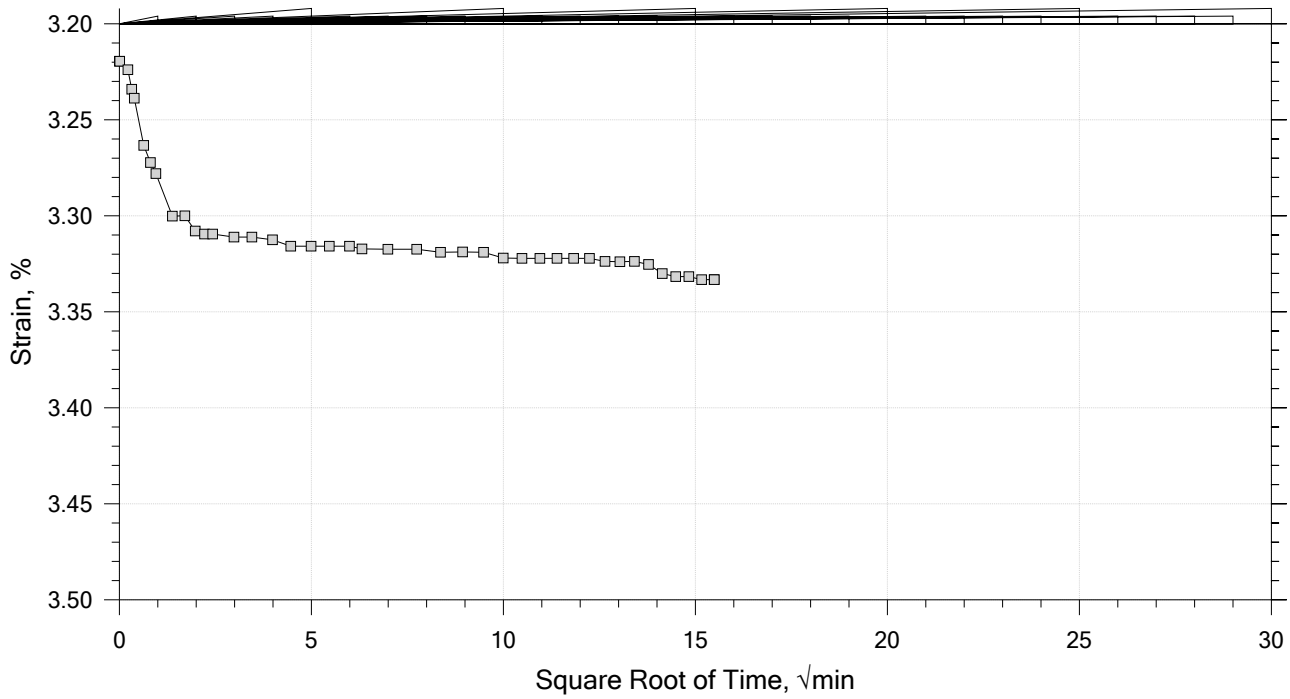


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

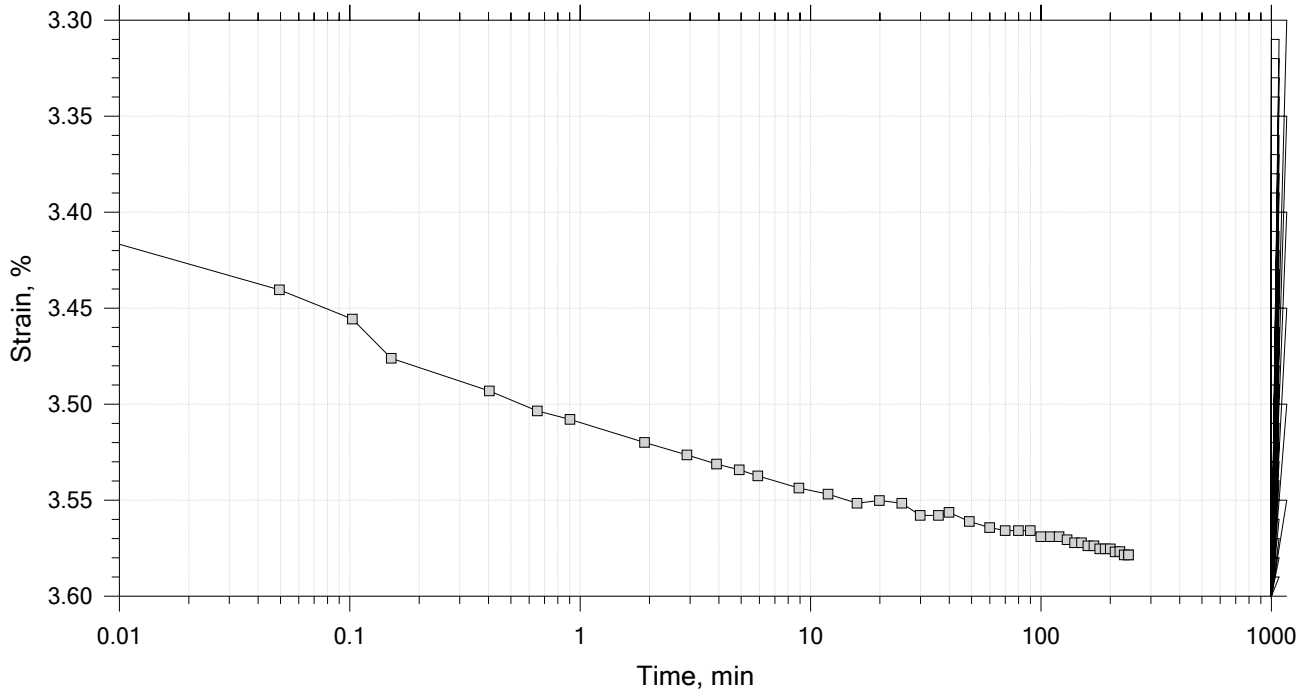


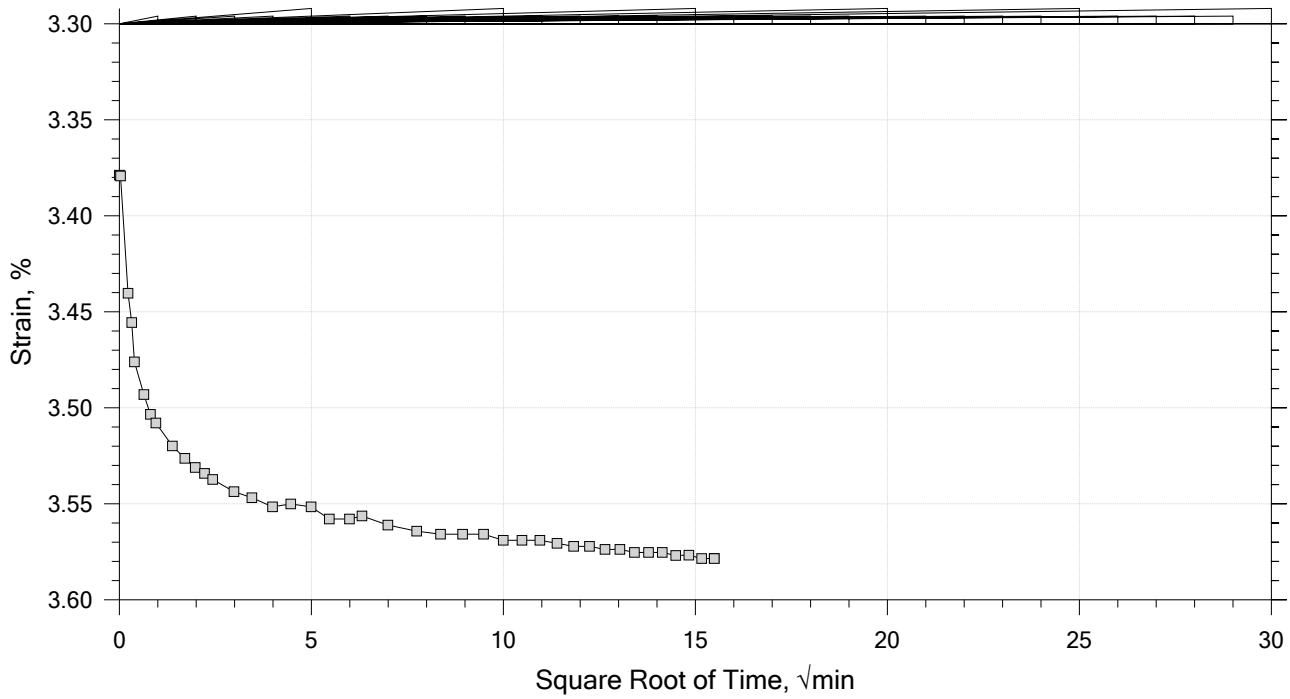


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

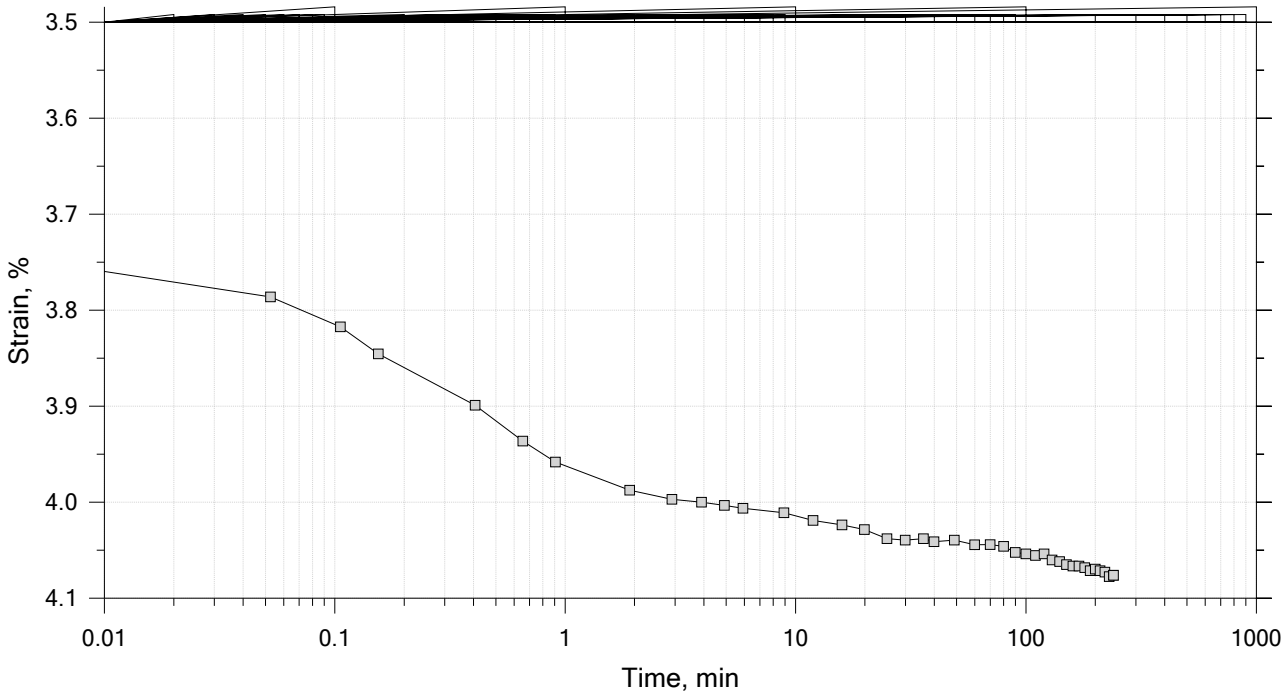


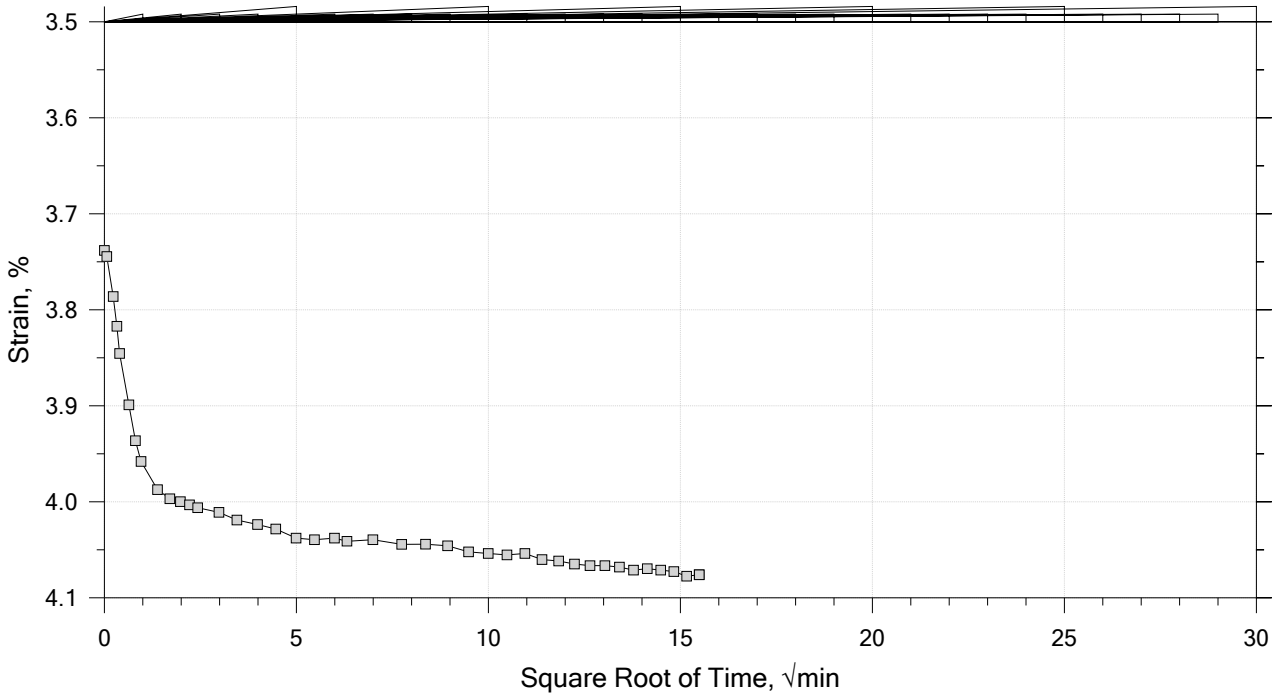


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf

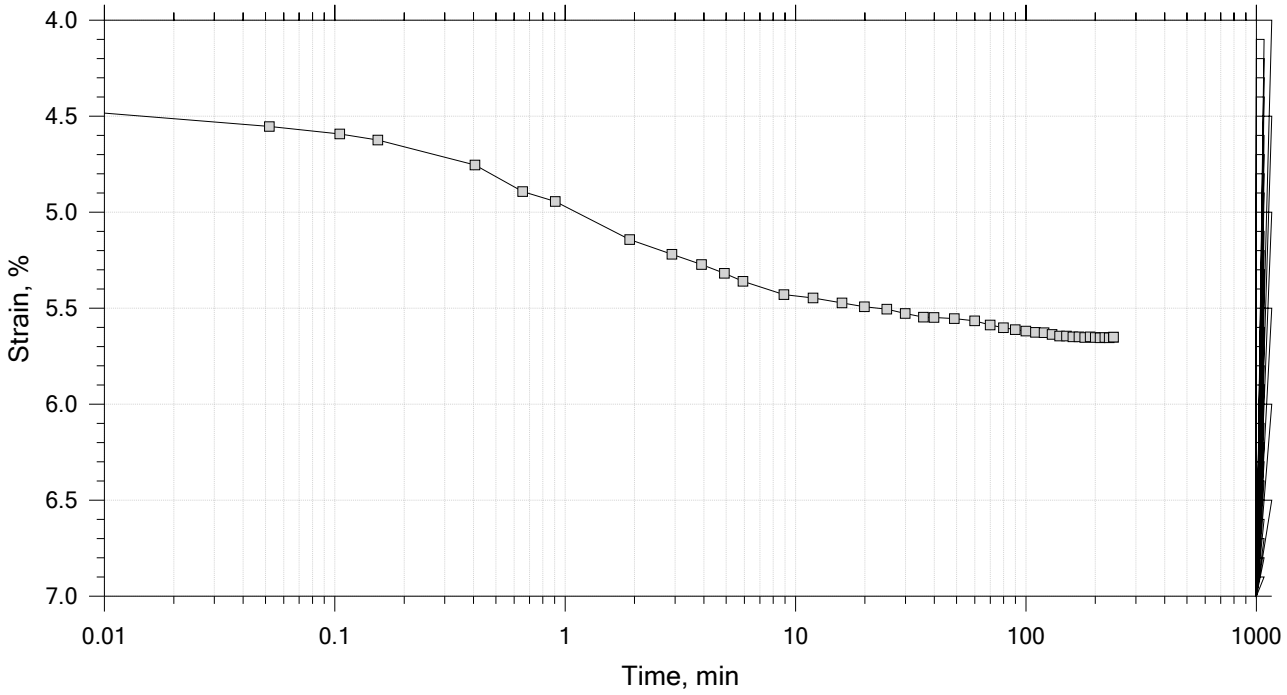


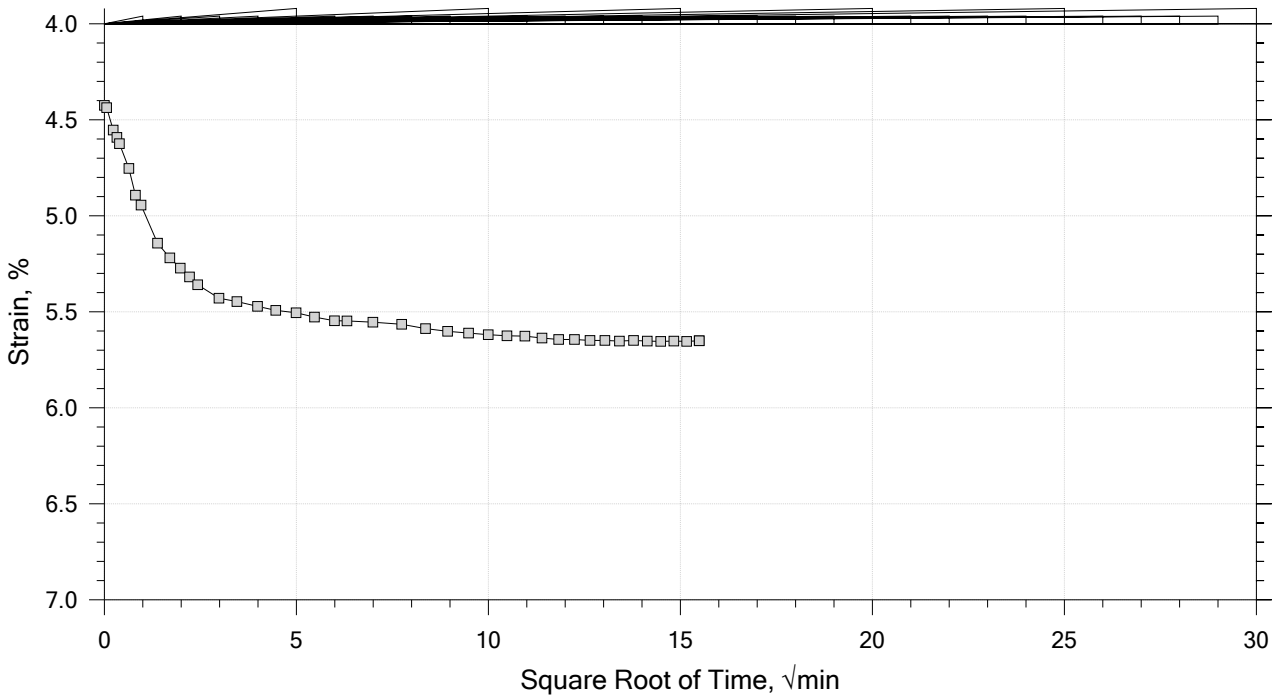


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14
Constant Load Step
Stress: 8 tsf

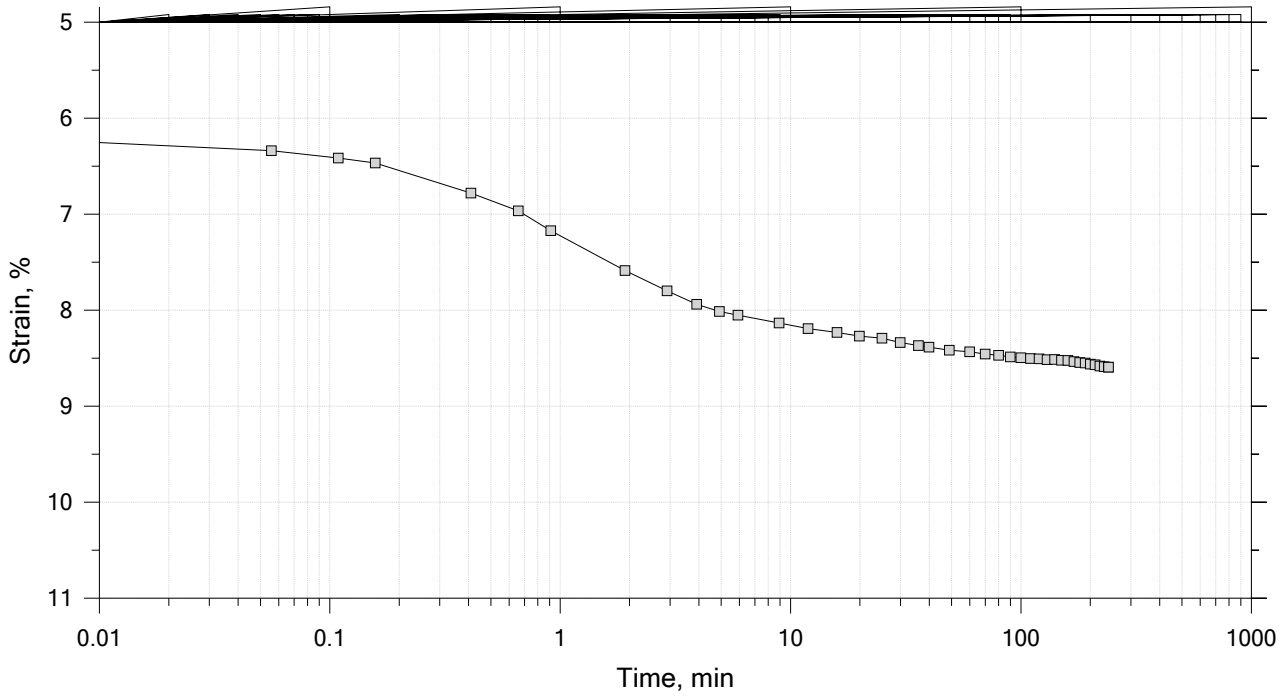


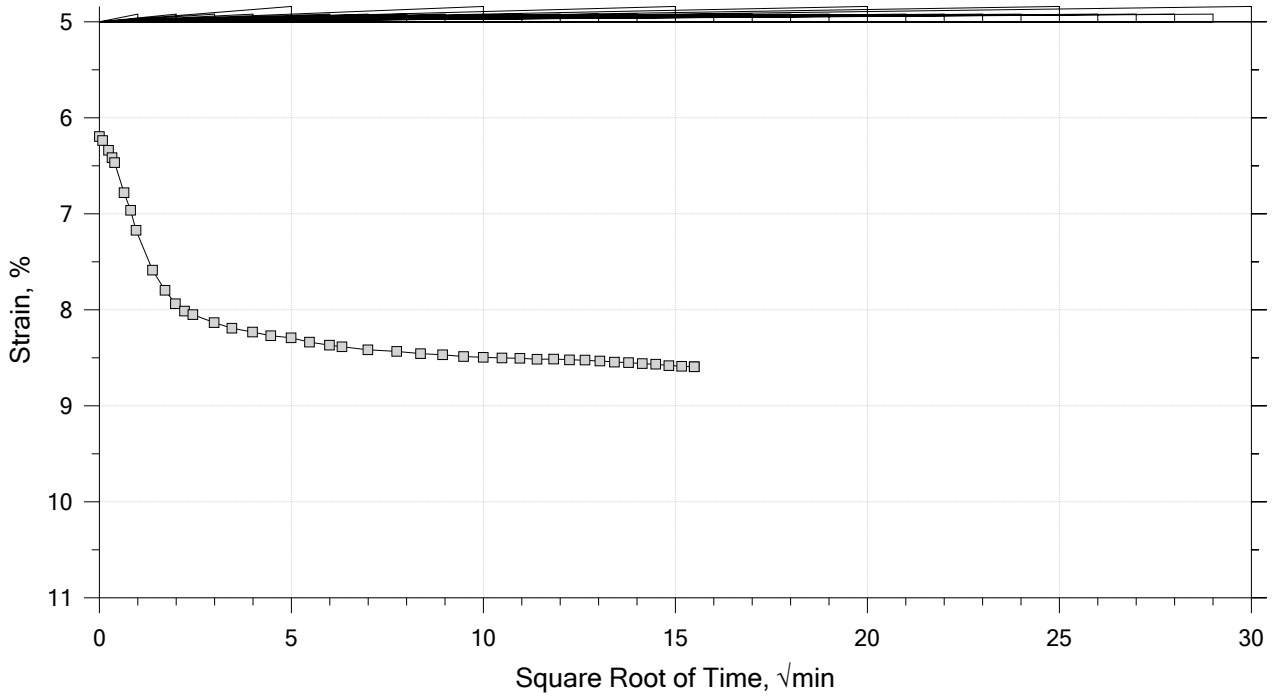


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 14
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.74	Liquid Limit: 35
Initial Height: 1.00 in	Initial Void Ratio: 0.792	Plastic Limit: 20
Final Height: 0.93 in	Final Void Ratio: 0.667	Plasticity Index: 15

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-4074	RING		E5135
Mass Container, gm	8.69	109.06	109.06	8.42
Mass Container + Wet Soil, gm	100.45	266.66	262	160.82
Mass Container + Dry Soil, gm	80.21	232.07	232.07	131
Mass Dry Soil, gm	71.52	123.01	123.01	122.58
Water Content, %	28.30	28.12	24.33	24.33
Void Ratio	---	0.79	0.67	---
Degree of Saturation, %	---	97.27	100.00	---
Dry Unit Weight, pcf	---	95.469	102.66	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Square Root of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.125	0.01148	0.772	1.15	---	---	9.19e-02	---
2	0.250	0.01278	0.769	1.28	10.887	3.17e-04	1.04e-02	6.15e-05
3	0.500	0.01378	0.768	1.38	12.407	2.77e-04	3.99e-03	2.07e-05
4	1.00	0.01823	0.760	1.82	4.989	6.86e-04	8.90e-03	1.14e-04
5	2.00	0.02765	0.743	2.76	6.017	5.61e-04	9.42e-03	9.89e-05
6	4.00	0.03914	0.722	3.91	4.330	7.62e-04	5.75e-03	8.21e-05
7	2.00	0.03751	0.725	3.75	220.825	1.48e-05	8.16e-04	2.26e-07
8	1.00	0.03523	0.729	3.52	1.907	1.72e-03	2.28e-03	7.34e-05
9	0.500	0.03199	0.735	3.20	6.250	5.28e-04	6.49e-03	6.42e-05
10	1.00	0.03333	0.732	3.33	1.892	1.75e-03	2.69e-03	8.81e-05
11	2.00	0.03579	0.728	3.58	2.860	1.15e-03	2.45e-03	5.29e-05
12	4.00	0.04076	0.719	4.08	1.887	1.73e-03	2.49e-03	8.07e-05
13	8.00	0.05651	0.691	5.65	4.975	6.43e-04	3.94e-03	4.74e-05
14	16.0	0.08593	0.638	8.59	5.739	5.31e-04	3.68e-03	3.66e-05

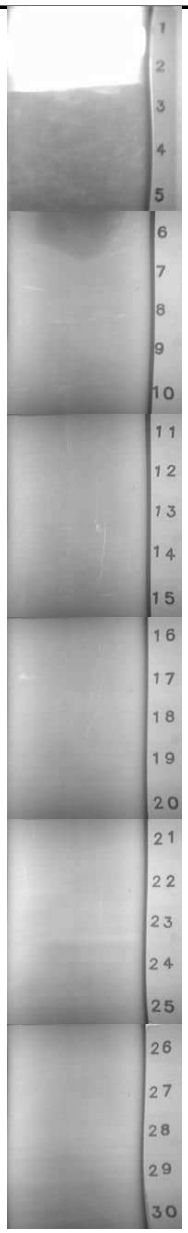
GeoTesting EXPRESS

Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-E		
Displacement at End of Increment		

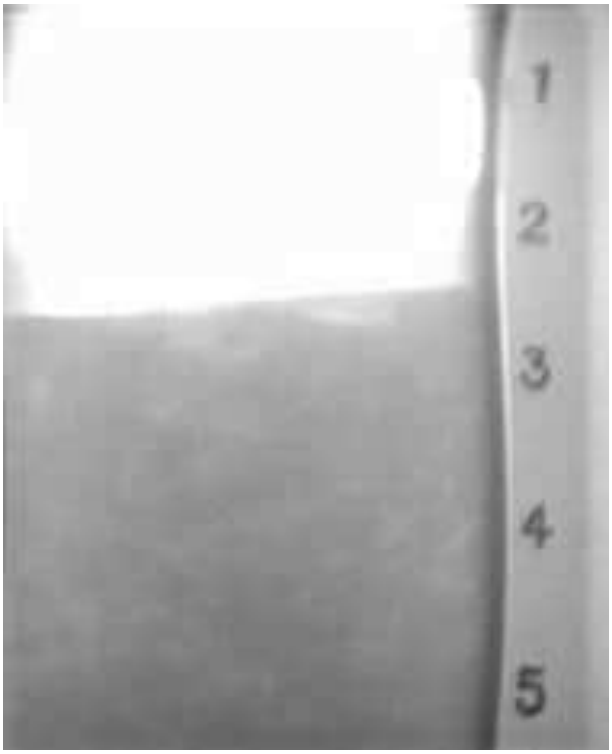
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



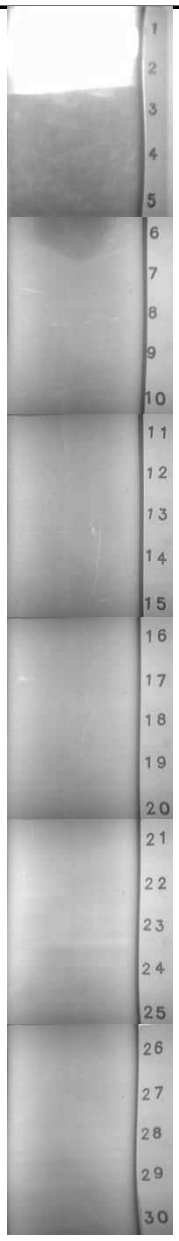
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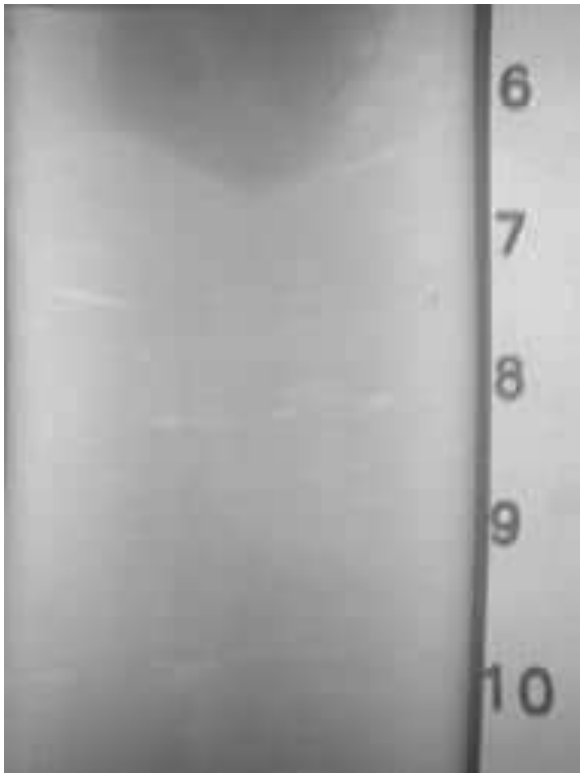
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube



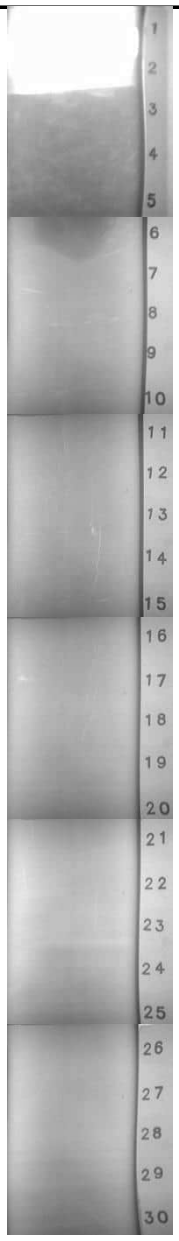
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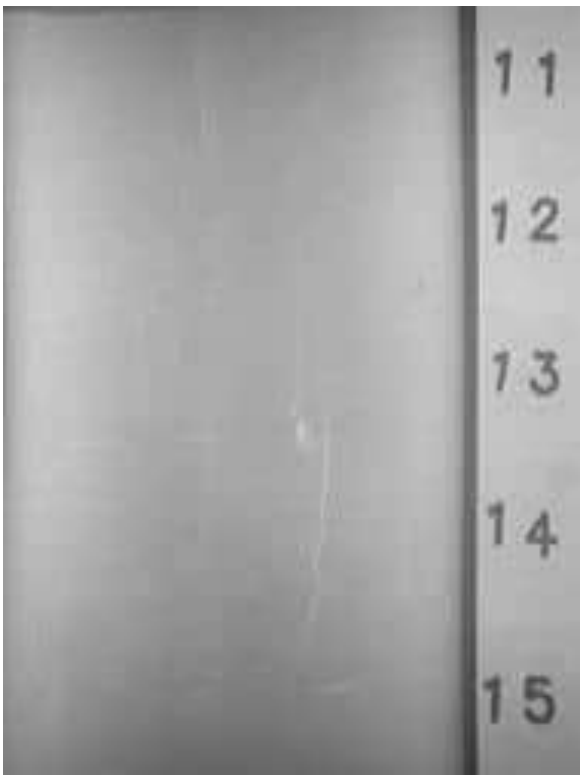
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
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	Bottom of Tube



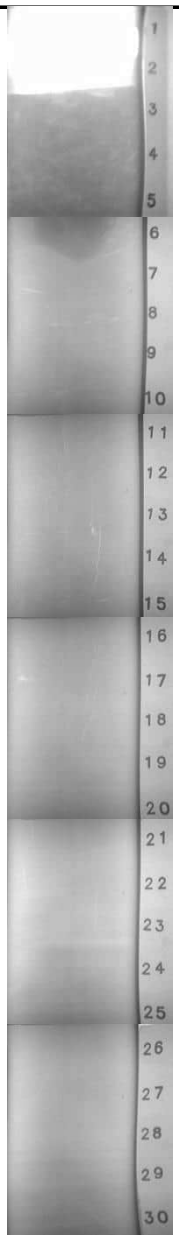
SECTION 3



Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



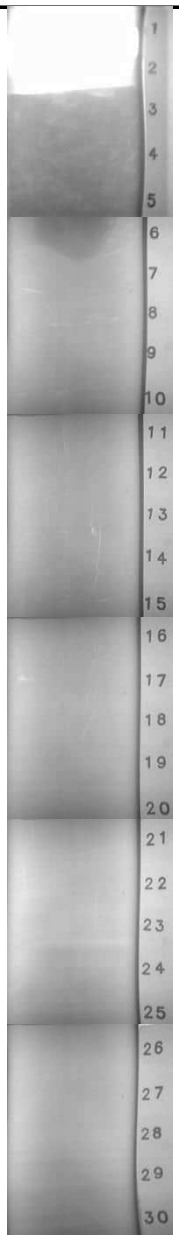
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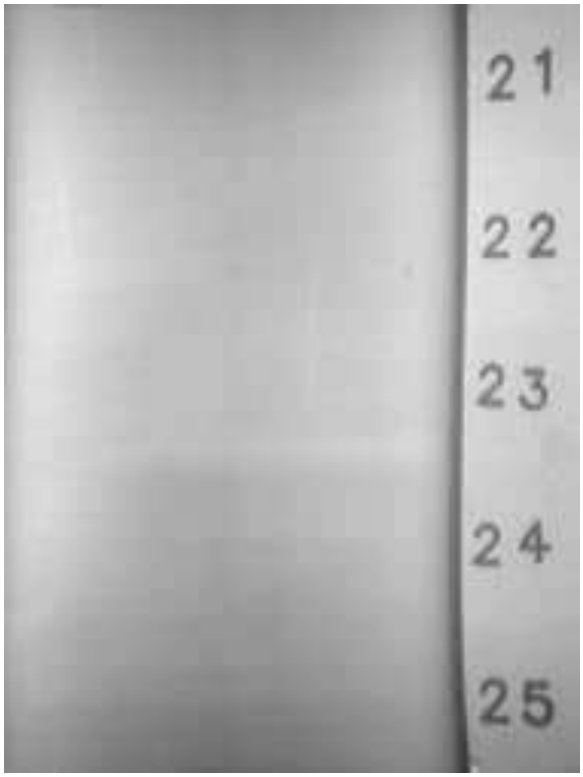
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Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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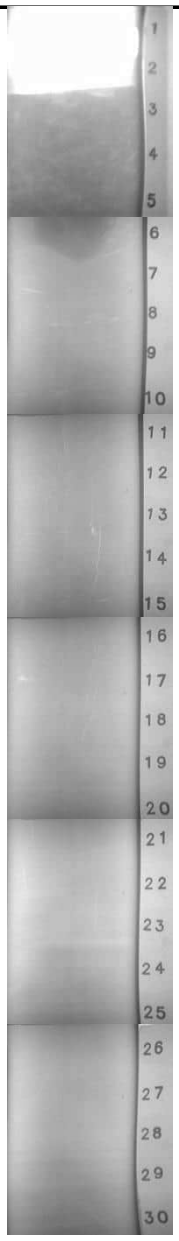
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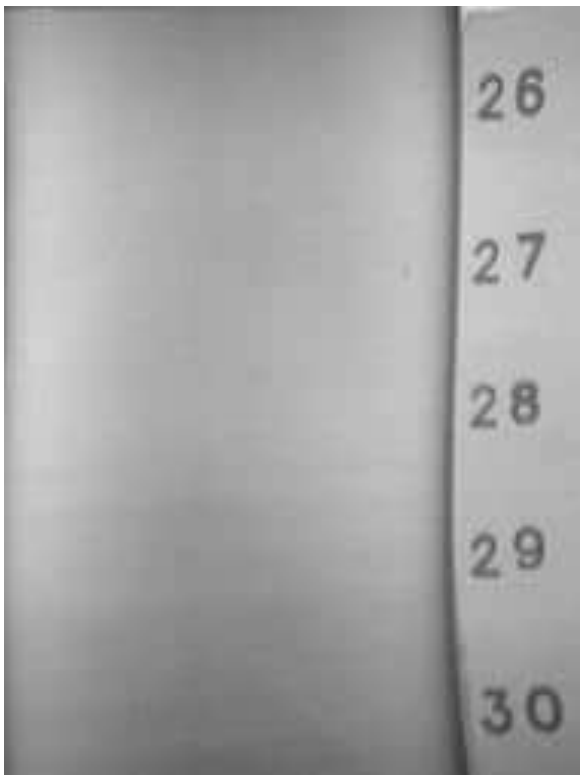
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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2	
3	
4	
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6	
	Bottom of Tube

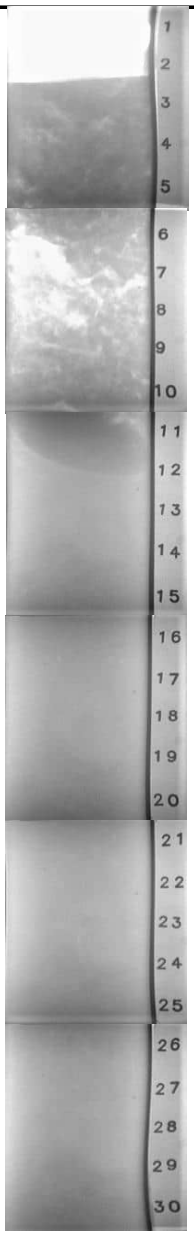


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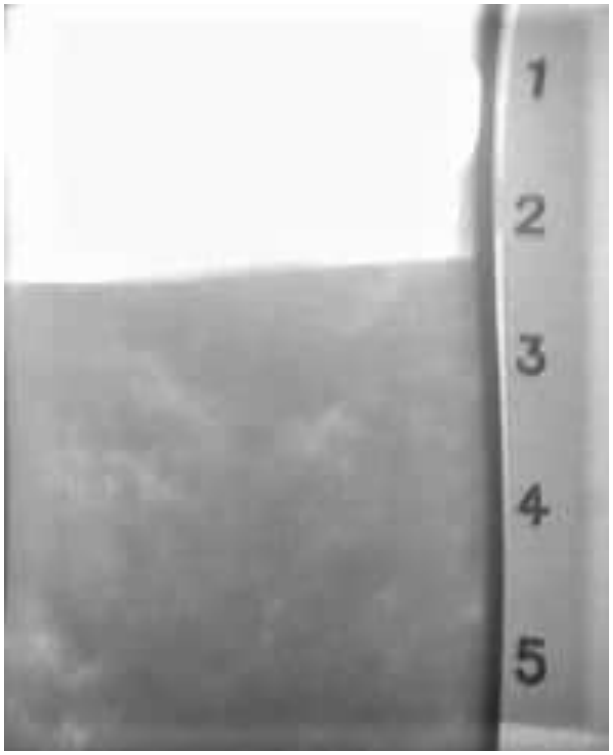


Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
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	Bottom of Tube

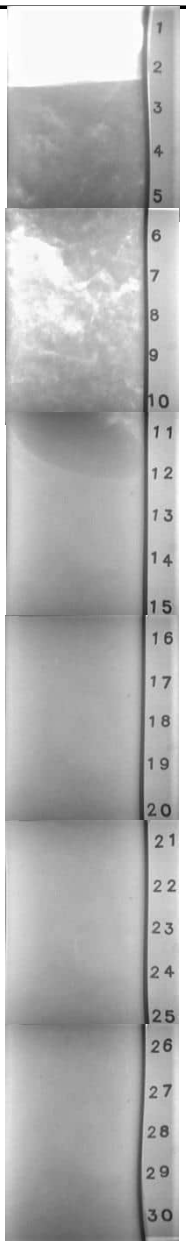
SECTION 1



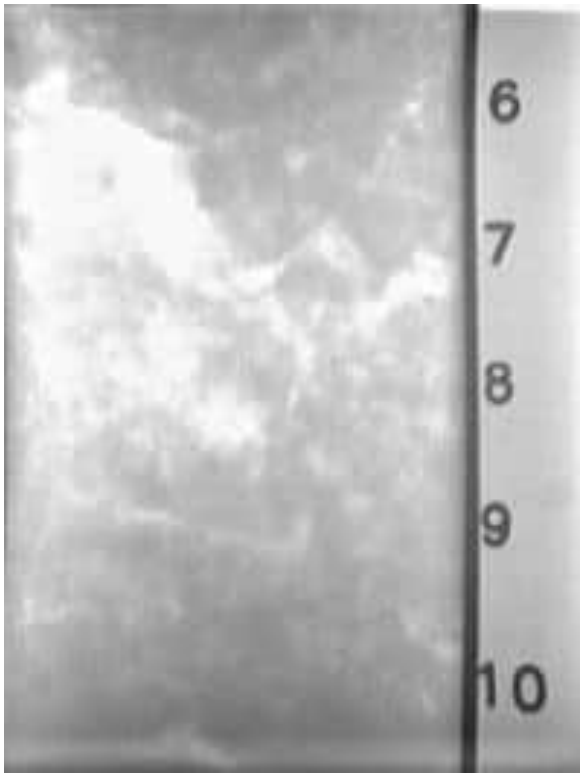
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube



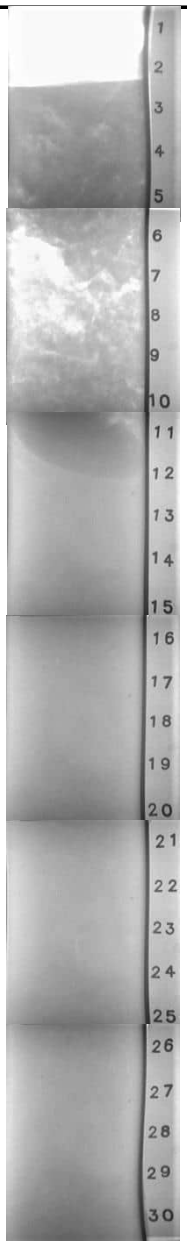
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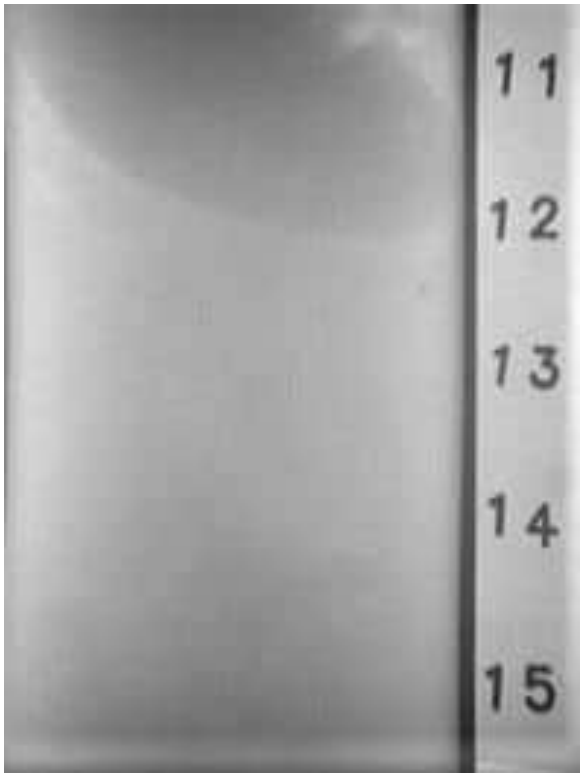
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Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube



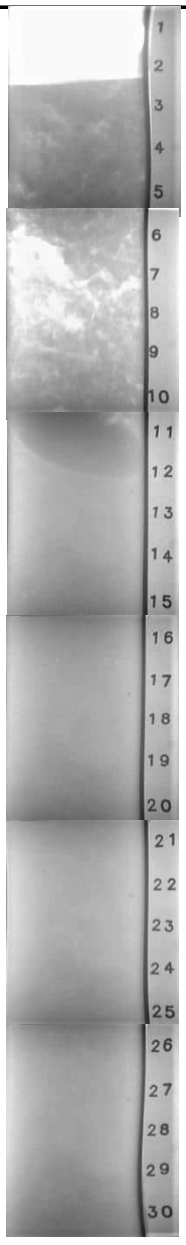
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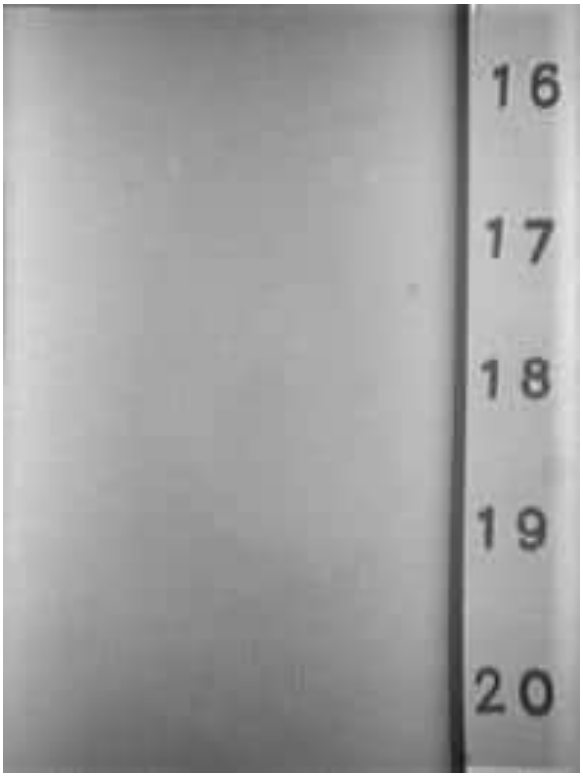
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Project Location:	Bangor, ME
GTX #:	315483
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Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

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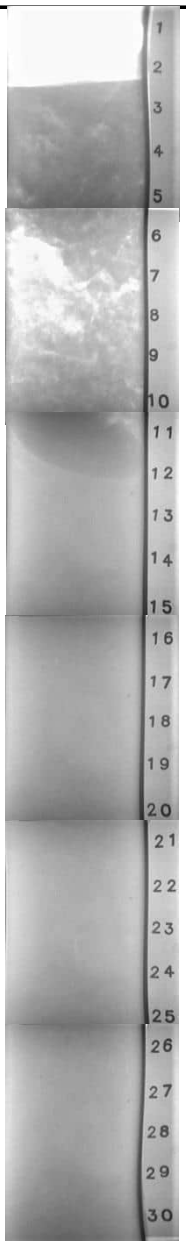
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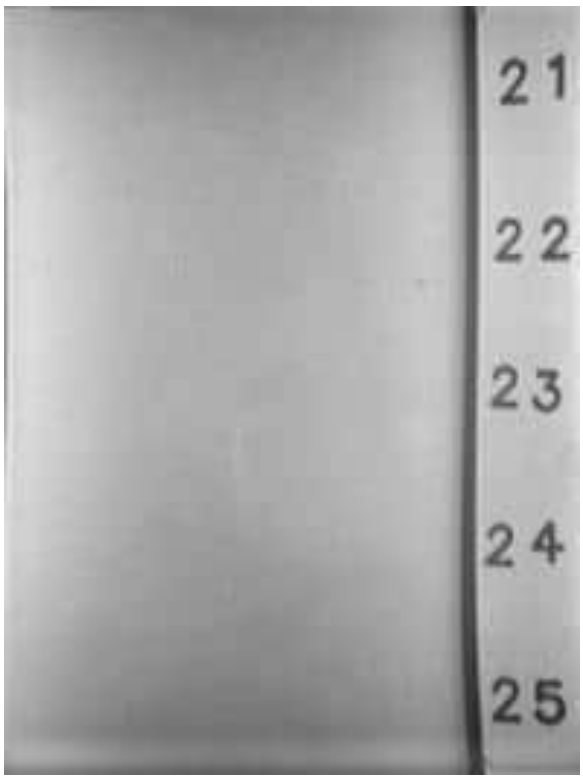
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Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube



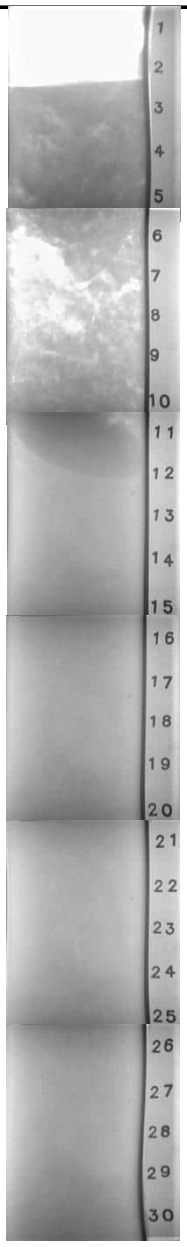
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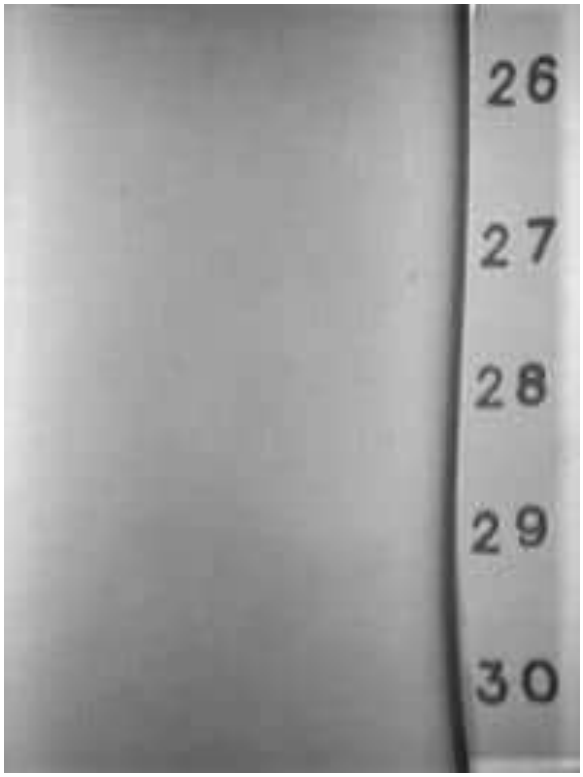
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
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Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube

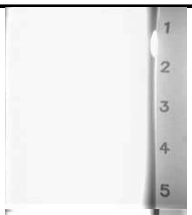
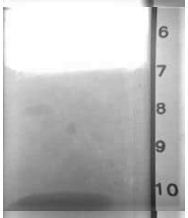
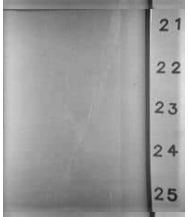
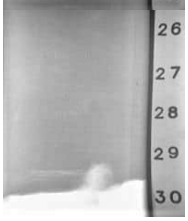


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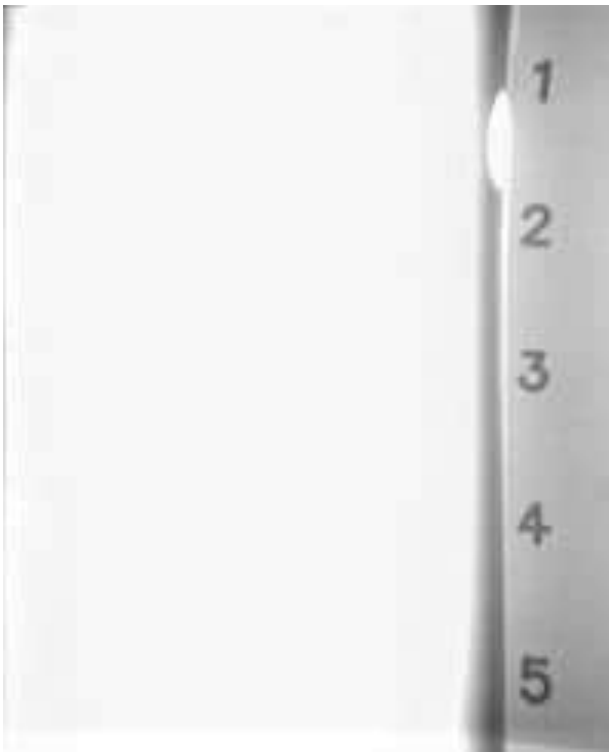


Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-108
Sample ID:	1U
Depth:	80-10 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube

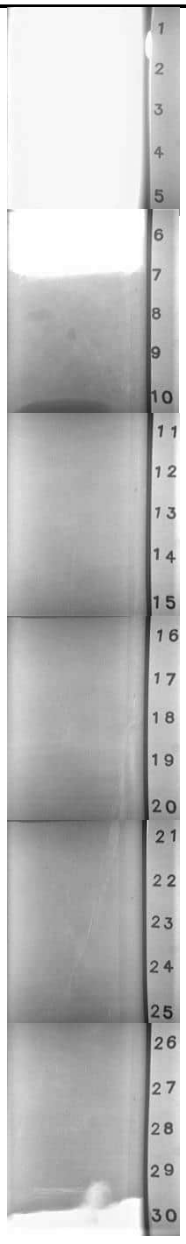
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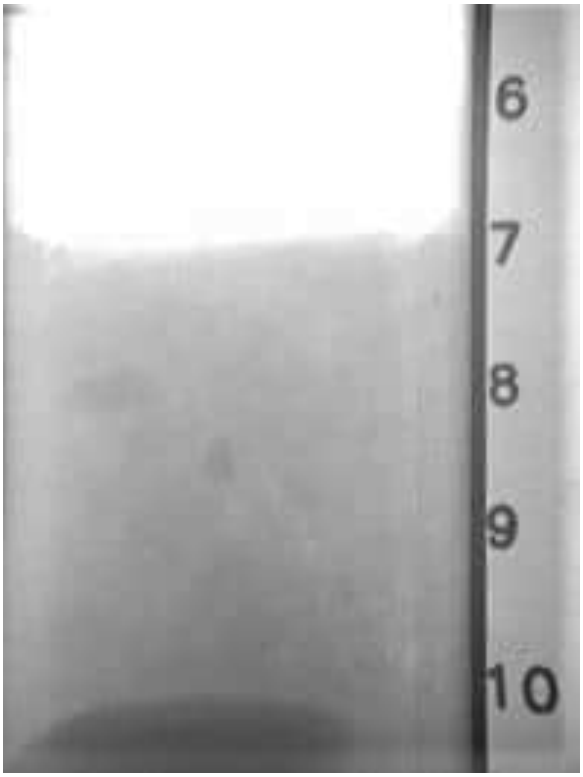
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Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-108
Sample ID:	1U
Depth:	80-10 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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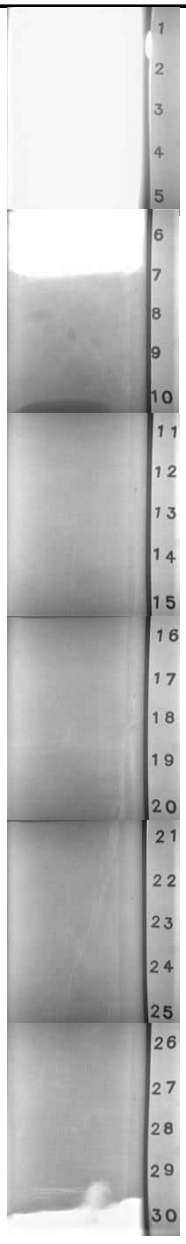
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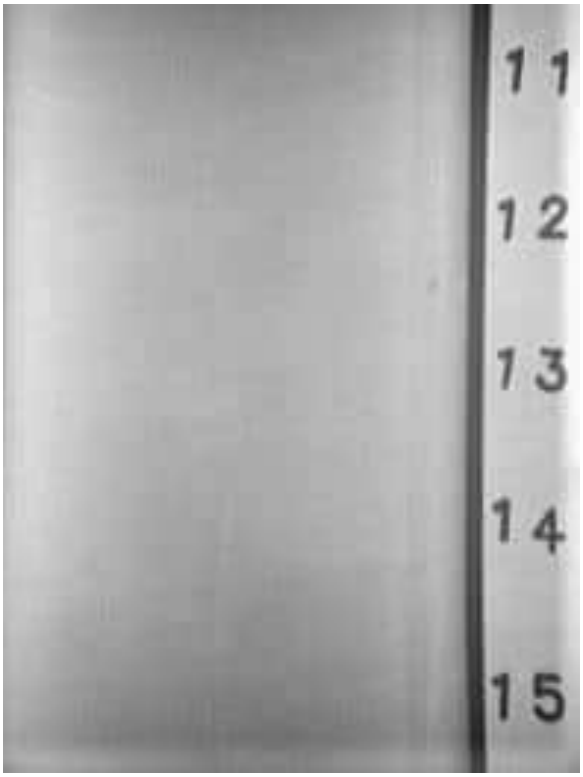
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Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
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Boring ID:	HB-BHR-108
Sample ID:	1U
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X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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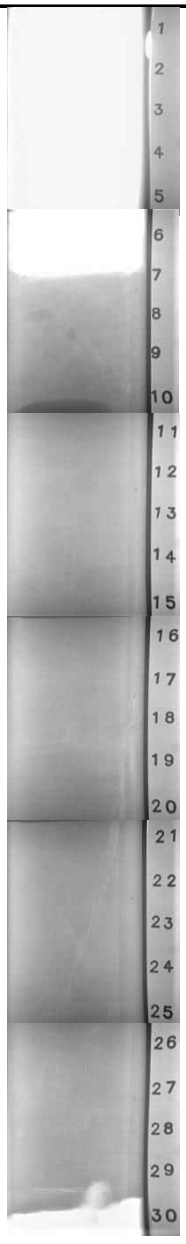
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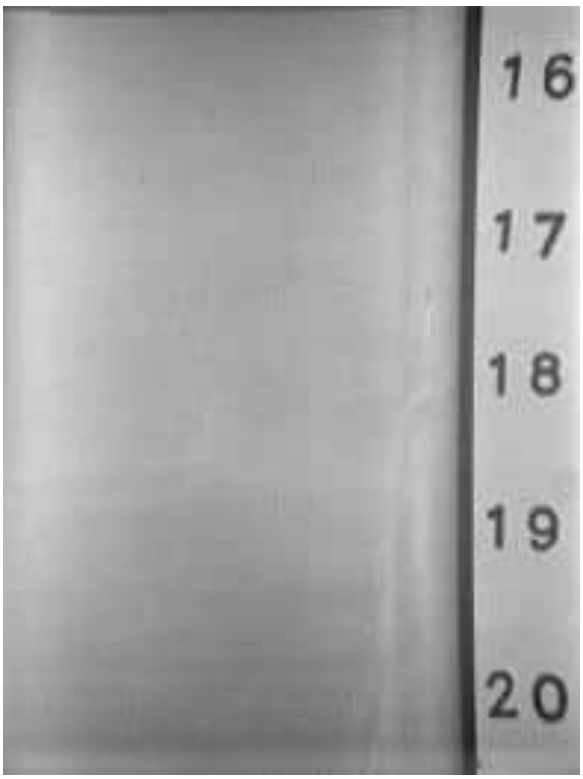
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
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Boring ID:	HB-BHR-108
Sample ID:	1U
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X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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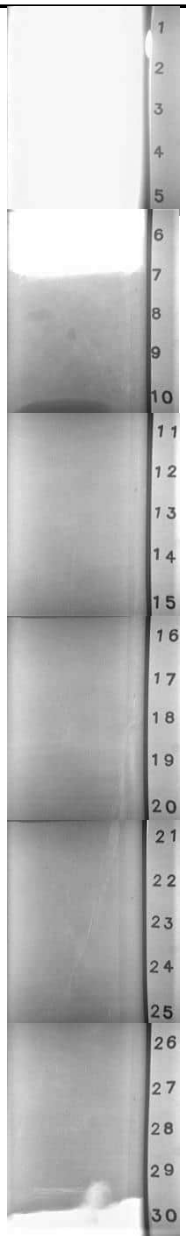
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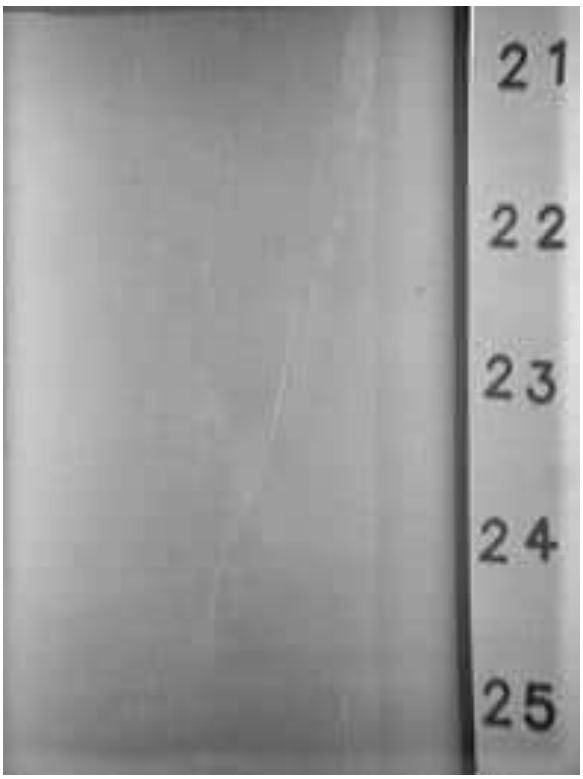
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
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X-Ray of Soil Sample by ASTM D4452

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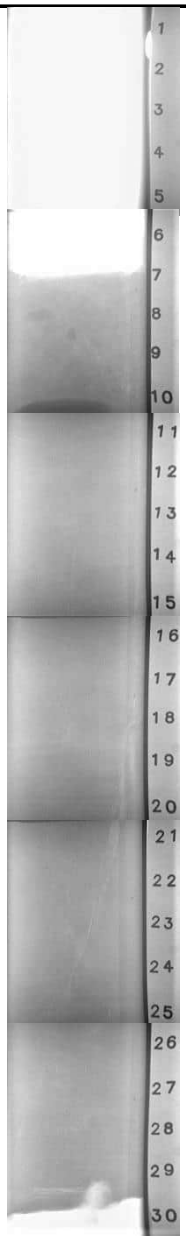
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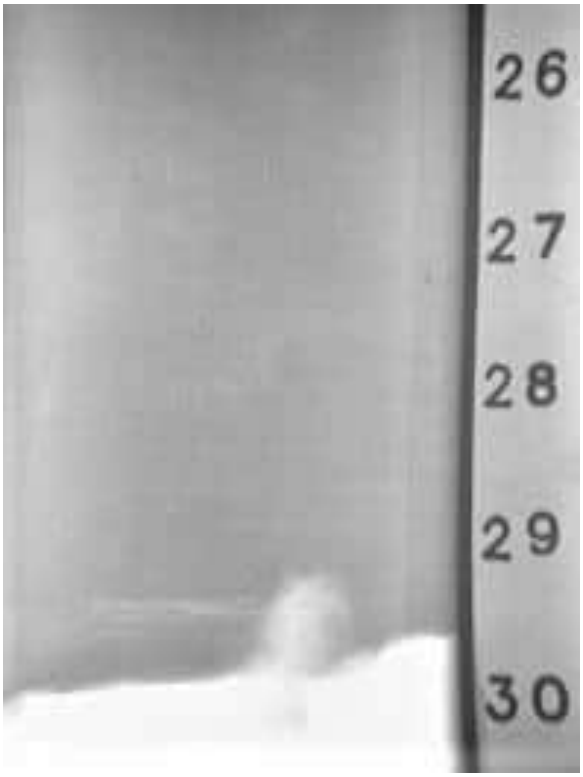
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
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Tested By:	md
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Boring ID:	HB-BHR-108
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Depth:	80-10 ft

X-Ray of Soil Sample by ASTM D4452

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	Bottom of Tube

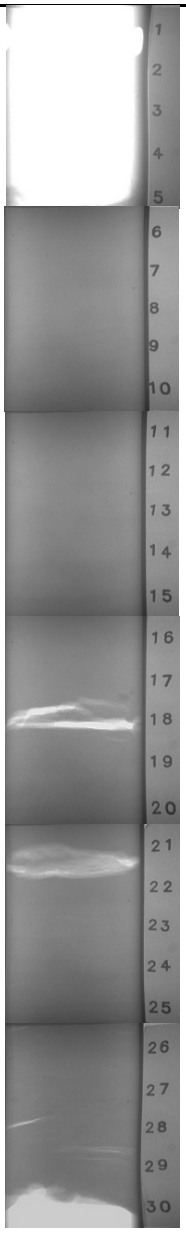


SECTION 6



Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube

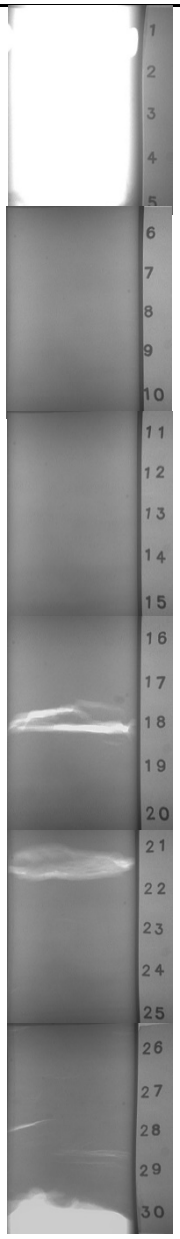
SECTION 1



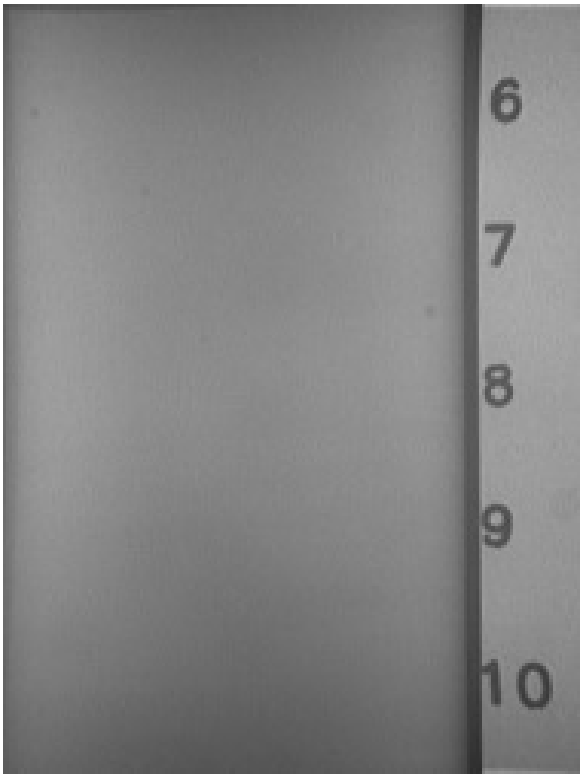
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Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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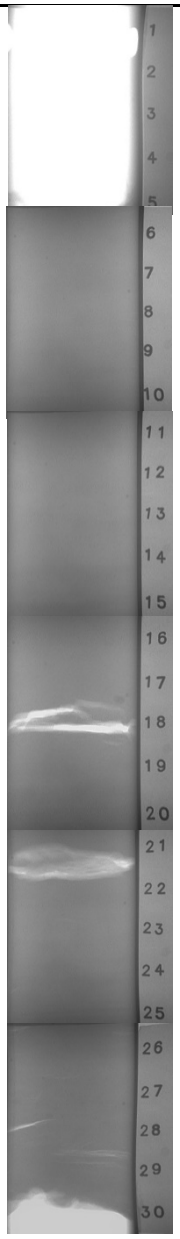
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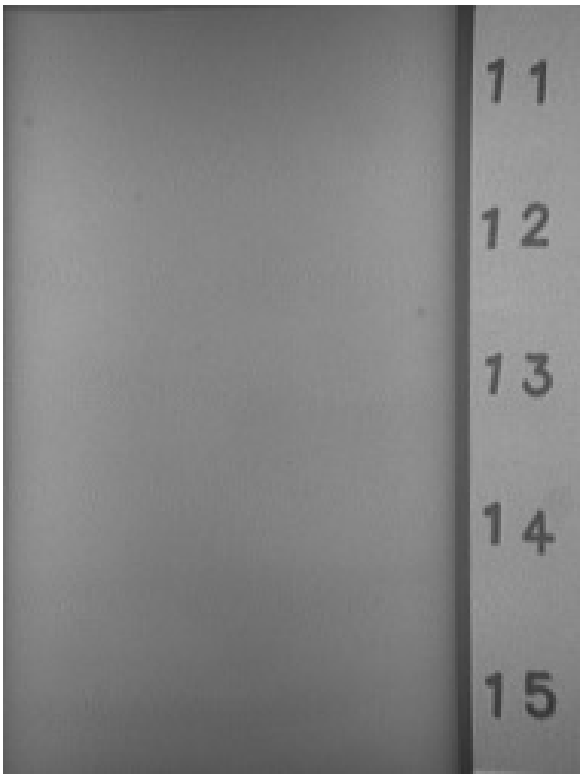
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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	Bottom of Tube



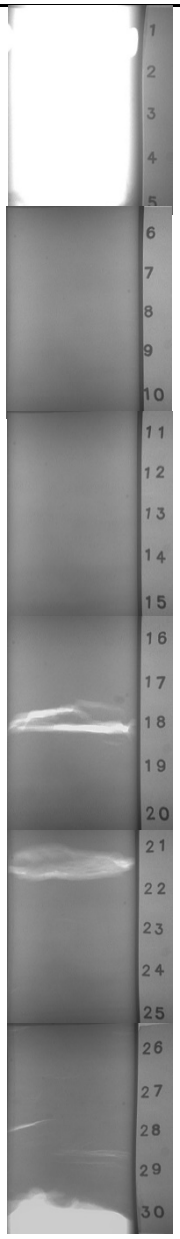
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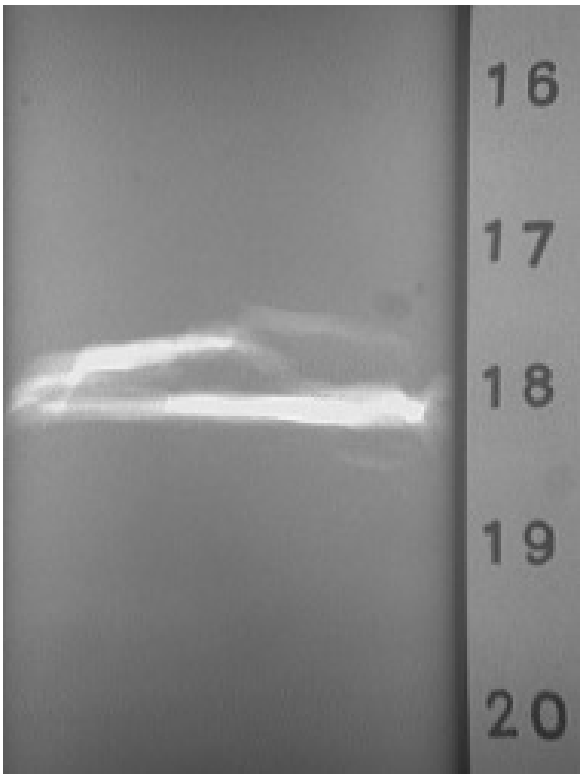
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

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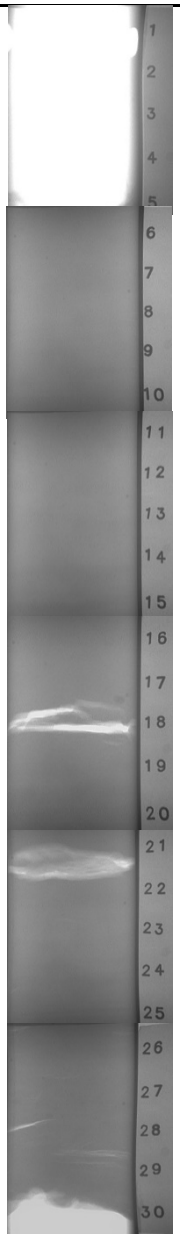
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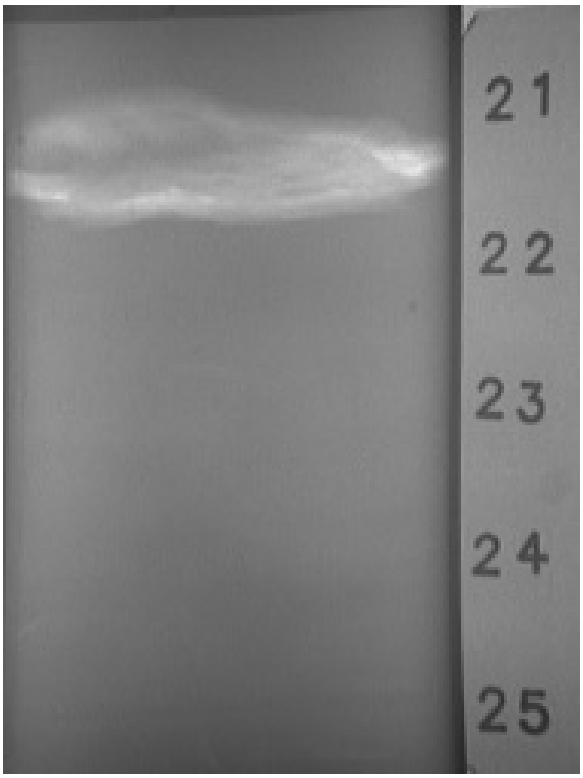
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Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

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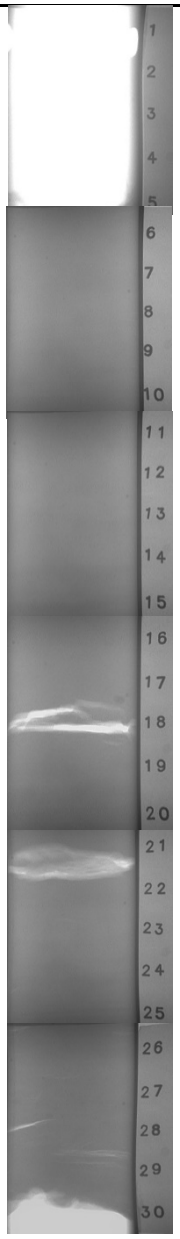
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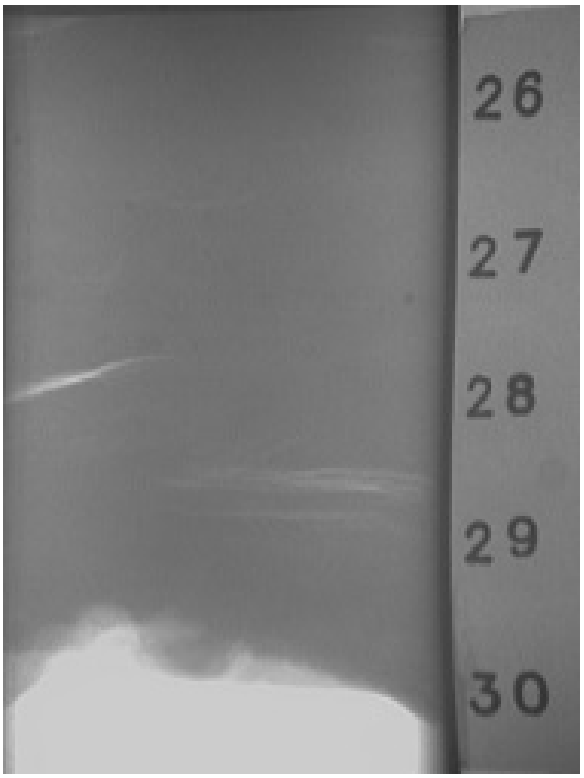
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
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4	
5	
6	
	Bottom of Tube



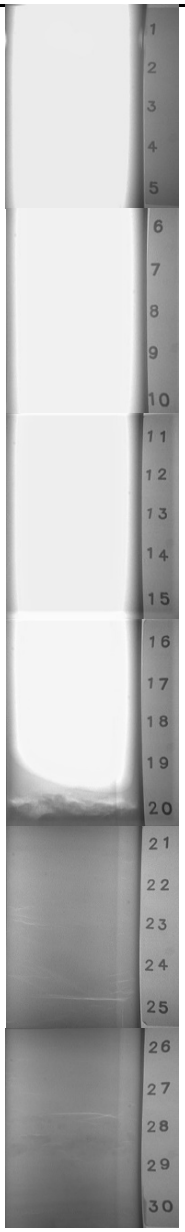
SECTION 6



Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
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4	
5	
6	
	Bottom of Tube



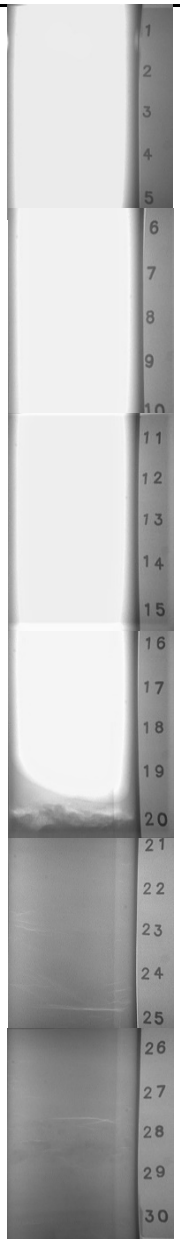
SECTION 1



Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



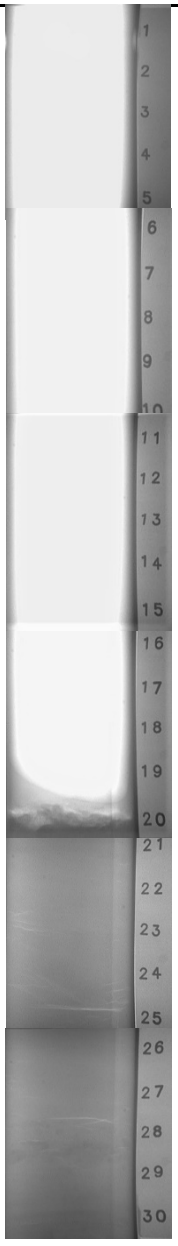
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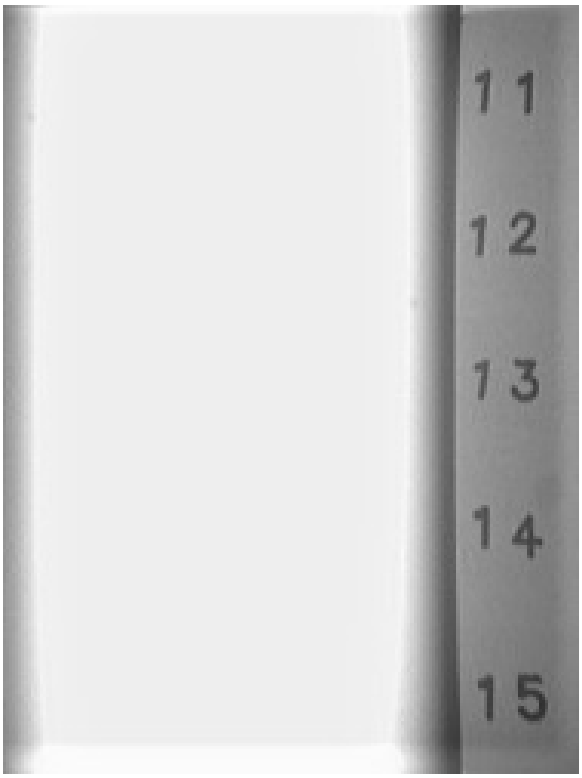
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Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



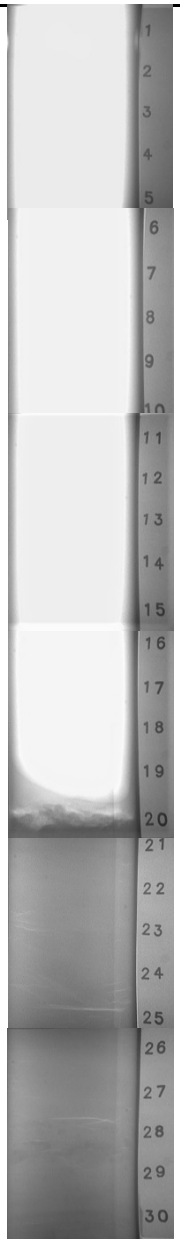
SECTION 3



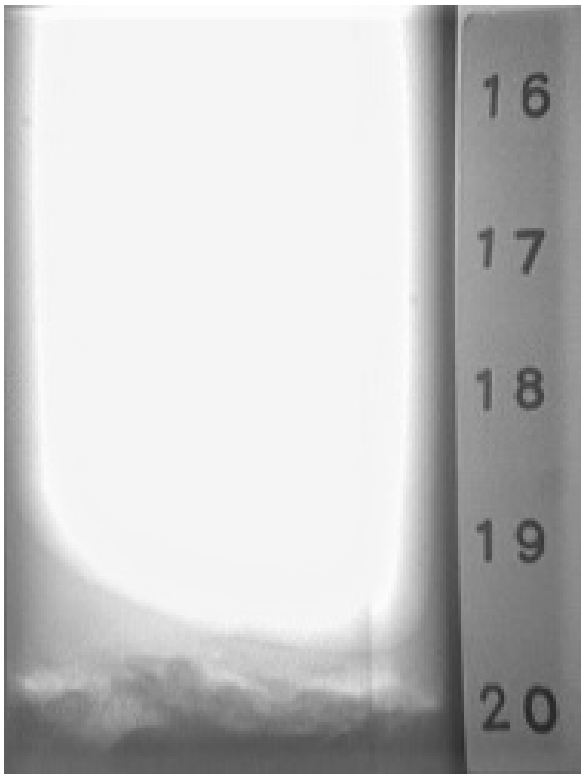
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



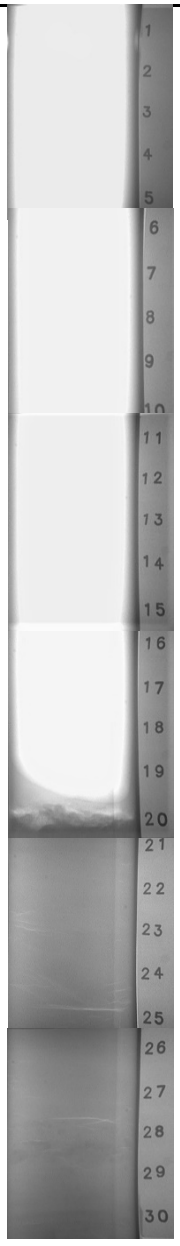
SECTION 4



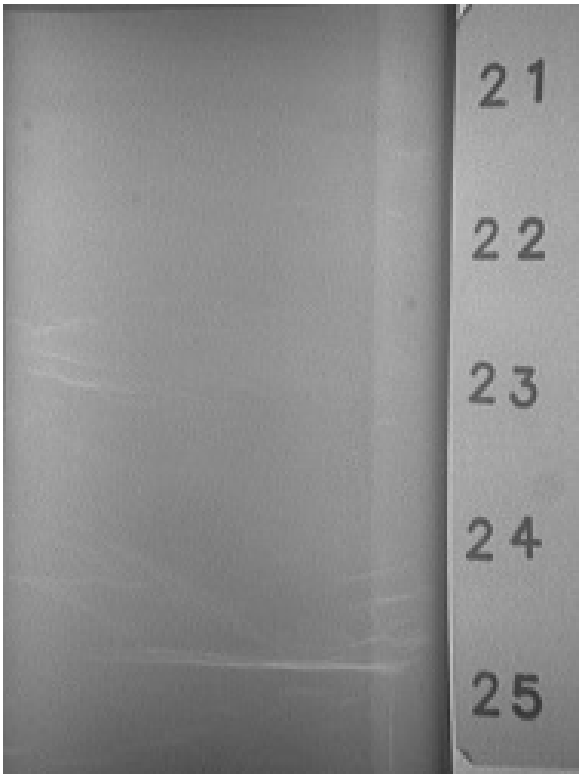
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



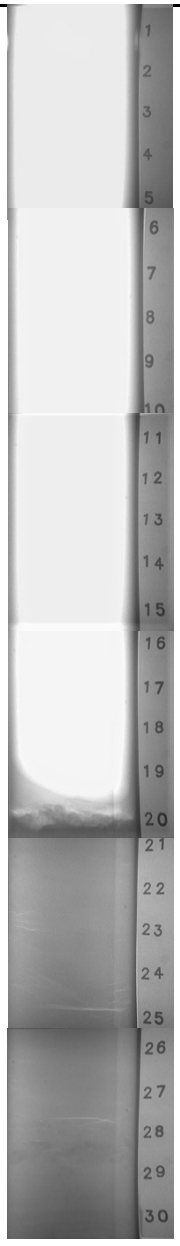
SECTION 5



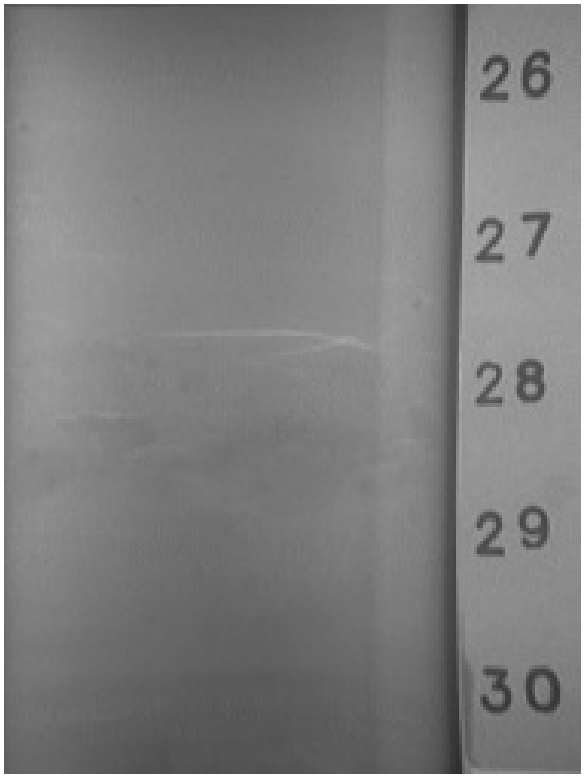
Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



SECTION 6



Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils by ASTM D6528

Client:	Golder Associates USA, Inc.	GTX#:	315431
Project Name:	Hogan Bridge Road	Test Date:	5/17/22
Project Location:	Bangor, ME		

Boring ID: BB-BHR-101
Sample ID: 1U
Depth, ft: 15-17

Visual Description: Moist, gray clay

Test Equipment: Top and bottom box (circular) = 2.50 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch. Stacked rings used. Set up included porous stones with pins.

Test Condition: Inundated prior to consolidating to 1,175 psf.

Sample Type and Preparation: density: Extruded from tube, cut, trimmed and placed into apparatus at as-received and moisture content.

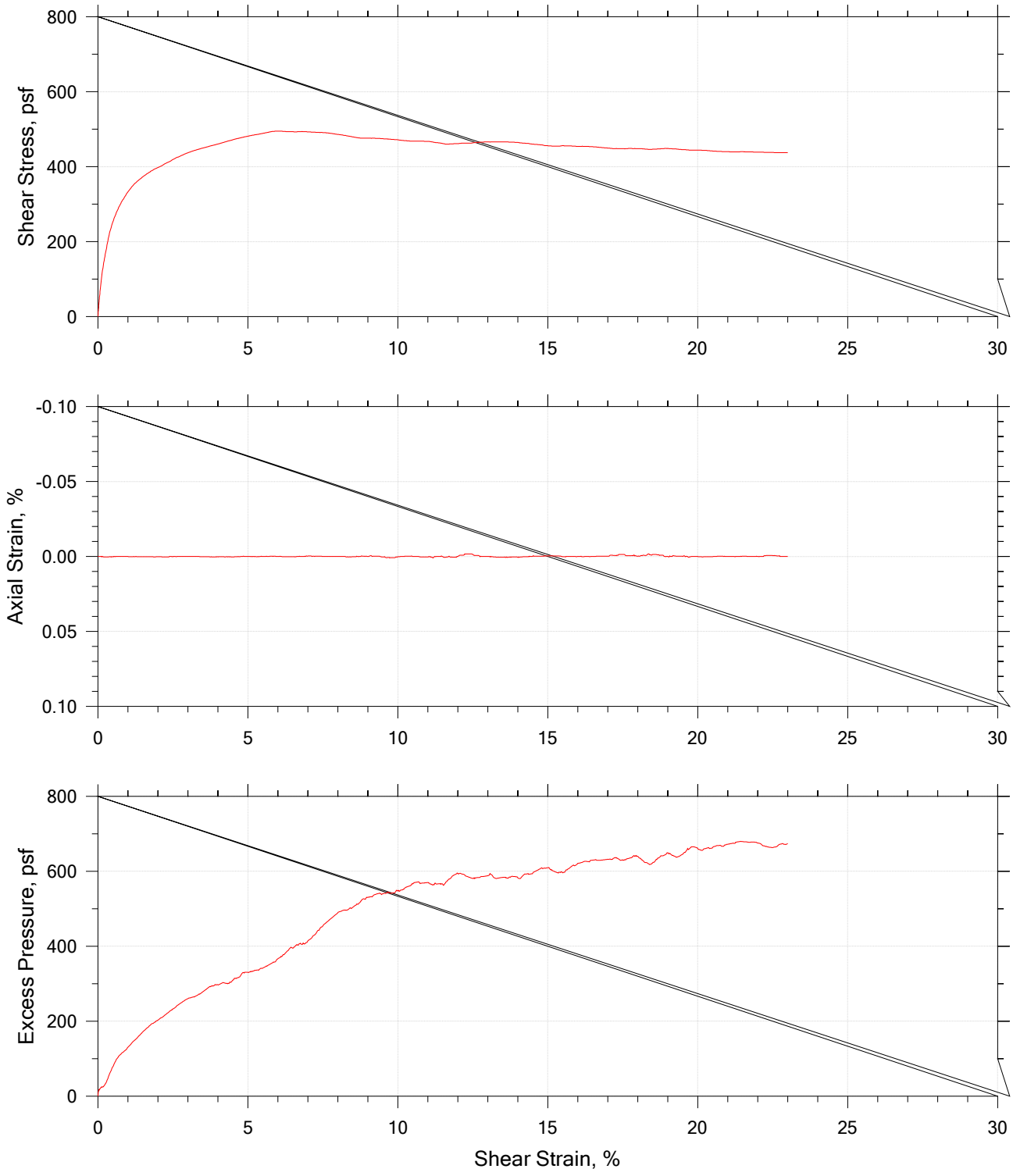
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	30.1				
Initial Dry Density, pcf	91.5				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	1,175				
Vertical Consolidation Stress at time of shear, psf	1,175				
Final Moisture Content, %	28.8				
Measured Peak Shear Stress, psf	493.74				
Shear Strain at Peak Shear Stress, %	6.9				
Membrane Correction, psf	0.35				
Corrected Peak Shear Stress, psf	493.39				
S_u / σ'_{vc}	0.42				

Comments:

Tested By: md Checked By: njh

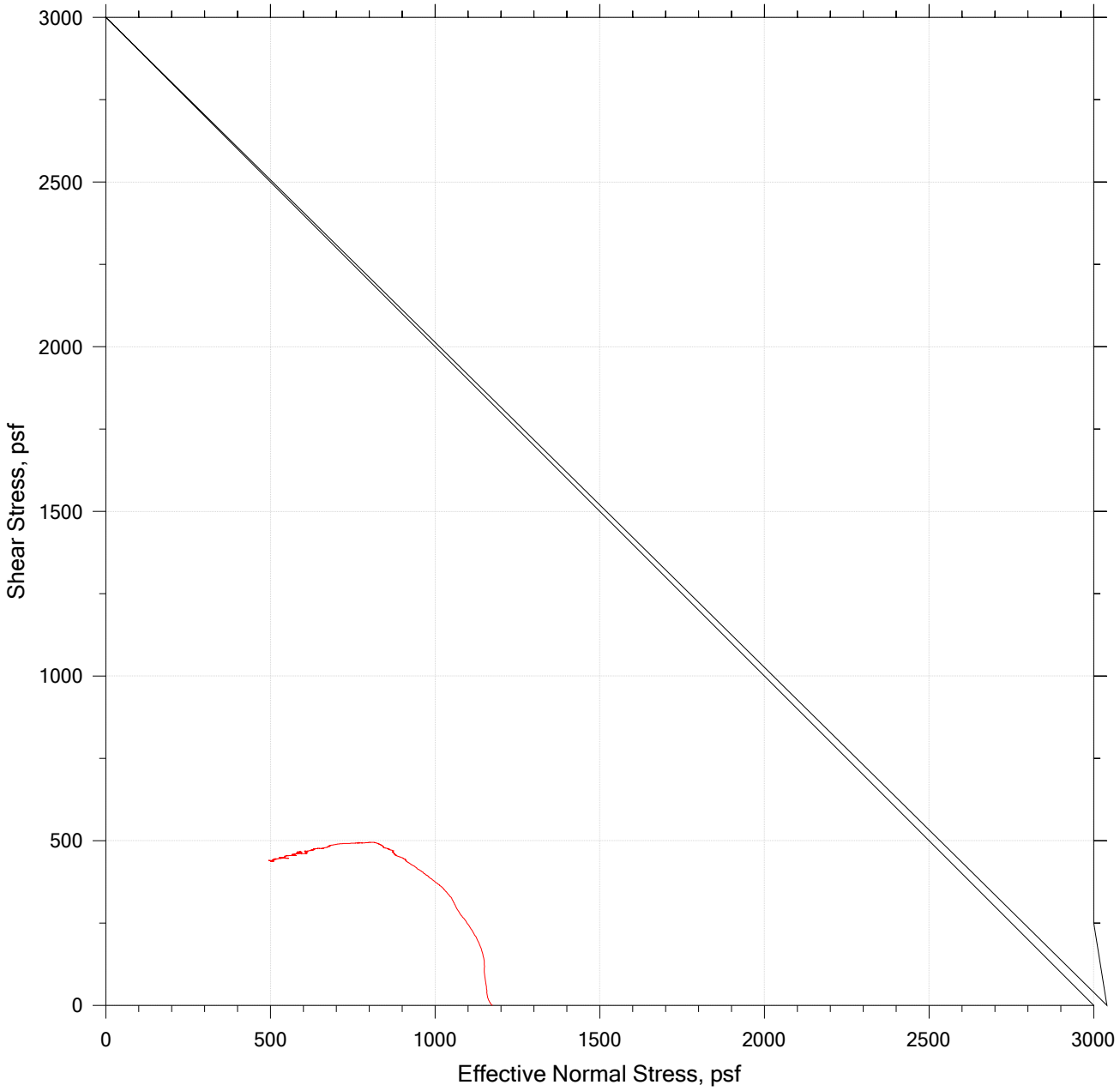
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

DIRECT SIMPLE SHEAR TEST by ASTM D6528



GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 05/17/22	Depth: 15-17
	Test No.: DSS-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks:		

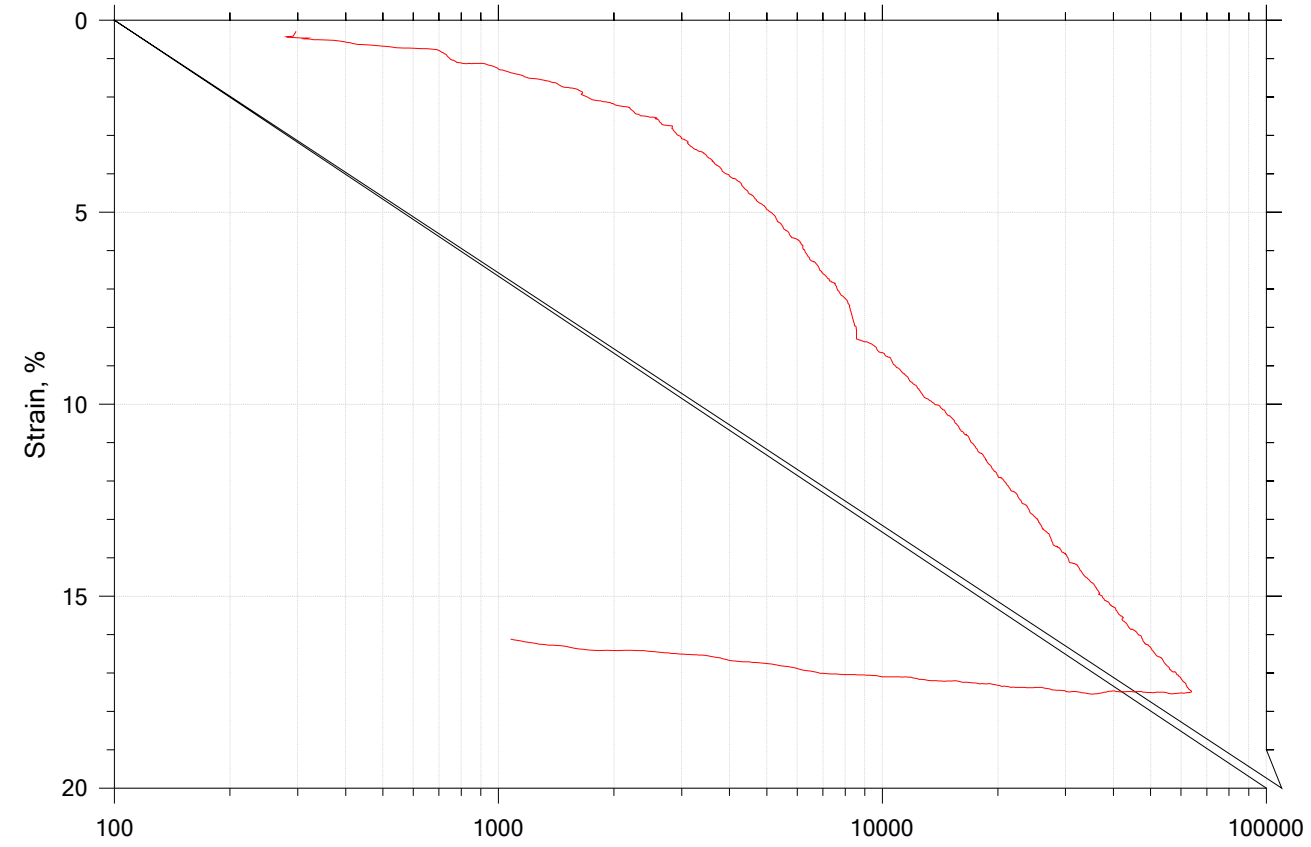
DIRECT SIMPLE SHEAR TEST by ASTM D6528

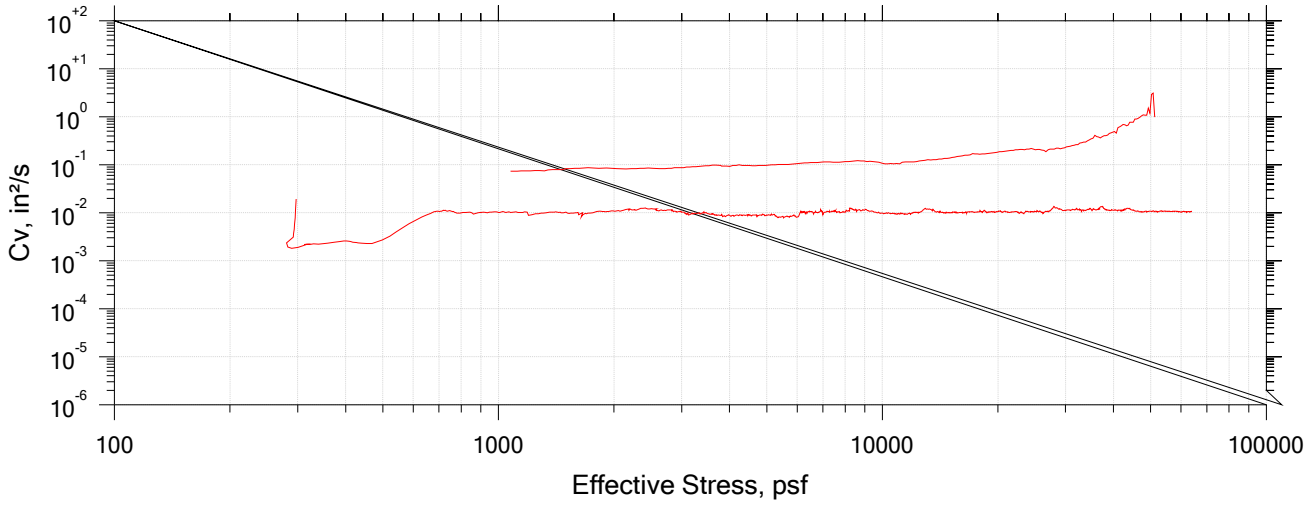


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 05/17/22	Depth: 15-17
	Test No.: DSS-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks:		

CRC Test

Summary

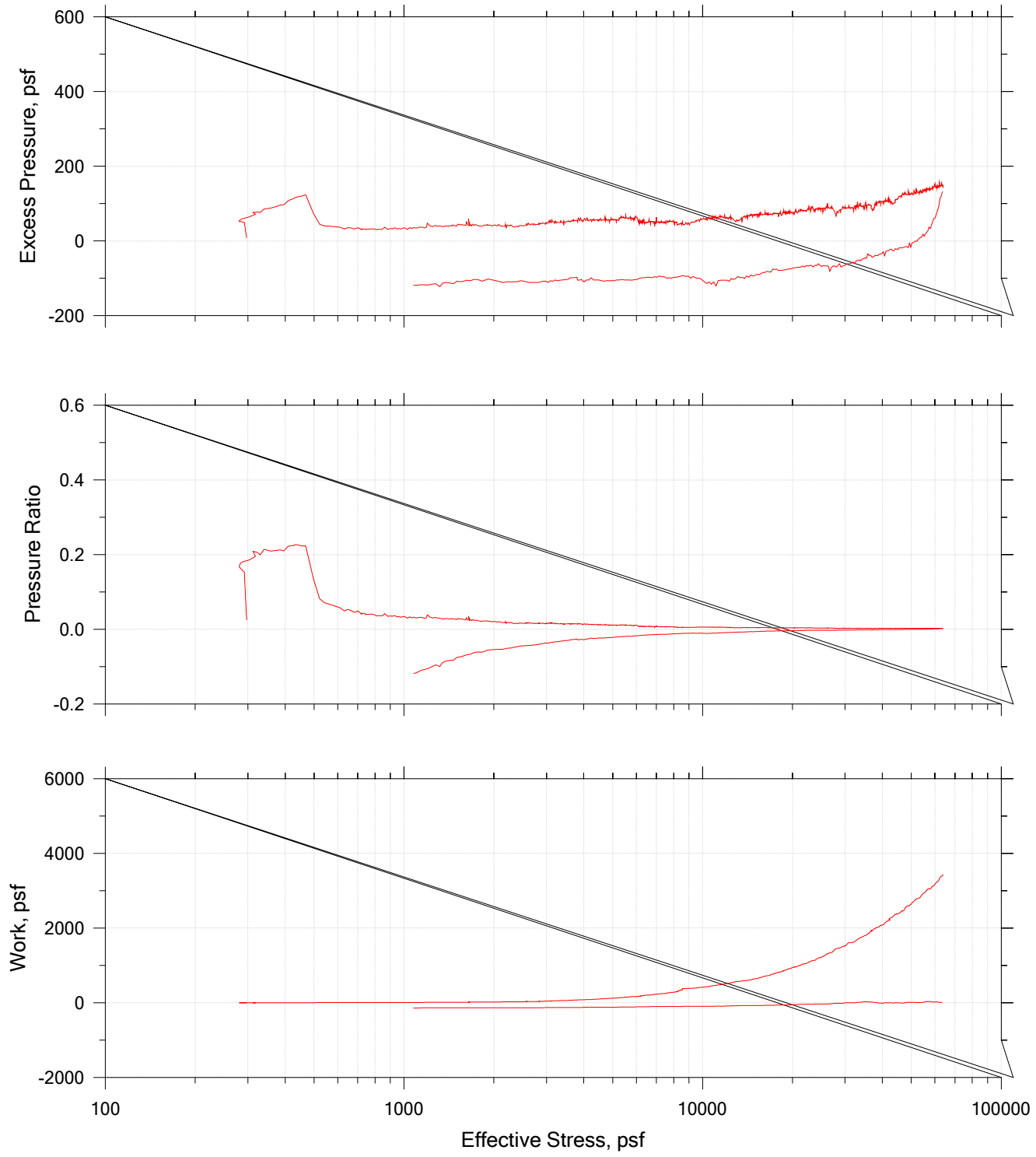




Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-31315431
Boring Number: BB-BHR-101	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 05/17/22	Depth: 15-17
Test Number: CRC-1	Preparation: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-003		

CRC Test

Pressure Curves



GeoTesting EXPRESS	Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-31315431
	Boring Number: BB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/17/22	Depth: 15-17
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-003		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.76 (Estimated)	Liquid Limit: 31
Specimen Height, in: 1.00	Initial Void Ratio: 0.819	Plastic Limit: 18
Final Height, in: 0.88	Final Void Ratio: 0.601	Plasticity Index: 13

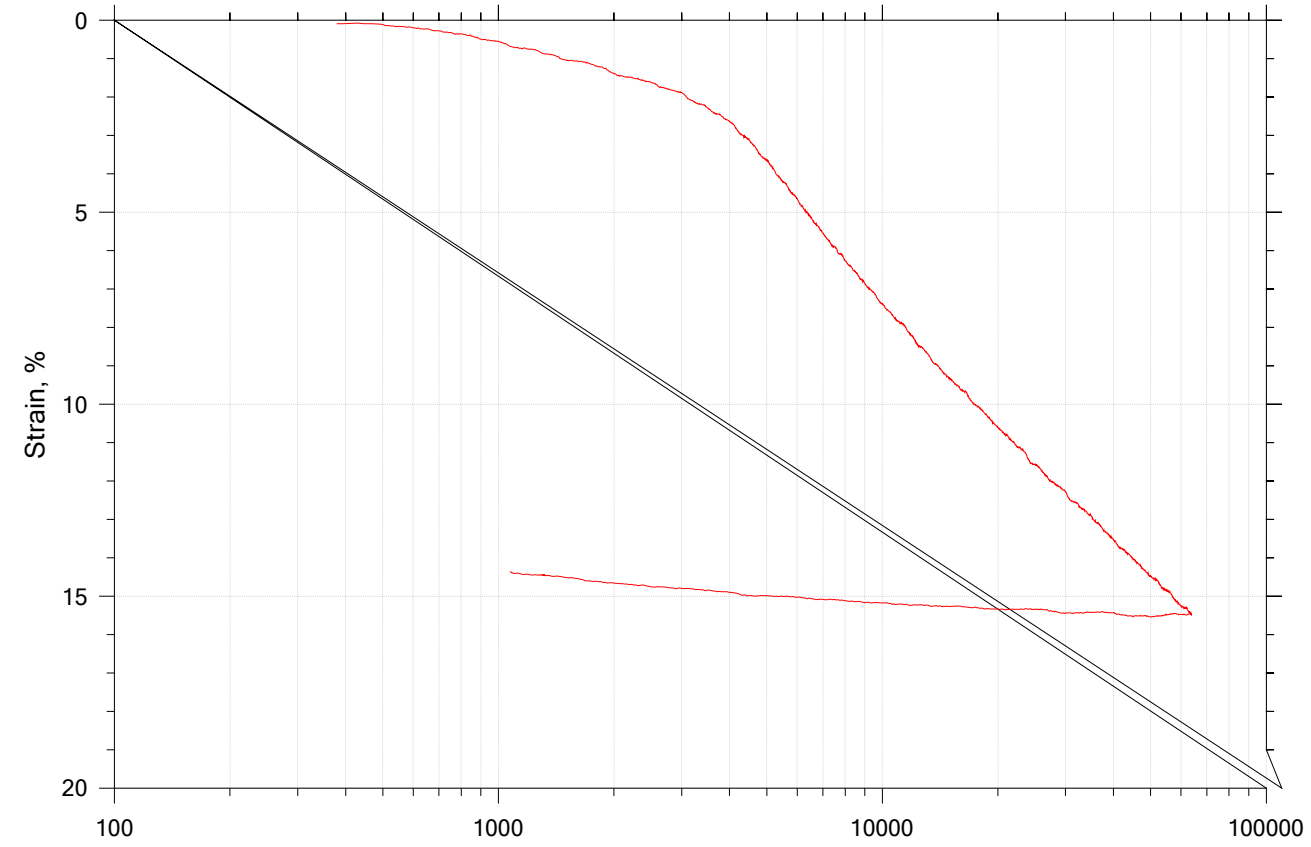
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-2020	---		E3267
Mass Container, gm	8.29	111.06	111.06	8.2
Mass Container + Wet Soil, gm	210	268.99	259.6	155.29
Mass Container + Dry Soil, gm	163.3	233.03	233.03	128.98
Mass Dry Soil, gm	155.01	121.97	121.97	120.78
Water Content, %	30.13	29.48	21.78	21.78
Void Ratio	---	0.82	0.60	---
Degree of Saturation, %	---	99.27	100.00	---
Dry Unit Weight, pcf	---	94.659	107.57	---

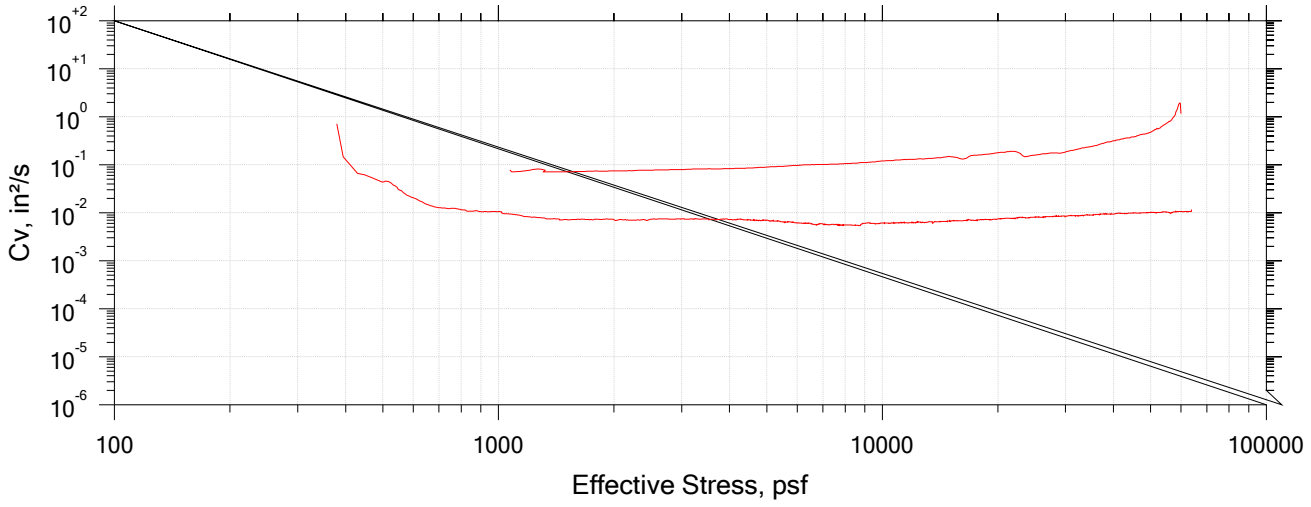
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-31315431
	Boring Number: BB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/17/22	Depth: 15-17
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-003		

CRC Test

Summary

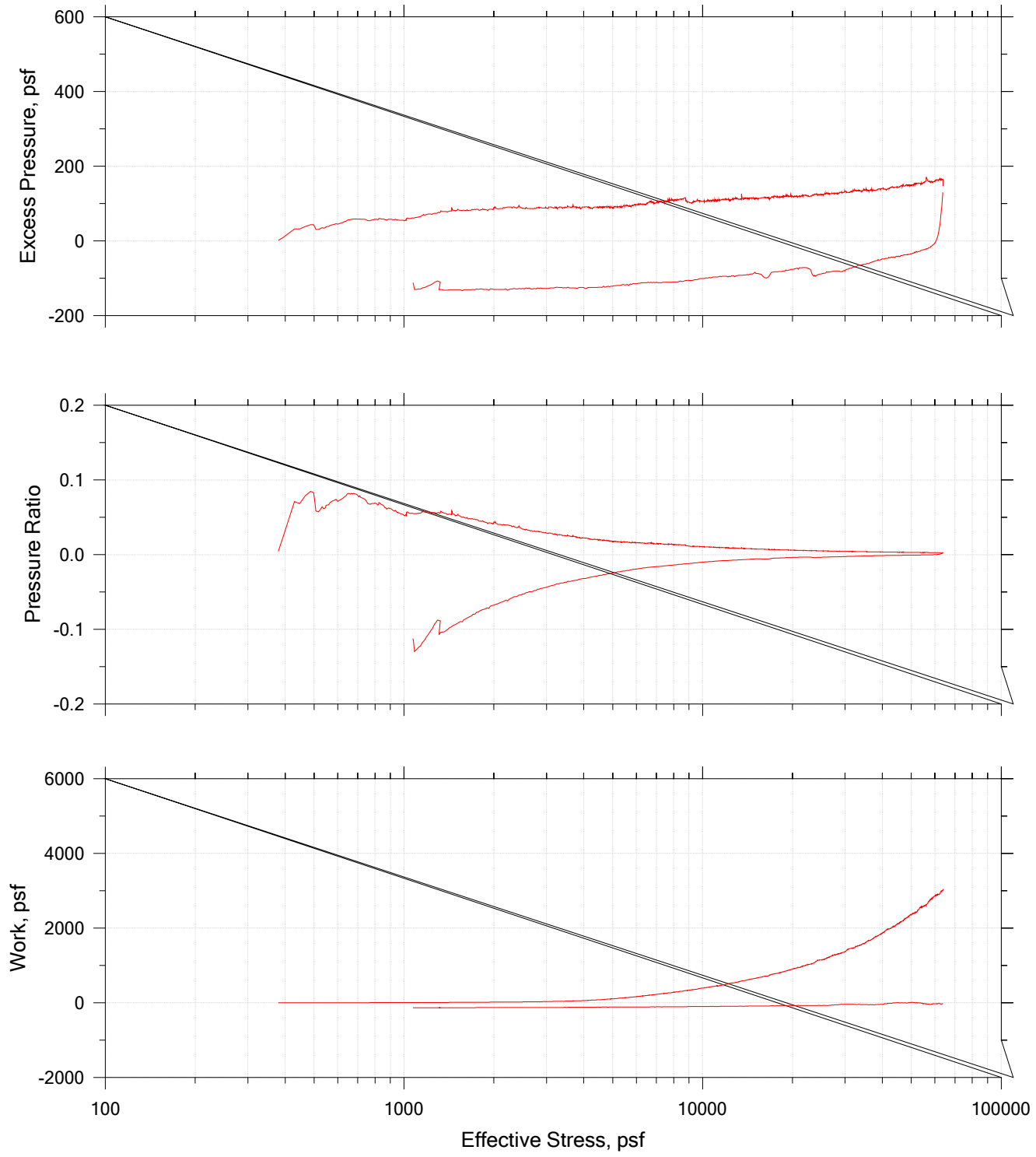




GeoTesting EXPRESS	Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-315431
	Boring Number: BB-BHR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/17/22	Depth: 15-16.2
	Test Number: CRC-2	Preparation: intact	Elevation:
	Description: Moist, gray clay		
	Remarks: System JJ		

CRC Test

Pressure Curves



GeoTesting EXPRESS	Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-315431
	Boring Number: BB-BHR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/17/22	Depth: 15-16.2
	Test Number: CRC-2	Preparation: intact	Elevation:
	Description: Moist, gray clay		
	Remarks: System JJ		

CRC Test

Specimen Diameter, in: 2.50	Specific Gravity: 2.76 (Estimated)	Liquid Limit: 27
Specimen Height, in: 1.00	Initial Void Ratio: 0.789	Plastic Limit: 15
Final Height, in: 0.84	Final Void Ratio: 0.503	Plasticity Index: 12

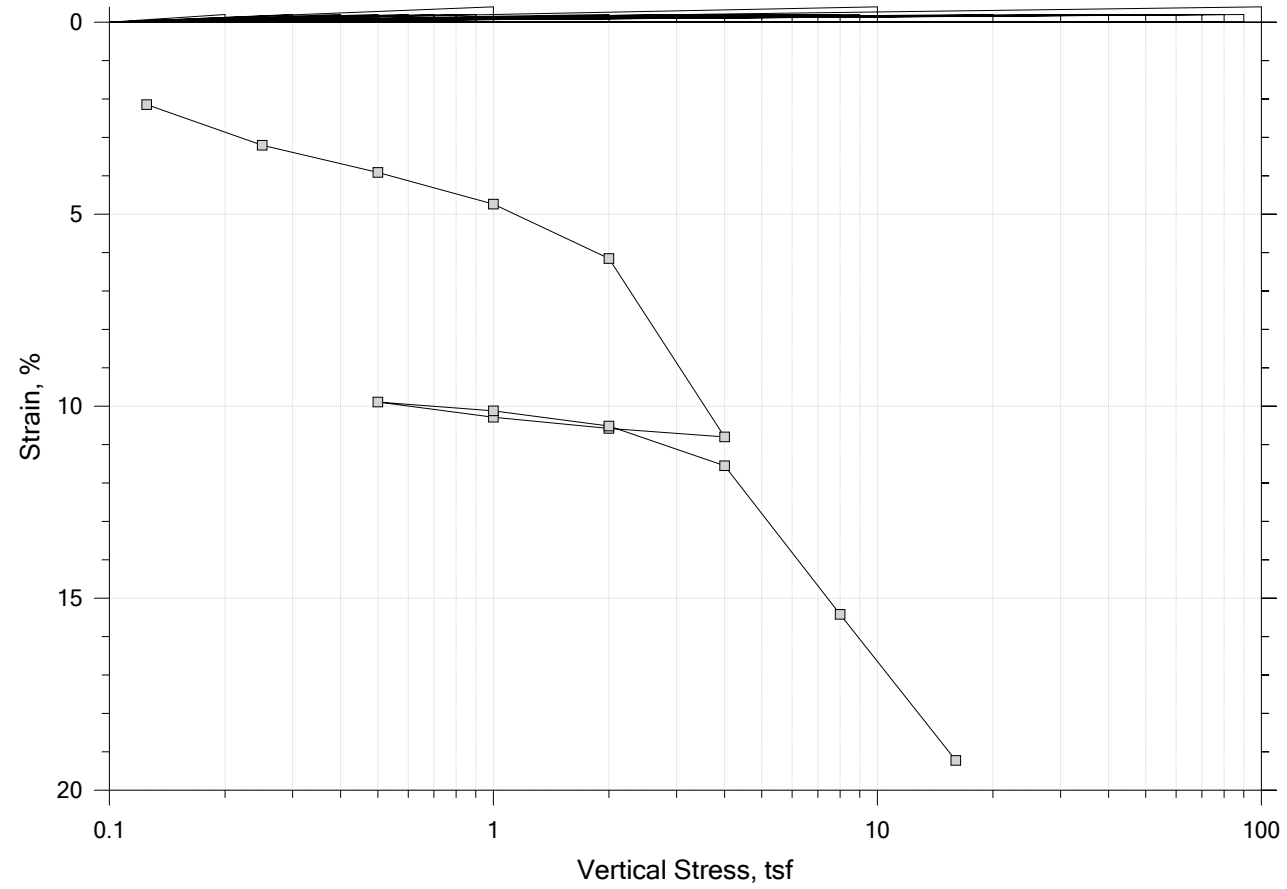
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-4975	---		E3466
Mass Container, gm	8.01	109.76	109.76	8.32
Mass Container + Wet Soil, gm	235.9	268.97	256.54	156.54
Mass Container + Dry Soil, gm	187.49	233.92	233.92	133.7
Mass Dry Soil, gm	179.48	124.16	124.16	125.38
Water Content, %	26.97	28.23	18.22	18.22
Void Ratio	---	0.79	0.50	---
Degree of Saturation, %	---	98.76	100.00	---
Dry Unit Weight, pcf	---	96.36	114.71	---

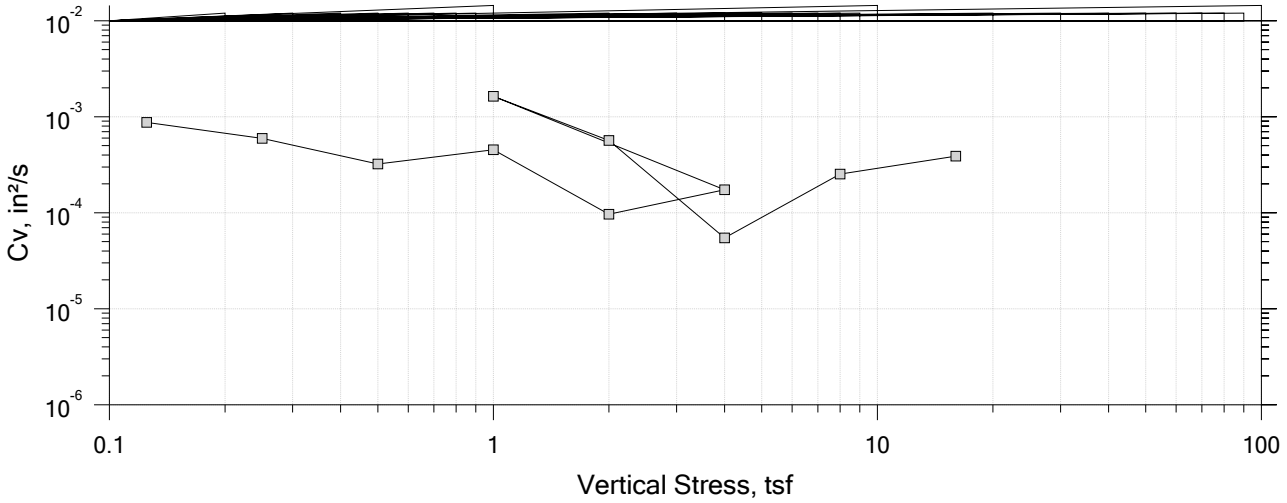
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-315431
	Boring Number: BB-BHR-103	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/17/22	Depth: 15-16.2
	Test Number: CRC-2	Preparation: intact	Elevation:
	Description: Moist, gray clay		
	Remarks: System JJ		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

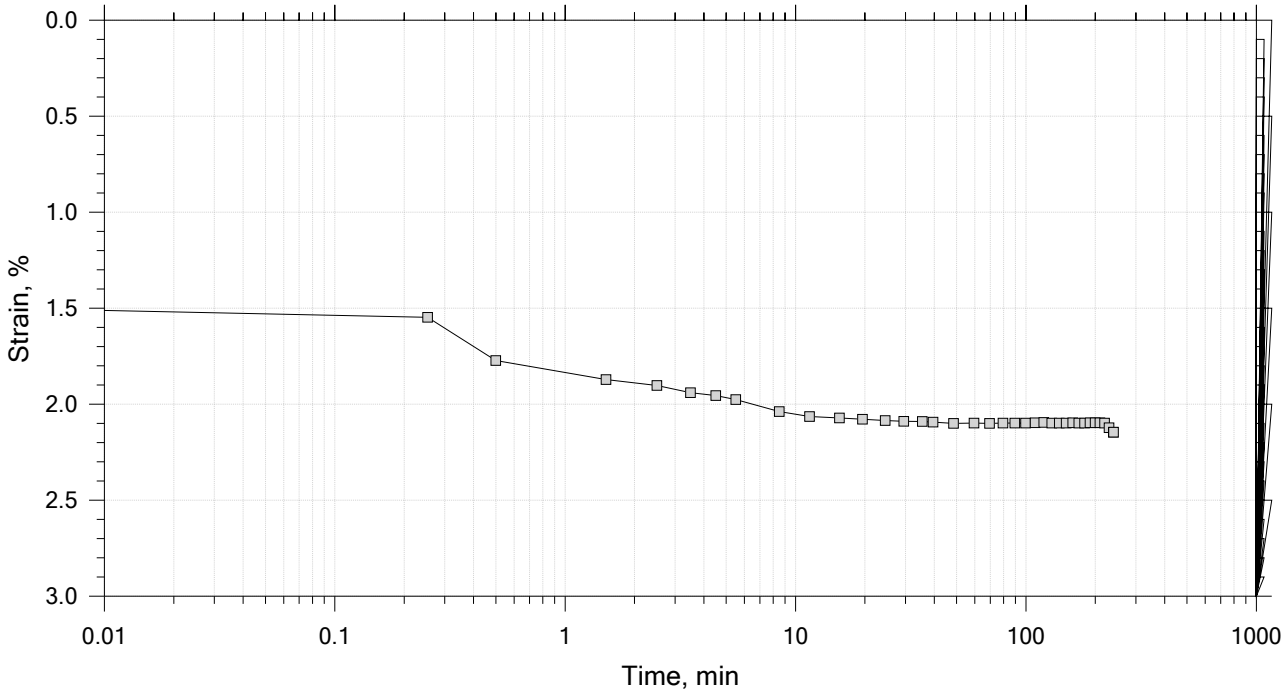


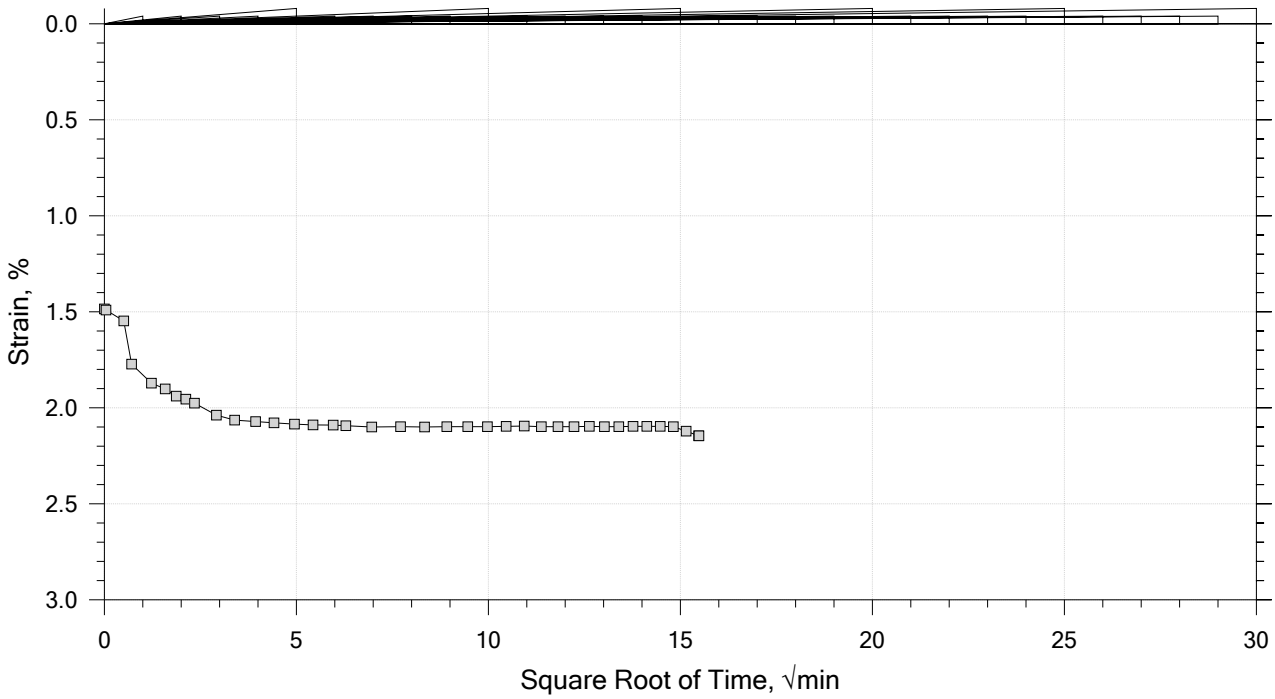


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 14
Constant Load Step
Stress: 0.125 tsf





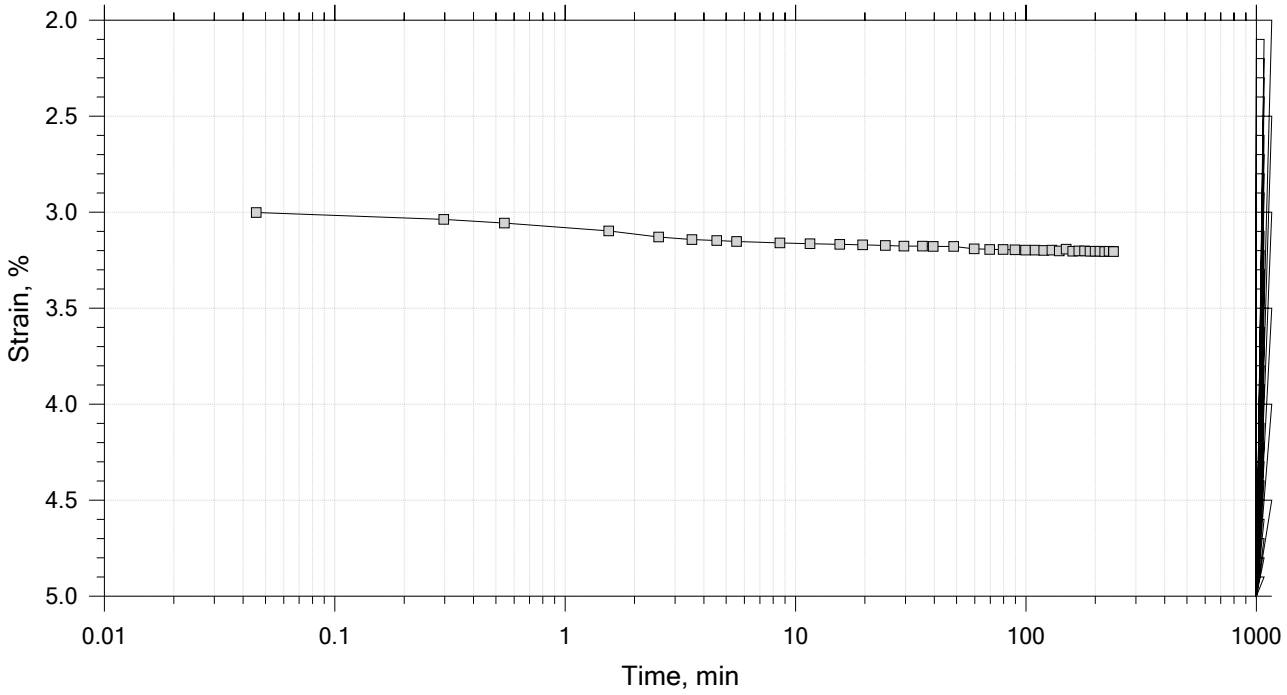
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

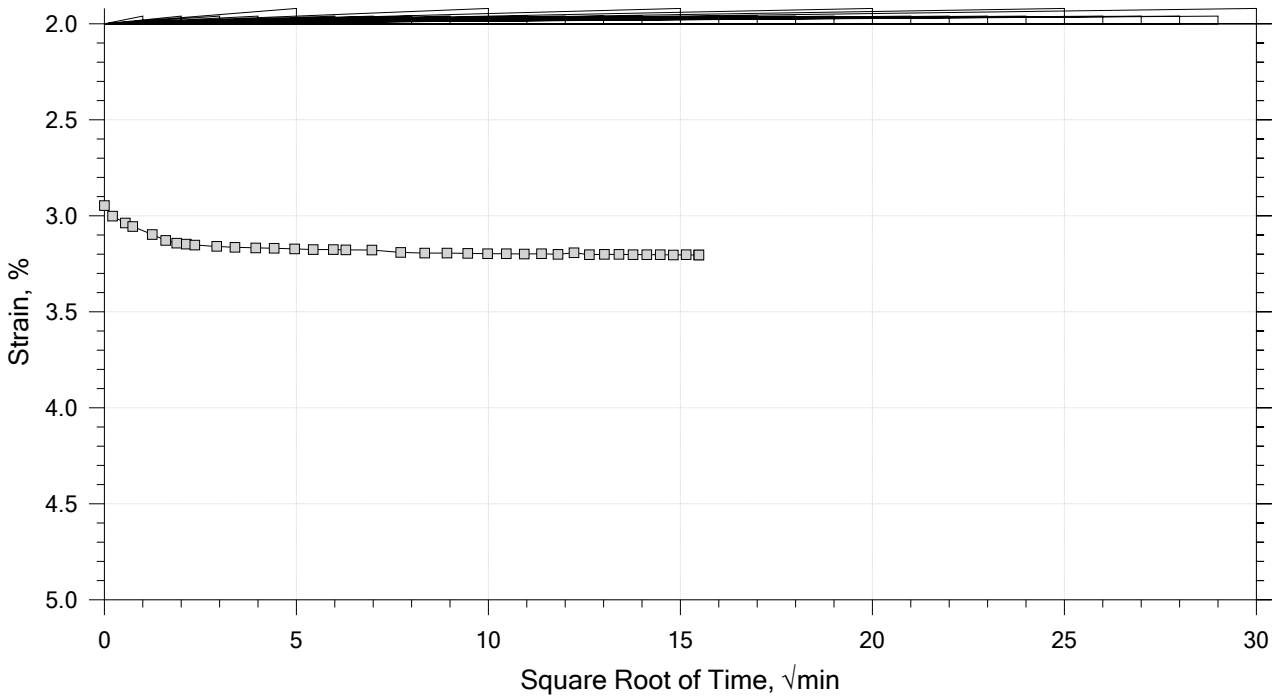
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14

Constant Load Step

Stress: 0.25 tsf

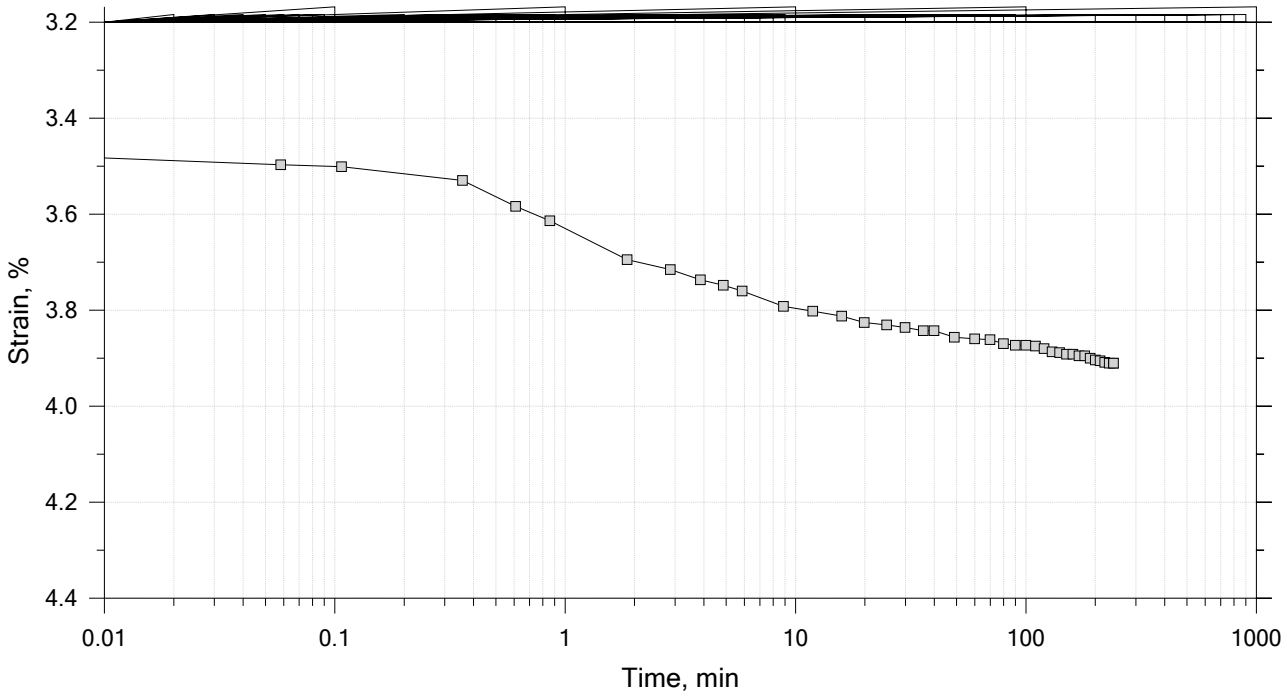


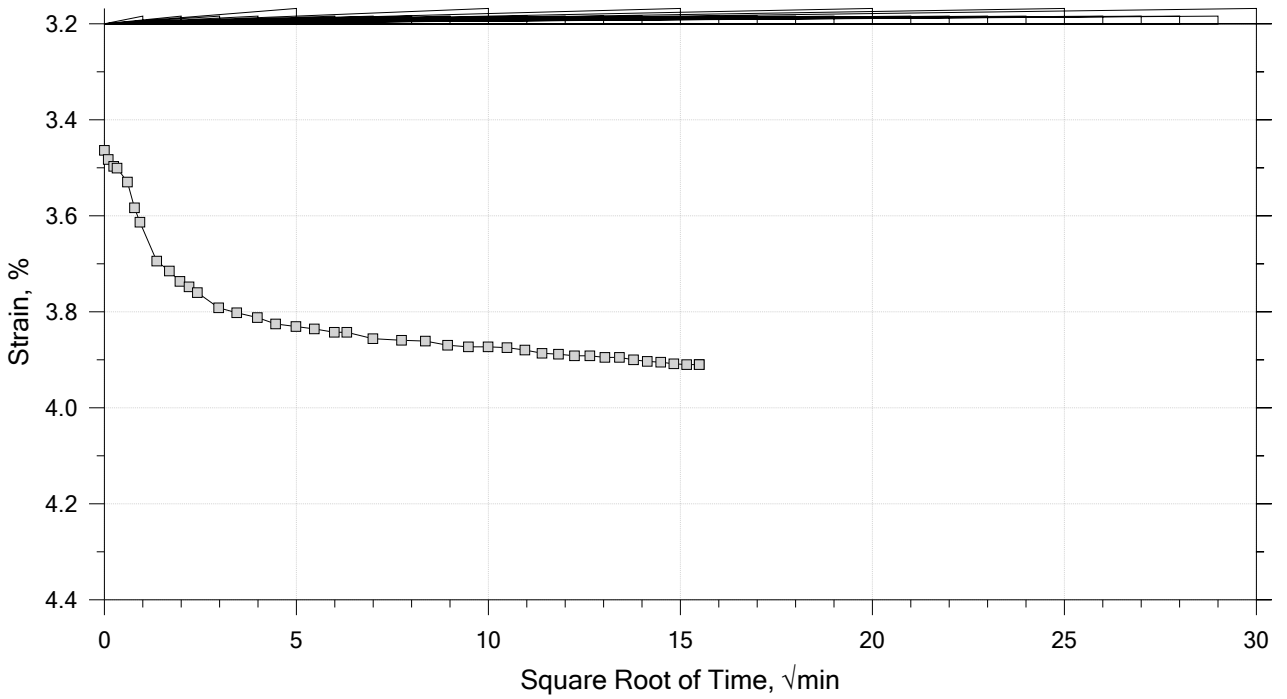


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf





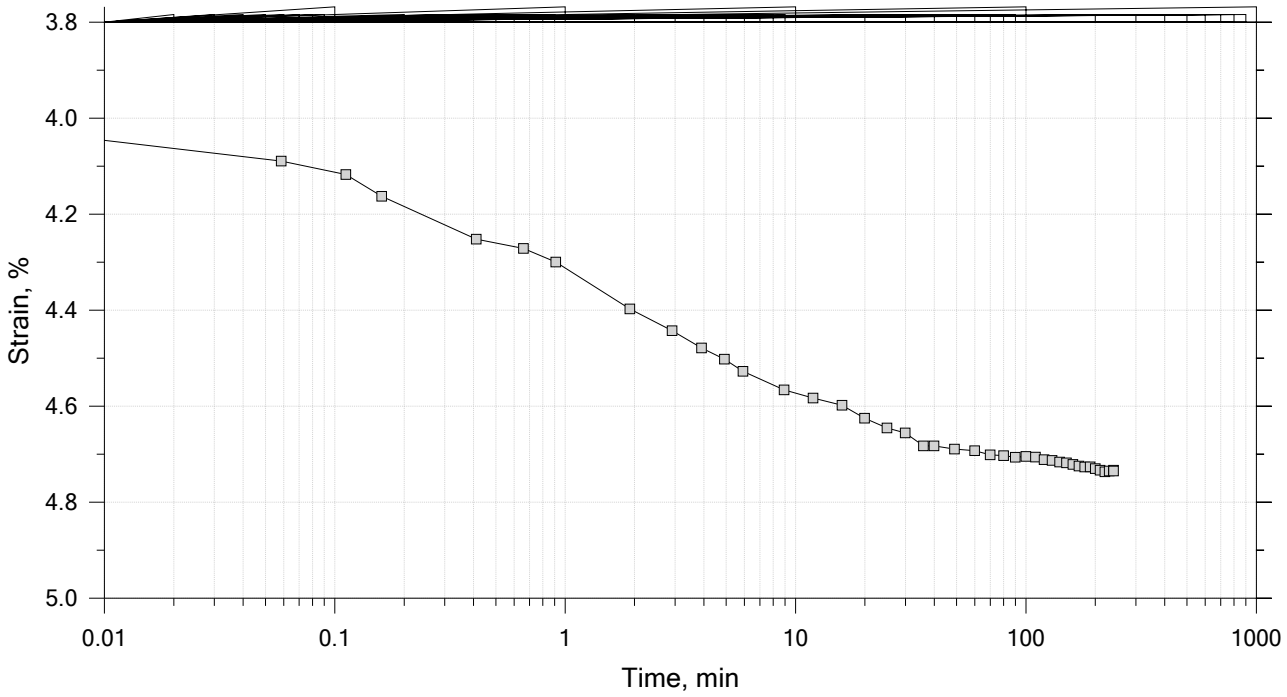
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

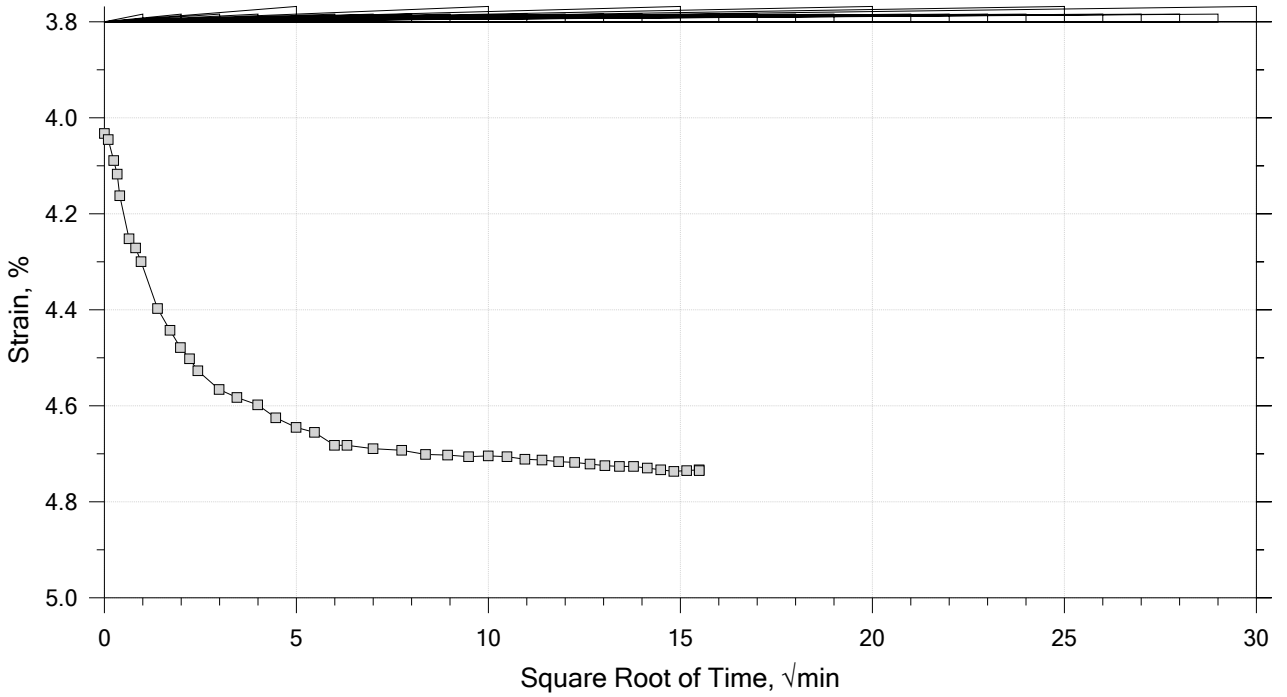
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 1 tsf

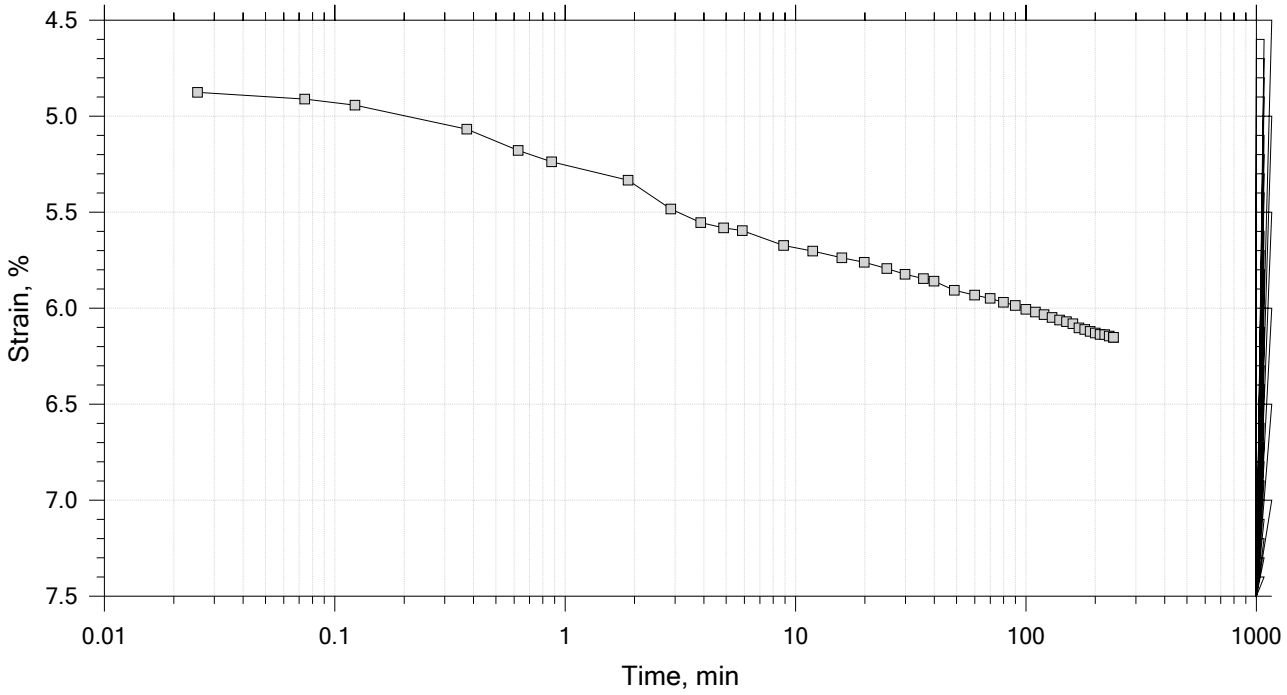


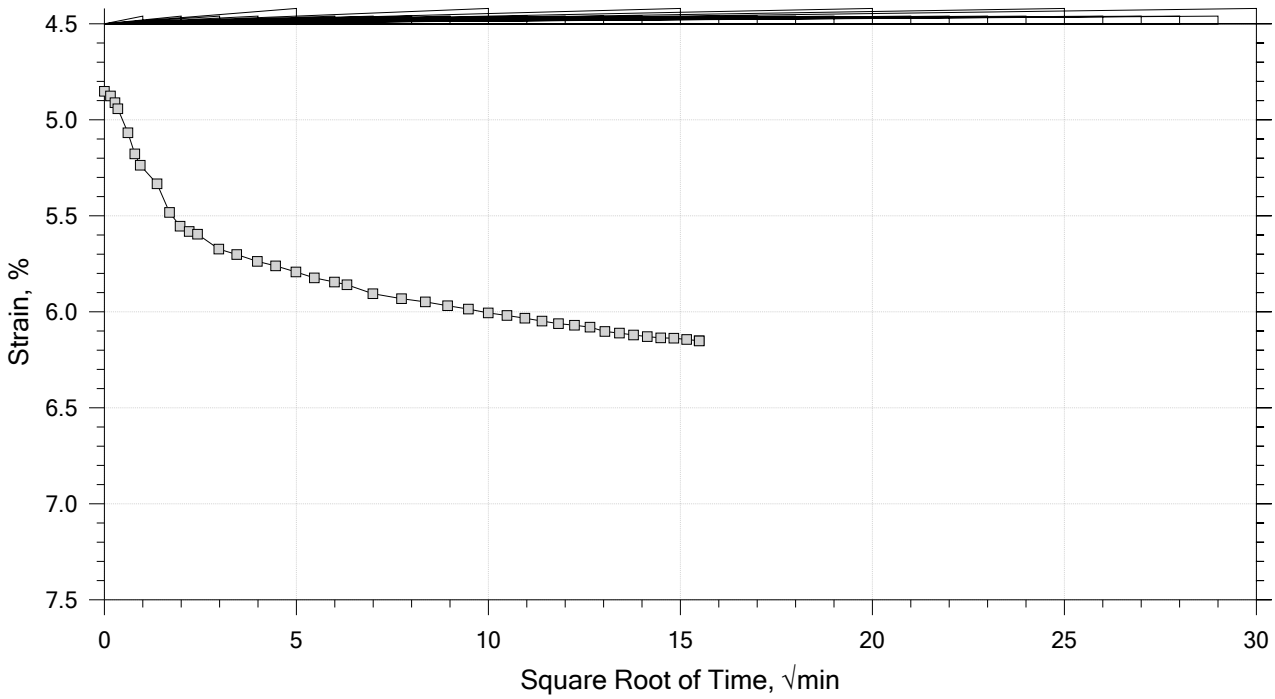


Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14
Constant Load Step
Stress: 2 tsf





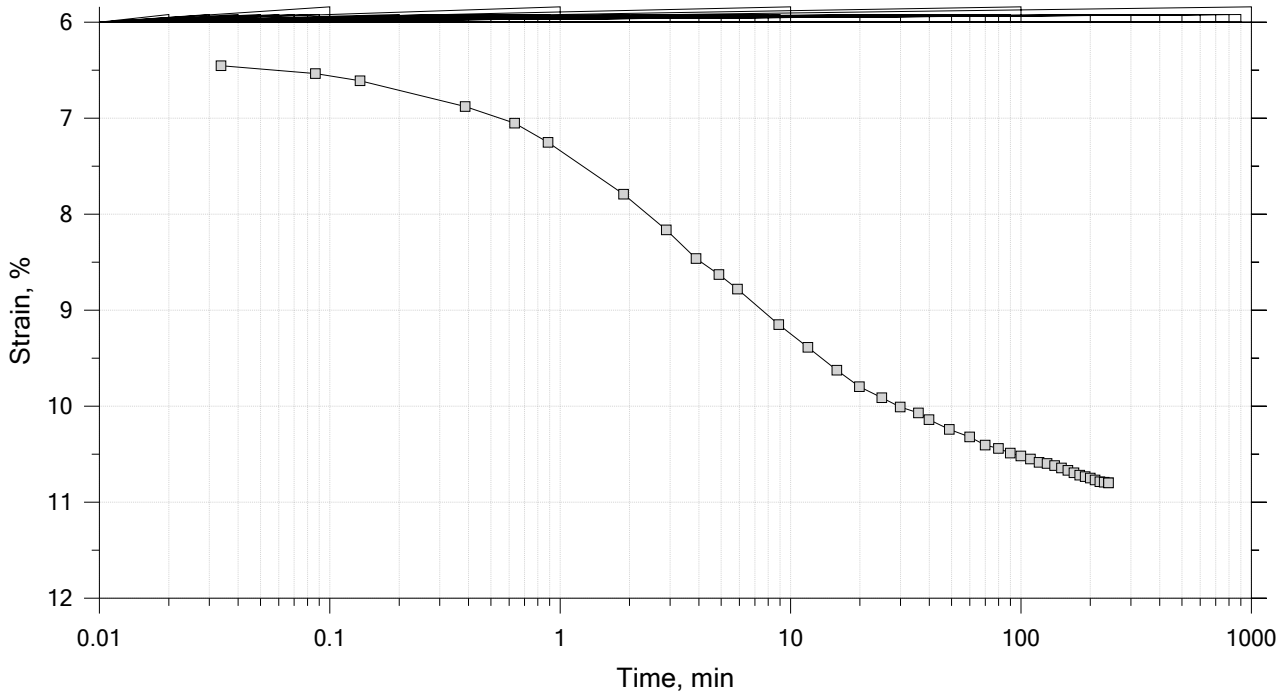
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

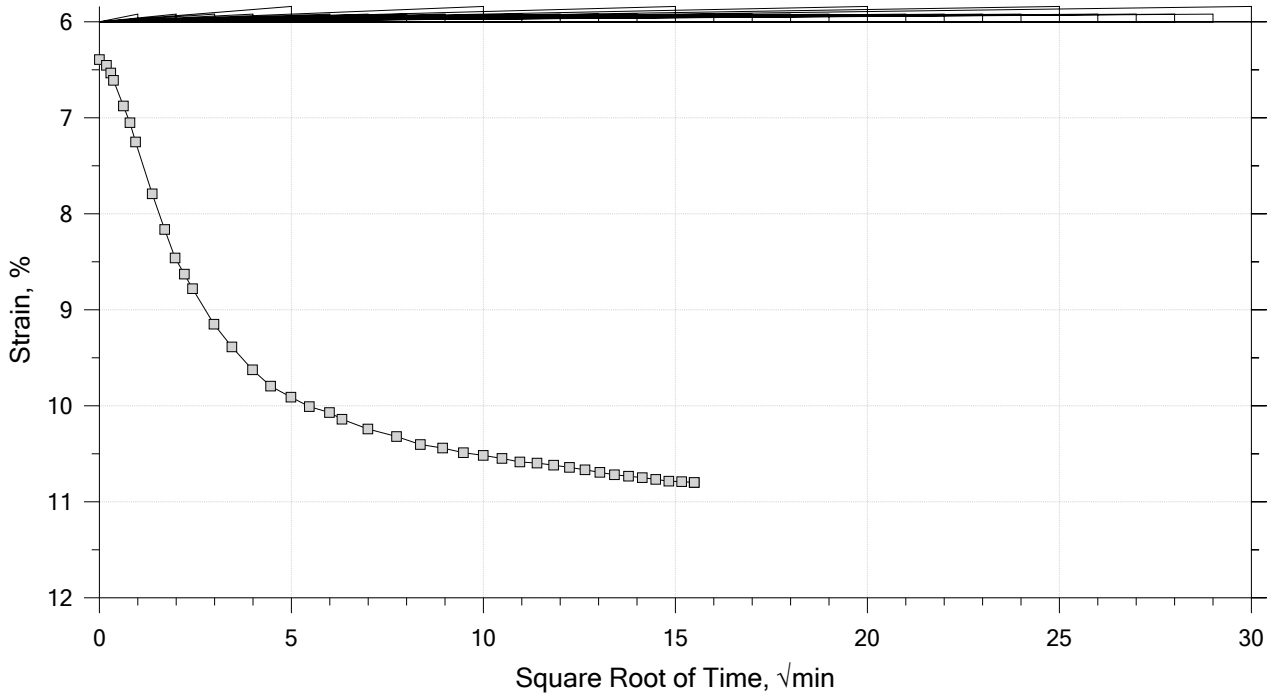
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 4 tsf





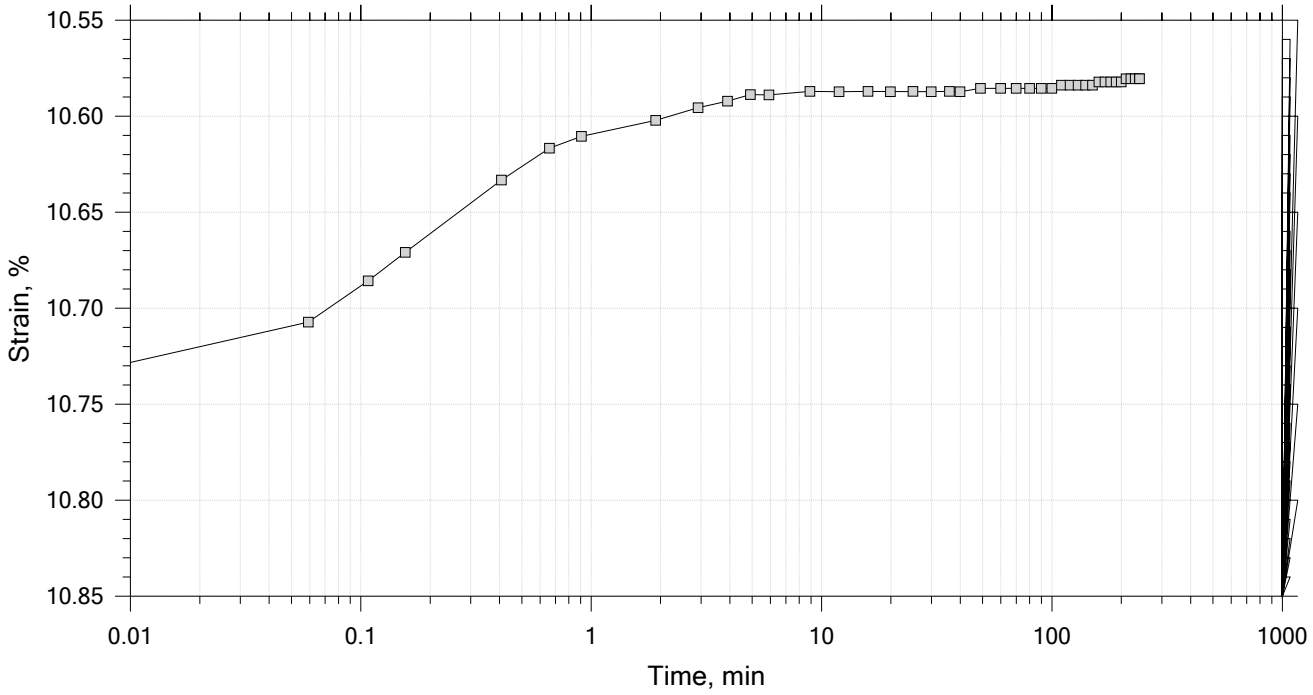
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

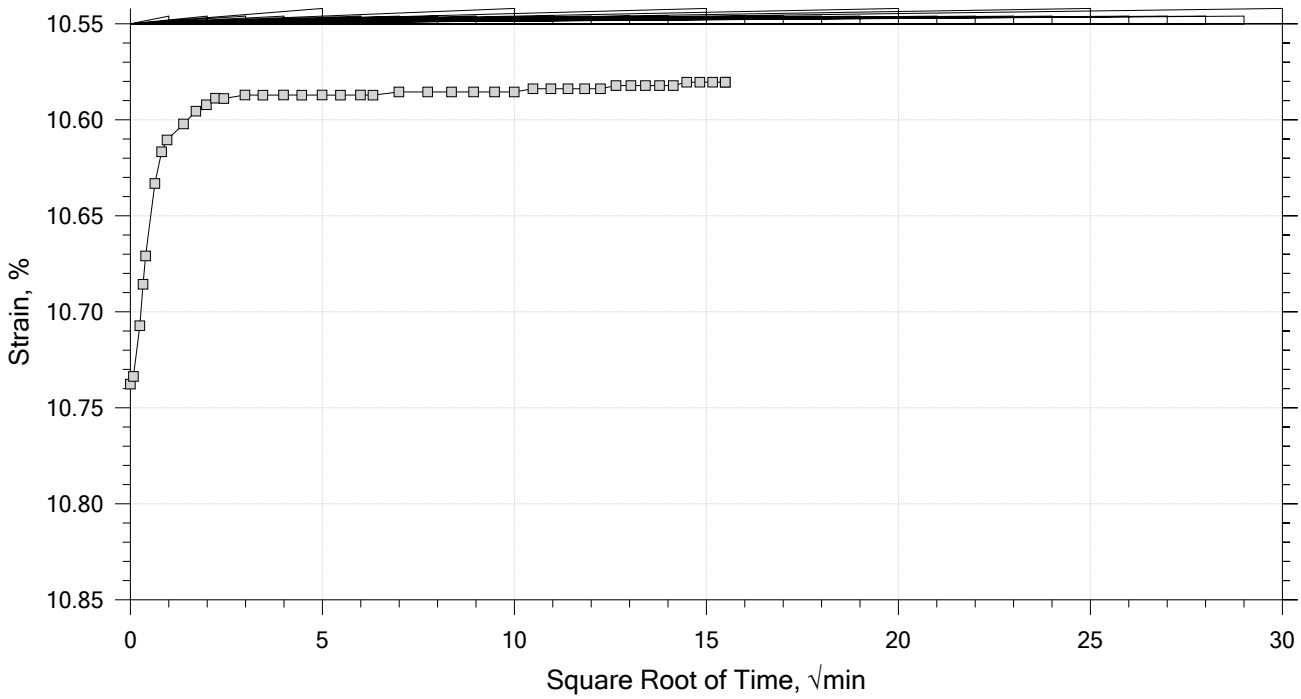
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14

Constant Load Step

Stress: 2 tsf





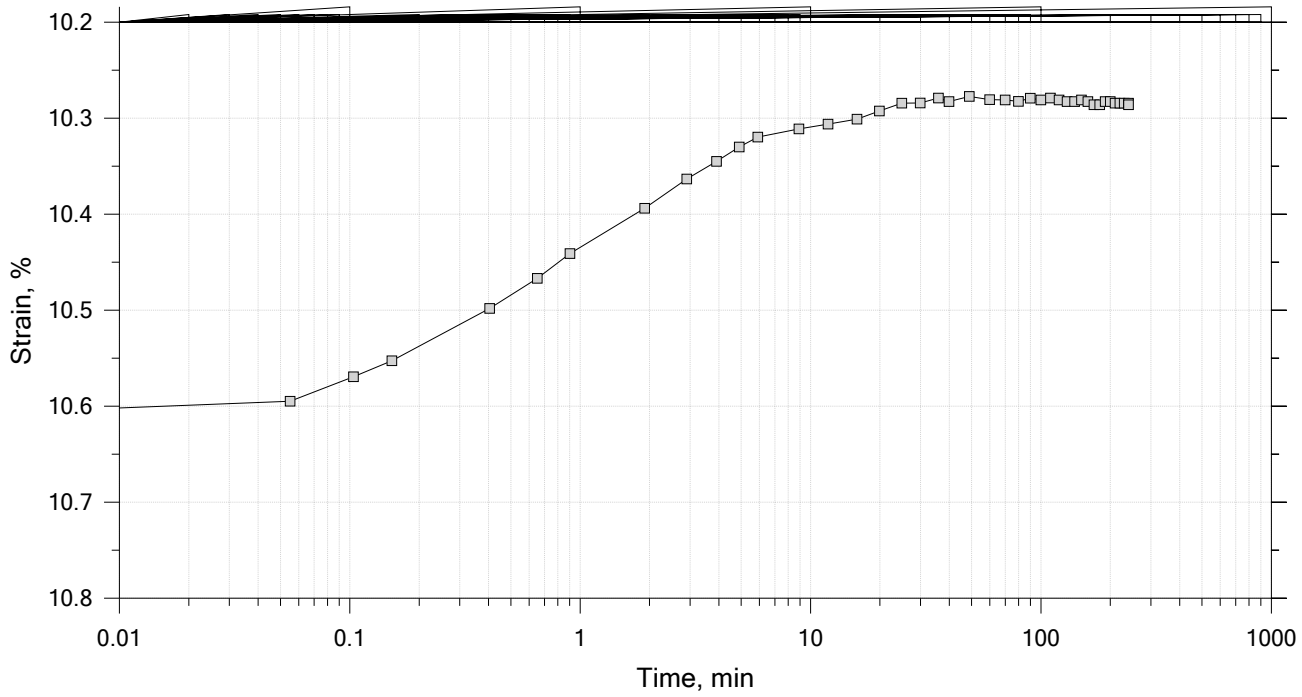
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

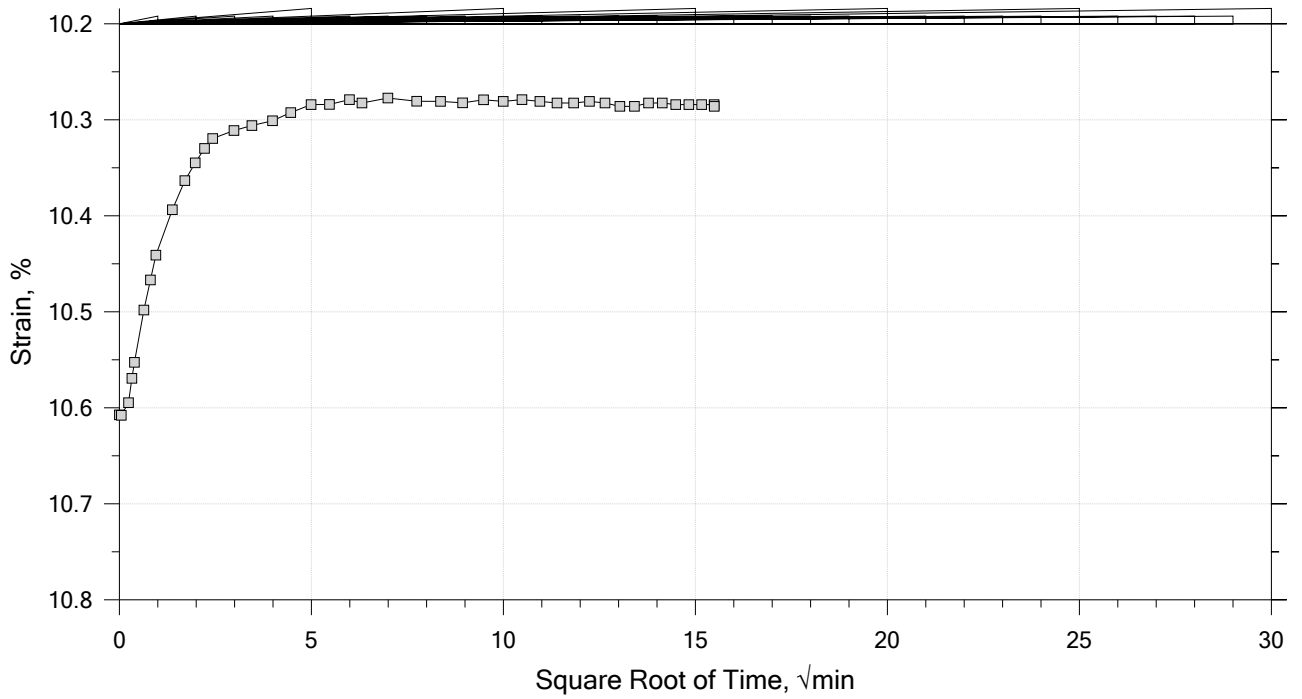
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf

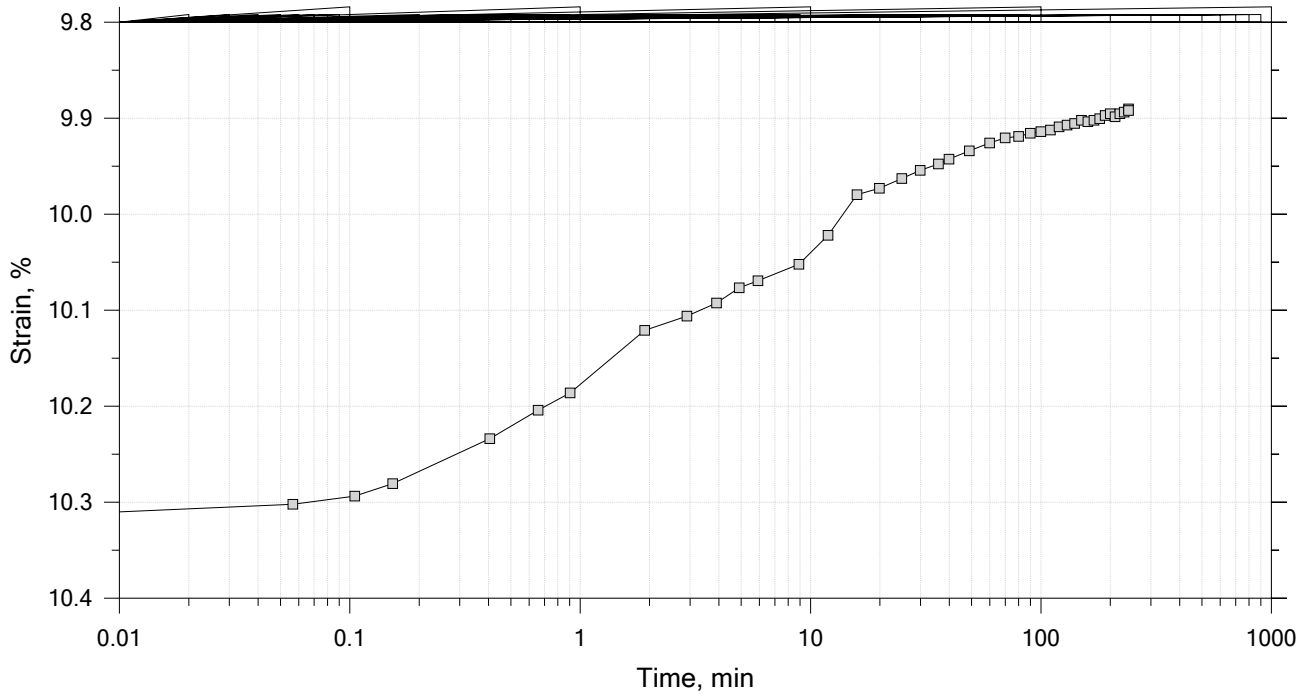


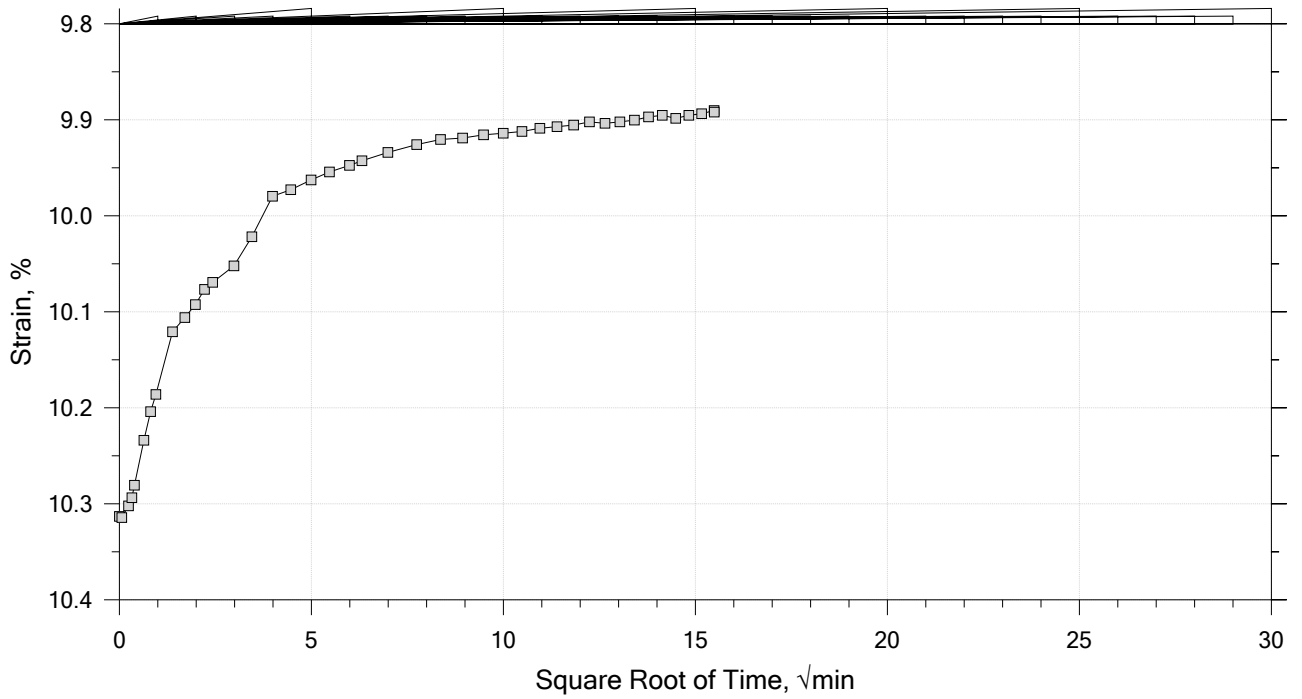


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

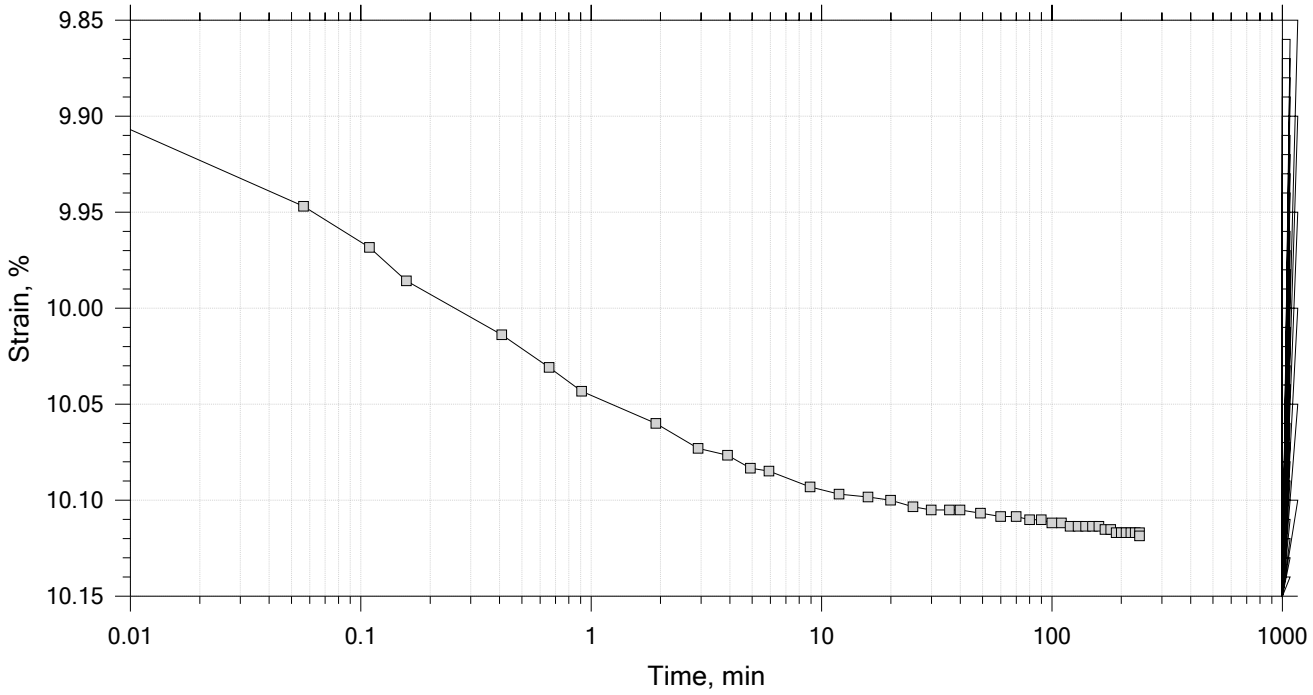


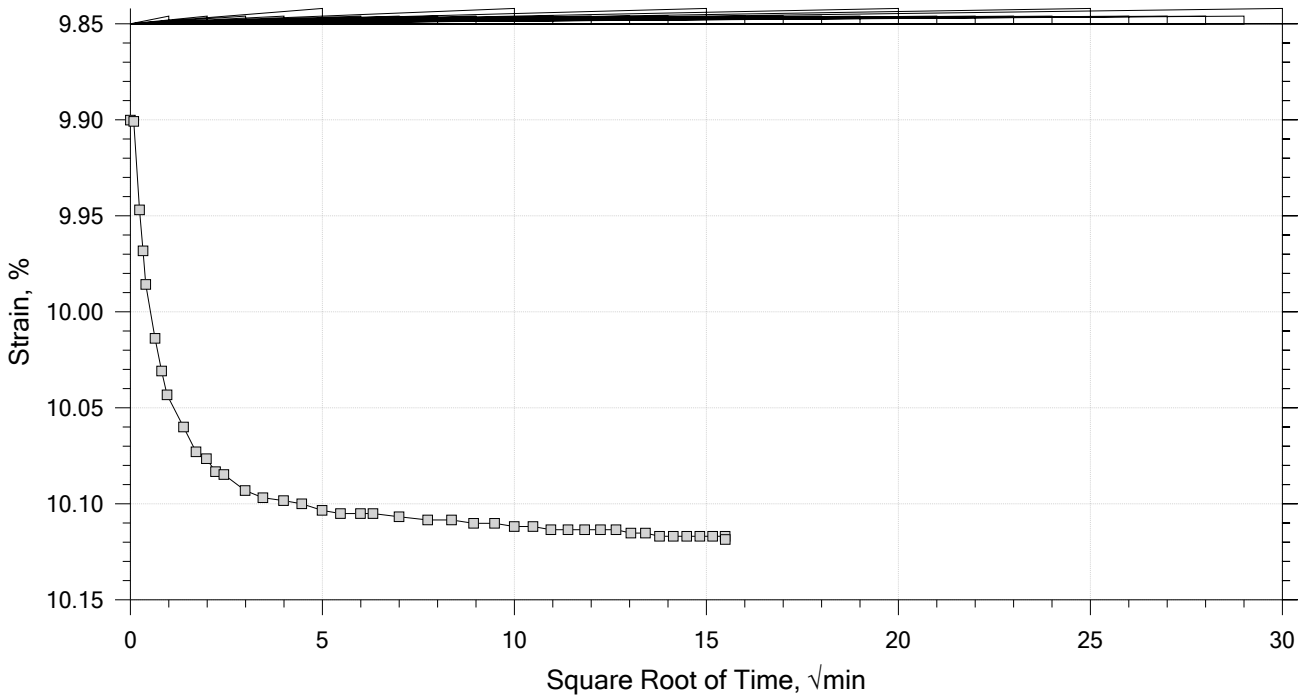


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

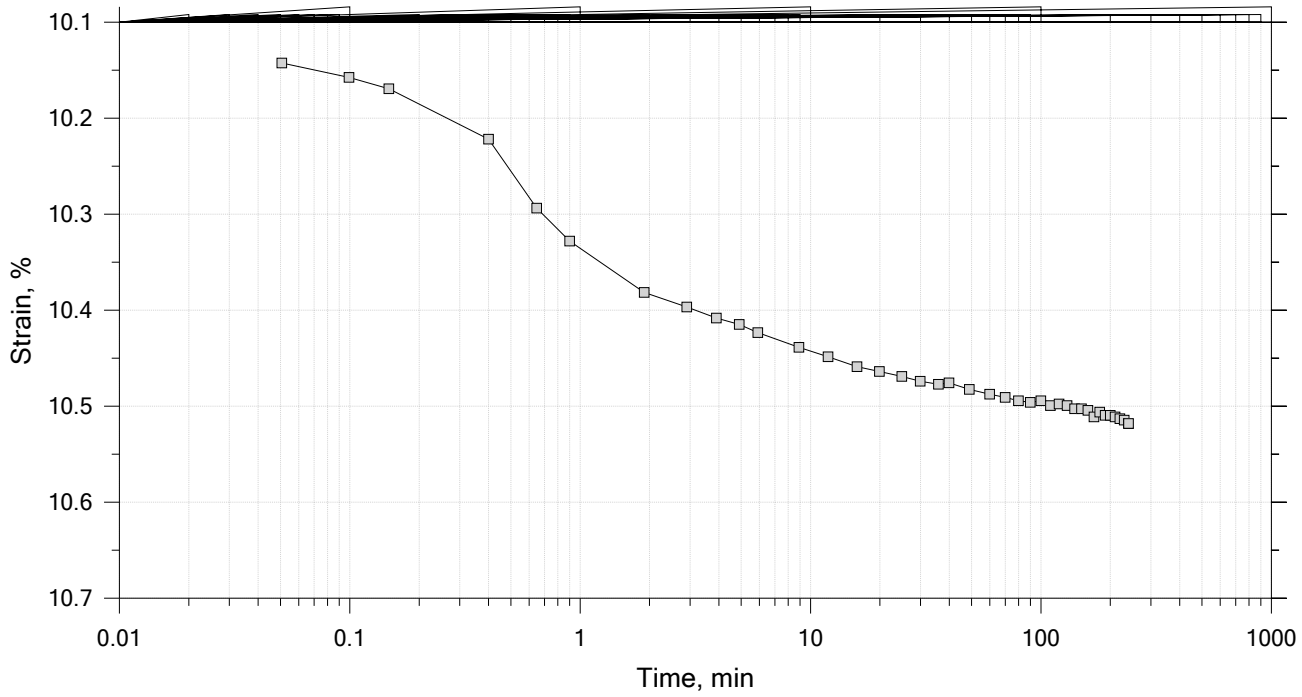


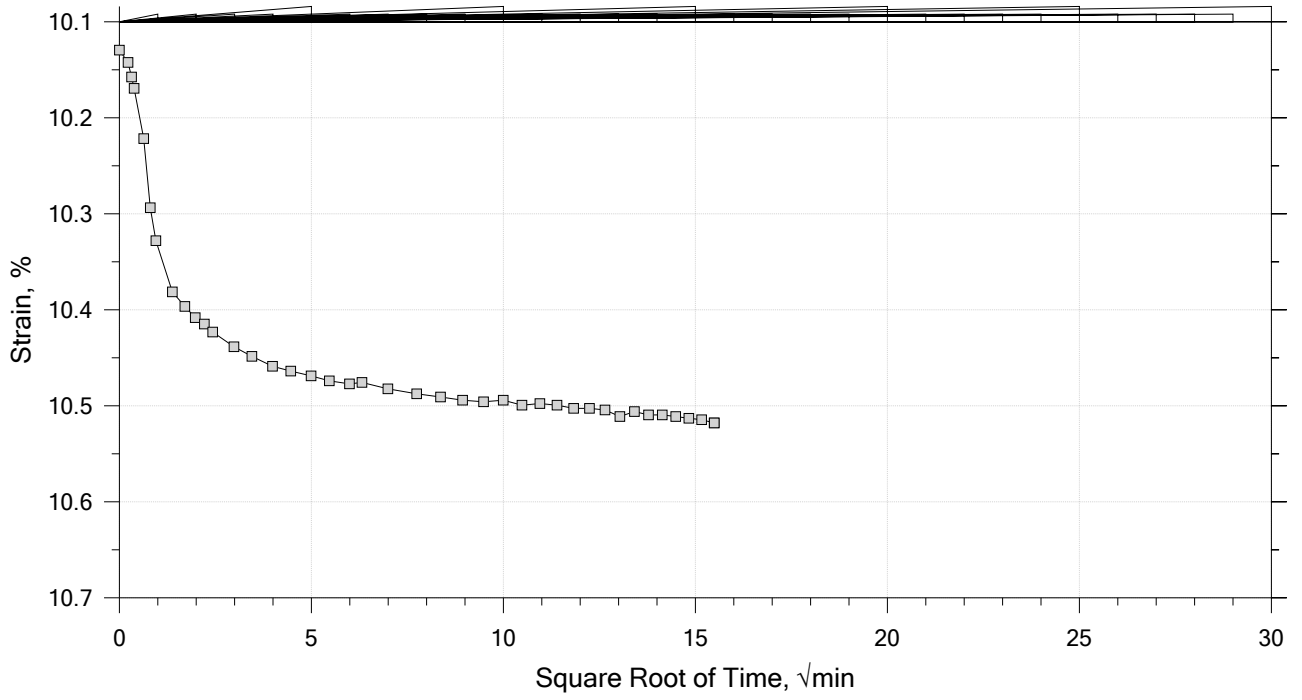


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

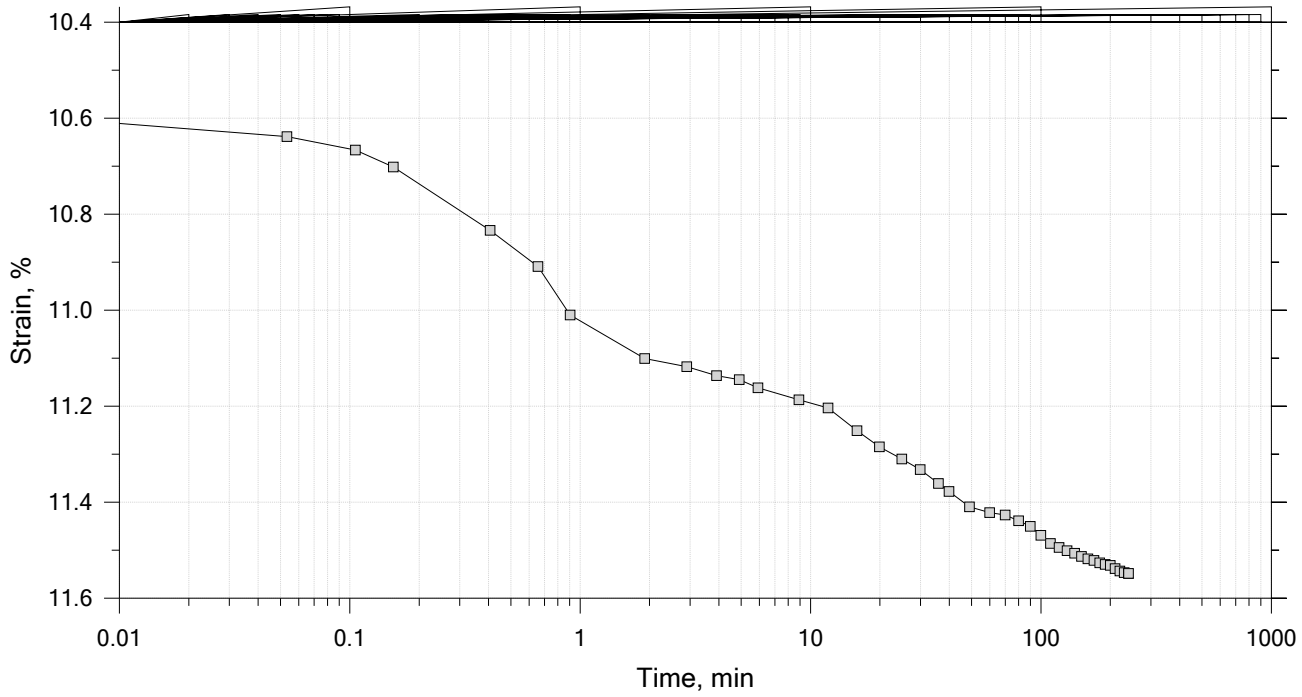


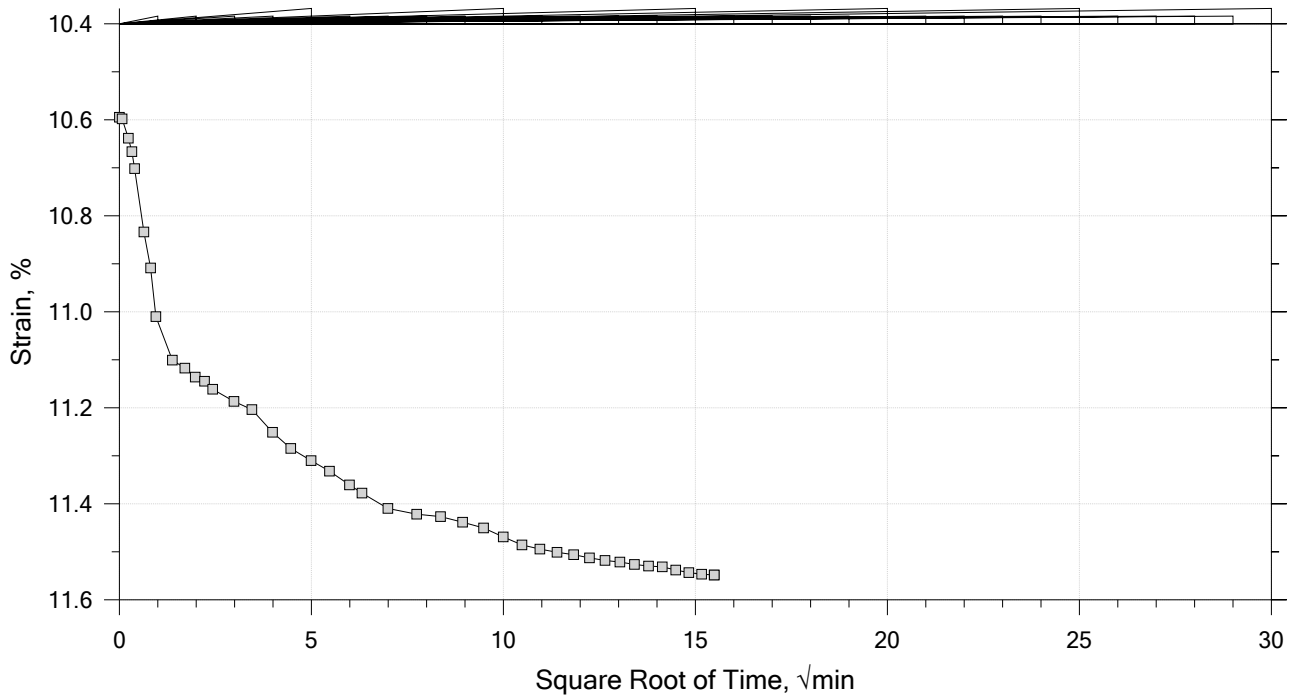


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf

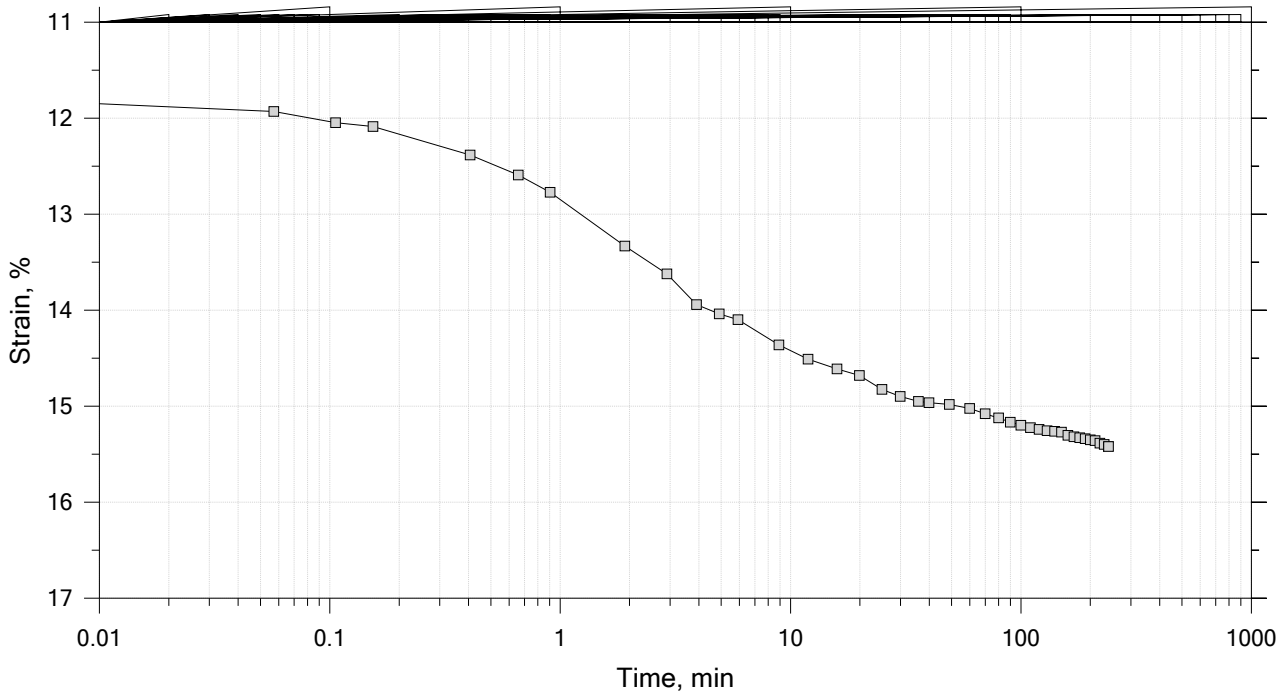


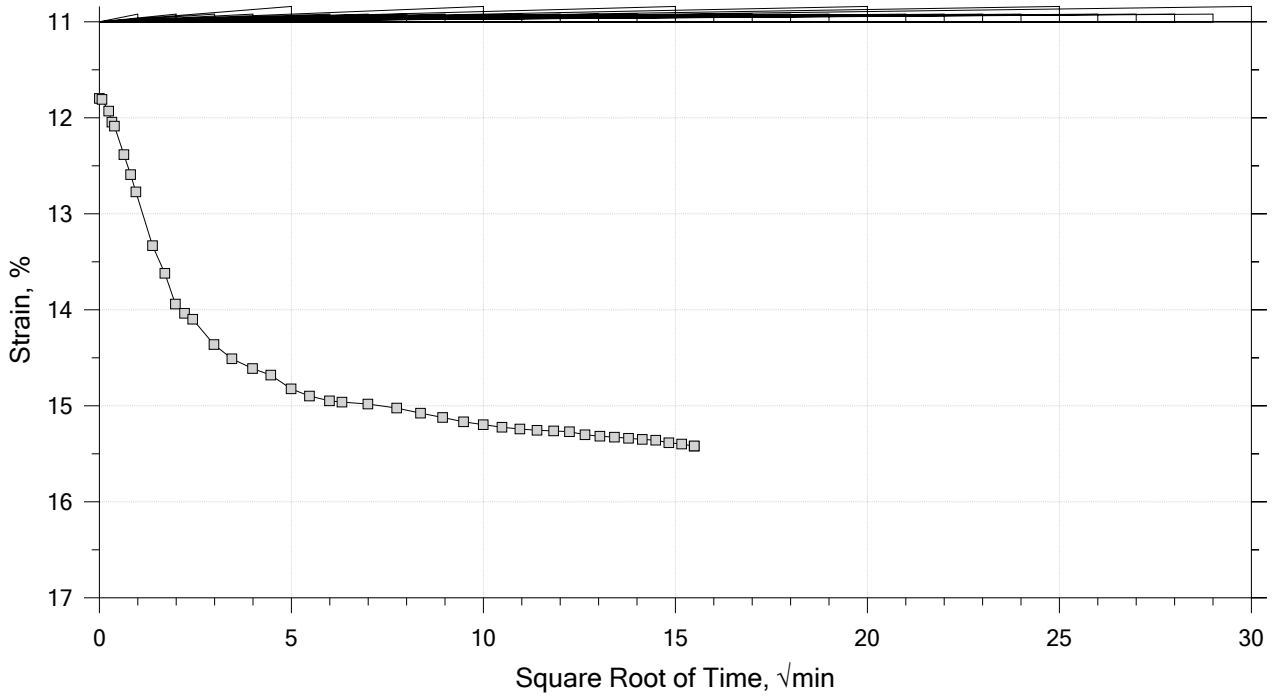


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14
Constant Load Step
Stress: 8 tsf

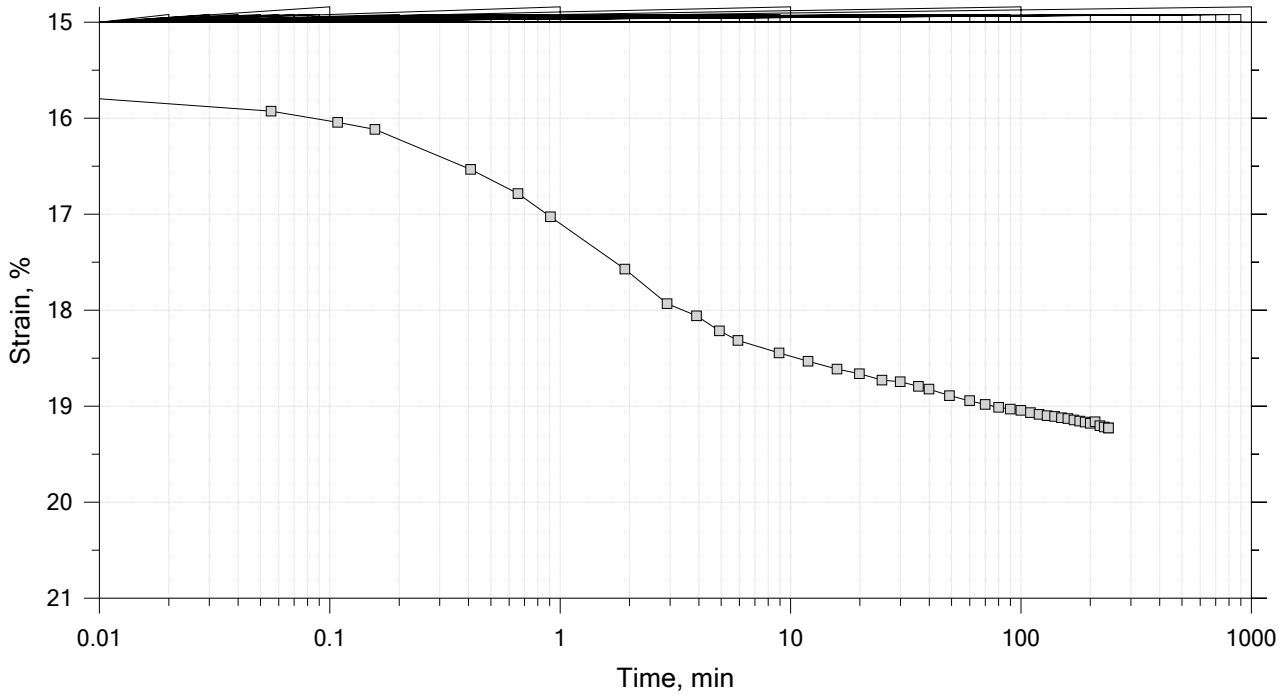


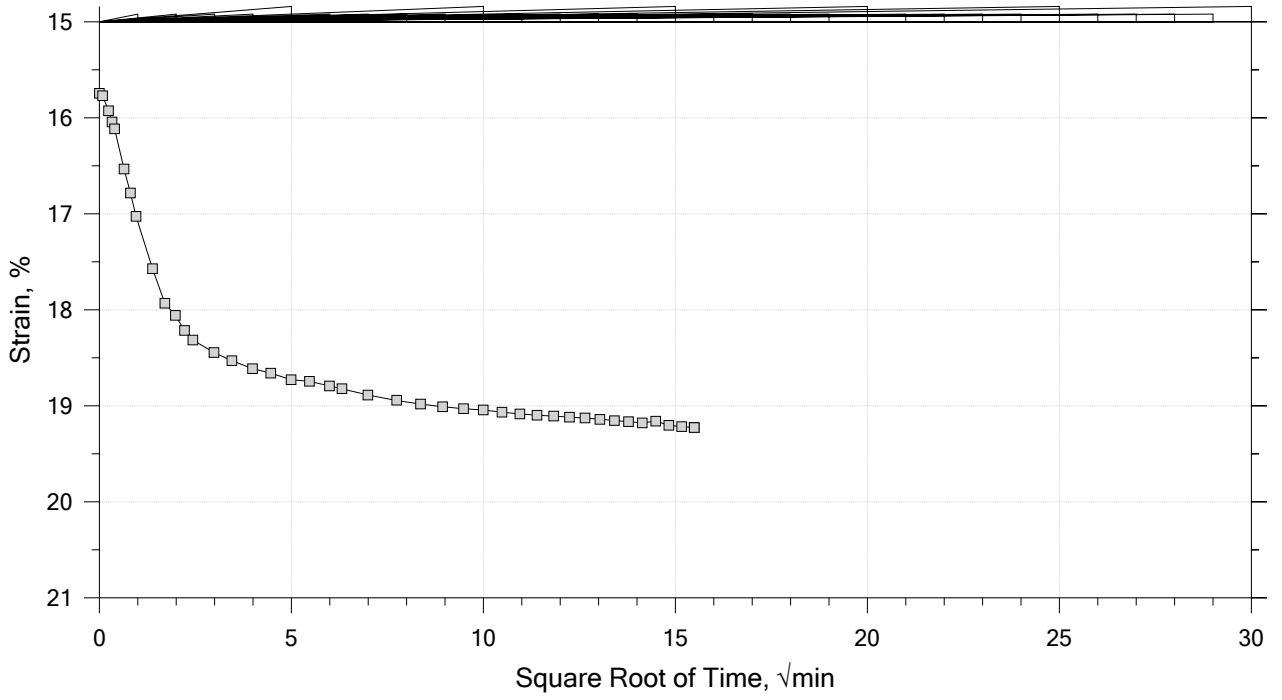


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 14
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.74	Liquid Limit: 31
Initial Height: 1.00 in	Initial Void Ratio: 0.842	Plastic Limit: 18
Final Height: 0.85 in	Final Void Ratio: 0.566	Plasticity Index: 13

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-5615	RING		E1532
Mass Container, gm	8.27	109.65	109.65	8.34
Mass Container + Wet Soil, gm	109.56	265.47	254	154.3
Mass Container + Dry Soil, gm	86.23	229.3	229.3	129.32
Mass Dry Soil, gm	77.96	119.65	119.65	120.98
Water Content, %	29.93	30.23	20.65	20.65
Void Ratio	---	0.84	0.57	---
Degree of Saturation, %	---	98.38	100.00	---
Dry Unit Weight, pcf	---	92.855	109.24	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Square Root of Time Coefficients

Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.125	0.02146	0.802	2.15	3.958	8.74e-04	1.72e-01	2.81e-03
2	0.250	0.03204	0.783	3.20	5.620	5.95e-04	8.47e-02	9.44e-04
3	0.500	0.03910	0.770	3.91	10.209	3.22e-04	2.82e-02	1.70e-04
4	1.00	0.04735	0.755	4.73	7.122	4.54e-04	1.65e-02	1.40e-04
5	2.00	0.06151	0.729	6.15	32.683	9.67e-05	1.42e-02	2.56e-05
6	4.00	0.1080	0.643	10.8	17.050	1.74e-04	2.32e-02	7.55e-05
7	2.00	0.1058	0.647	10.6	1.617	1.74e-03	1.09e-03	3.55e-05
8	1.00	0.1029	0.653	10.3	5.027	5.64e-04	2.95e-03	3.11e-05
9	0.500	0.09892	0.660	9.89	18.947	1.51e-04	7.88e-03	2.22e-05
10	1.00	0.1012	0.656	10.1	1.753	1.63e-03	4.53e-03	1.39e-04
11	2.00	0.1052	0.648	10.5	5.018	5.66e-04	3.99e-03	4.24e-05
12	4.00	0.1155	0.629	11.5	51.087	5.47e-05	5.15e-03	5.28e-06
13	8.00	0.1542	0.558	15.4	10.474	2.53e-04	9.68e-03	4.58e-05
14	16.0	0.1923	0.488	19.2	6.210	3.89e-04	4.76e-03	3.47e-05

GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		
	Displacement at End of Increment		

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/03/23	Checked By:	ank
Depth :	---	Test Id:	741455		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-203	S- 2	5-7'	Moist, grayish brown clay	23.8
HB-BHR-204	S- 2	5-7'	Moist, brown clay	24.0
HB-BHR-204	S- 3	10-12'	Moist, olive brown sandy silt	12.4
HB-BHR-205	S- 2	5-7'	Moist, brown clay	27.7
HB-BHR-205	S- 3	10-12'	Moist, brown clay	24.8
HB-BHR-206	S- 3	10-12'	Moist, olive gray clay	22.7
HB-BHR-206	S- 5	20-22'	Moist, olive gray clay with sand and gravel	14.6
HB-BHR-207	S- 4	15-17'	Moist, olive brown clay	27.8
HB-BHR-208	S- 2	5-7'	Moist, brown clay	24.5
HB-BHR-208	S- 4	15-17'	Wet, gray silty clay	28.2

Notes: Temperature of Drying : 110º Celsius

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/03/23	Checked By:	ank
Depth :	---	Test Id:	741465		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-209	S- 2	5-7'	Moist, brown silt with sand	10.1
HB-BHR-209	S- 4	15-17'	Moist, olive gray silty sand with gravel	10.2
HB-BHR-210	S- 2	5-7'	Moist, olive brown silt with gravel	10.5
HB-BHR-210	S- 4	15-17'	Moist, brown silt	12.0
HB-BHR-210	S- 6	25-27'	Moist, olive gray clay	23.6
HB-BHR-210	S- 9	40-42'	Moist, gray gravel with silt and sand	6.8
HB-BHR-211	S- 2	5-7'	Moist, brown clay	22.1
HB-BHR-211	S- 4	15-17'	Moist, olive clay	30.1
HB-BHR-211	S- 5	20-22'	Moist, brown silty sand with gravel	11.8
HB-BHR-212	S- 4	15-17'	Moist, olive brown clay	24.7

Notes: Temperature of Drying : 110º Celsius

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/03/23	Checked By:	ank
Depth :	---	Test Id:	741473		

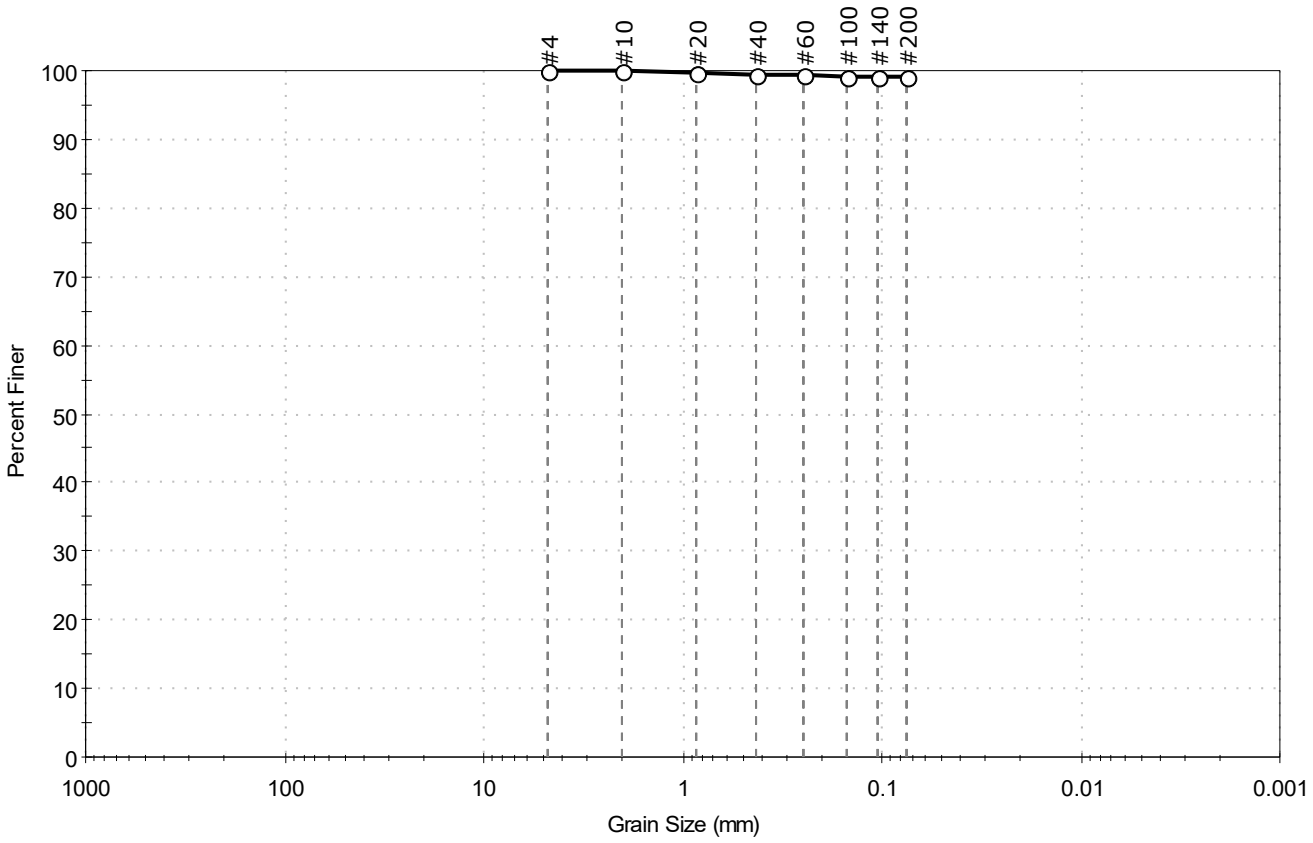
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
HB-BHR-212	S- 3	10-12'	Moist, olive clay	21.4
HB-BHR-212	S- 5	20-22'	Moist, brown sandy clay with gravel	11.5
HB-BHR-213	S- 4	15-17'	Moist, gray clay	13.9
HB-BHR-214	S- 6	25-27'	Moist, gray clay with sand and gravel	12.8
HB-BHR-215	S- 2	5-7'	Moist, olive brown silt	17.9
HB-BHR-215	S- 5	20-22'	Moixst, gray clay with gravel	9.9
HB-BHR-216	S- 3	10-12'	Moist, dark brown clay	15.6
HB-BHR-216	S- 6	25-27'	Moist, yellowish brown silty sand with gravel	9.7

Notes: Temperature of Drying : 110º Celsius

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-203	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741421		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	1.0	99.0

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

Coefficients

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

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (12))

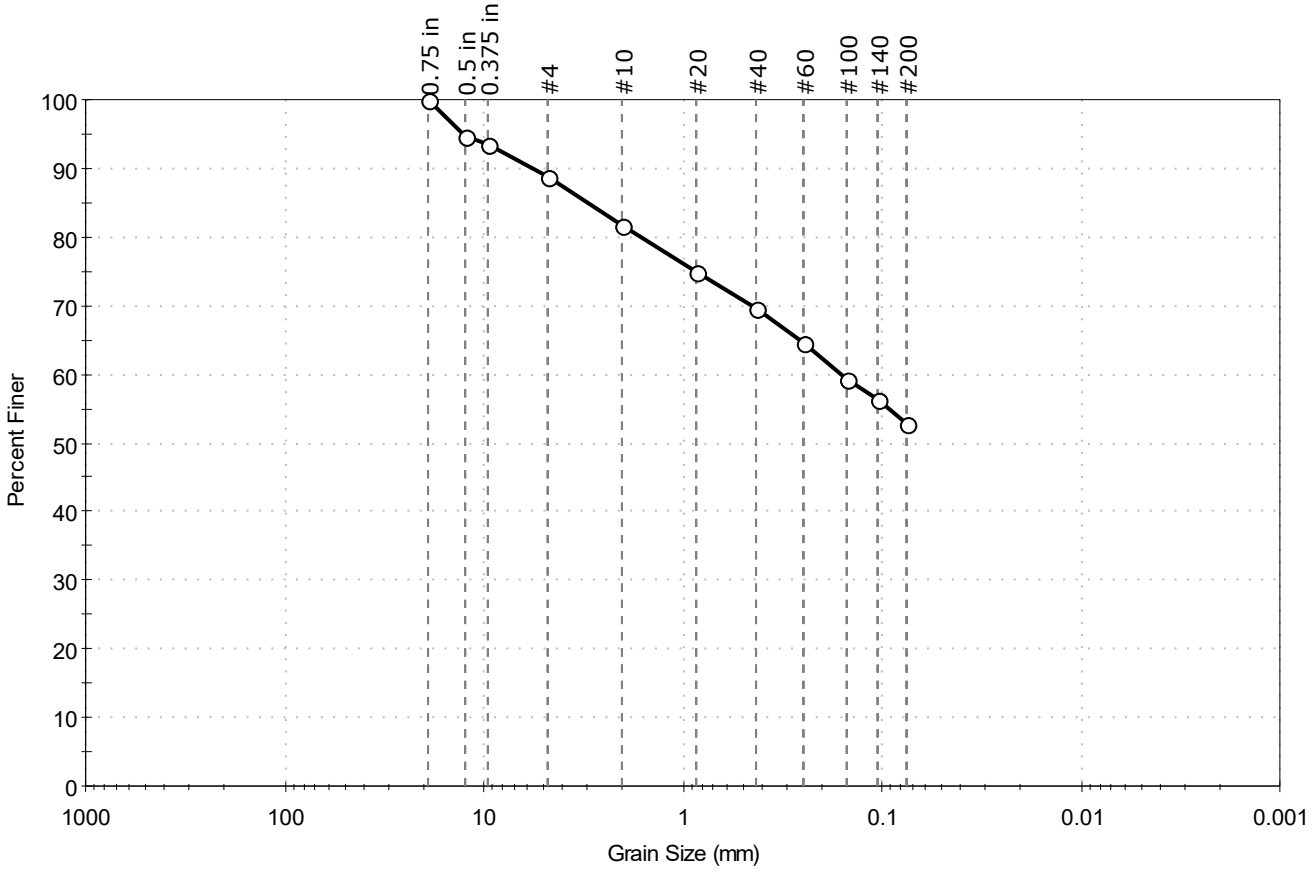
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-204	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3	Test Date:	11/03/23	Checked By:	ank
Depth :	10-12'	Test Id:	741422		
Test Comment:	---				
Visual Description:	Moist, olive brown sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	11.1	36.1	52.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	94		
#4	4.75	89		
#10	2.00	82		
#20	0.85	75		
#40	0.42	70		
#60	0.25	65		
#100	0.15	59		
#140	0.11	56		
#200	0.075	53		

Coefficients

D_{85} = 2.9991 mm	D_{30} = N/A
D_{60} = 0.1587 mm	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

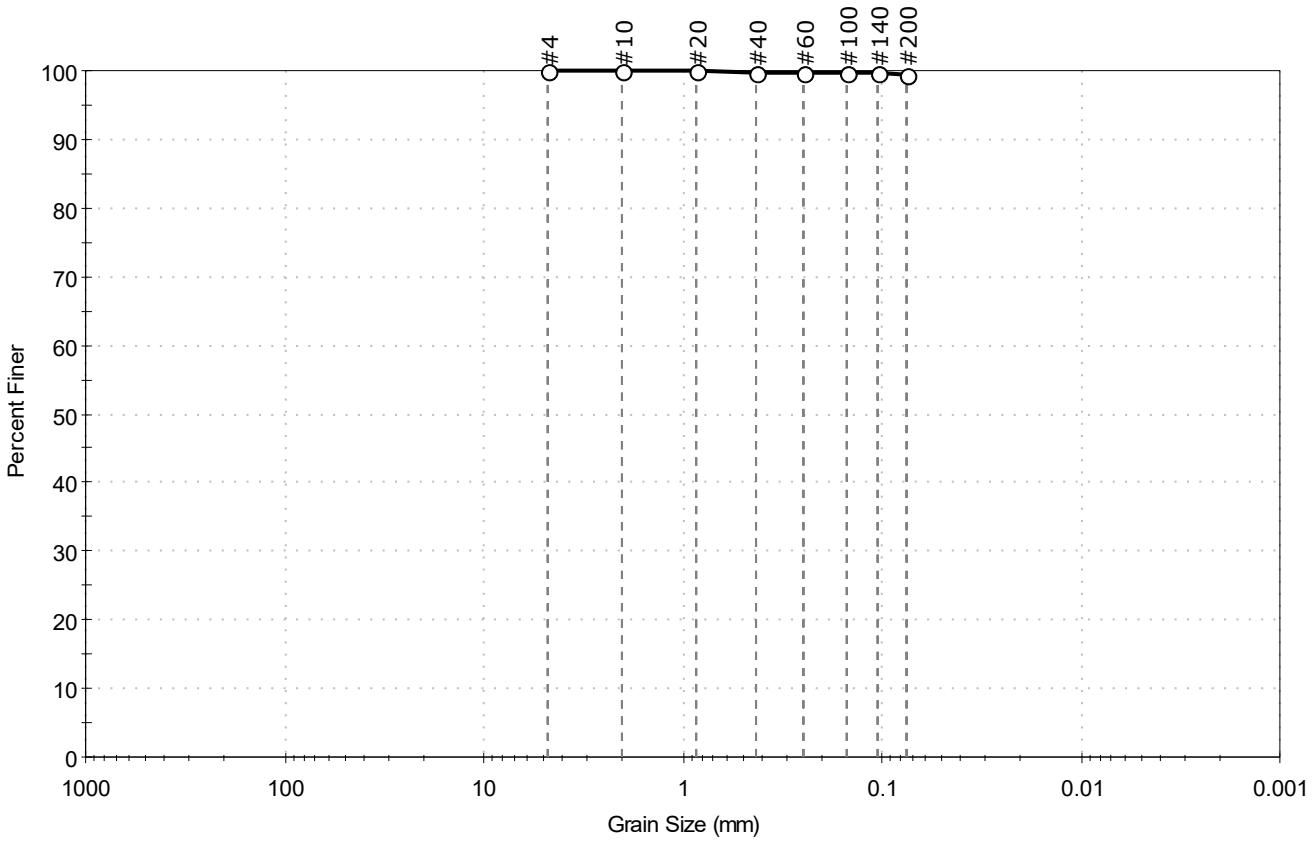
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-205	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741423		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.5	99.5

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

Coefficients

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

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

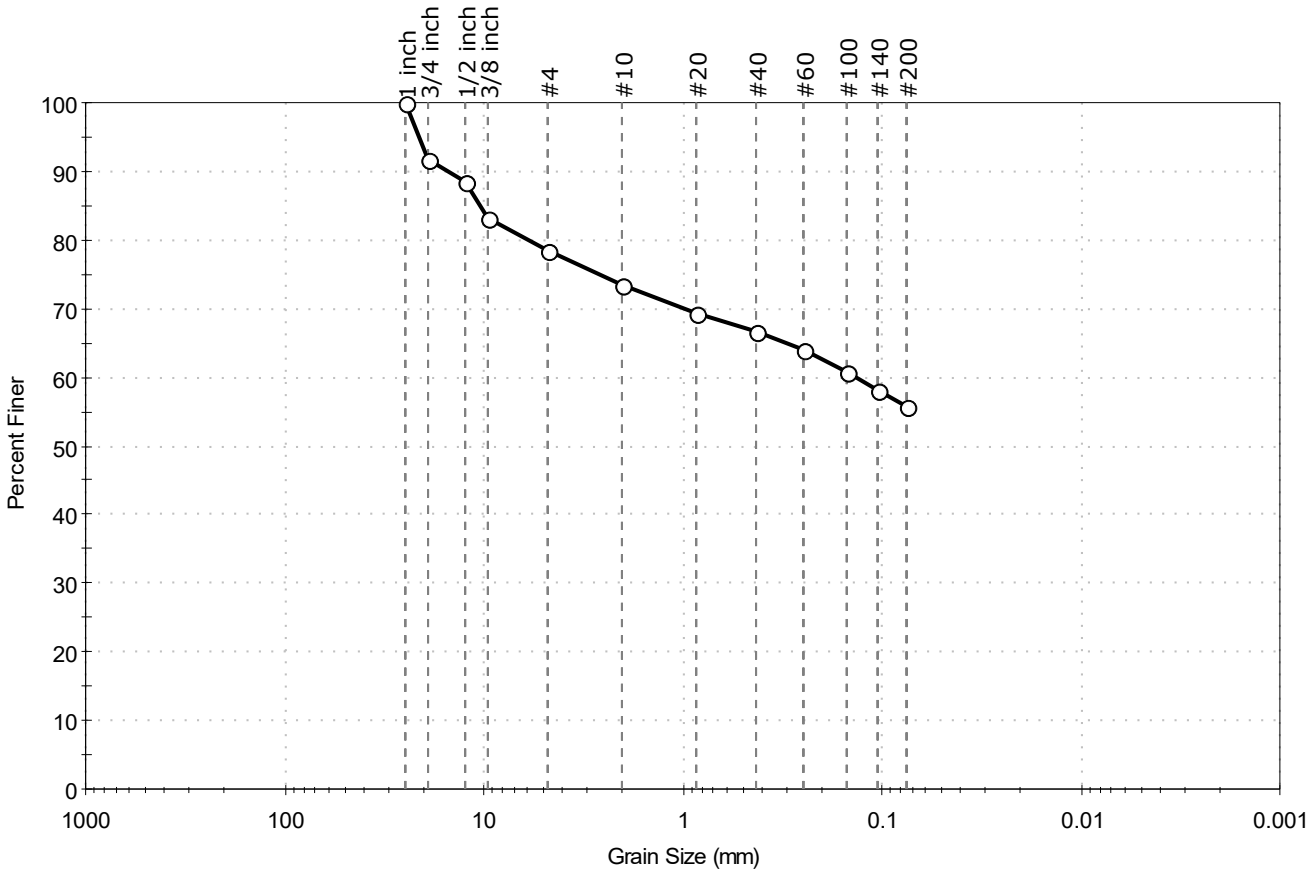
Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-206	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-5	Test Date:	11/02/23	Checked By:	ank
Depth :	20-22'	Test Id:	741424		
Test Comment:	---				
Visual Description:	Moist, olive gray clay with sand and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	21.6	22.7	55.7

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	89		
3/8 inch	9.50	83		
#4	4.75	78		
#10	2.00	74		
#20	0.85	69		
#40	0.42	67		
#60	0.25	64		
#100	0.15	61		
#140	0.11	58		
#200	0.075	56		

Coefficients

D_{85} = 10.4434 mm D_{30} = N/A

D_{60} = 0.1348 mm D_{15} = N/A

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

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

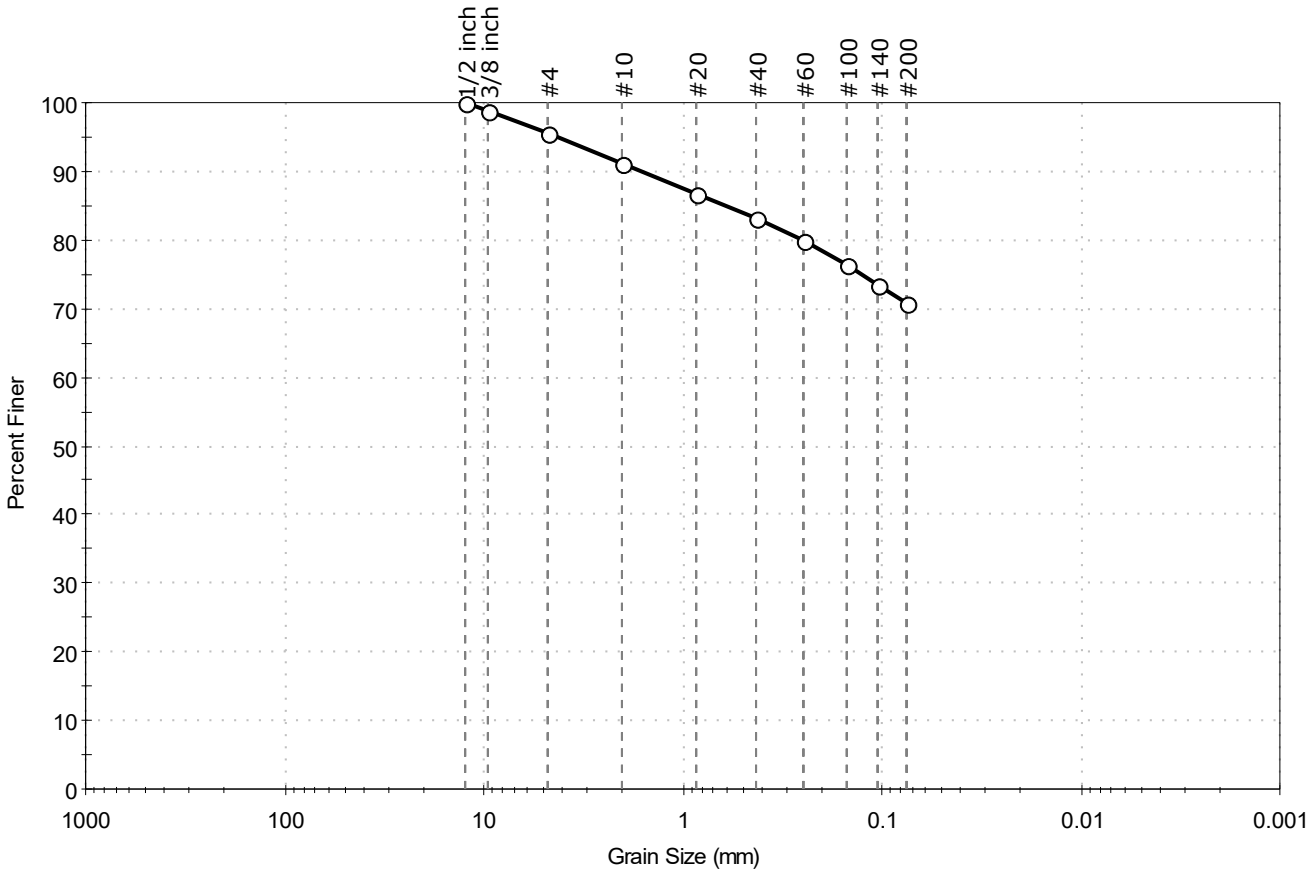
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-207	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741425		
Test Comment:	---				
Visual Description:	Moist, brown silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.5	24.7	70.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1/2 inch	12.50	100		
3/8 inch	9.50	99		
#4	4.75	96		
#10	2.00	91		
#20	0.85	87		
#40	0.42	83		
#60	0.25	80		
#100	0.15	76		
#140	0.11	73		
#200	0.075	71		

Coefficients

D_{85} = 0.6080 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

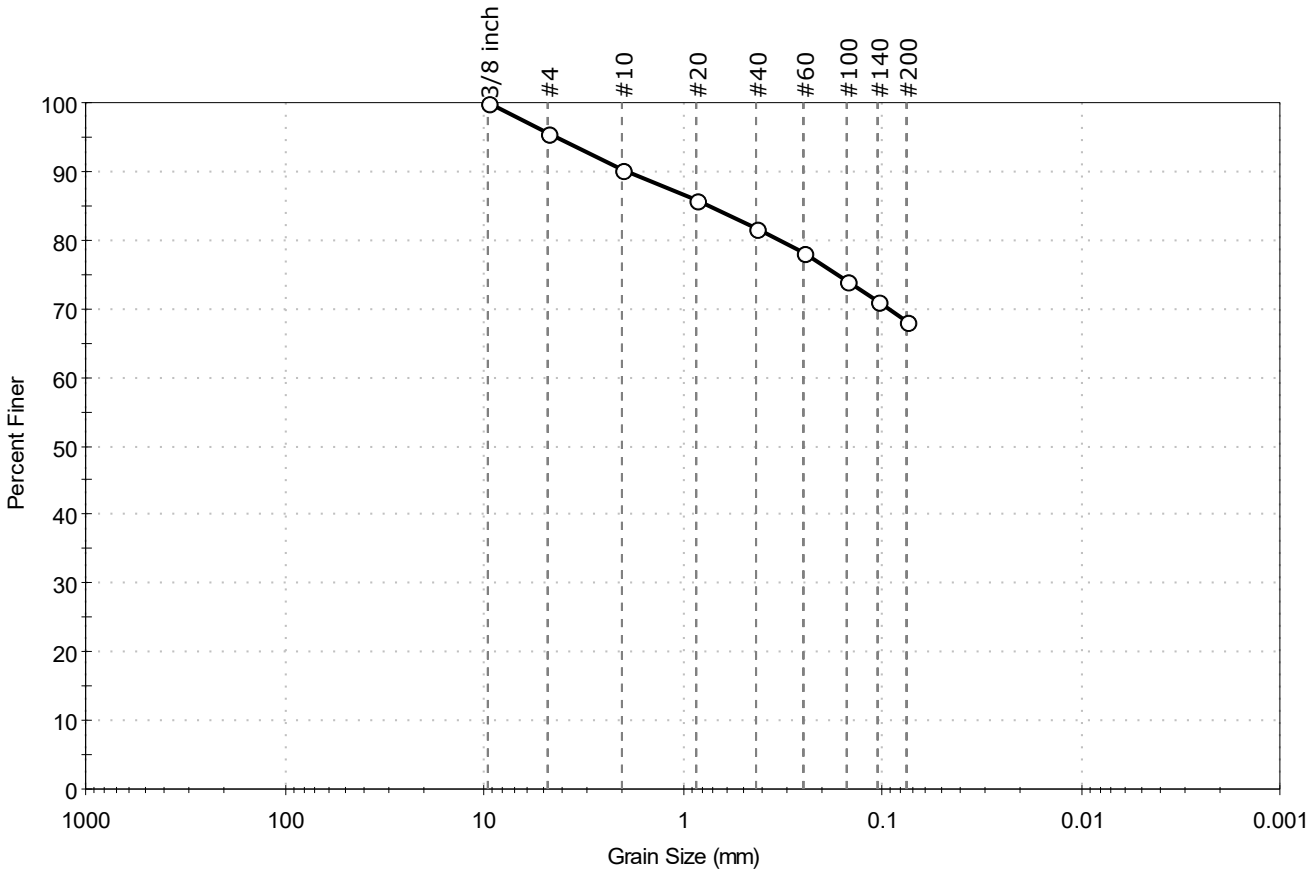
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-209	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741426		
Test Comment:	---				
Visual Description:	Moist, brown silt with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.4	27.6	68.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	96		
#10	2.00	90		
#20	0.85	86		
#40	0.42	82		
#60	0.25	78		
#100	0.15	74		
#140	0.11	71		
#200	0.075	68		

Coefficients

D_{85} = 0.7402 mm	D_{30} = N/A
D_{60} = N/A	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

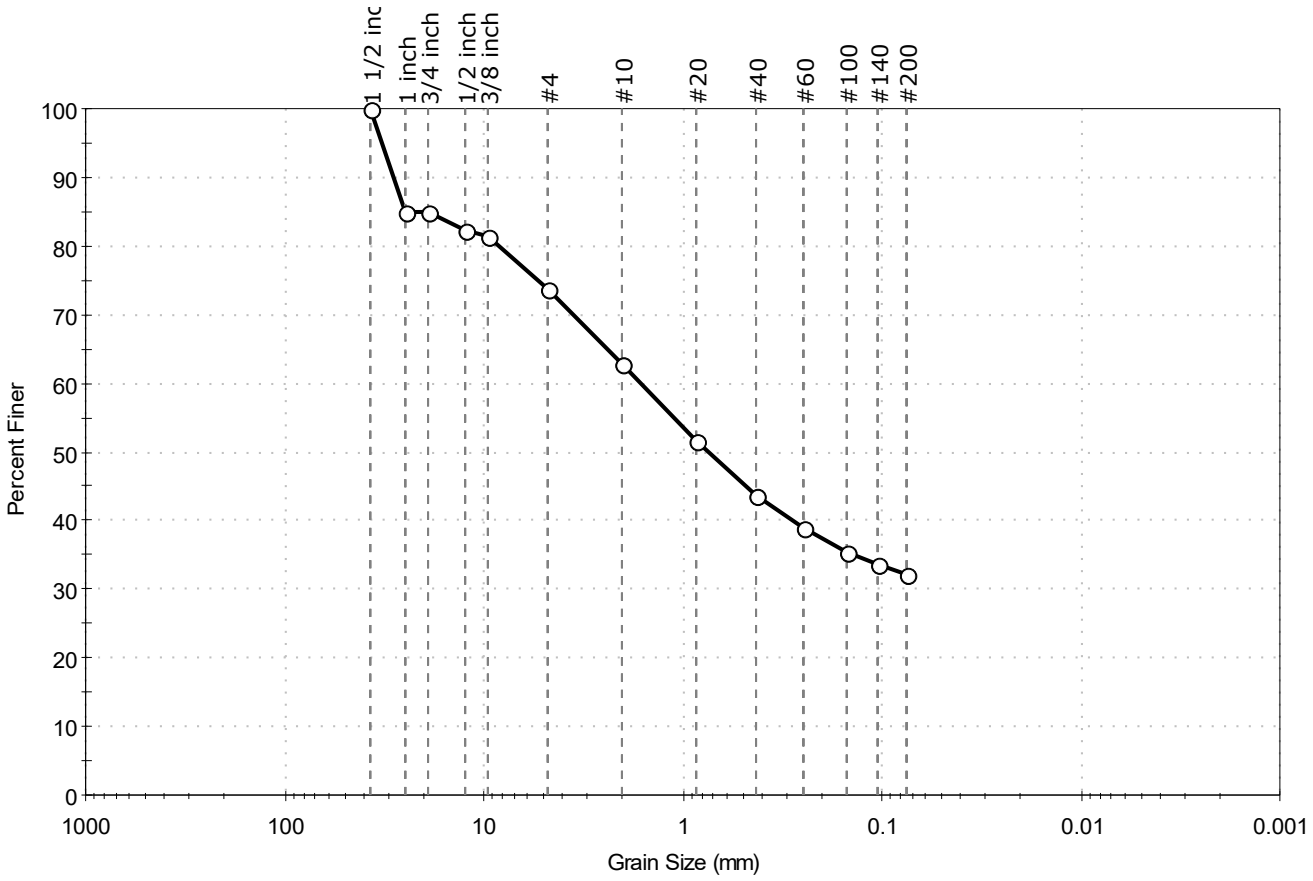
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-209	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-4	Test Date:	11/02/23	Checked By:	ank
Depth :	15-17'	Test Id:	741427		
Test Comment:	---				
Visual Description:	Moist, olive gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	26.4	41.4	32.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	85		
3/4 inch	19.00	85		
1/2 inch	12.50	82		
3/8 inch	9.50	82		
#4	4.75	74		
#10	2.00	63		
#20	0.85	52		
#40	0.42	44		
#60	0.25	39		
#100	0.15	35		
#140	0.11	34		
#200	0.075	32		

Coefficients

D_{85} = 25.0734 mm D_{30} = N/A

D_{60} = 1.6125 mm D_{15} = N/A

D_{50} = 0.7395 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

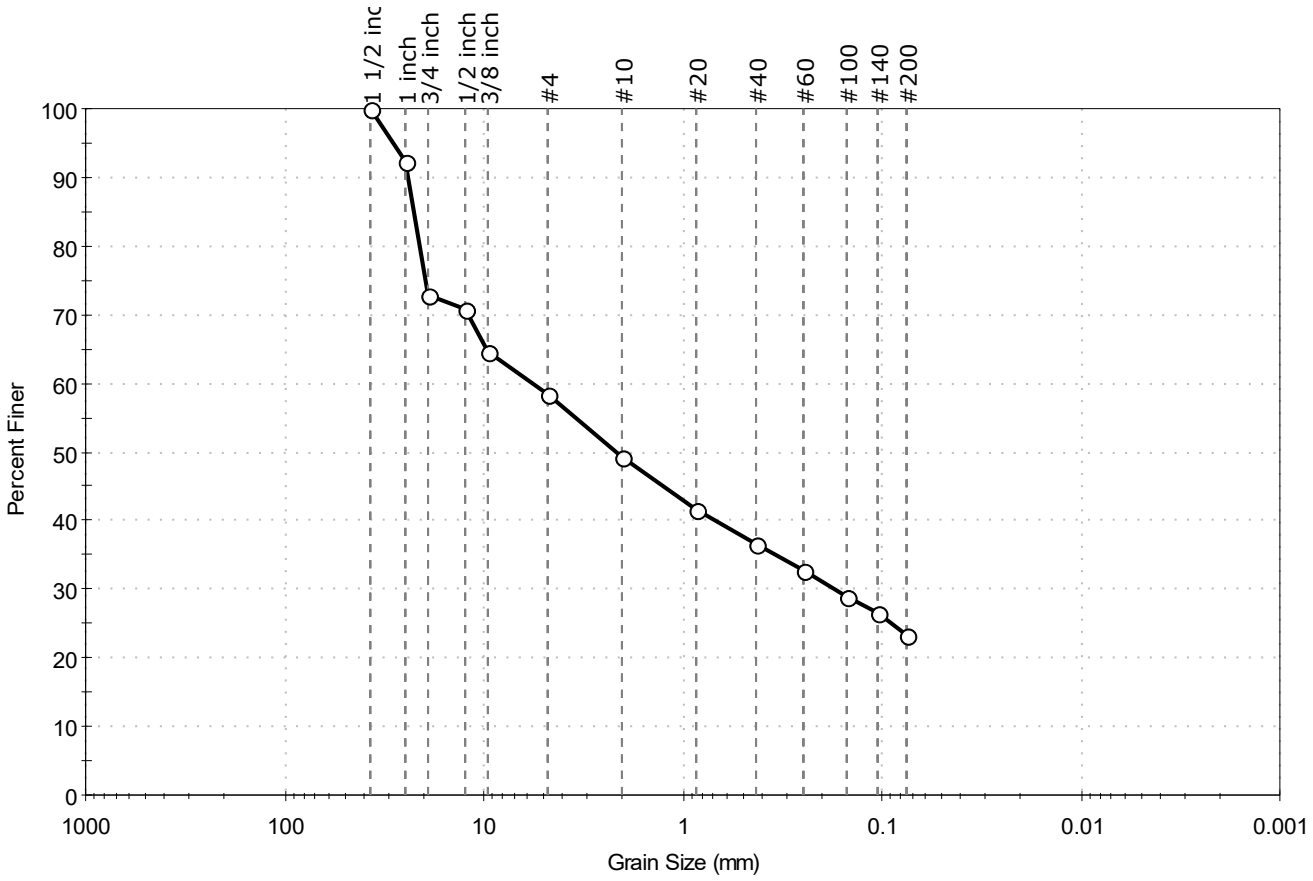
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-210	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-9	Test Date:	11/02/23	Checked By:	ank
Depth :	40-42'	Test Id:	741428		
Test Comment:	---				
Visual Description:	Moist, gray gravel with silt and sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	41.7	34.9	23.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	92		
3/4 inch	19.00	73		
1/2 inch	12.50	71		
3/8 inch	9.50	65		
#4	4.75	58		
#10	2.00	49		
#20	0.85	42		
#40	0.42	37		
#60	0.25	33		
#100	0.15	29		
#140	0.11	26		
#200	0.075	23		

Coefficients

D_{85} = 22.5187 mm	D_{30} = 0.1712 mm
D_{60} = 5.7029 mm	D_{15} = N/A
D_{50} = 2.1732 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

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

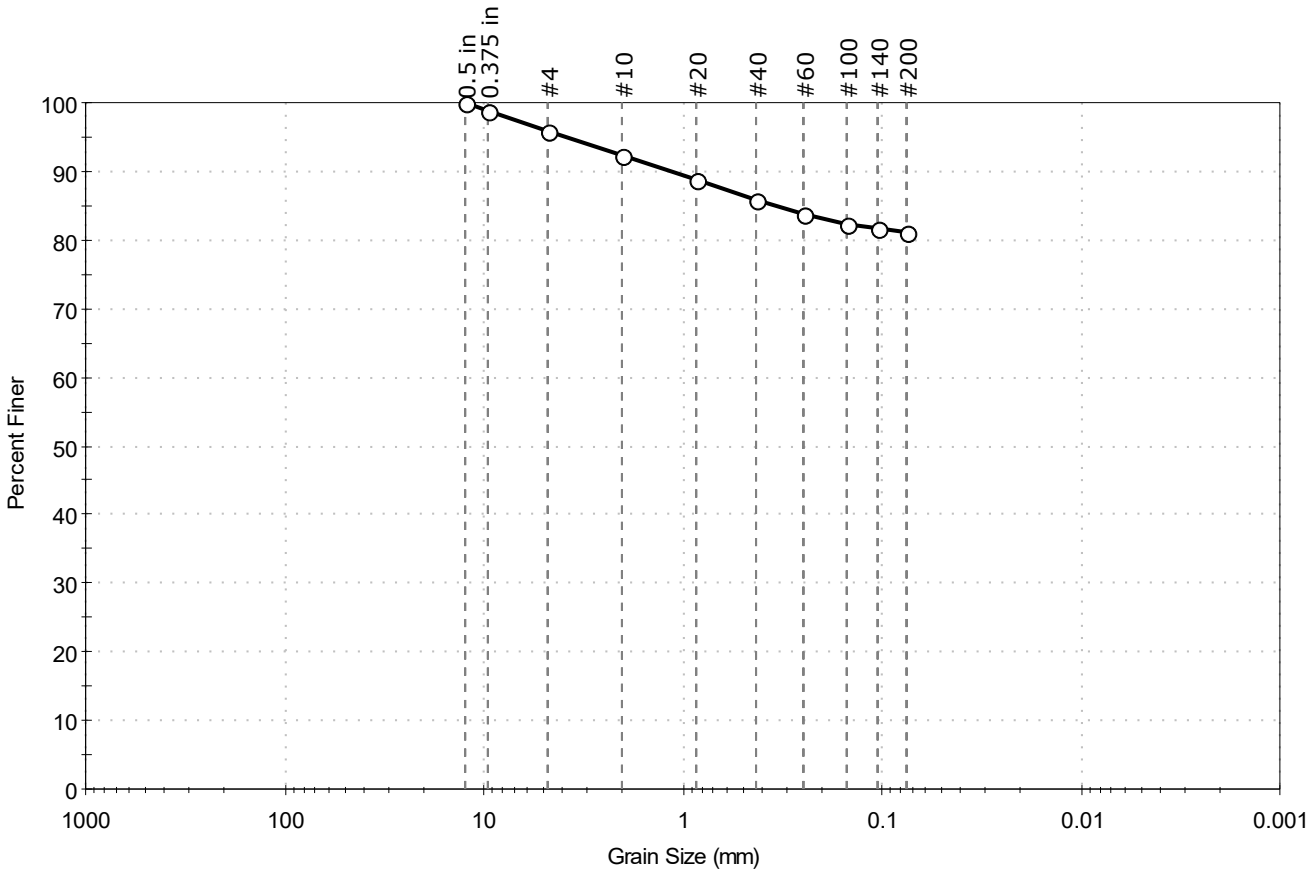
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-211	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741429		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	4.1	14.9	81.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	96		
#10	2.00	92		
#20	0.85	89		
#40	0.42	86		
#60	0.25	84		
#100	0.15	82		
#140	0.11	82		
#200	0.075	81		

Coefficients

D_{85} = 0.3412 mm D_{30} = N/A
D_{60} = N/A D_{15} = N/A
D_{50} = N/A D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

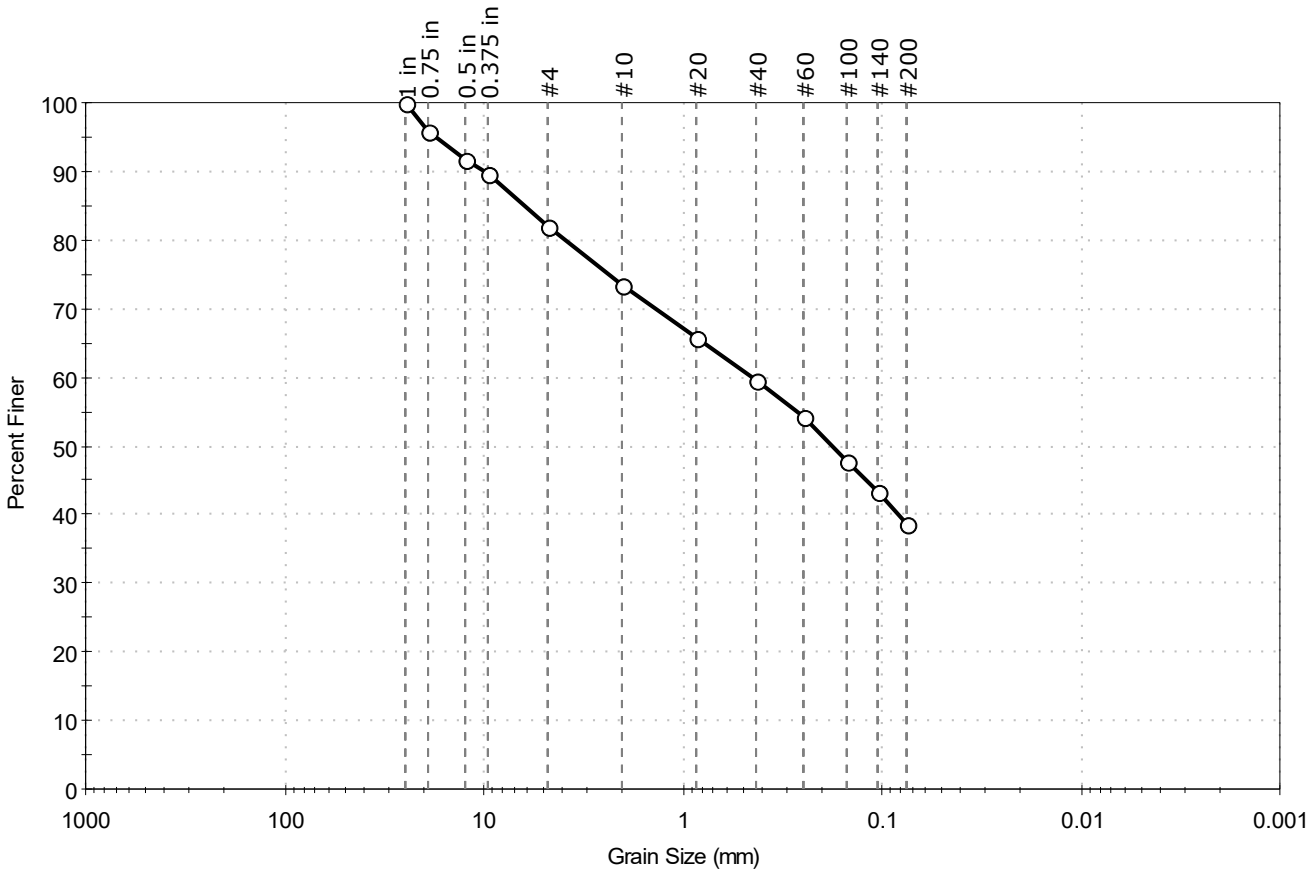
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-211	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-5	Test Date:	11/02/23	Checked By:	ank
Depth :	20-22'	Test Id:	741430		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	18.0	43.3	38.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	96		
0.5 in	12.50	92		
0.375 in	9.50	90		
#4	4.75	82		
#10	2.00	73		
#20	0.85	66		
#40	0.42	60		
#60	0.25	54		
#100	0.15	48		
#140	0.11	43		
#200	0.075	39		

Coefficients

D_{85} = 6.2137 mm D_{30} = N/A

D_{60} = 0.4402 mm D_{15} = N/A

D_{50} = 0.1768 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

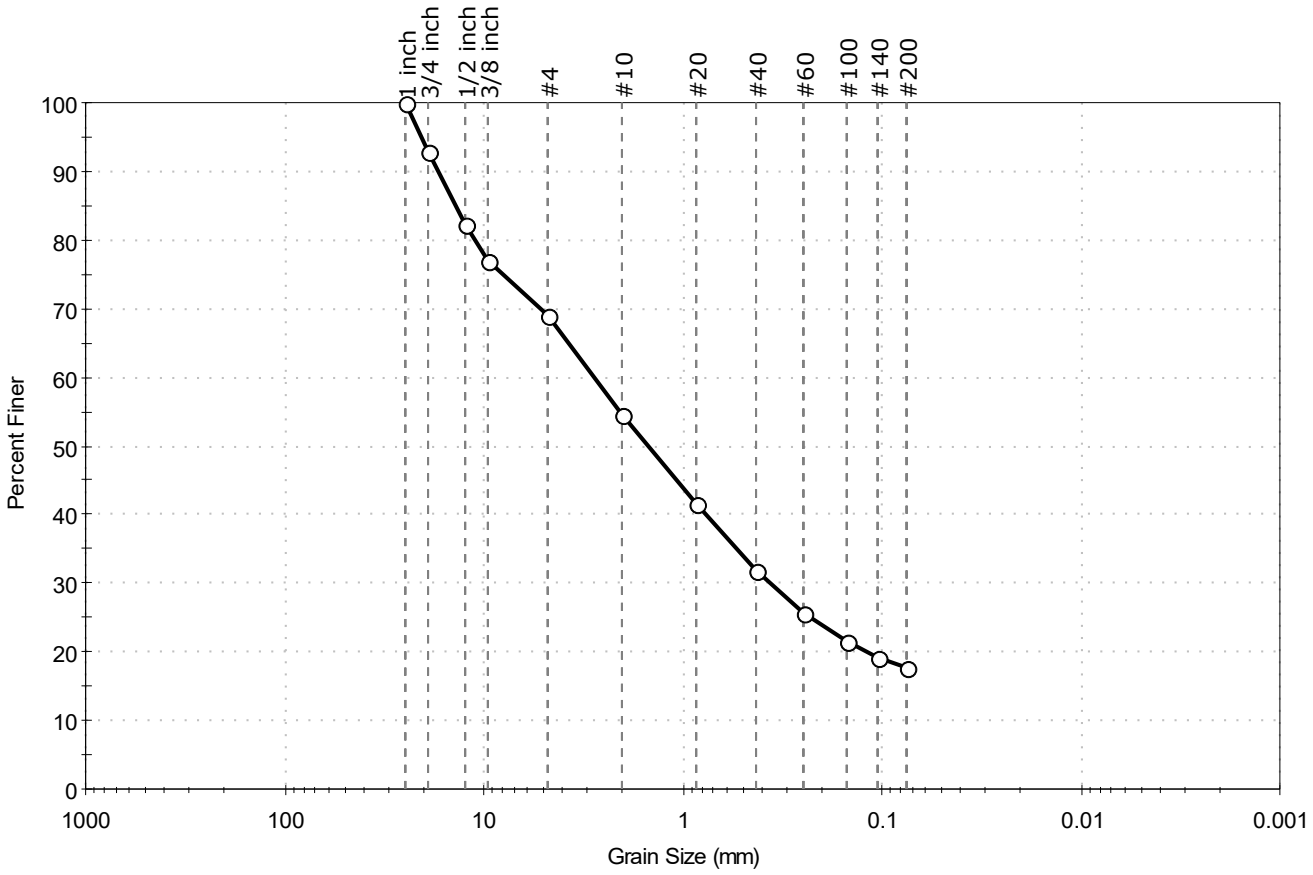
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-212	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741431		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	30.9	51.4	17.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	93		
1/2 inch	12.50	82		
3/8 inch	9.50	77		
#4	4.75	69		
#10	2.00	55		
#20	0.85	42		
#40	0.42	32		
#60	0.25	26		
#100	0.15	21		
#140	0.11	19		
#200	0.075	18		

Coefficients

D_{85} = 13.8847 mm	D_{30} = 0.3623 mm
D_{60} = 2.7543 mm	D_{15} = N/A
D_{50} = 1.4693 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

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

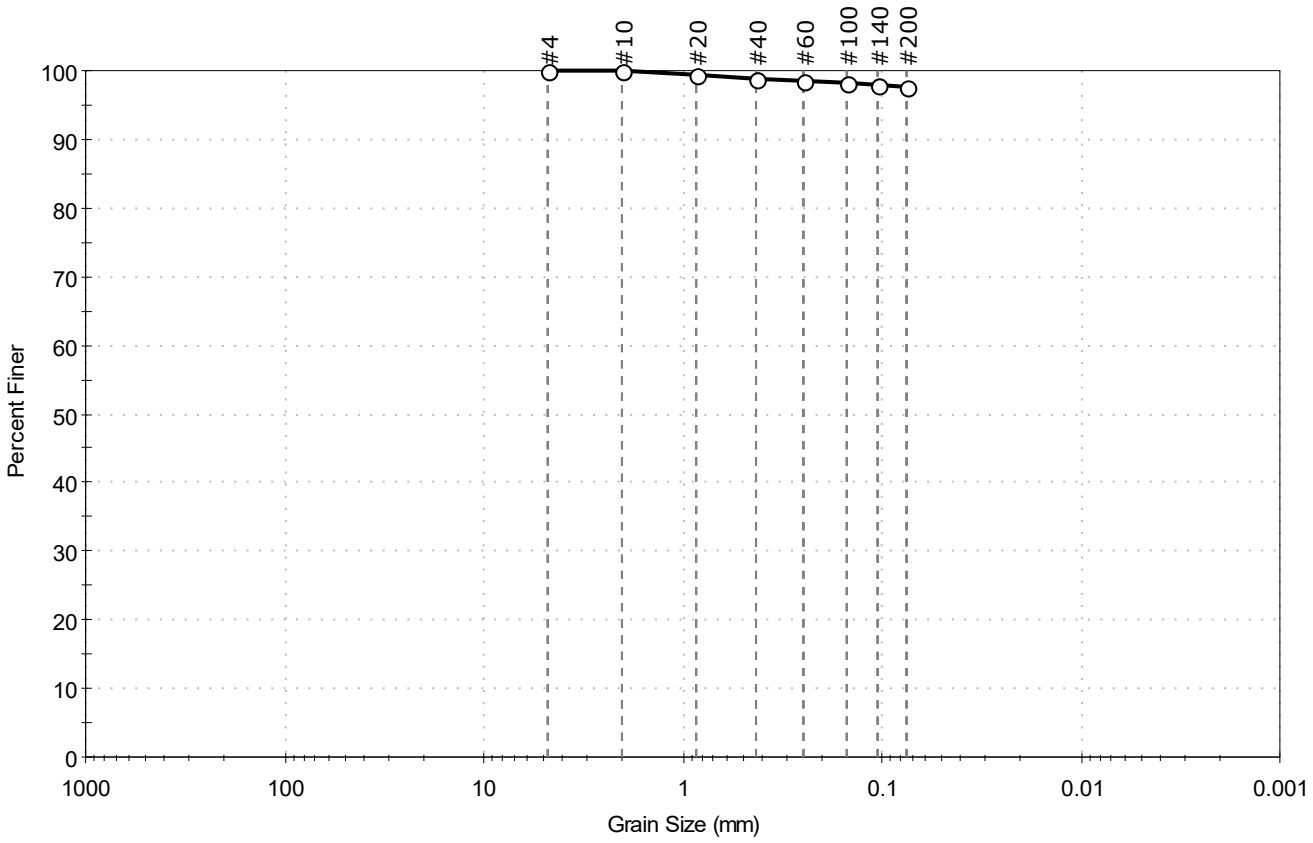
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-212	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-4	Test Date:	11/02/23	Checked By:	ank
Depth :	15-17'	Test Id:	741432		
Test Comment:	---				
Visual Description:	Moist, olive brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	2.3	97.7

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	99		
#60	0.25	99		
#100	0.15	98		
#140	0.11	98		
#200	0.075	98		

Coefficients

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

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (19))

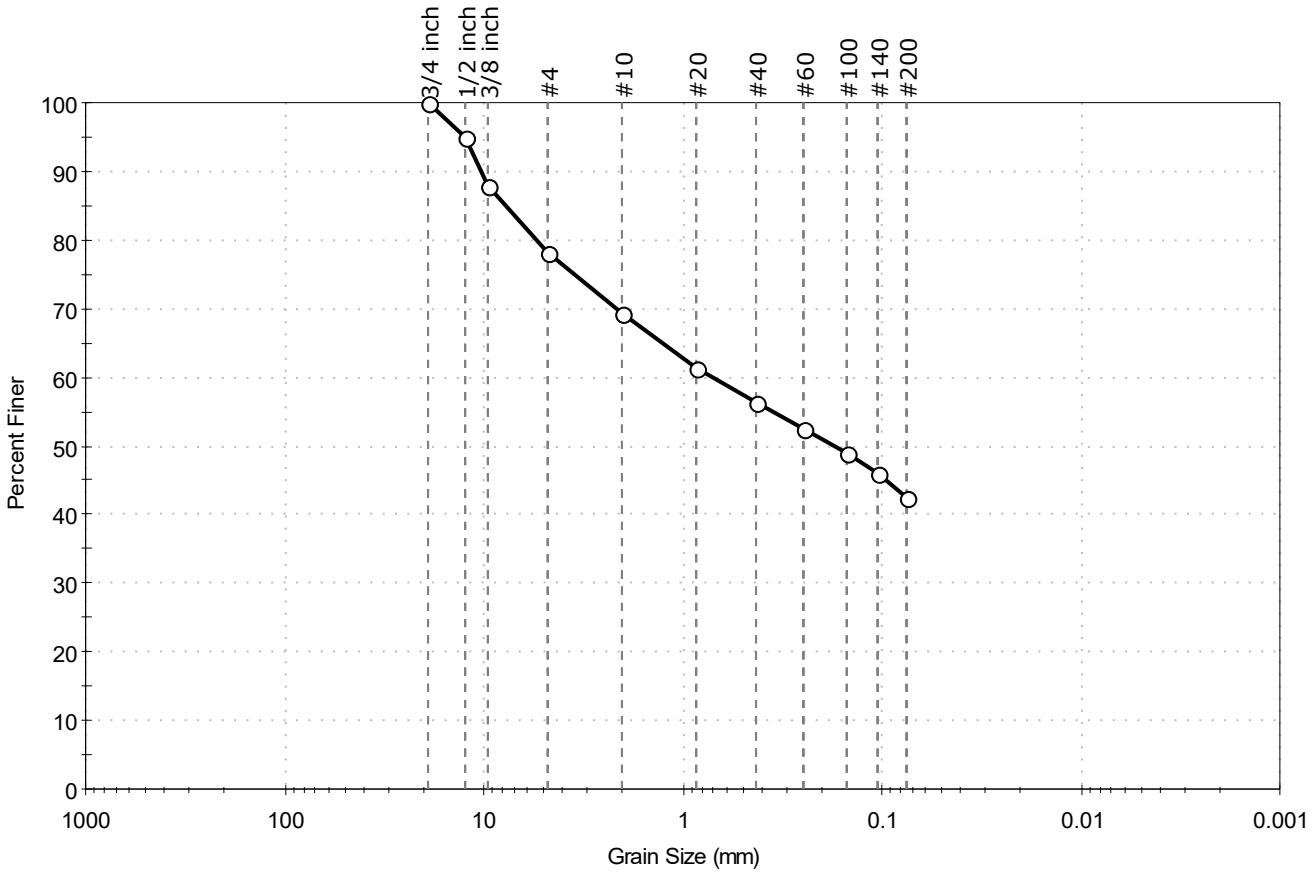
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-212	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-5	Test Date:	11/02/23	Checked By:	ank
Depth :	20-22'	Test Id:	741433		
Test Comment:	---				
Visual Description:	Moist, brown sandy clay with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	21.7	35.9	42.4

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	88		
#4	4.75	78		
#10	2.00	69		
#20	0.85	61		
#40	0.42	56		
#60	0.25	53		
#100	0.15	49		
#140	0.11	46		
#200	0.075	42		

Coefficients

D_{85} = 7.7116 mm D_{30} = N/A

D_{60} = 0.6980 mm D_{15} = N/A

D_{50} = 0.1761 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

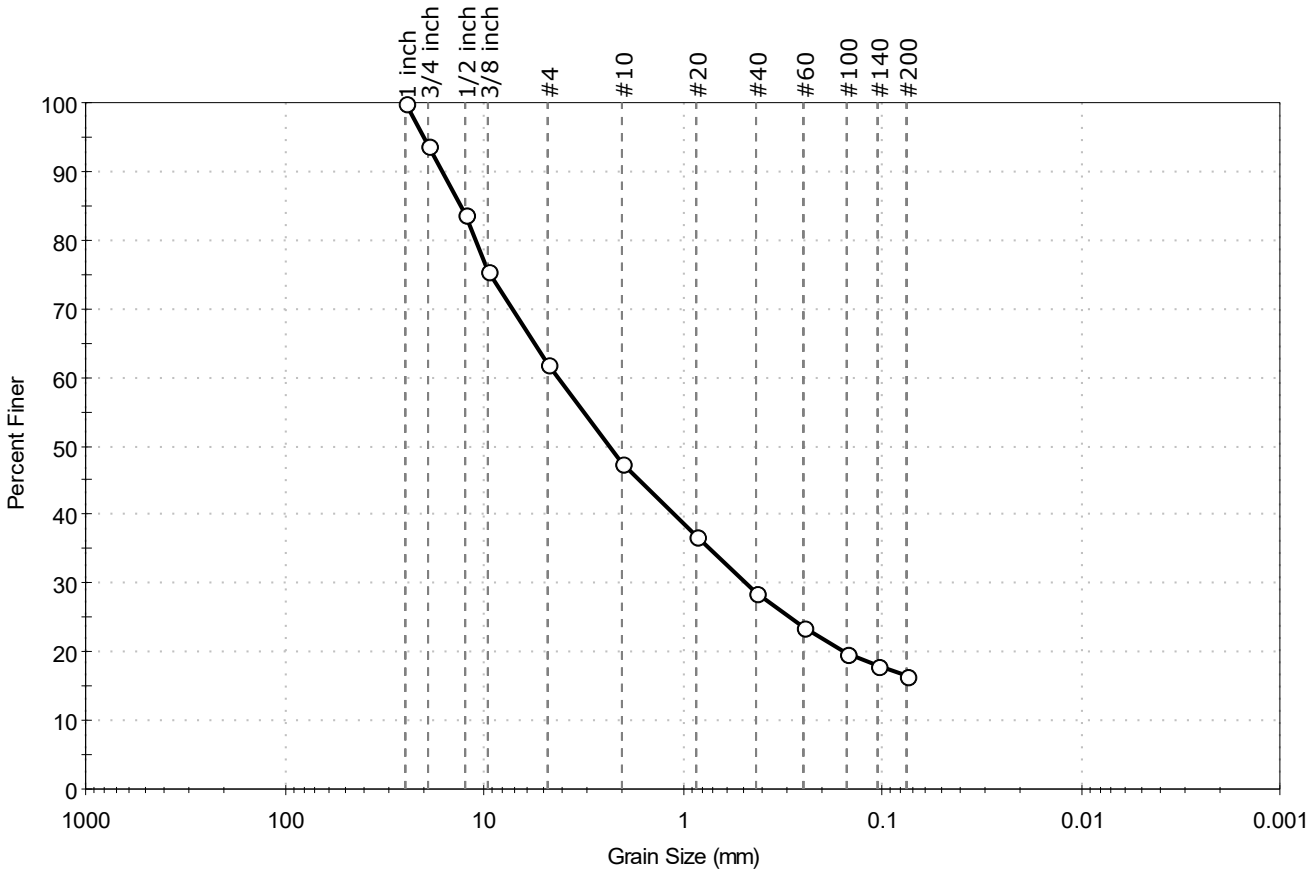
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-213	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741434		
Test Comment:	---				
Visual Description:	Moist, gray silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	38.0	45.6	16.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	94		
1/2 inch	12.50	84		
3/8 inch	9.50	75		
#4	4.75	62		
#10	2.00	48		
#20	0.85	37		
#40	0.42	29		
#60	0.25	24		
#100	0.15	20		
#140	0.11	18		
#200	0.075	16		

Coefficients

D_{85} = 13.2039 mm D_{30} = 0.4743 mm

D_{60} = 4.2227 mm D_{15} = N/A

D_{50} = 2.3208 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

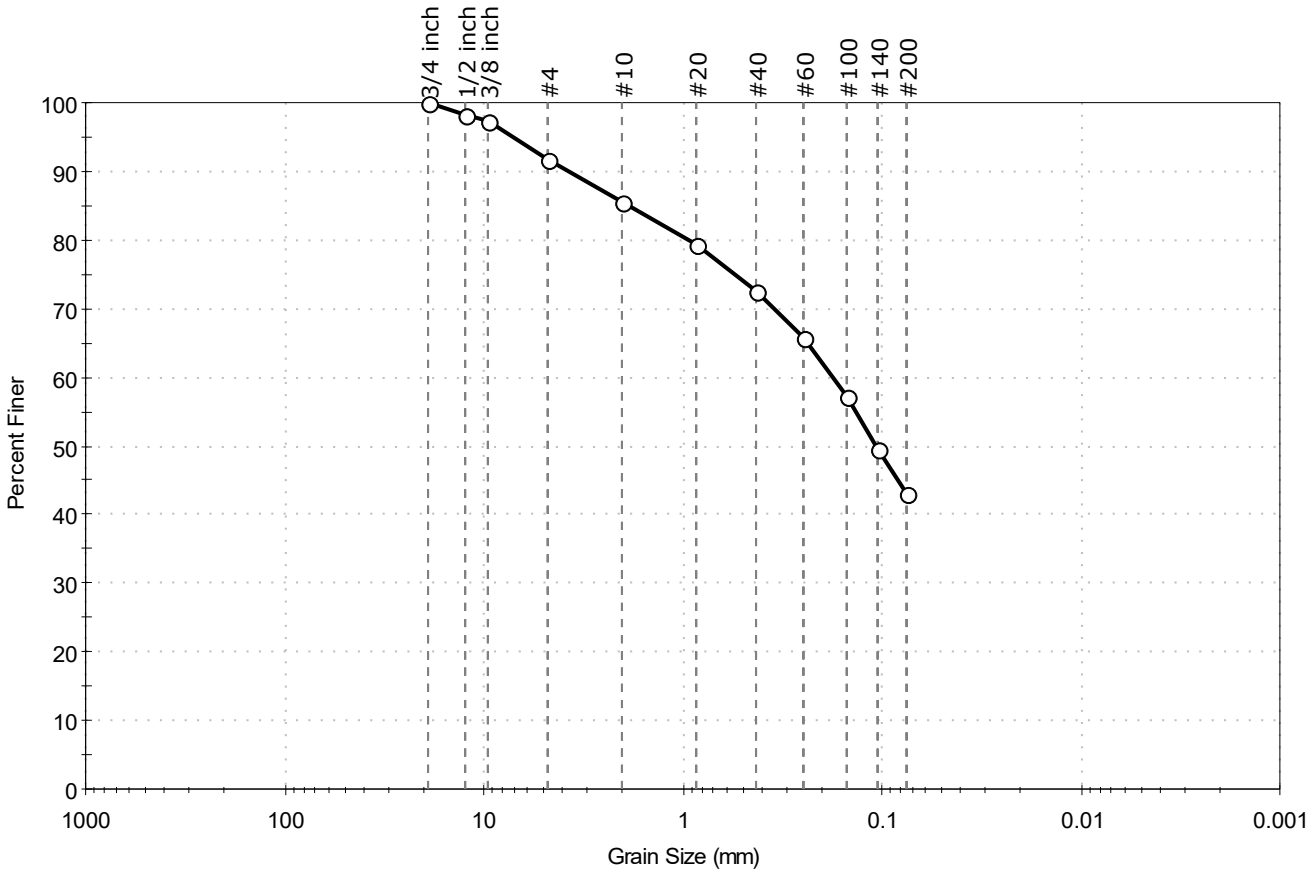
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-214	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3	Test Date:	11/02/23	Checked By:	ank
Depth :	10-12'	Test Id:	741435		
Test Comment:	---				
Visual Description:	Moist, brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	8.1	48.9	43.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/4 inch	19.00	100		
1/2 inch	12.50	98		
3/8 inch	9.50	97		
#4	4.75	92		
#10	2.00	86		
#20	0.85	79		
#40	0.42	73		
#60	0.25	66		
#100	0.15	57		
#140	0.11	50		
#200	0.075	43		

Coefficients

D_{85} = 1.8343 mm D_{30} = N/A

D_{60} = 0.1775 mm D_{15} = N/A

D_{50} = 0.1075 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

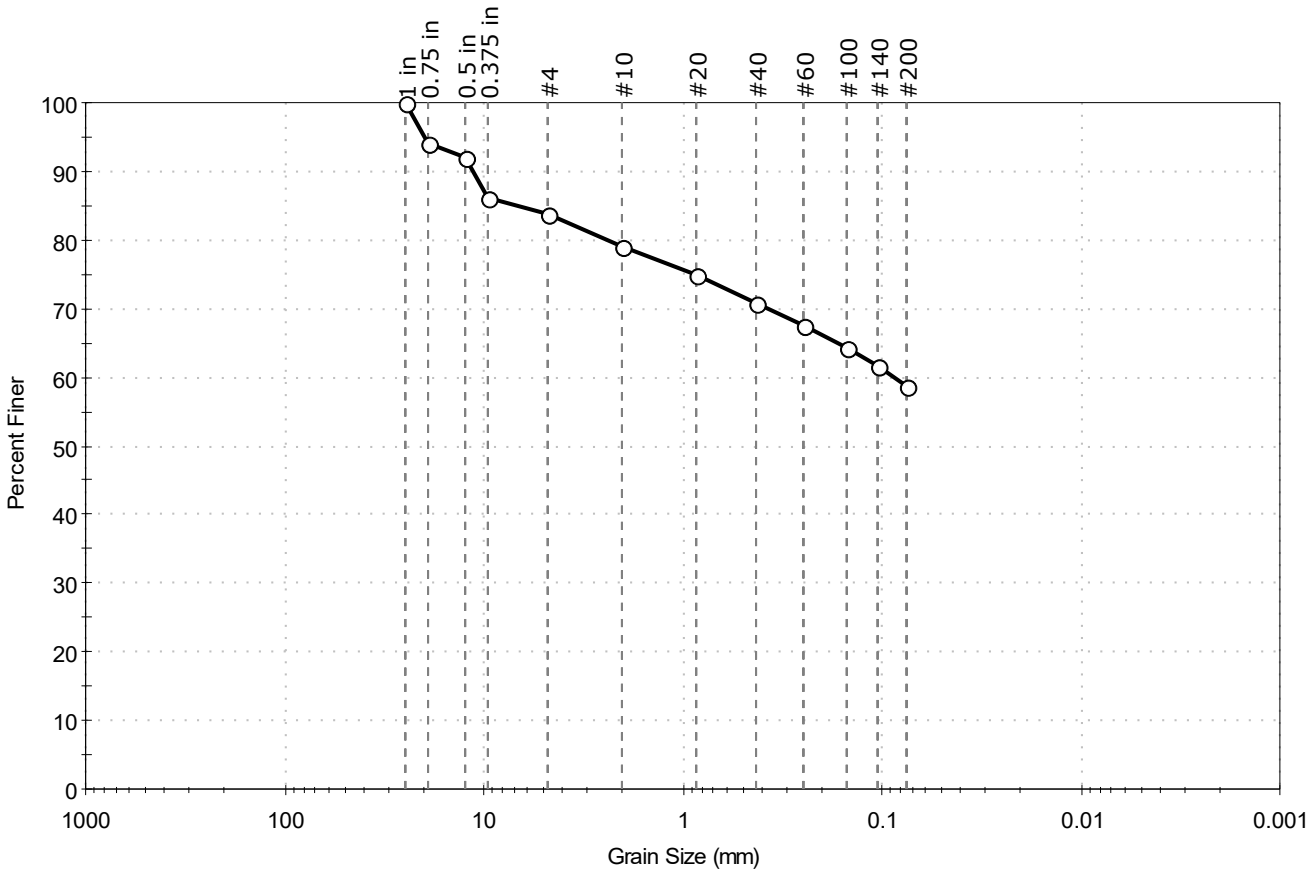
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-214	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-6	Test Date:	11/01/23	Checked By:	ank
Depth :	25-27'	Test Id:	741436		
Test Comment:	---				
Visual Description:	Moist, gray clay with sand and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	16.1	25.2	58.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	92		
0.375 in	9.50	86		
#4	4.75	84		
#10	2.00	79		
#20	0.85	75		
#40	0.42	71		
#60	0.25	68		
#100	0.15	64		
#140	0.11	62		
#200	0.075	59		

Coefficients

D_{85} = 6.5913 mm	D_{30} = N/A
D_{60} = 0.0876 mm	D_{15} = N/A
D_{50} = N/A	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

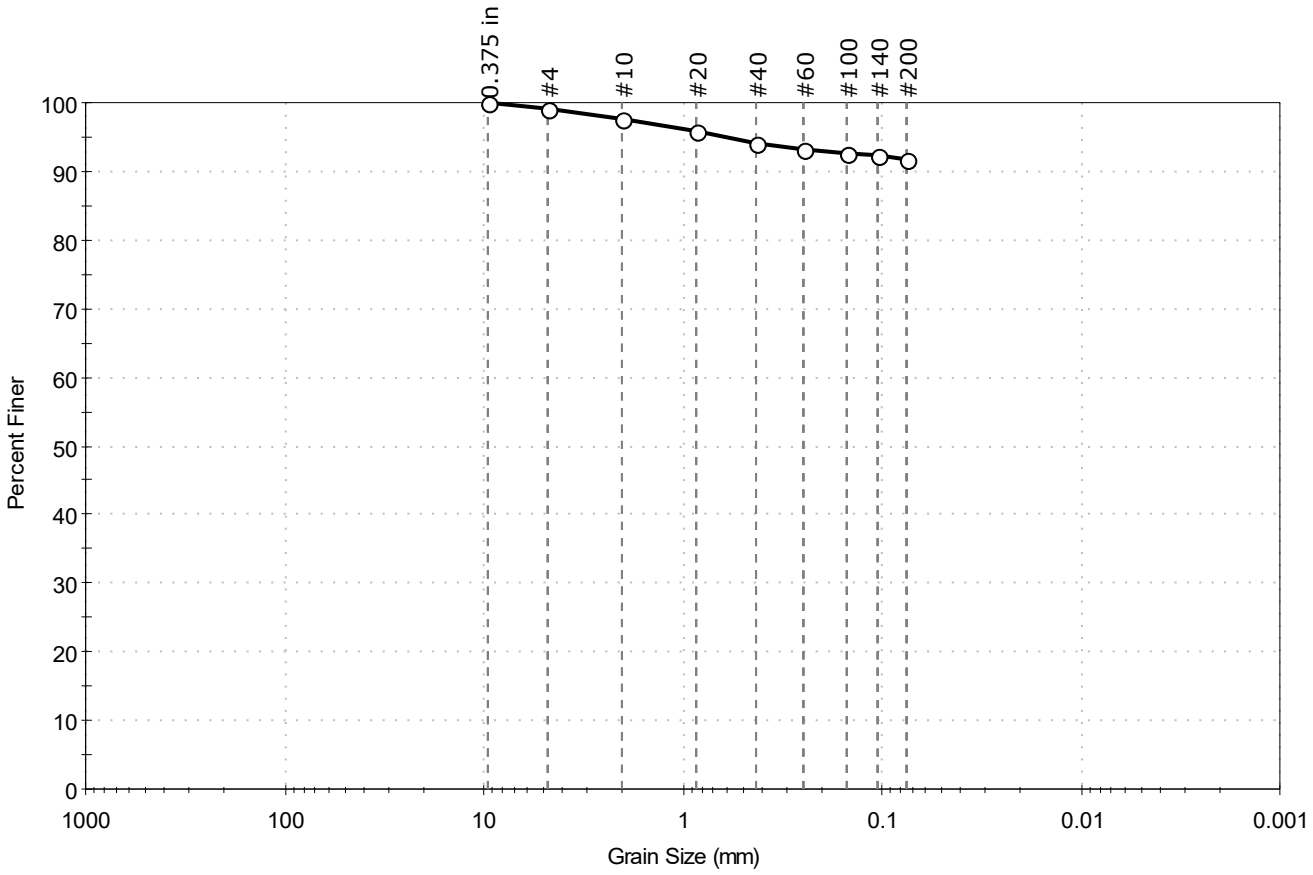
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-215	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/01/23	Checked By:	ank
Depth :	5-7'	Test Id:	741437		
Test Comment:	---				
Visual Description:	Moist, olive brown silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.8	7.4	91.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	96		
#40	0.42	94		
#60	0.25	93		
#100	0.15	93		
#140	0.11	92		
#200	0.075	92		

Coefficients

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

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

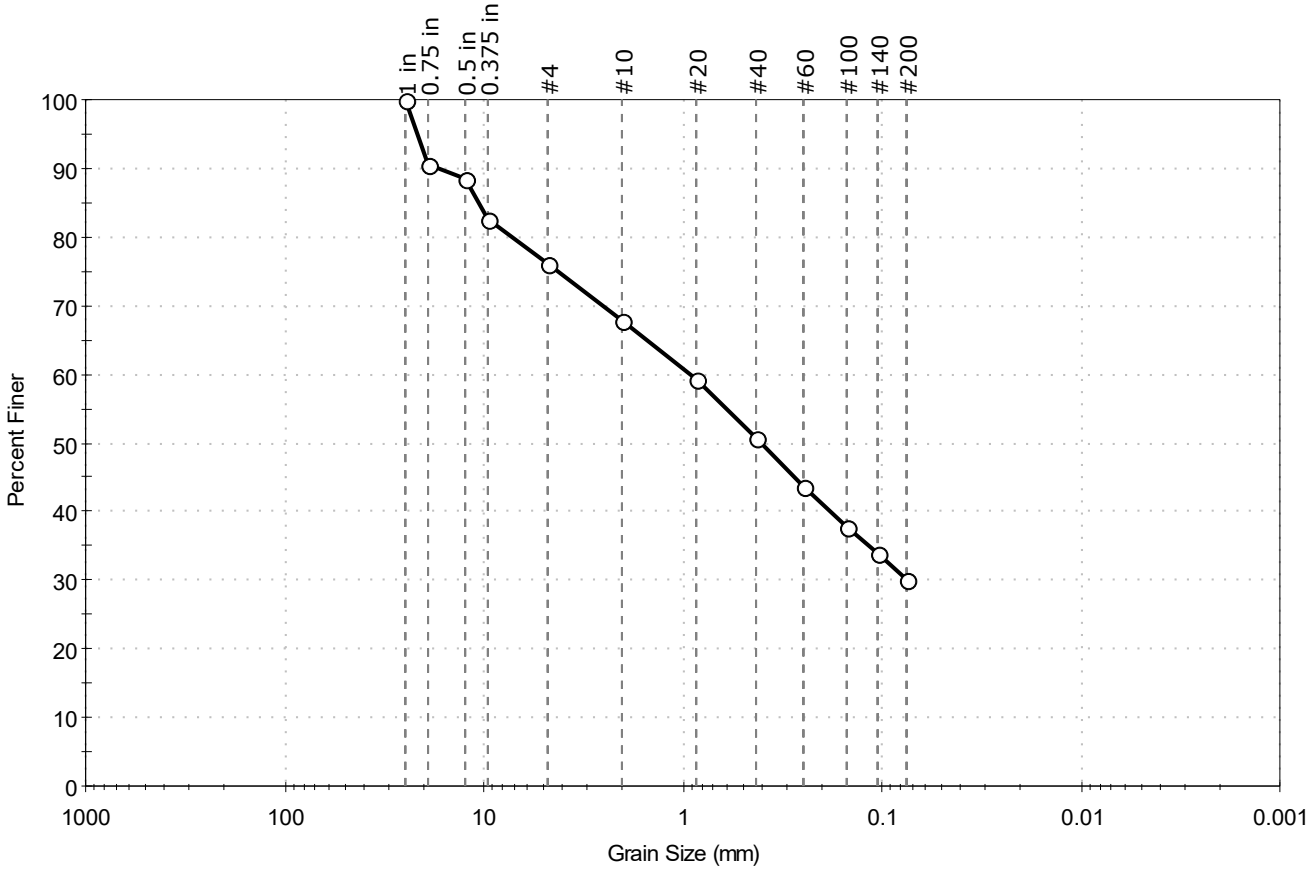
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-216	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-6	Test Date:	11/01/23	Checked By:	ank
Depth :	25-27'	Test Id:	741438		
Test Comment:	---				
Visual Description:	Moist, yellowish brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	23.9	46.0	30.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	89		
0.375 in	9.50	83		
#4	4.75	76		
#10	2.00	68		
#20	0.85	59		
#40	0.42	51		
#60	0.25	44		
#100	0.15	38		
#140	0.11	34		
#200	0.075	30		

Coefficients

D_{85} = 10.6324 mm D_{30} = N/A

D_{60} = 0.9034 mm D_{15} = N/A

D_{50} = 0.4048 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

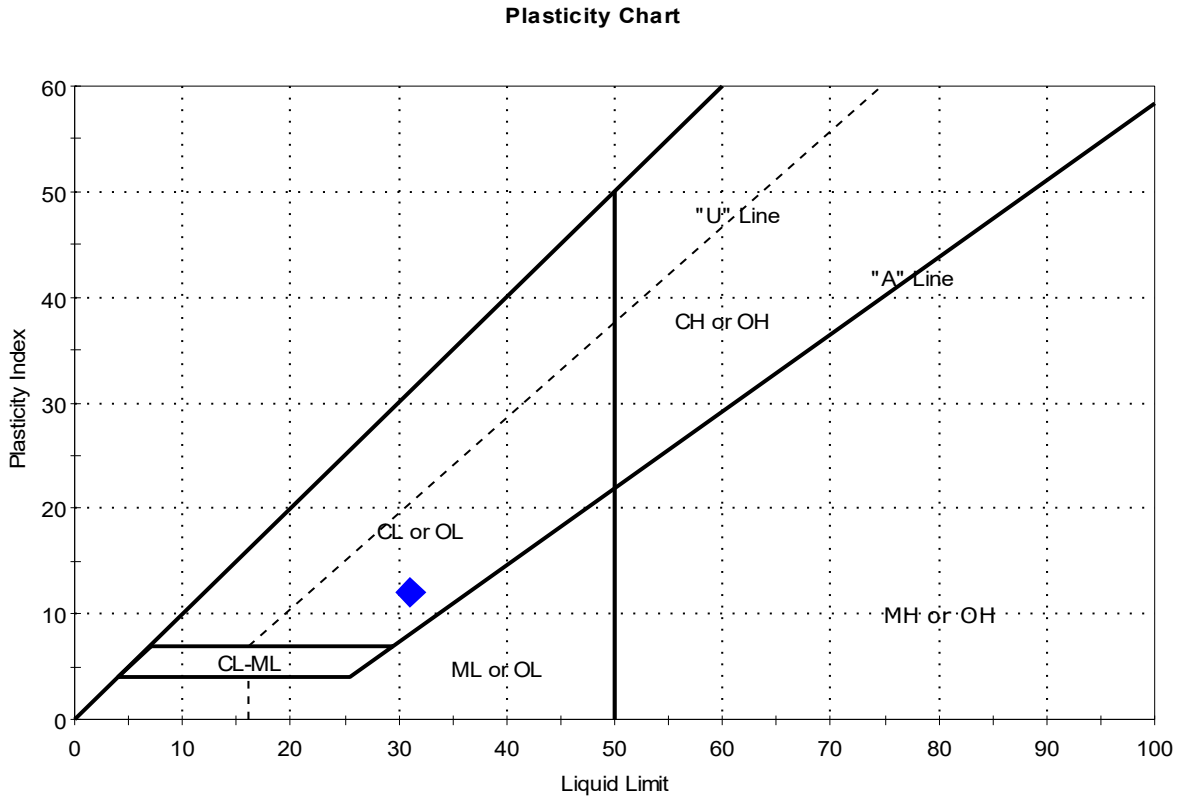
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-203	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	11/03/23	Checked By:	ank
Depth :	5-7'	Test Id:	741386		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

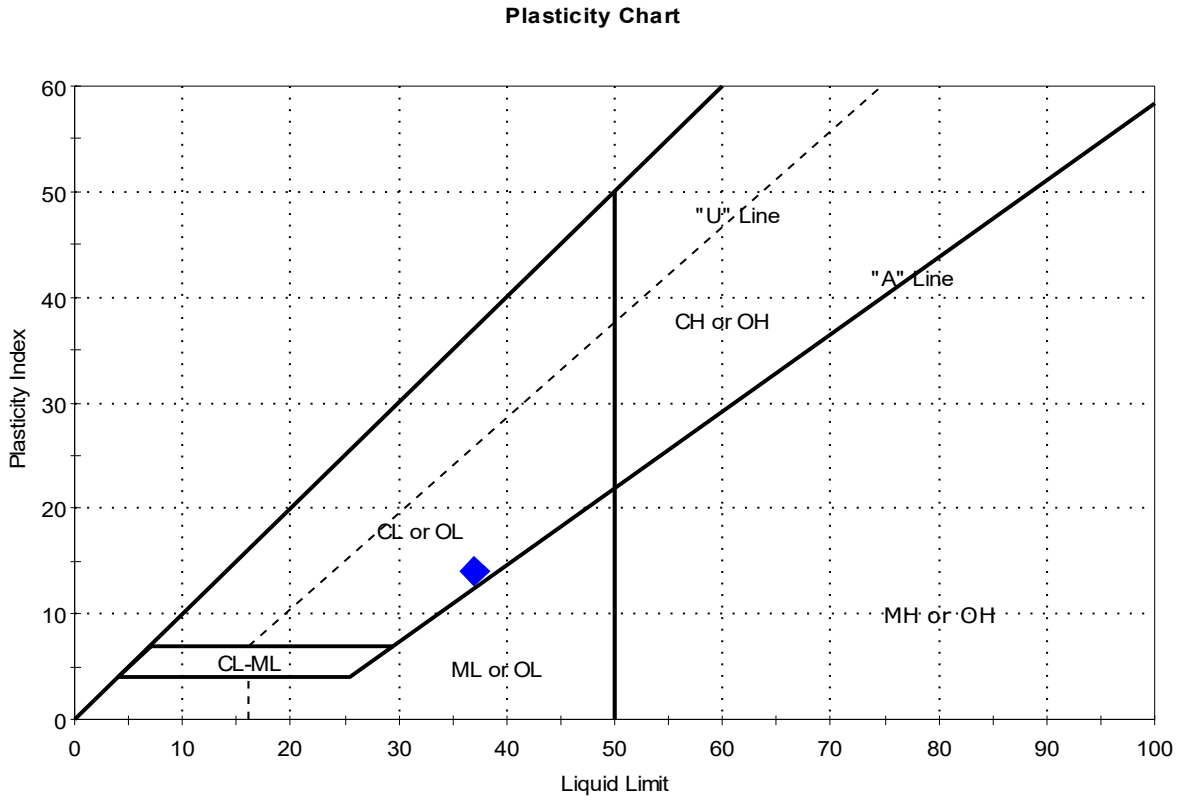


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	24	31	19	12	0.4	Lean CLAY (CL)

Sample Prepared using the WET method
1% Retained on #40 Sieve
Dry Strength: HIGH
Dilatancy: SLOW
Toughness: MEDIUM

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-204	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	10/31/23	Checked By:	ank
Depth :	5-7'	Test Id:	741387		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	24	37	23	14	0.1	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

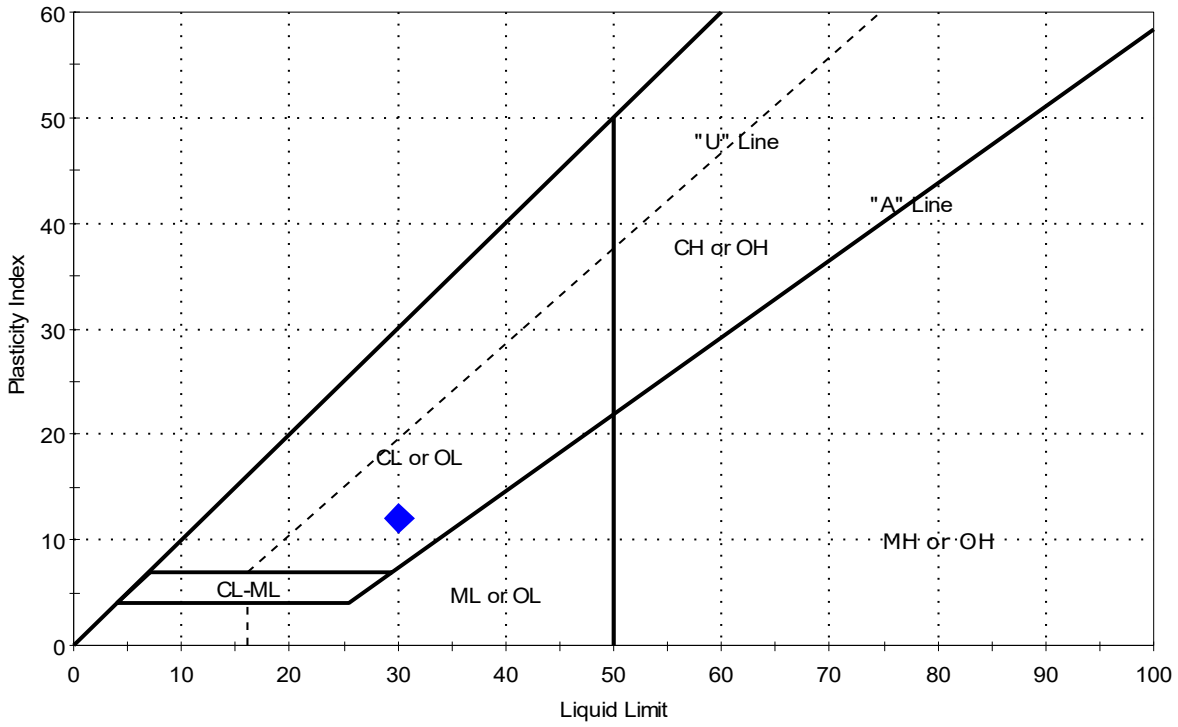
Toughness: LOW

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GeoTesting EXPRESS

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-205	Sample Type:	jar	Tested By:	GA
Sample ID:	S-3	Test Date:	11/03/23	Checked By:	ank
Depth :	10-12'	Test Id:	741388		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-3	B-BHR-20	10-12'	25	30	18	12	0.6	

Sample Prepared using the WET method

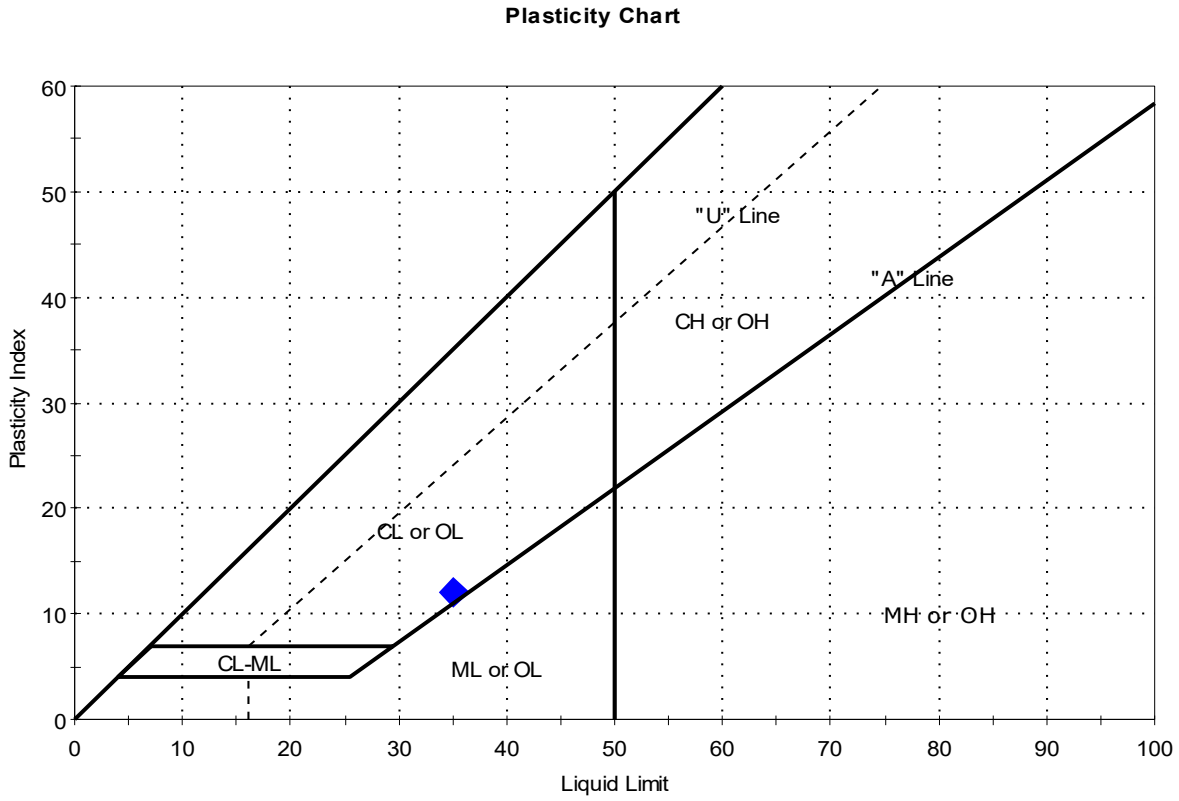
Dry Strength: HIGH

Dilatancy: SLOW

Toughness: MEDIUM

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-206	Sample Type:	jar	Tested By:	GA
Sample ID:	S-3	Test Date:	11/03/23	Checked By:	ank
Depth :	10-12'	Test Id:	741389		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-3	B-BHR-20	10-12'	23	35	23	12	0	

Sample Prepared using the WET method

Dry Strength: HIGH

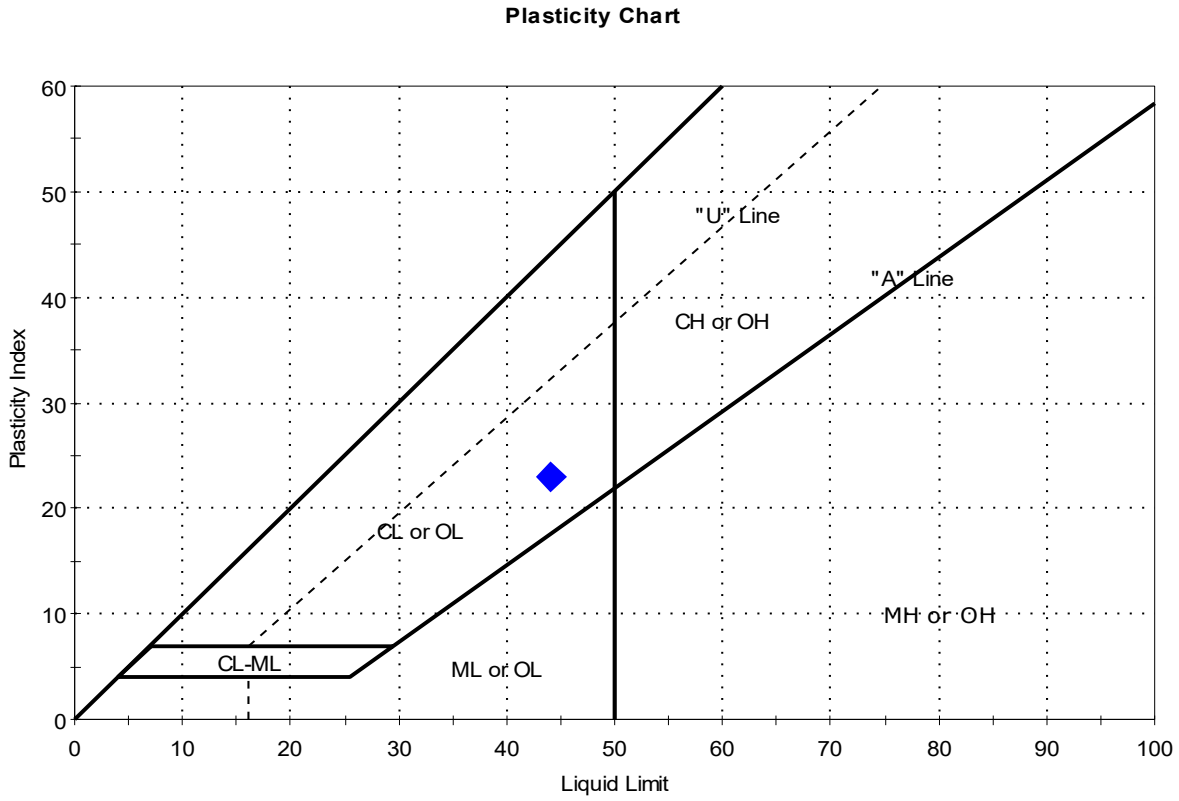
Dilatancy: SLOW

Toughness: MEDIUM

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Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-207	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	11/01/23	Checked By:	ank
Depth :	15-17'	Test Id:	741390		
Test Comment:	---				
Visual Description:	Moist, olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-20	15-17'	28	44	21	23	0.3	

Sample Prepared using the WET method

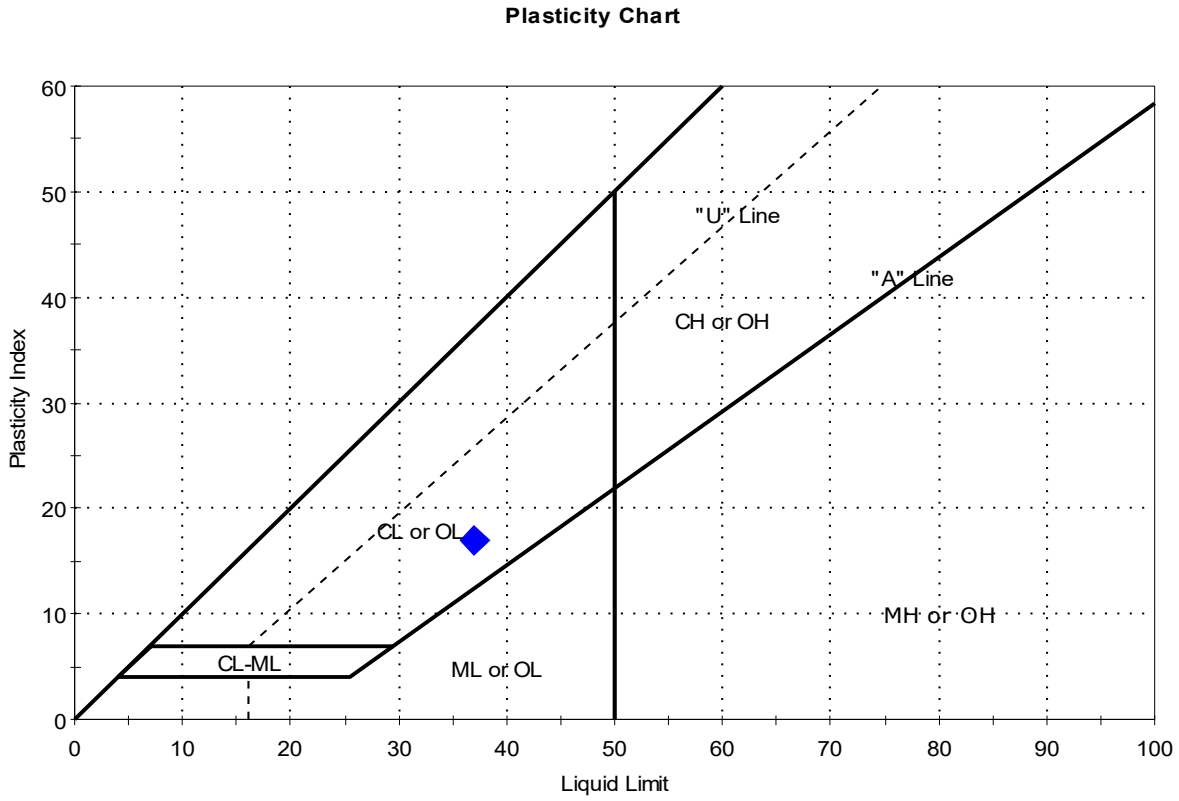
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-208	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	11/01/23	Checked By:	ank
Depth :	5-7'	Test Id:	741391		
Test Comment:	---				
Visual Description:	Moist, brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	24	37	20	17	0.3	

Sample Prepared using the WET method

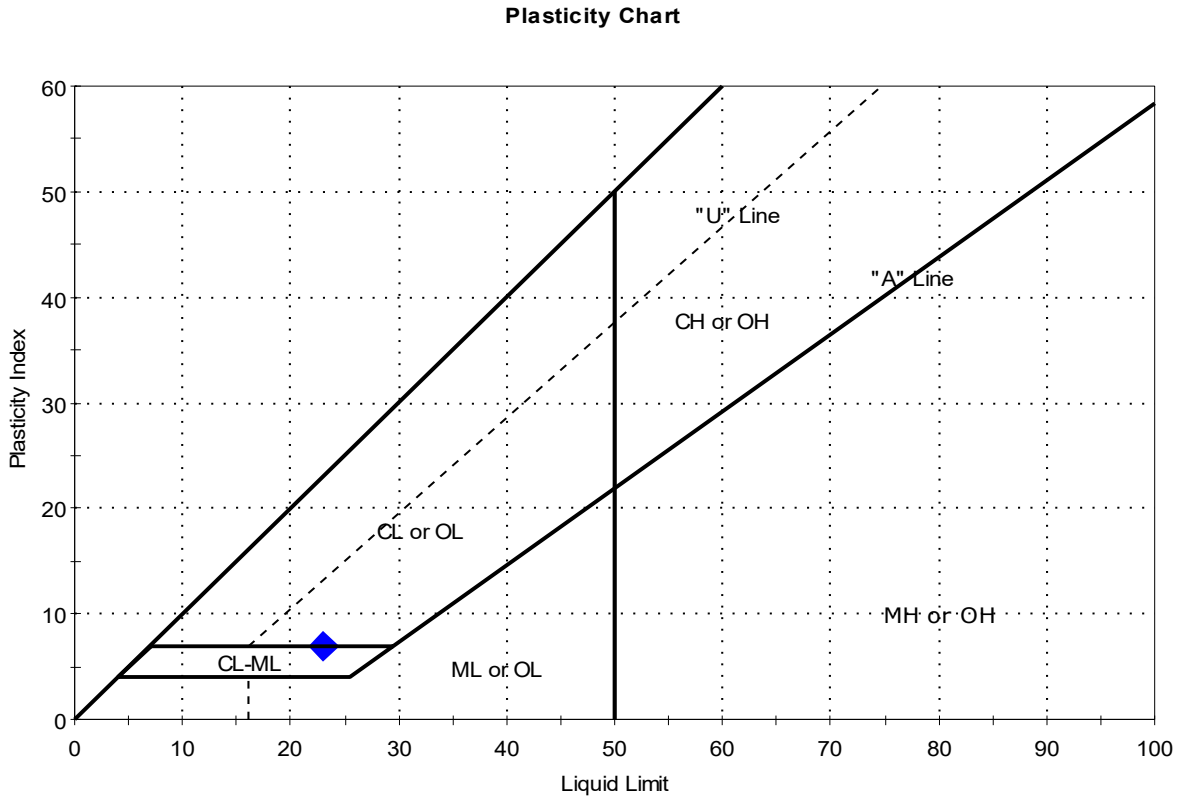
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-208	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	11/01/23	Checked By:	ank
Depth :	15-17'	Test Id:	741392		
Test Comment:	---				
Visual Description:	Wet, gray silty clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-20	15-17'	28	23	16	7	1.7	

Sample Prepared using the WET method

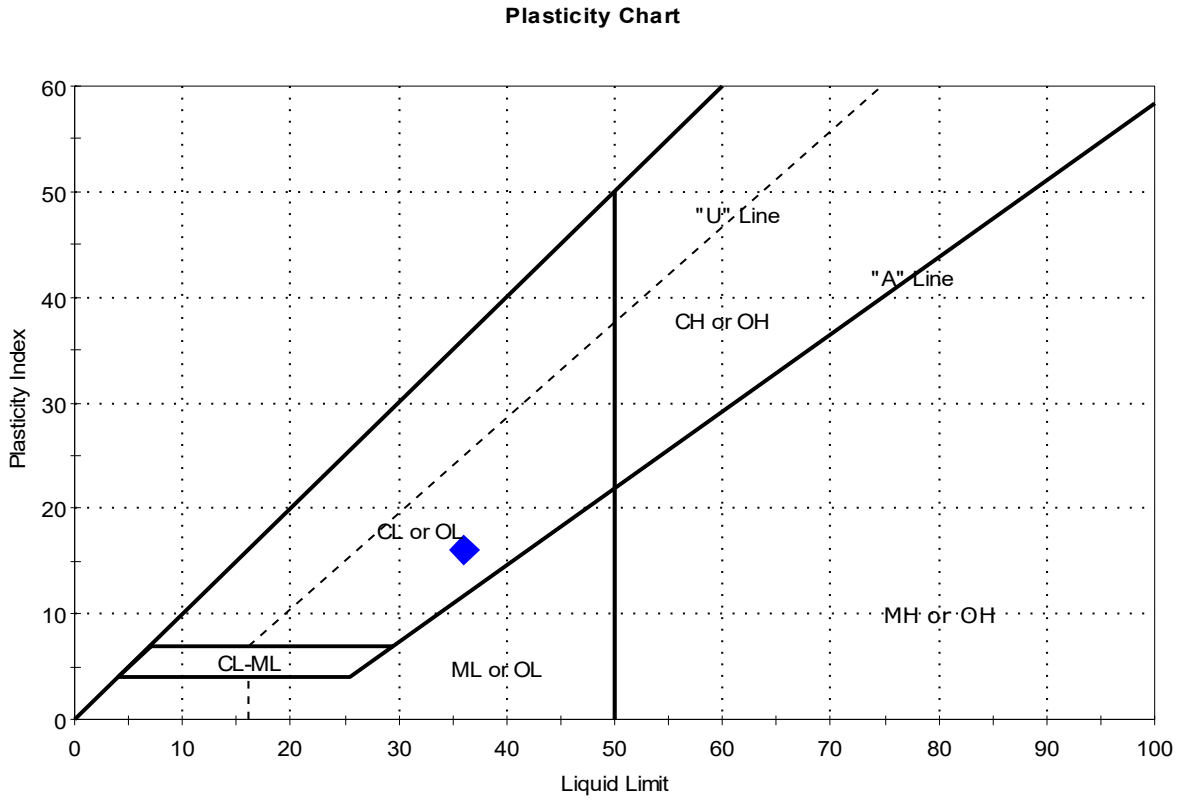
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-210	Sample Type:	jar	Tested By:	cam
Sample ID:	S-6	Test Date:	11/01/23	Checked By:	ank
Depth :	25-27'	Test Id:	741393		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-6	B-BHR-21	25-27'	24	36	20	16	0.2	

Sample Prepared using the WET method

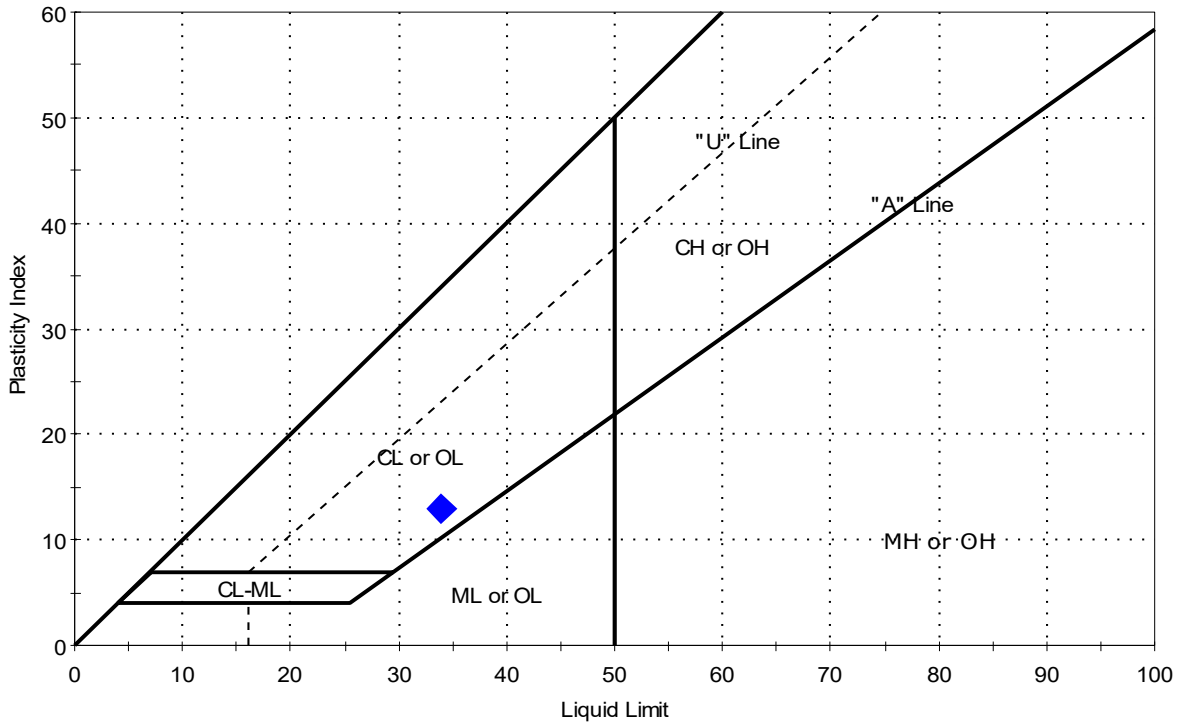
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-211	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	11/01/23	Checked By:	ank
Depth :	15-17'	Test Id:	741394		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-21	15-17'	30	34	21	13	0.7	

Sample Prepared using the WET method

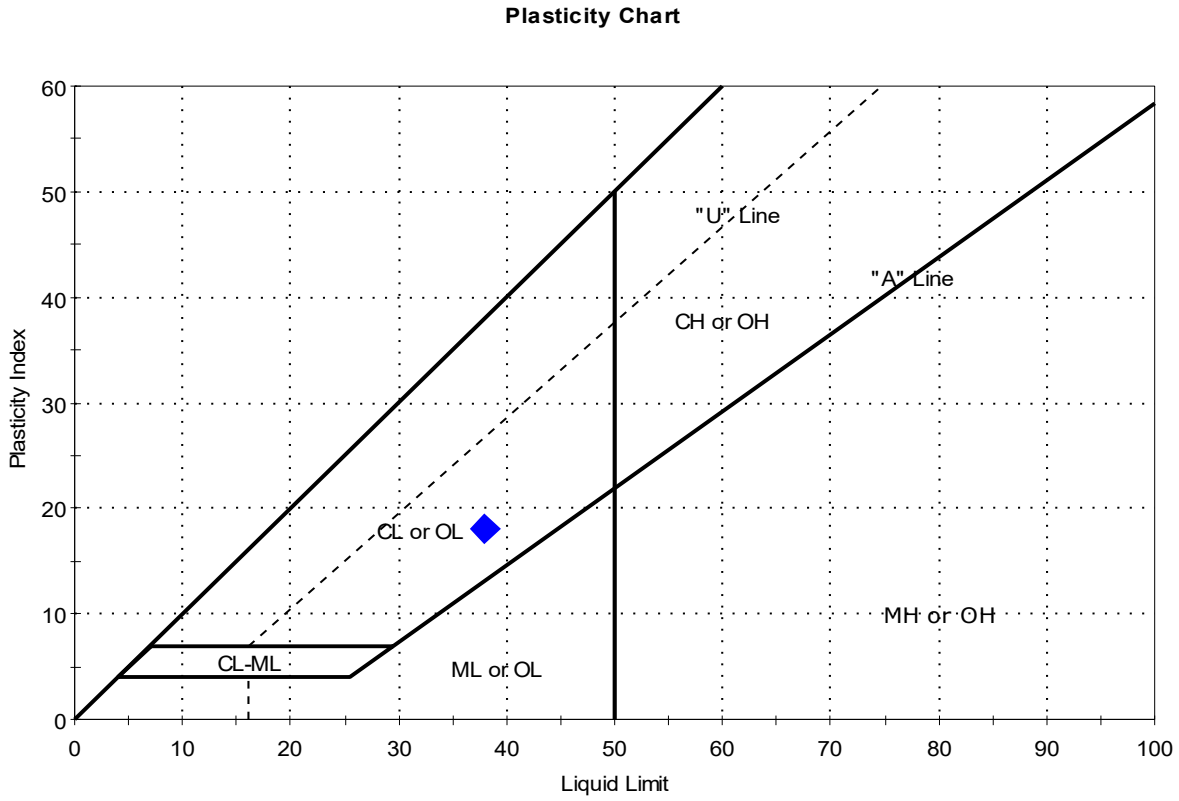
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-212	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	11/01/23	Checked By:	ank
Depth :	15-17'	Test Id:	741395		
Test Comment:	---				
Visual Description:	Moist, olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

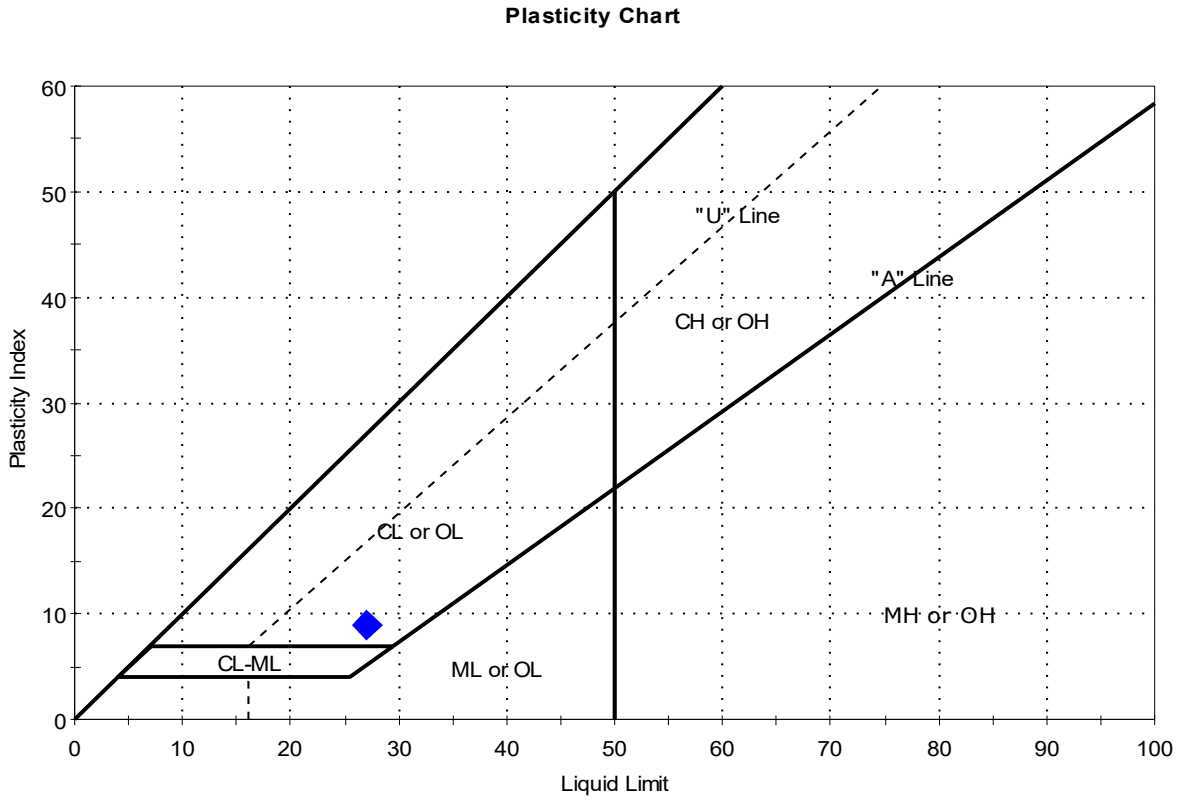


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-21	15-17'	25	38	20	18	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
1% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-212	Sample Type:	jar	Tested By:	cam
Sample ID:	S-3	Test Date:	10/31/23	Checked By:	ank
Depth :	10-12'	Test Id:	741396		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-3	B-BHR-21	10-12'	21	27	18	9	0.4	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

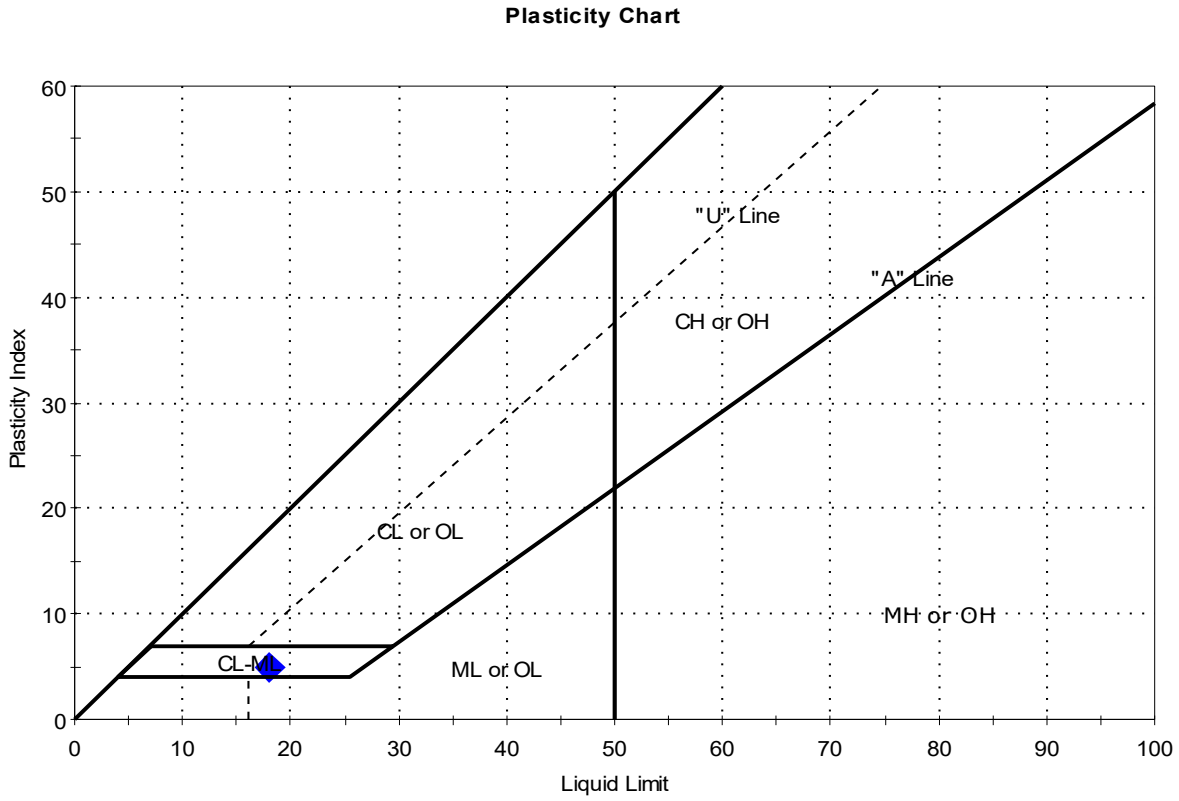
Dilatancy: SLOW

Toughness: LOW

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Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-213	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	10/31/23	Checked By:	ank
Depth :	15-17'	Test Id:	741397		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-21	15-17'	14	18	13	5	0.2	

Sample Prepared using the WET method

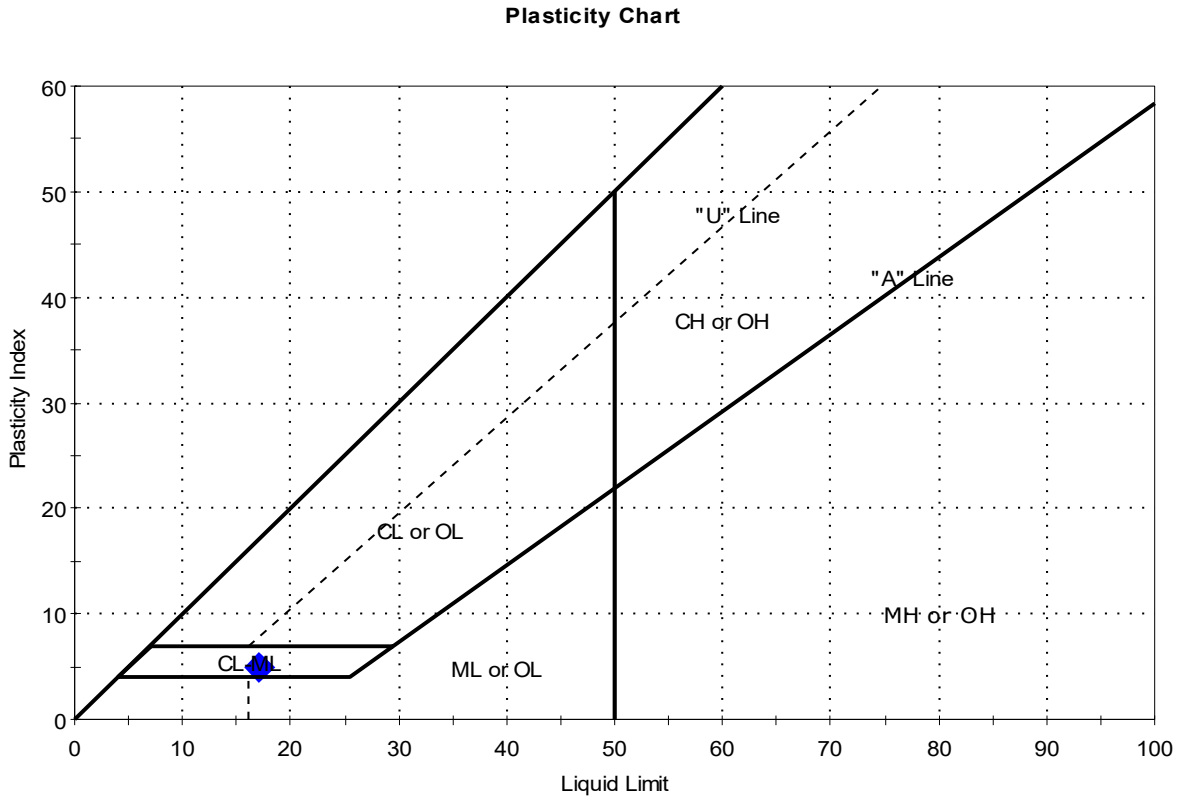
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-215	Sample Type:	jar	Tested By:	cam
Sample ID:	S-5	Test Date:	10/31/23	Checked By:	ank
Depth :	20-22'	Test Id:	741398		
Test Comment:	---				
Visual Description:	Moixst, gray clay with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-5	B-BHR-21	20-22'	10	17	12	5	-0.4	

Sample Prepared using the WET method

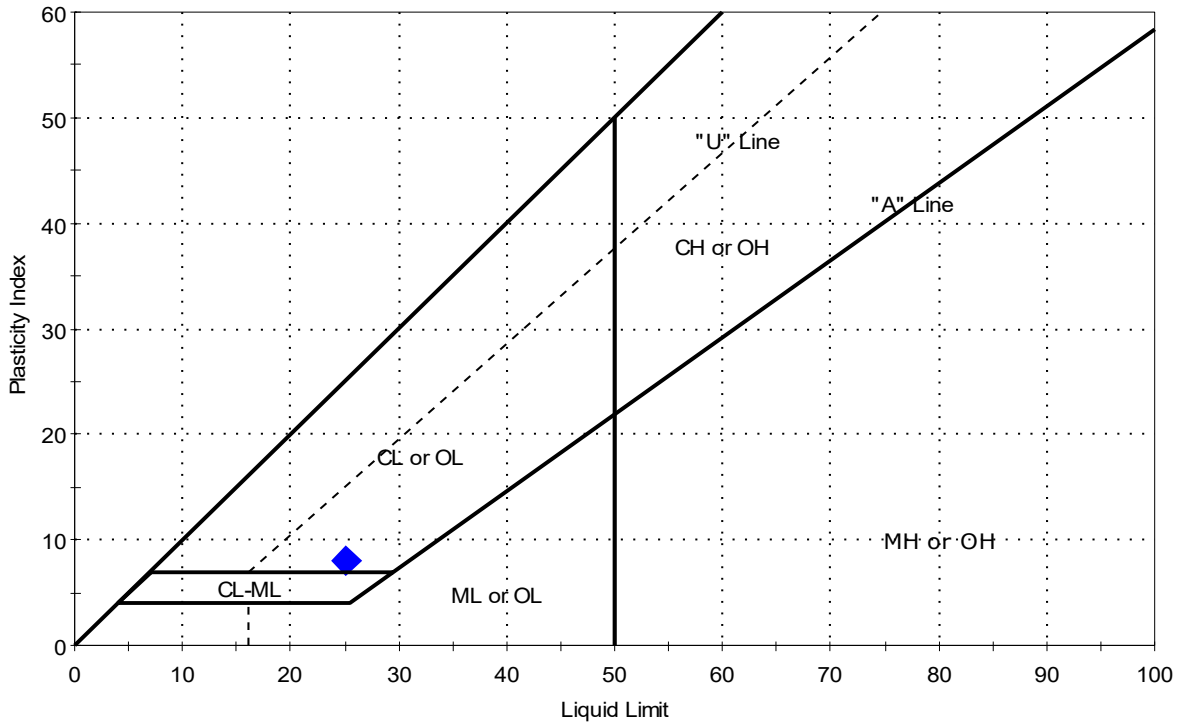
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-317984
Boring ID:	HB-BHR-216	Sample Type:	jar	Tested By:	cam
Sample ID:	S-3	Test Date:	10/31/23	Checked By:	ank
Depth :	10-12'	Test Id:	741399		
Test Comment:	---				
Visual Description:	Moist, dark brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-3	B-BHR-21	10-12'	16	25	17	8	-0.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

APPENDIX D

Global Stability Calculations

CALCULATIONS

Date:	6/16/2025	**Made by:**	TJ/LMP/JKW
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Global Stability Analysis for Hogan Rd Highway Interchange	**Reviewed by:**	CCB
Project:	MaineDOT WIN 018595.10 Hogan Road Diverging Diamond Interchange.		

OBJECTIVE

Calculate global factor of safety for the embankments at Hogan Road EB Station 106+00 (Hogan Road EB Embankment Fill) and 119+00 (Spur C Embankment Fill), Ramp B Station 808+00 (Embankment Fill), Ramp C Station 1007+50 (Embankment Fill). The slope stability evaluations were made for surficial failure surfaces with a minimum depth of 1 ft, and deep seated surfaces with a minimum depth of 10 ft.

REFERENCES

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

2. VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated March 10, 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, Geotechincal Design Report).

5. Rocscience Inc. Slide2 software package, version 9.038, build date February 28, 2025.

6. United States Department of the Interior Geological Survey. 1963. Porosity and Bulk Density of Sedimentary Rocks. Geological Survey Bulletin 1144-E.

7. WSP, Material properties table named “Lab Field Data Summary Plots V4 31404817.000.xlsx"

8. MaineDOT Bridge Design Guide, prepared by Guertin Elkerton & Associates, August 2003, Updated 2018

9. Nicks, J. E., Ghaaowd, I. I., & Adams, M. T. 2024. Strength-Deformation Characteristics of Two Different Foamed Glass Aggregates under a Static Design Load. Geotechnical Testing Journal, 47(1).

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 ft equivalent load of soil (Reference 1, Table 3.11.6.4-1): 2 ft x 135 pcf (Aggregate Subbase - Gravel Borrow) = 270 psf.

2. A minimum target factor of safety (FS) of FS ≥ 1.3 for static loading is recommended for potential failure surfaces per AASHTO LRFD and MaineDOT standards 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. Deep failure surfaces evaluation include the global minimum FS with surfaces deeper than 10 feet.

5. The existing stratigraphy and water table surfaces are estimated from samples and groundwater measurements encountered during drilling and interpreted in Reference 4.

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. The Topsoil will be stripped and removed from the site prior to the placement of any construction material. Thus, if topsoil was encountered in the statigraphy, it was replaced by fill.

9. Granular borrow will be used as the proposed slope fill to cover on the front slope of the existing fill.

10. The clay will be divided into two layers based on the strenght: A medium stiff clay and a stiff clay layer.

CALCULATIONS

Date:	6/16/2025	**Made by:**	TJ/LMP/JKW
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Global Stability Analysis for Hogan Rd Highway Interchange	**Reviewed by:**	CCB
Project:	MaineDOT WIN 018595.10 Hogan Road Diverging Diamond Interchange.		

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.

a. The cohesion parameters for the stiff to very stiff clay and silt layers were based off the undrained shear strength values from correlation from the SPT results and plasticity index values obtained from lab testing the field vane shear testing summarized in Tab "Hwy N60" of Ref. 7. The cohesion parameters for soft to medium soft clay layers were based off the field vane test results as summarized in Tab "Hwy N60" of Ref. 7.

b. The friction angle parameters for the existing fill, silt, sand & gravel, sand, 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 = $27.1 + 0.3*N_{60} - 0.00054*(N_{60})^2$

c. The unit weight parameters for the existing fill, silt, sand & gravel, sand, and glacial till layers were selected based on local engineering experience. The unit weight parameter for the clay layer was selected as the 30th percentile from the consolidation lab testing results (Ref. 3).

d. 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. 8.

e. The unit weight parameter for the metawacke bedrock encountered at the site was selected as 158 pcf based on Ref. 6, Table 1.

f. 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.

g. The silt material in the shallow subsurface is located both above and below the water table. This material is field characterized as silt, and few samples are characterized from the lab as low plasticity clay (CL). Thus, we modeled the silt layer as having either drained or undrained properties for comparison of stability.

h. The friction angle and unit weight parameters for the Foamed Glass Aggregate were selected based on engineering judgement and Ref. 9.

Table 1: In Situ Soil Properties (Ref. 7)

Station Number	Borings Present	Material Name	Unit Weight (pcf)	Strength Type	Cohesion[1] (psf)	Friction Angle[2] (°)
106+00	HB-101, HB-102, HB-206, and HB-207	Clay (Stiff)	122	Undrained	1343	0
		Silt (Drained)	125	Mohr-Coulomb	0	32
		Silt (Undrained)	125	Undrained	1000	0
		Clay (Soft to Medium Stiff)	121	Undrained	913	0
		Glacial Till	130	Mohr-Coulomb	0	38
119+00	HB-109, HB-110, HB-211, and HB-212	Sand and Gravel	130	Mohr-Coulomb	0	38
		Clay (Stiff)	122	Undrained	1343	0
		Clay (Soft to Medium Stiff)	121	Undrained	913	0
		Silt (Drained)	125	Mohr-Coulomb	0	32
		Silt (Undrained)	125	Undrained	1000	0
		Glacial Till	130	Mohr-Coulomb	0	38
808+00	HB-112, HB-113, HB-114, and HB-217	Silt (Drained)	125	Mohr-Coulomb	0	32
		Silt (Undrained)	125	Undrained	1000	0
		Sand and Gravel	130	Mohr-Coulomb	0	38
		Silty Clay (Stiff)	122	Undrained	1343	0

CALCULATIONS

Date: 6/16/2025 **Made by:** TJ/LMP/JKW
Project No.: 31404817.003 **Checked by:** LMP
Subject: Global Stability Analysis for Hogan Rd Highway Interchange **Reviewed by:** CCB
Project: MaineDOT WIN 018595.10 Hogan Road Diverging Diamond Interchange.

Table 1 Continued						
808+00	HB-112, HB-113, HB-114, and HB-217	Glacial Till	130	Mohr-Coulomb	0	38
1007+50	HB-108, HB-109, and HB-210	Clay (Very Stiff)	122	Undrained	1343	0
		Sand and Gravel	130	Mohr-Coulomb	0	38
		Glacial Till	130	Mohr-Coulomb	0	38

Note: [1]Measurement from field vane shear test for soft to medium stiff clay or correlated from N60 using Stroud (1974) and Stroud (1989) from FHWA Geotechnical Engineering Circular No. 5, Evaluation of Soil and Rock Properties, Table 33, April 2002 for use in an undrained analysis of the stiff to very stiff clay and silt below the water table. [2]Correlated from N60 using Peck, Hanson, & Thornburn (1974) from Wolff (1989) for use in a drained analysis of the soil above and below the water table.

The attached Figure 1 shows the material properties and coloring for all the materials used in the Slide2 models.

Table 2: In Situ Rock Material

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle (°)
Bedrock	158	Infinite Strength	-	-

Table 3: Fill Materials

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)	Friction Angle[1] (°)
Existing Fill	125	Mohr-Coulomb	0	34
Granular Borrow	125	Mohr-Coulomb	0	32
Aggregate Subbase (Gravel Borrow)	135	Mohr-Coulomb	0	36
Foamed Glass Aggregate (FGA)	18	Mohr-Coulomb	0	35.5

Note: [1] Friction 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. 8). Friction angles for the FGA fill material were selected based of values provided in Reference 9.

Table 4: Proposed Geometry and Stratigraphy

Station Number	Proposed Structure	Attachment
106+00	Hogan Road EB Embankment Fill	A.0
119+00	Spur C Embankment Fill	B.0
808+00	Ramp B Embankment Fill	C.0
1007+50	Ramp C Embankment Fill	D.0

CALCULATIONS

Date:	6/16/2025	**Made by:**	TJ/LMP/JKW
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Global Stability Analysis for Hogan Rd Highway Interchange	**Reviewed by:**	CCB
Project:	MaineDOT WIN 018595.10 Hogan Road Diverging Diamond Interchange.		

Station Selection: These stations 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 attachments A.0 through D.2 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 either the half way point between the two split spoon samples with differentiating soil types or 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.

Model Extents: The Slide2 models were extended beyond the limits shown in the stratigraphy cross-sections, assuming the layers where horizontal beyond the limits of the topography shown in the CAD (Ref. 2).

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 scenarios at four different locations: Hogan Road EB Station 106+00 (Hogan Road EB Embankment Fill) and 119+00 (Hogan Road EB Embankment Fill), Ramp B Station 808+00 (Embankment Fill), Ramp C Station 1007+50 (Embankment Fill).

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	Silt Condition	Analysis Method	Factor of Safety	Figure # (Attach. 1)
Hogan Rd EB Embankment Fill	106+00	Undrained	Circular - Spencer[1]	1.26	-
			Non Circular - Spencer[1]	1.26	A.1
		Drained	Circular - Spencer[1]	1.26	-
			Non Circular - Spencer[1]	1.26	A.2
Hogan Road EB Embankment Fill	119+00	Undrained	Circular - Spencer[1]	1.25	-
			Non Circular - Spencer[1]	1.25	B.1
		Drained	Circular - Spencer[1]	1.25	-
			Non Circular - Spencer[1]	1.25	B.2
Ramp B Embankment Fill	808+00 (2:1 slope)	Undrained	Circular - Spencer[1]	1.25	-
			Non Circular - Spencer[1]	1.25	C.1
		Drained	Circular - Spencer[1]	1.25	-
			Non Circular - Spencer[1]	1.25	C.2
	808+00 (4:1 slope)	Undrained	Circular - Spencer[1]	2.48	-
			Non Circular - Spencer[1]	2.48	C.5
		Drained	Circular - Spencer[1]	2.48	-
			Non Circular - Spencer[1]	2.48	C.6
Ramp C Embankment Fill	1007+50	Drained	Circular - Spencer[1]	1.25	-
			Non Circular - Spencer[1]	1.25	D.1

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

CALCULATIONS

Date: 6/16/2025 **Made by:** TJ/LMP/JKW
Project No.: 31404817.003 **Checked by:** LMP
Subject: Global Stability Analysis for Hogan Rd Highway Interchange **Reviewed by:** CCB
Project: MaineDOT WIN 018595.10 Hogan Road Diverging Diamond Interchange.

Table 7: Deep Seated Failure - Static Loading

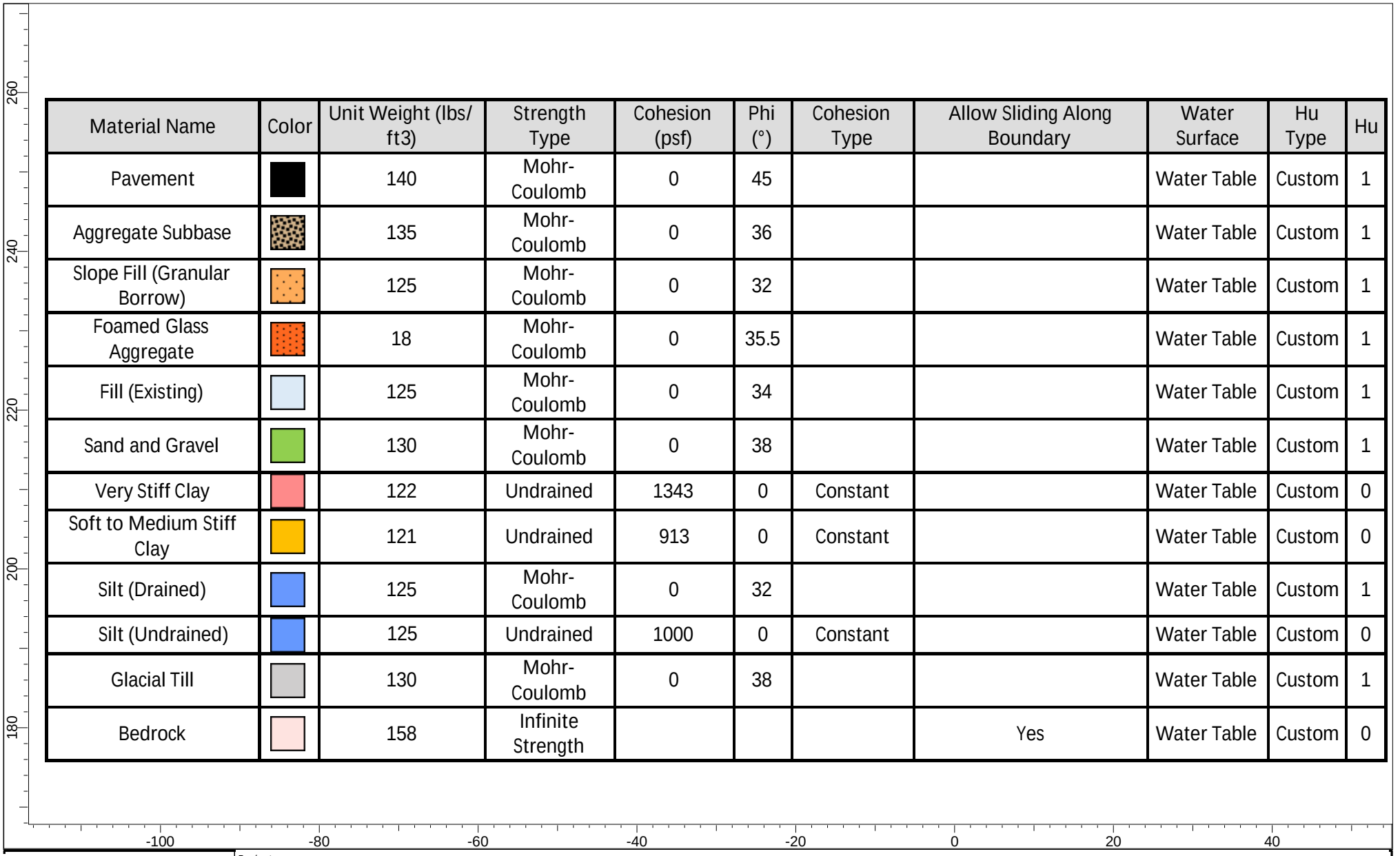
Structure	Station	Silt Condition	Analysis Method	Factor of Safety	Figure # (Attach. 1)
Hogan Rd EB Embankment Fill	106+00	Undrained	Circular - Spencer[1]	1.75	-
			Non Circular - Spencer[1]	1.69	A.3
		Drained	Circular - Spencer[1]	1.75	-
			Non Circular - Spencer[1]	1.69	A.4
Hogan Road EB Embankment Fill	119+00	Undrained	Circular - Spencer[1]	1.48	-
			Non Circular - Spencer[1]	1.45	B.3
		Drained	Circular - Spencer[1]	1.48	-
			Non Circular - Spencer[1]	1.45	B.4
Ramp B Embankment Fill	808+00 (2:1 slope)	Undrained	Circular - Spencer[1]	1.40	-
			Non Circular - Spencer[1]	1.41	C.3
		Drained	Circular - Spencer[1]	1.42	-
			Non Circular - Spencer[1]	1.40	C.4
	808+00 (4:1 slope)	Undrained	Circular - Spencer[1]	2.74	-
			Non Circular - Spencer[1]	2.69	C.7
		Drained	Circular - Spencer[1]	2.68	-
			Non Circular - Spencer[1]	2.66	C.8
Ramp C Embankment Fill	1007+50	Drained	Circular - Spencer[1]	1.49	-
			Non Circular - Spencer[1]	1.39	D.2

Note: Factor of safety for failures deeper than 10 feet. Shallower surfaces were not included in the analyses.

CONCLUSIONS

Static analyses; shallow failure surfaces: The proposed embankment fills at Stations 106+00, 119+00, 808+00 (2:1 slope) and 1007+50 produce global stability FS < 1.3 for potential slope failures in the proposed fill areas under static conditions. For these locations, the proposed fill engineering parameters used from the MaineDOT Bridge Design Guide section 5.9.2 for slopes and the combined 2:1 geometry are the cause of resultant factors of safety below 1.3.

Static analyses; deeper failure surfaces: The surfaces deeper than 10 feet analyzed, produced the minimum recommended FS of 1.3 or greater 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 (°)	Cohesion Type	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu
Pavement		140	Mohr-Coulomb	0	45			Water Table	Custom	1
Aggregate Subbase		135	Mohr-Coulomb	0	36			Water Table	Custom	1
Slope Fill (Granular Borrow)		125	Mohr-Coulomb	0	32			Water Table	Custom	1
Foamed Glass Aggregate		18	Mohr-Coulomb	0	35.5			Water Table	Custom	1
Fill (Existing)		125	Mohr-Coulomb	0	34			Water Table	Custom	1
Sand and Gravel		130	Mohr-Coulomb	0	38			Water Table	Custom	1
Very Stiff Clay		122	Undrained	1343	0	Constant		Water Table	Custom	0
Soft to Medium Stiff Clay		121	Undrained	913	0	Constant		Water Table	Custom	0
Silt (Drained)		125	Mohr-Coulomb	0	32			Water Table	Custom	1
Silt (Undrained)		125	Undrained	1000	0	Constant		Water Table	Custom	0
Glacial Till		130	Mohr-Coulomb	0	38			Water Table	Custom	1
Bedrock		158	Infinite Strength				Yes	Water Table	Custom	0

wsp

Project: Hogan Road Bridge Replacement, Bangor Maine

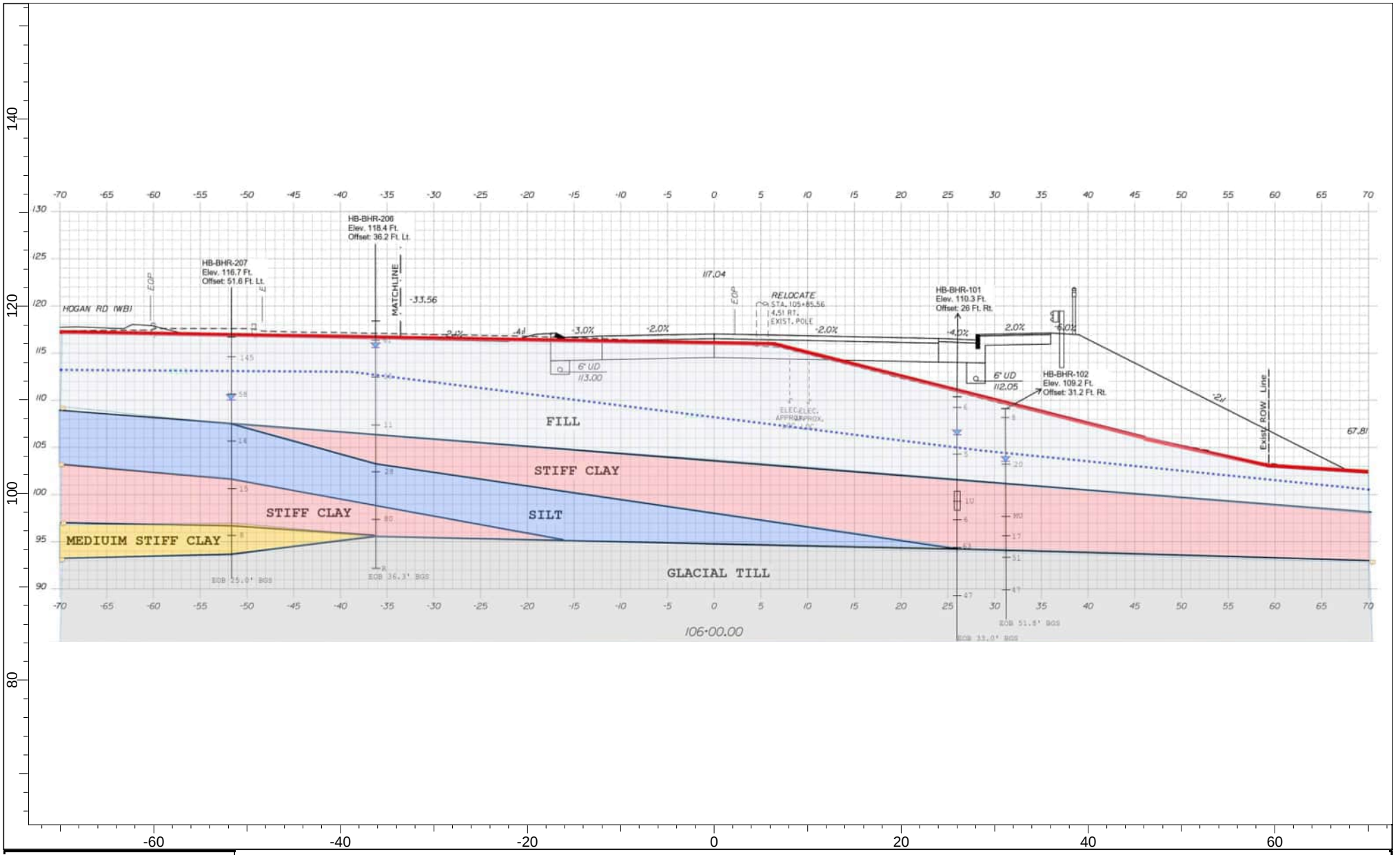
Analysis Description: Material Properties Table

Drawn By	Checked By	Reviewed By	Scale
TJ/LMP/JKW	LMP	CCB	1:200

Date	File Name
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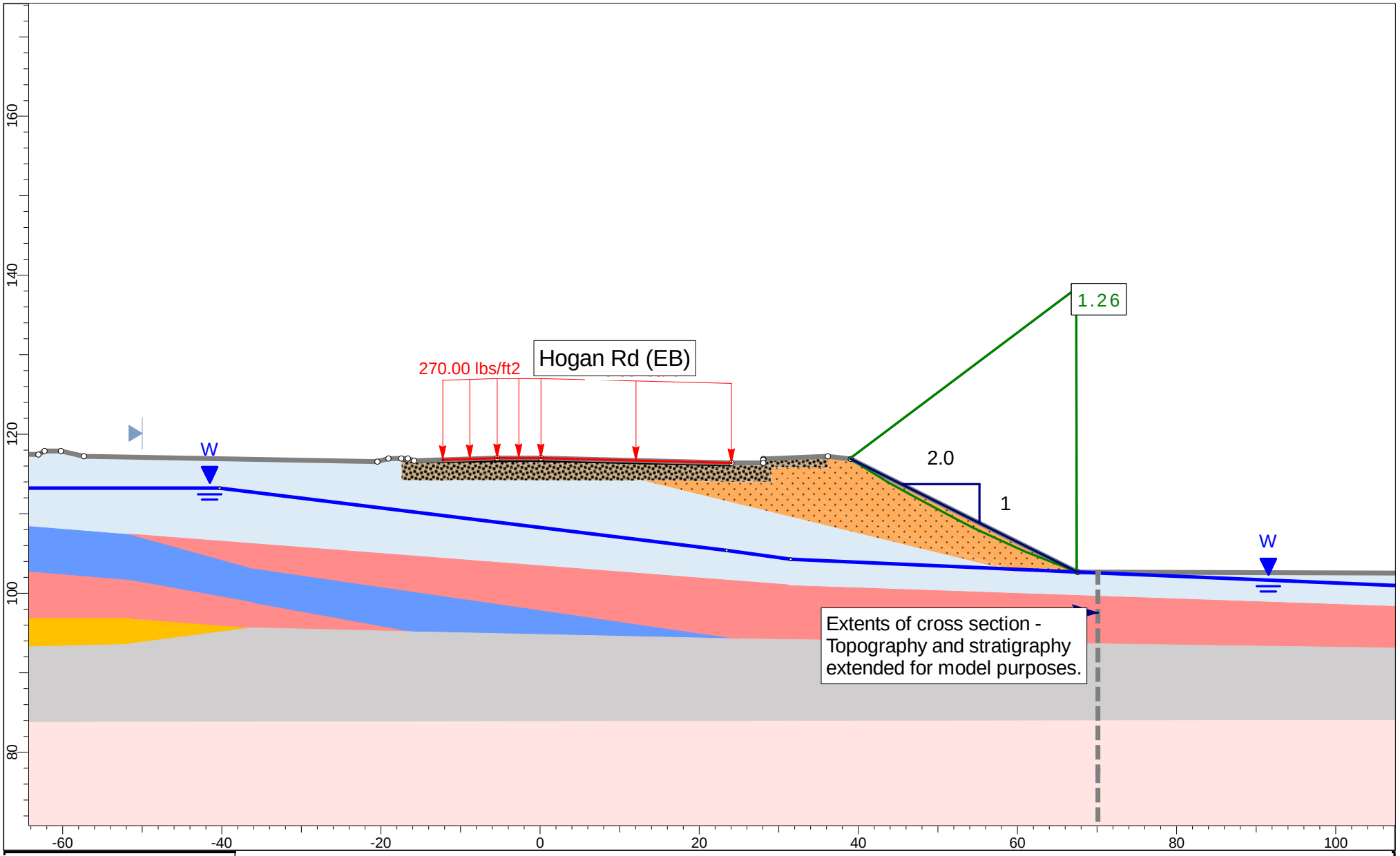
Figure 1

SLIDE 9.038



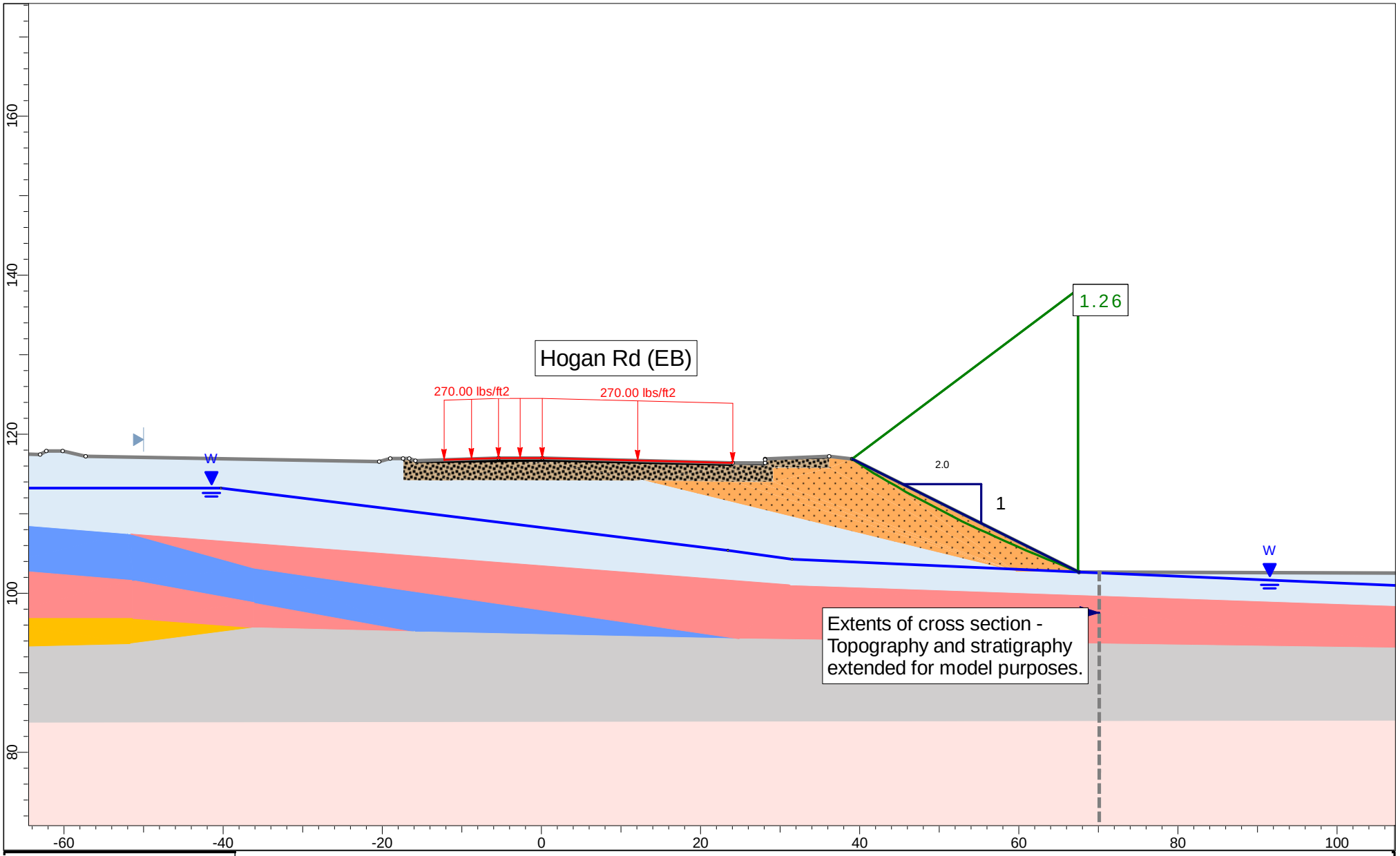
	Project	Hogan Road Bridge Replacement, Bangor, Maine		
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Drawn By TJ/LMP	Checked By JKW	Reviewed By CCB	Scale 1:170	Figure A.0
Date 2/20/2024	File Name Sta. 106+00.slmd			

SLIDEINTERPRET 9.020



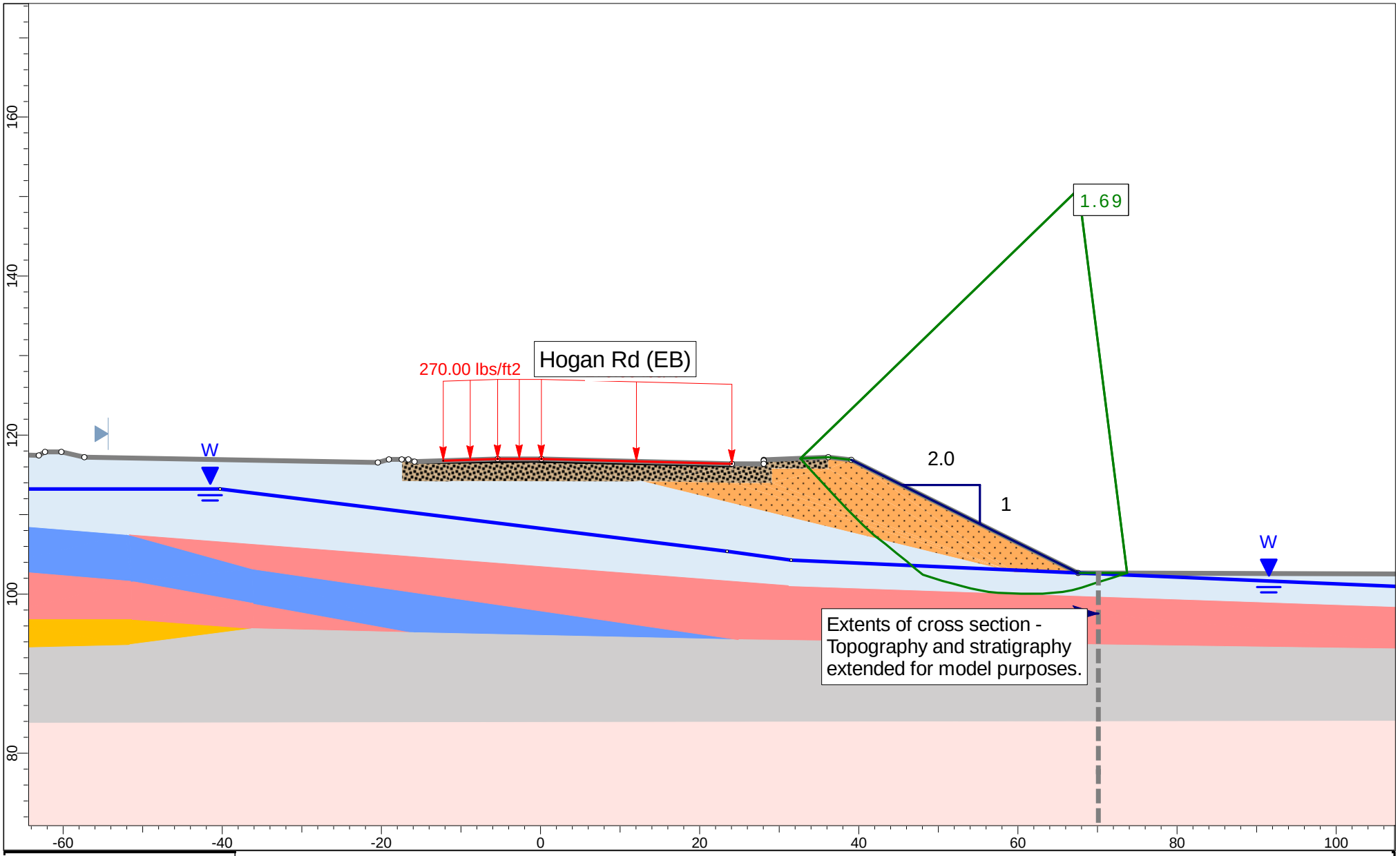
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Analysis Description	Sta. 106+00 - Noncircular Surifial Surfaces - Silt Undrained			
Drawn By TJ/LMP/JKW	Checked By JKW	Reviewed By CCB	Scale 1:200	Figure A.1
Date 6/18/2025	File Name 106+00.slmd			

SLIDEINTERPRET 9.038



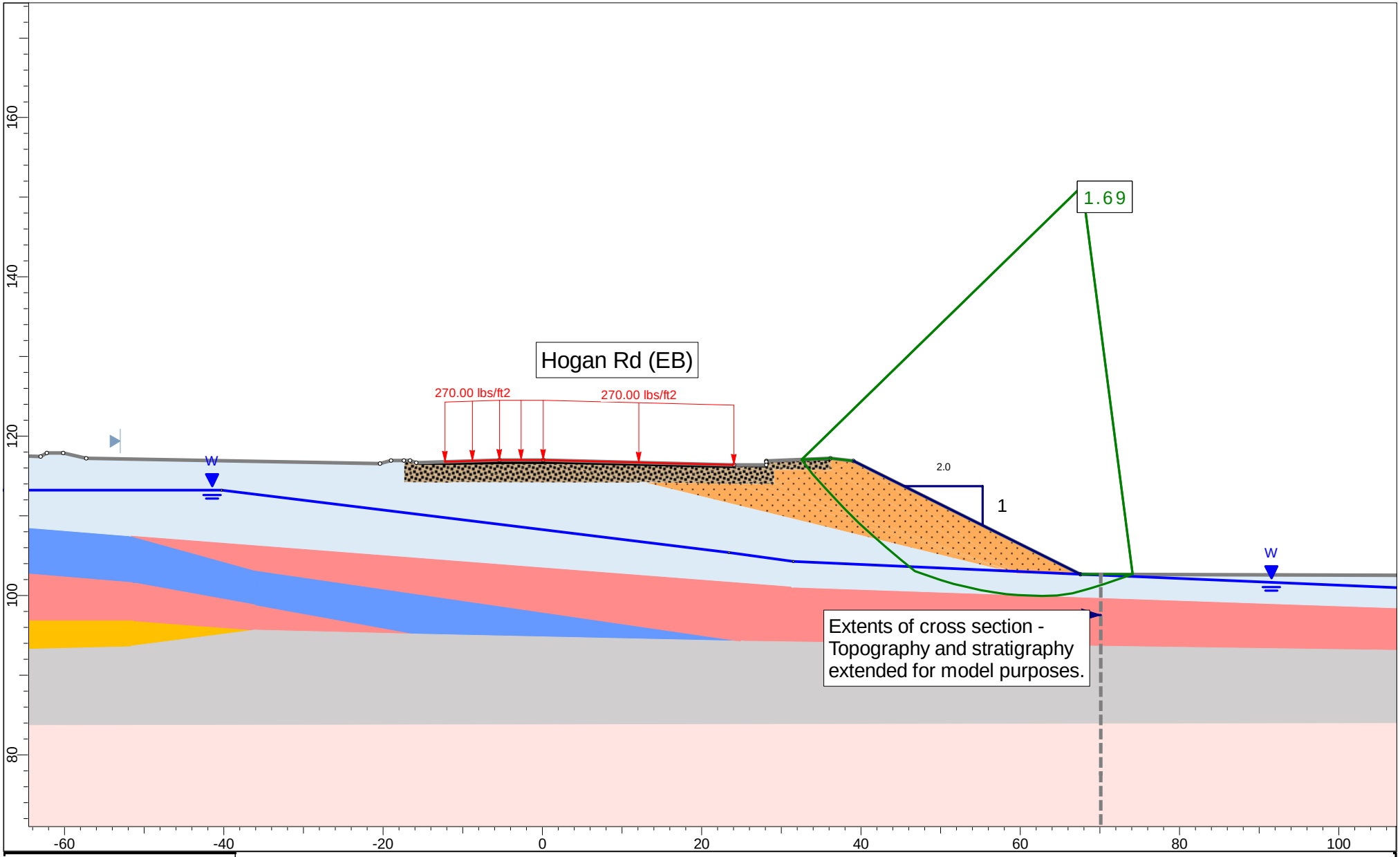
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Analysis Description	Sta. 106+00 - Noncircular Surifial Surfaces - Silt Drained			
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Date 6/18/2025	File Name 106+00.slmd			

SLIDEINTERPRET 9.038



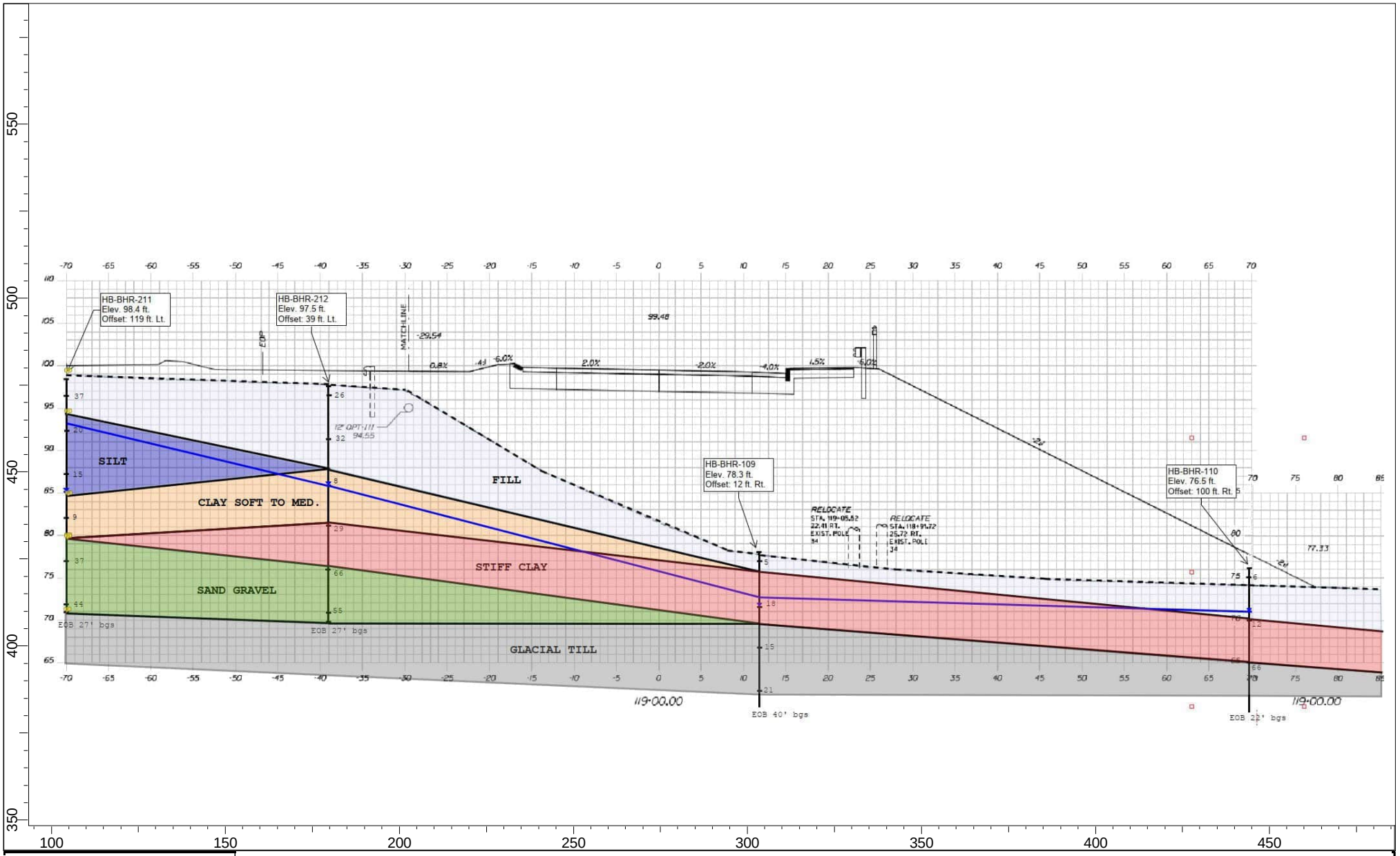
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Date 6/18/2025	File Name 106+00.slmd			

SLIDEINTERPRET 9.038



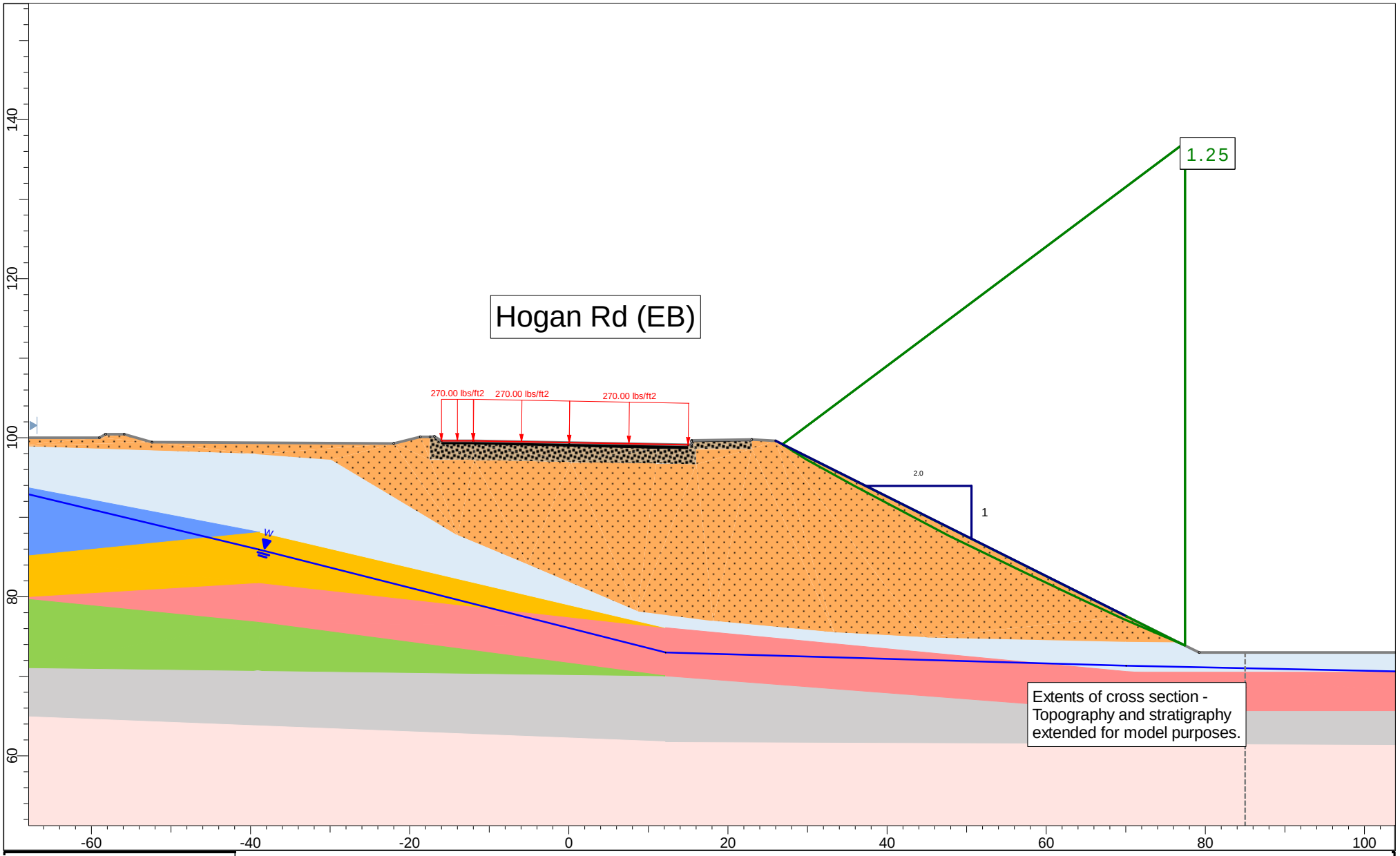
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Date 6/18/2025	File Name 106+00.slmd			

SLIDEINTERPRET 9.038



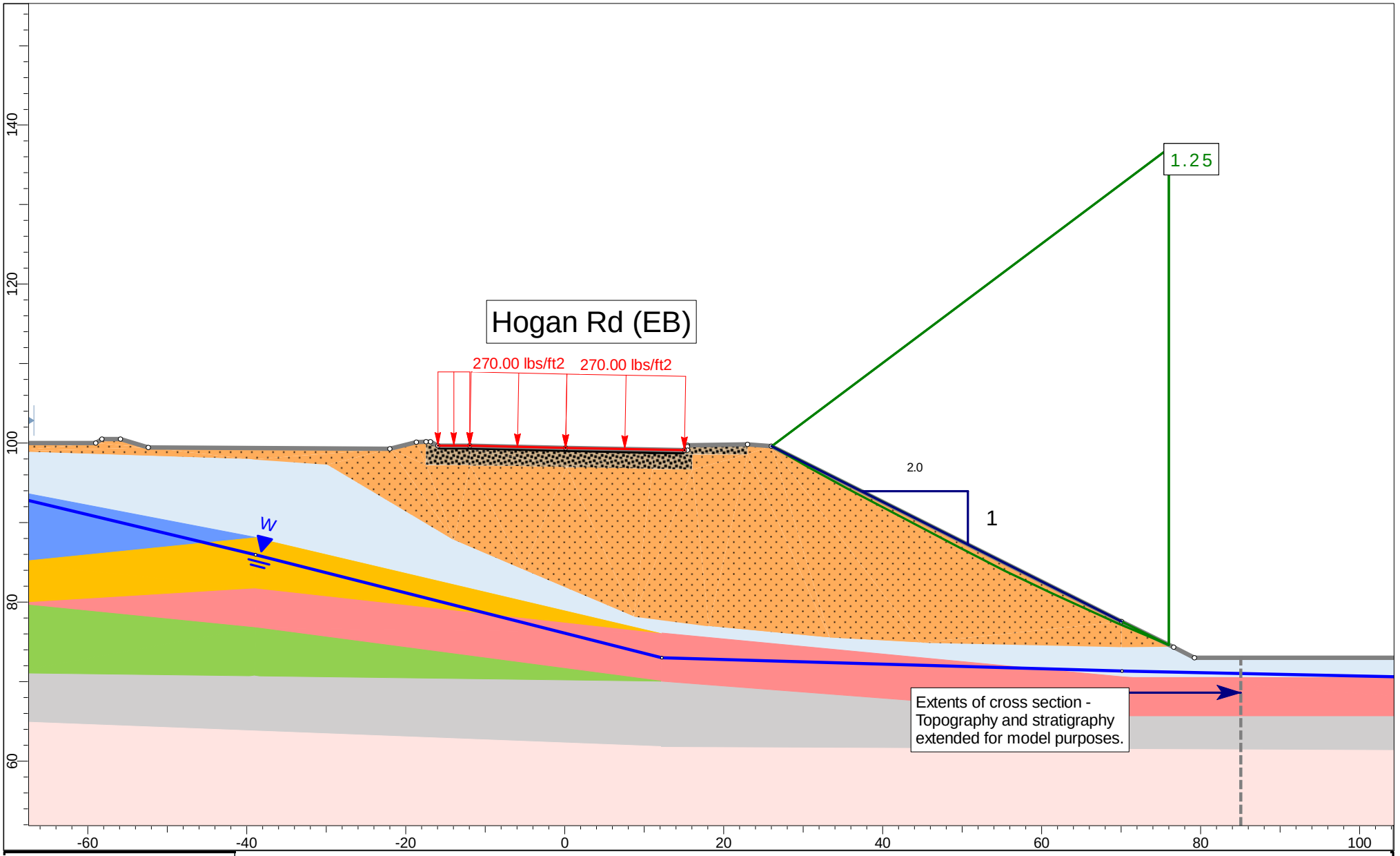
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	Analysis Description	Proposed Geometry		
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Date 6/20/2025	File Name 119+00.slmd			

SLIDE 9.038



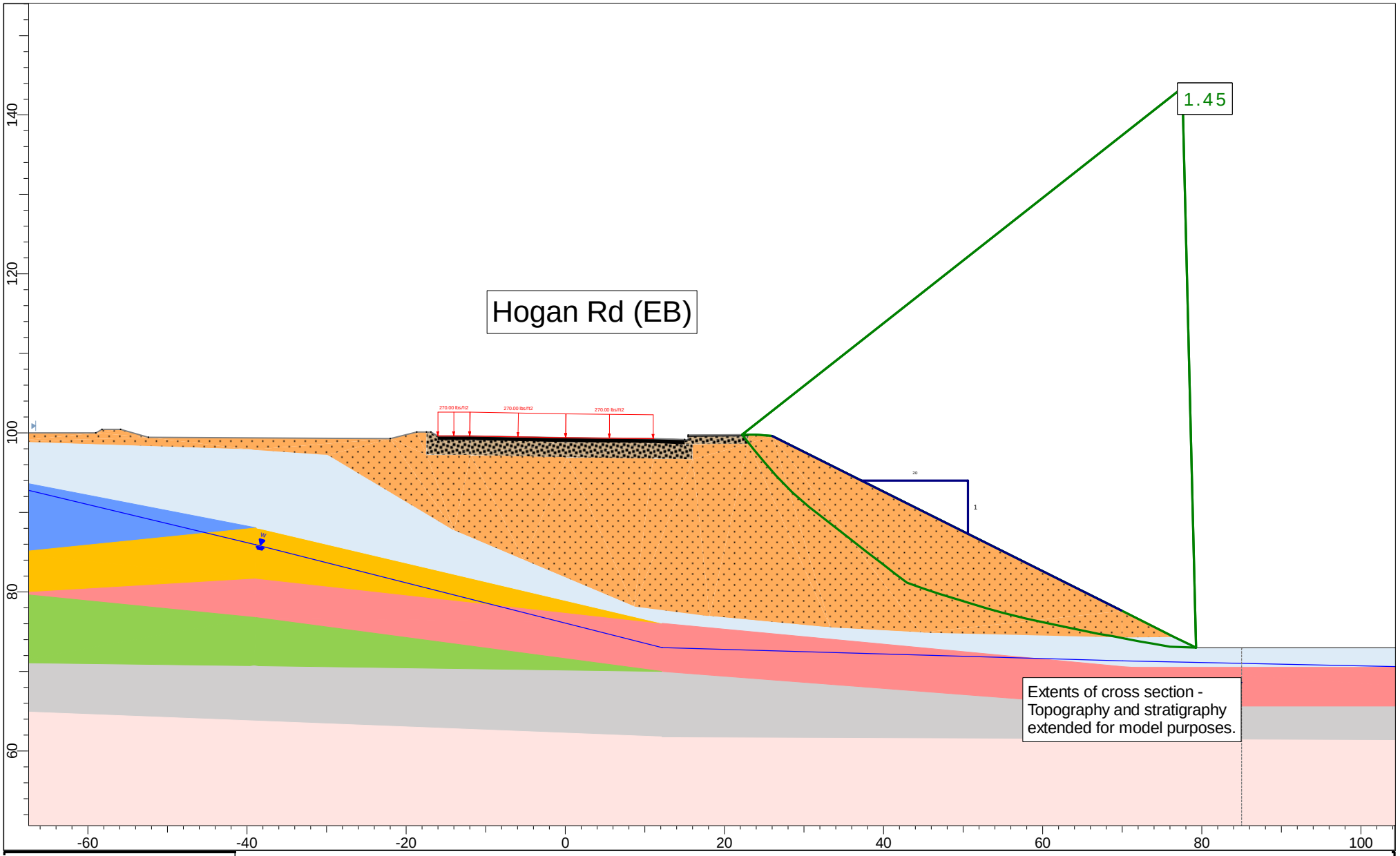
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Analysis Description	Sta. 119+00 - Noncircular Surficial Surfaces - Silt Undrained			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure B.1
Date 6/24/2025	File Name 119+00.slmd			

SLIDEINTERPRET 9.038



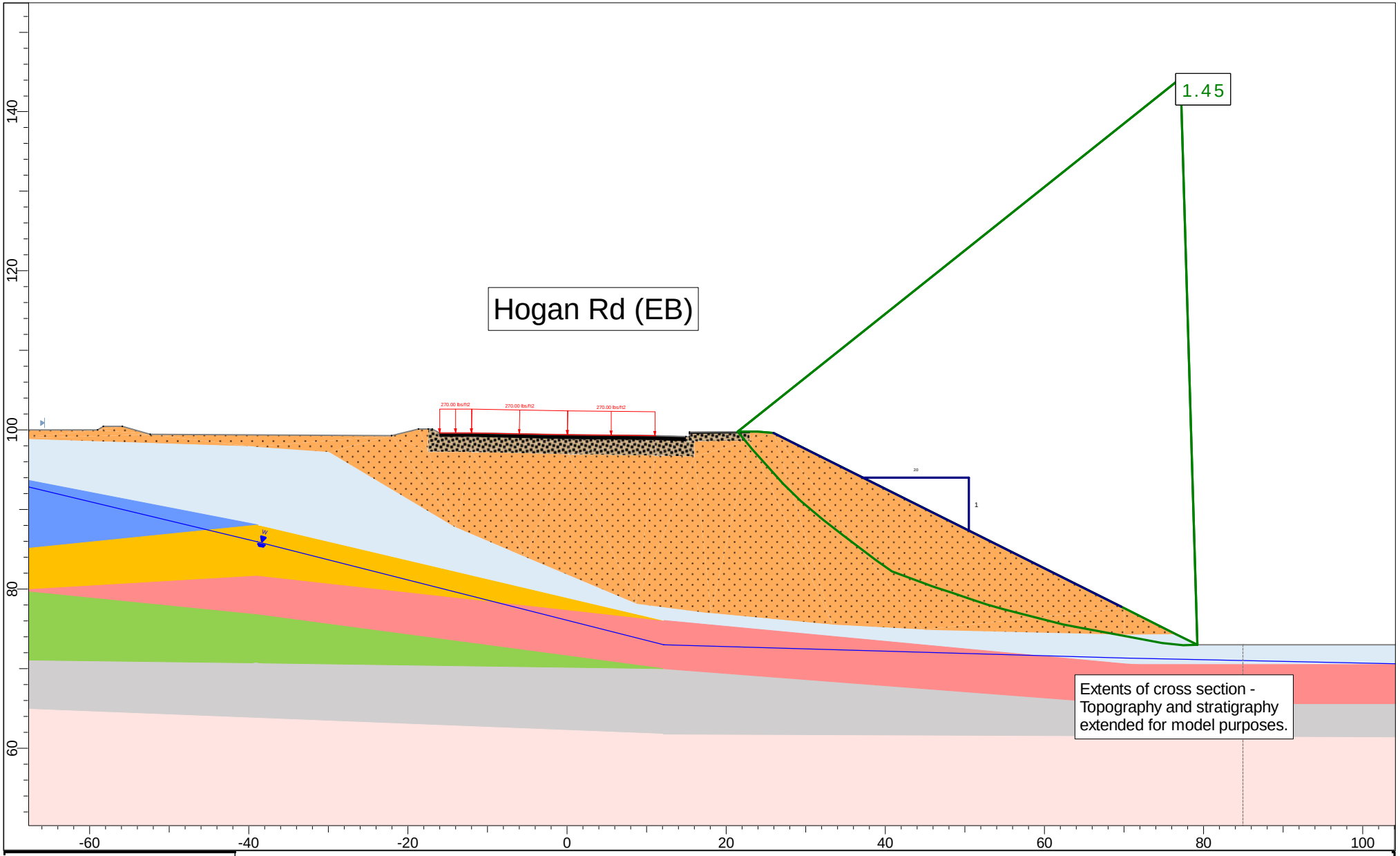
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Analysis Description	Sta. 119+00 - Noncircular Surficial Surfaces - Silt Drained			
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Date 6/24/2025	File Name 119+00.slmd			

SLIDEINTERPRET 9.038



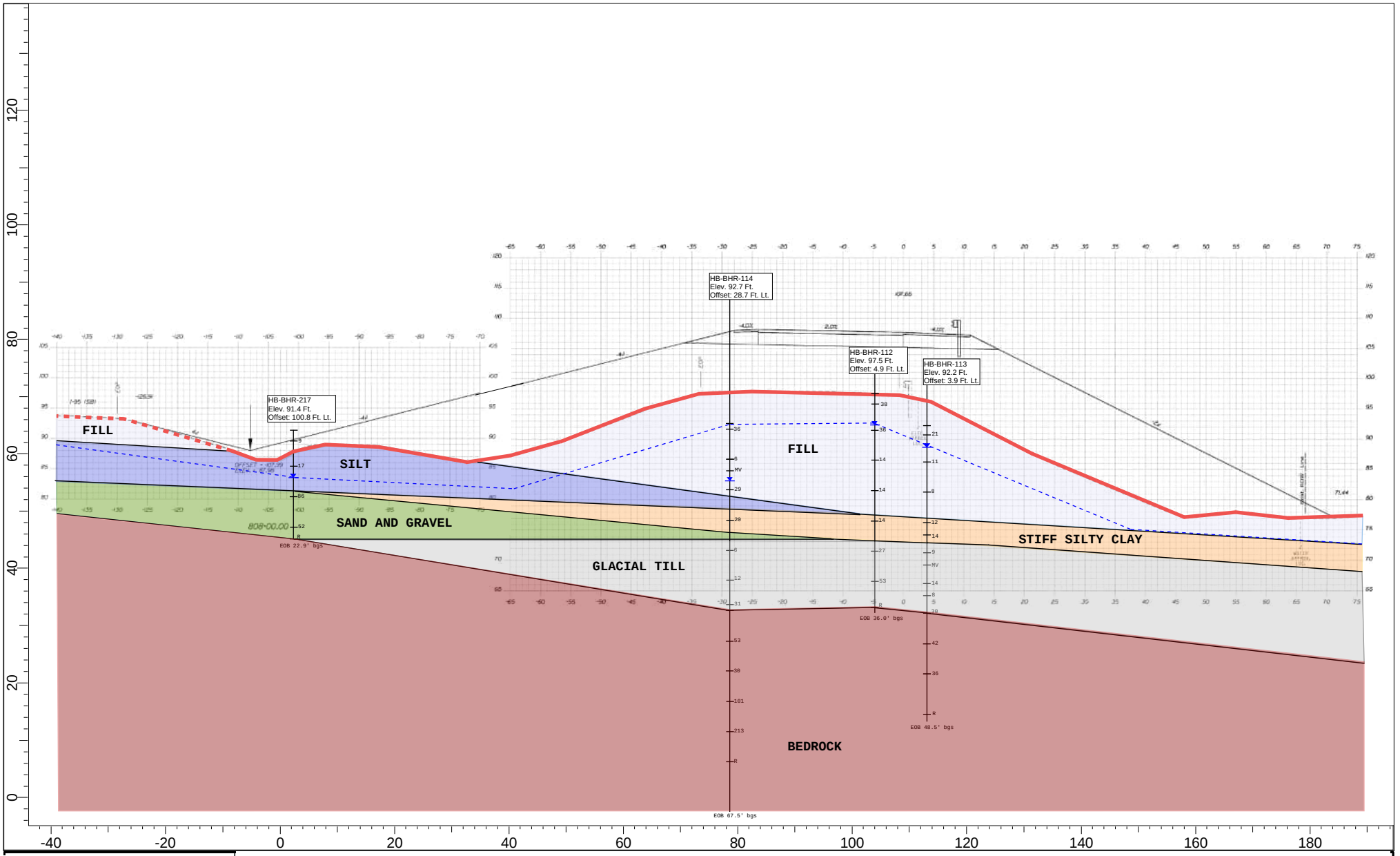
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Analysis Description	Sta. 119+00 - Noncircular Deep Surfaces - Silt Undrained			
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Date 6/24/2025	File Name 119+00.slmd			

SLIDEINTERPRET 9.038



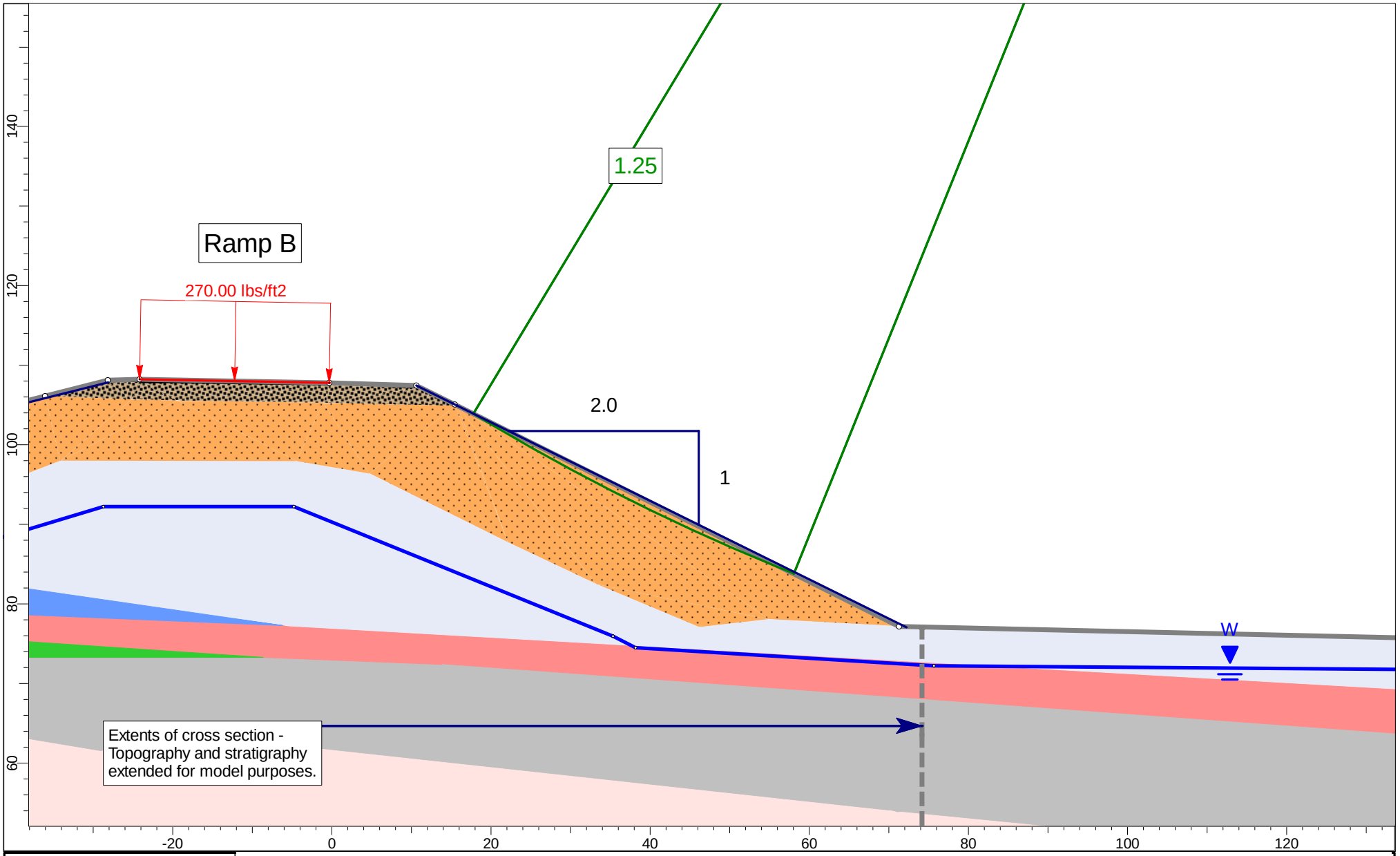
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Analysis Description	Sta. 119+00 - Noncircular Deepl Surfaces - Silt Drained			
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Date 6/24/2025	File Name 119+00.slmd			

SLIDEINTERPRET 9.038

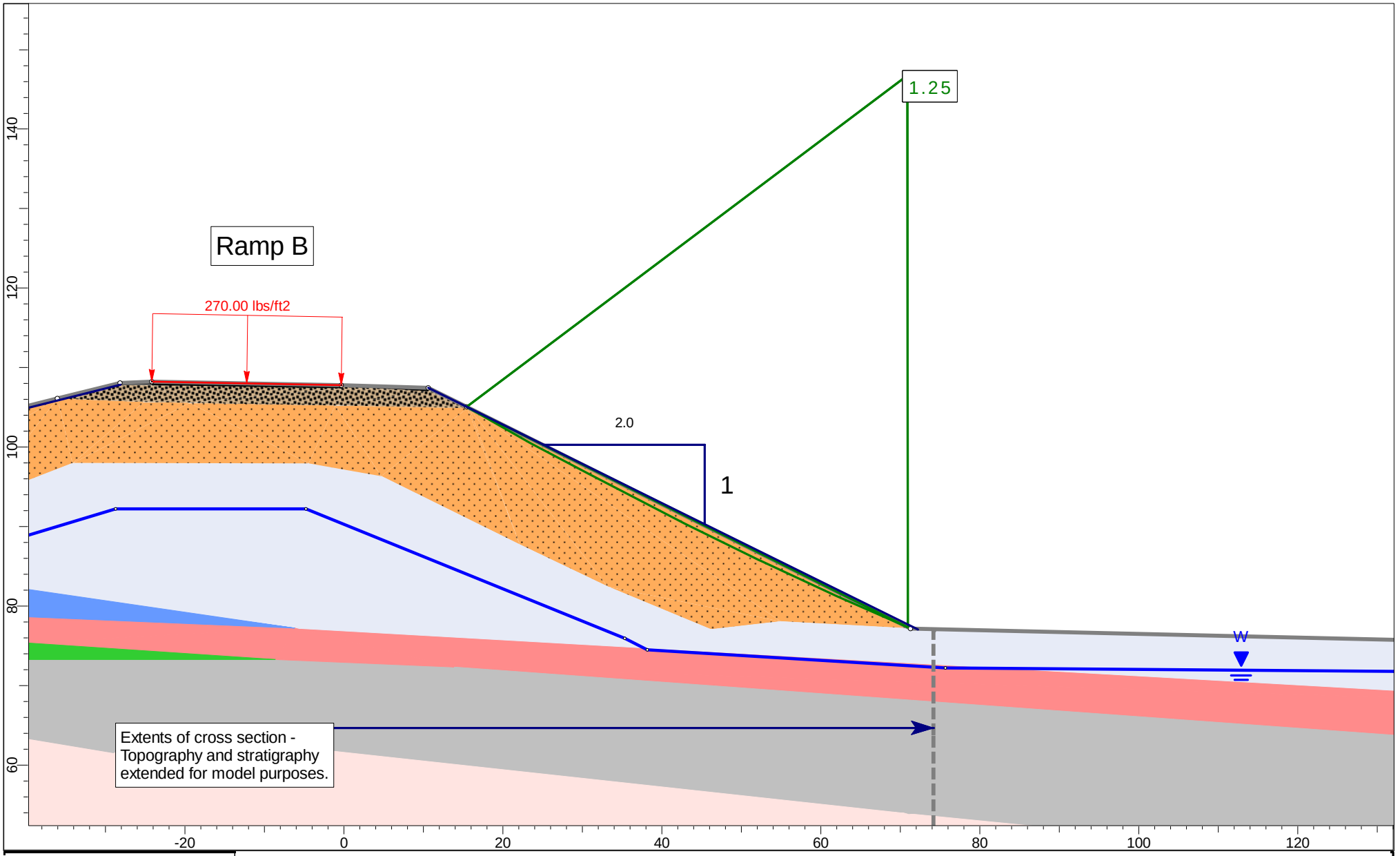


Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Proposed Geomerty			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:912	Figure C.0
Date 6/24/2025	File Name 808+00.slmd			

SLIDE 9.038

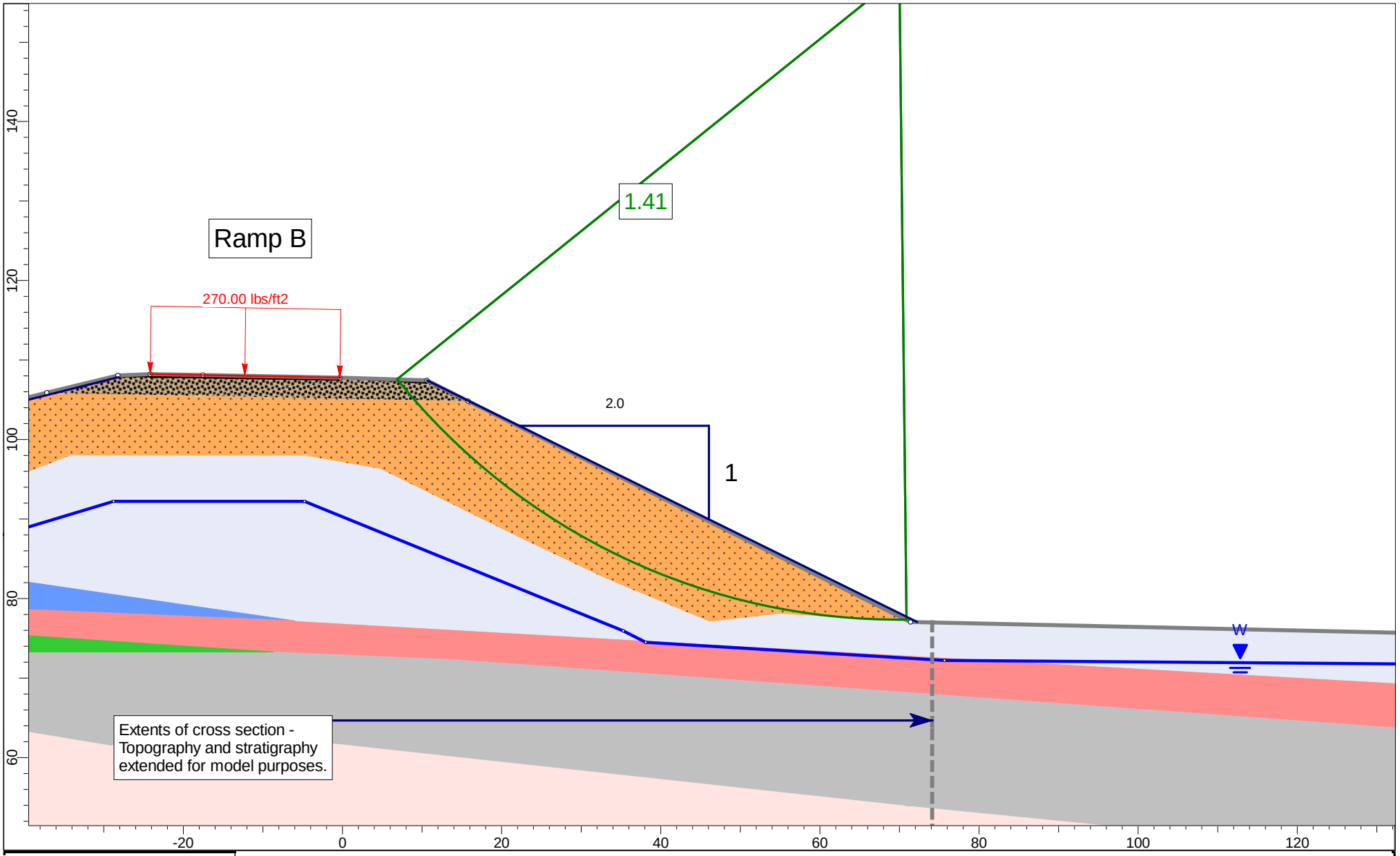


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	Analysis Description	Sta. 808+00 - Noncircular Surficial Surfaces - Silt Undrained - Left Slope		
	Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200
SLIDEINTERPRET 9.038	Date 6/24/2025	File Name 808+00.slmd		Figure C.1



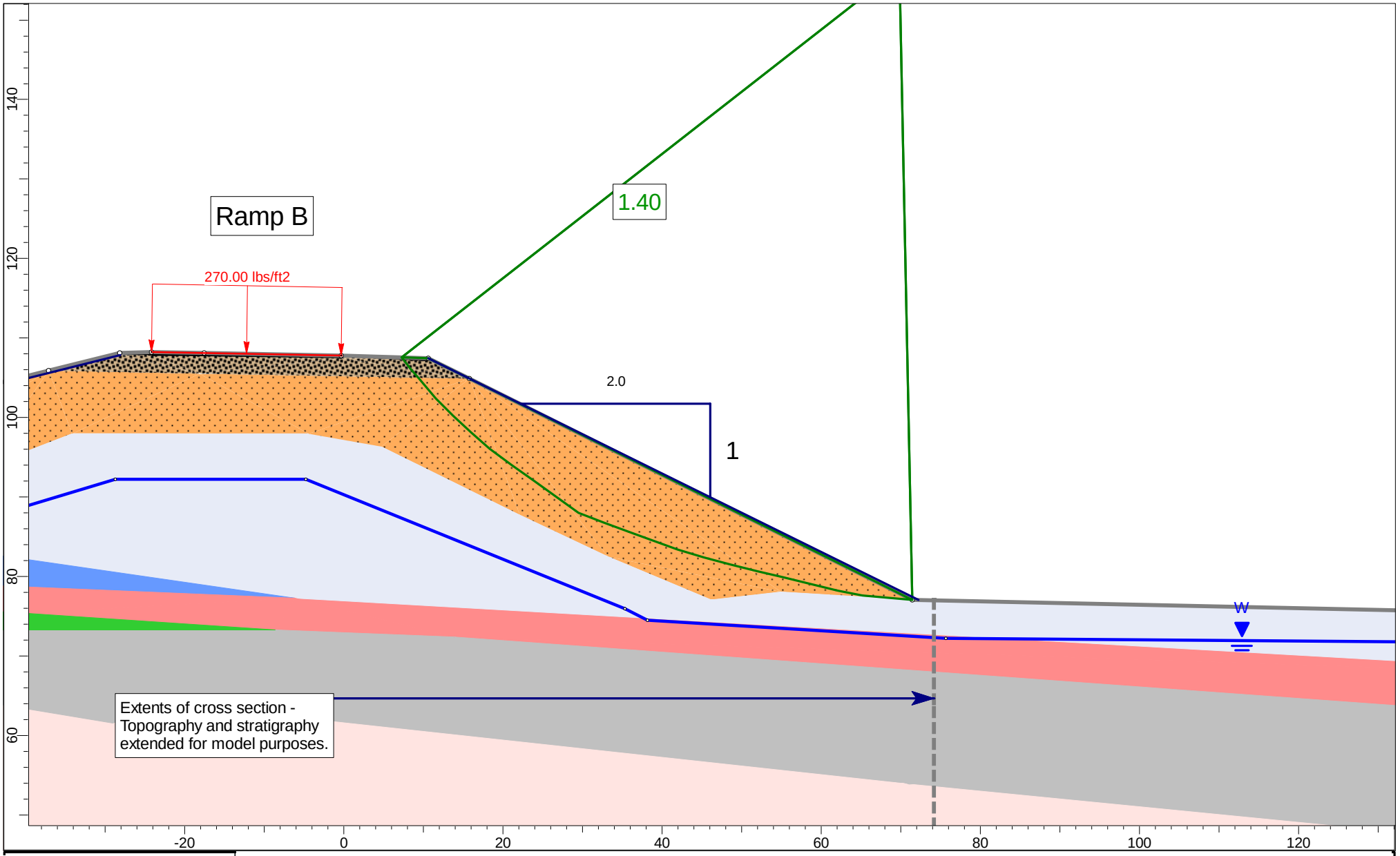
Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Sta. 808+00 - Noncircular Surficial Surfaces - Silt Drained - Right Slope			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure C.2
Date 6/18/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



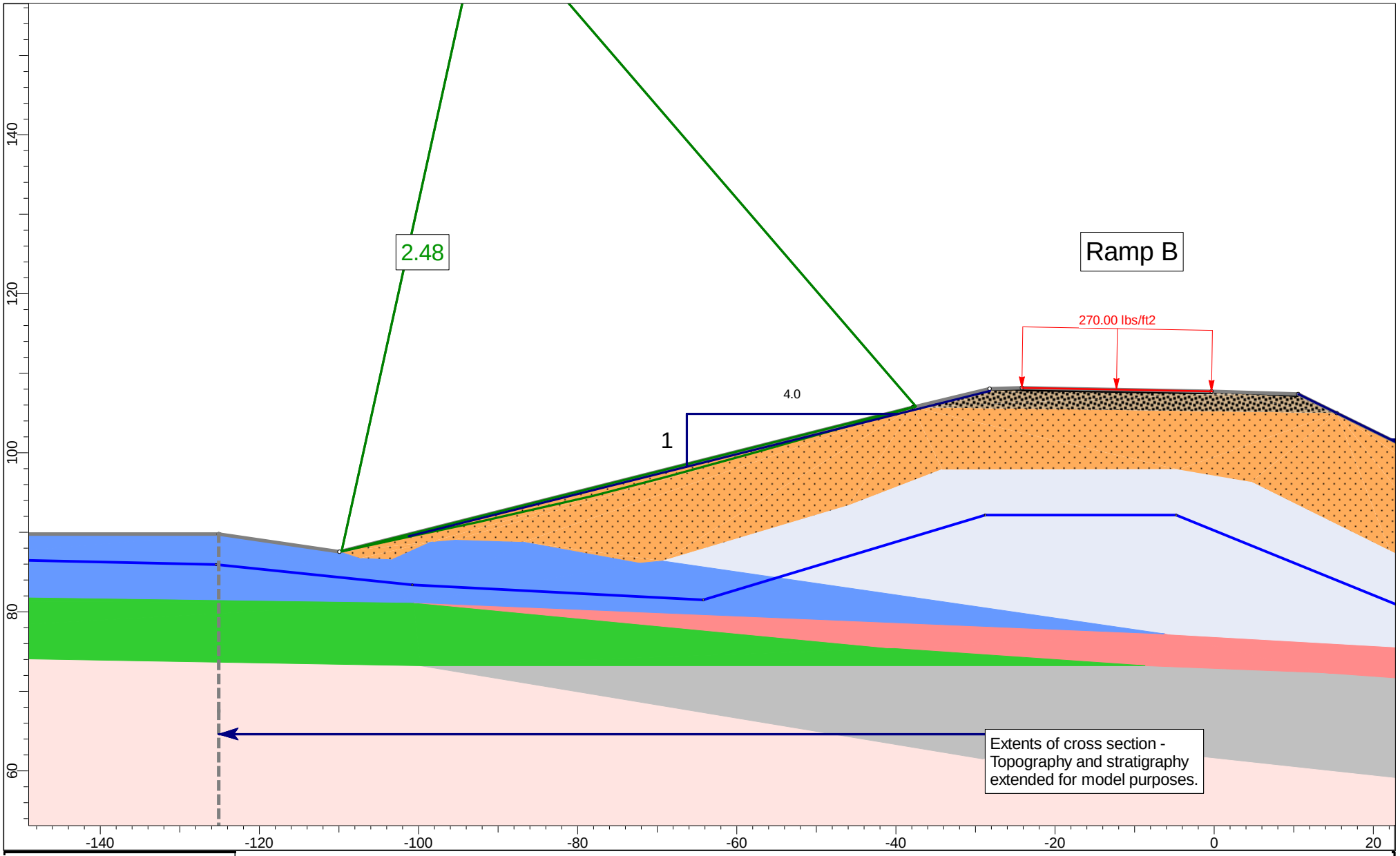
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Date 6/18/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



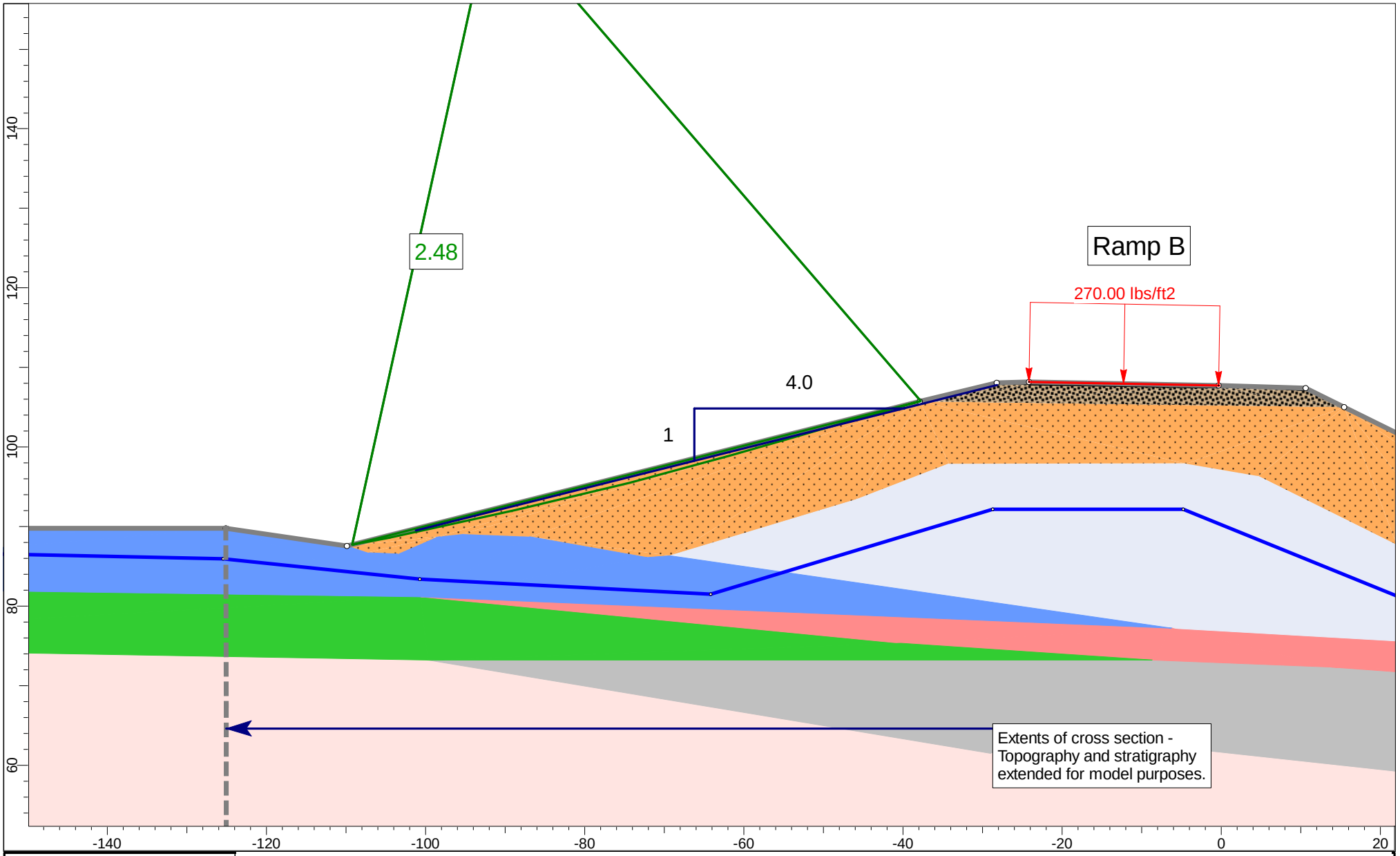
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Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure C.4
Date 6/18/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



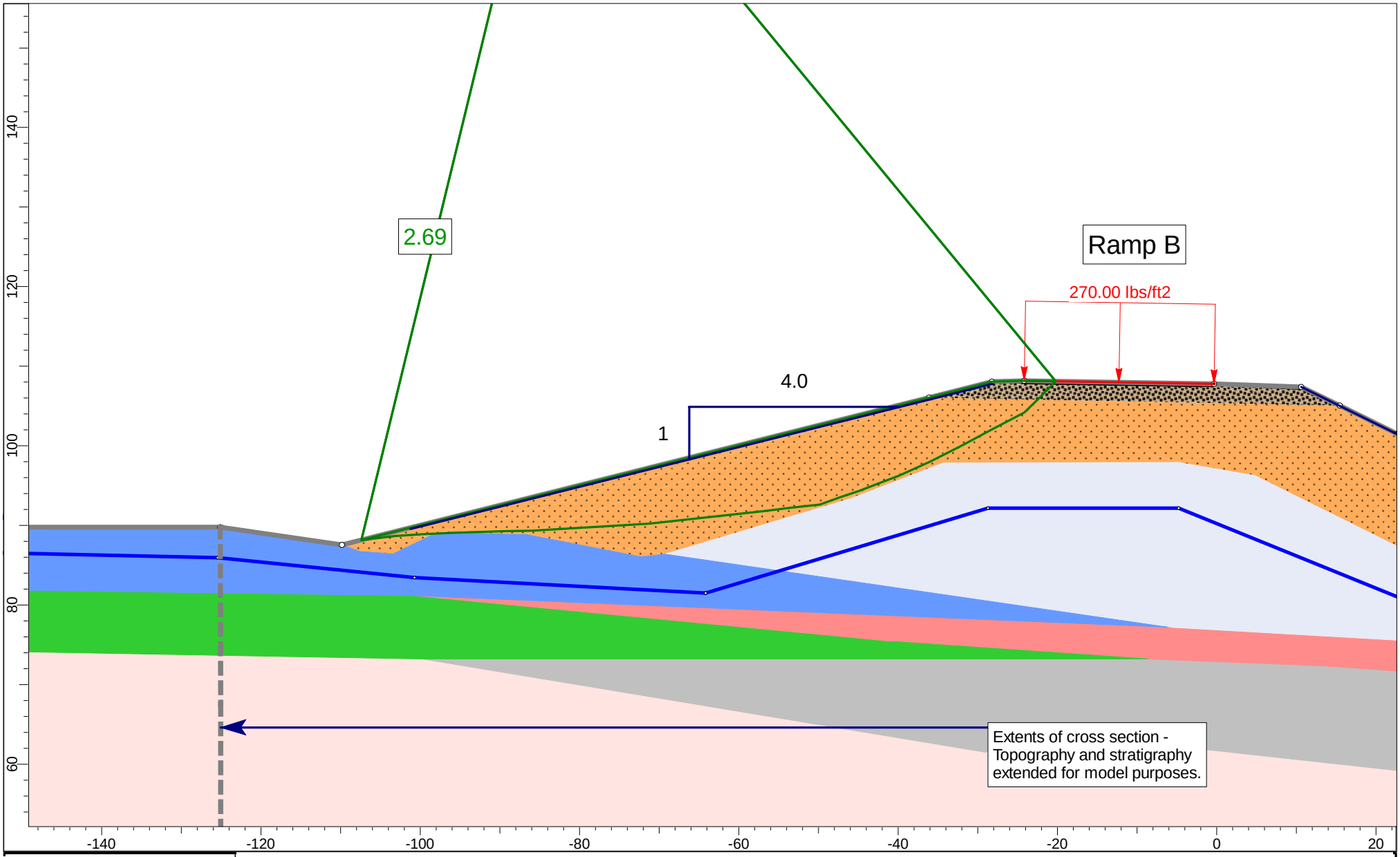
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Analysis Description	Sta. 808+00 - Noncircular Surficial Surfaces - Silt Undrained - Left Slope			
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Date 6/24/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



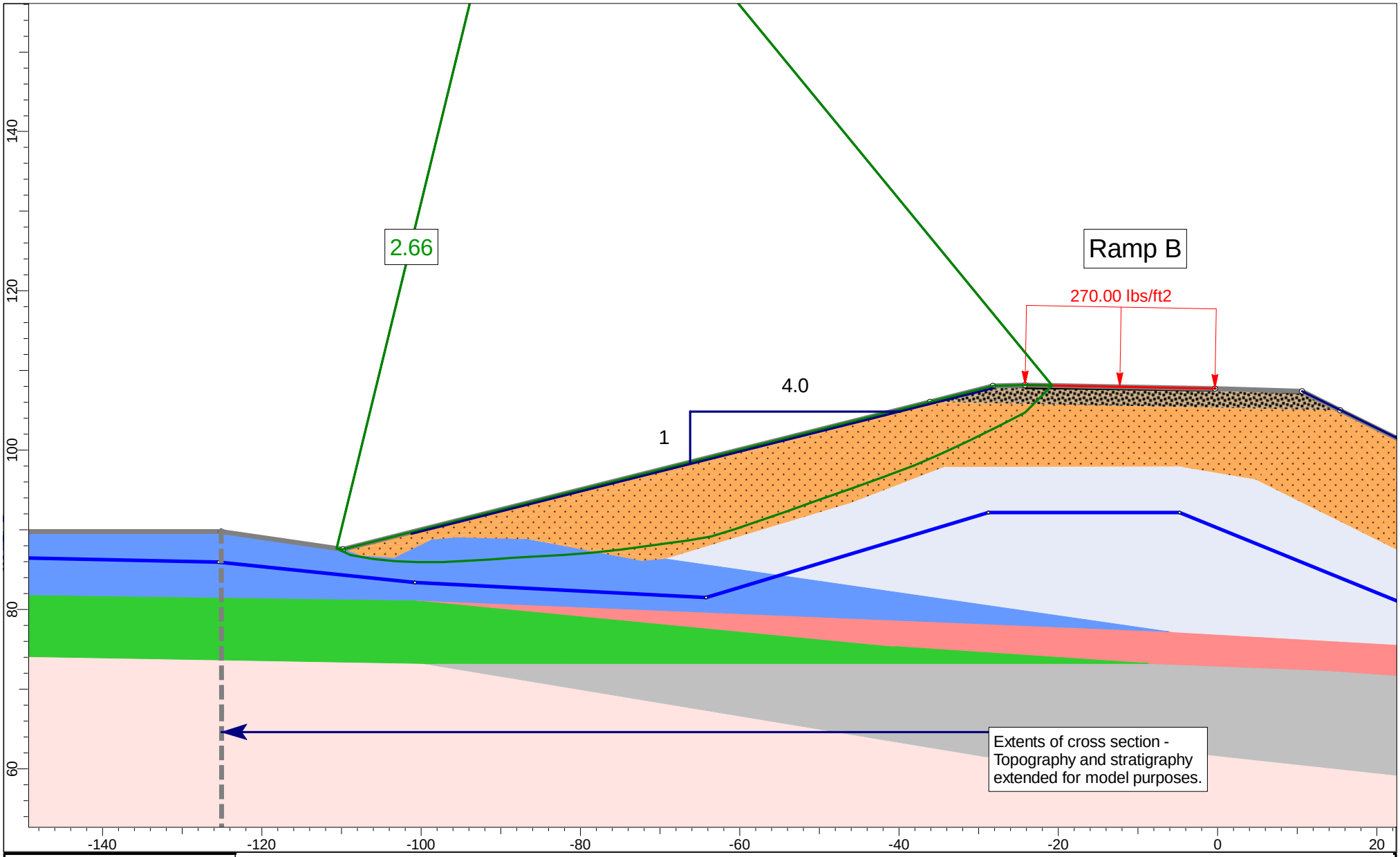
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Analysis Description	Sta. 808+00 - Noncircular Surficial Surfaces - Silt Drained - Left Slope			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure C.6
Date 6/24/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



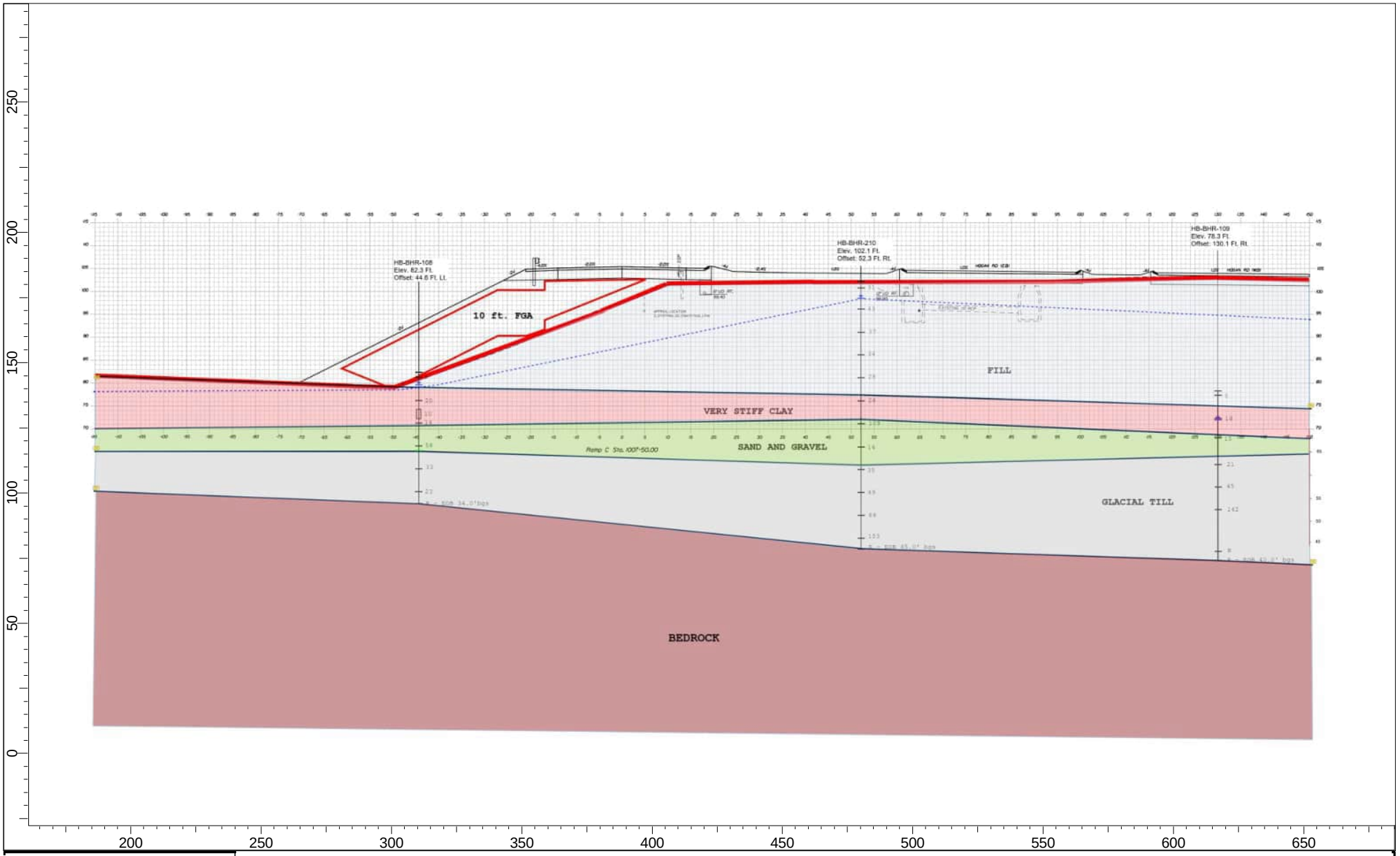
Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Sta. 808+00 - Noncircular Deep Surfaces - Silt Undrained - Left Slope			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure C.7
Date 6/24/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



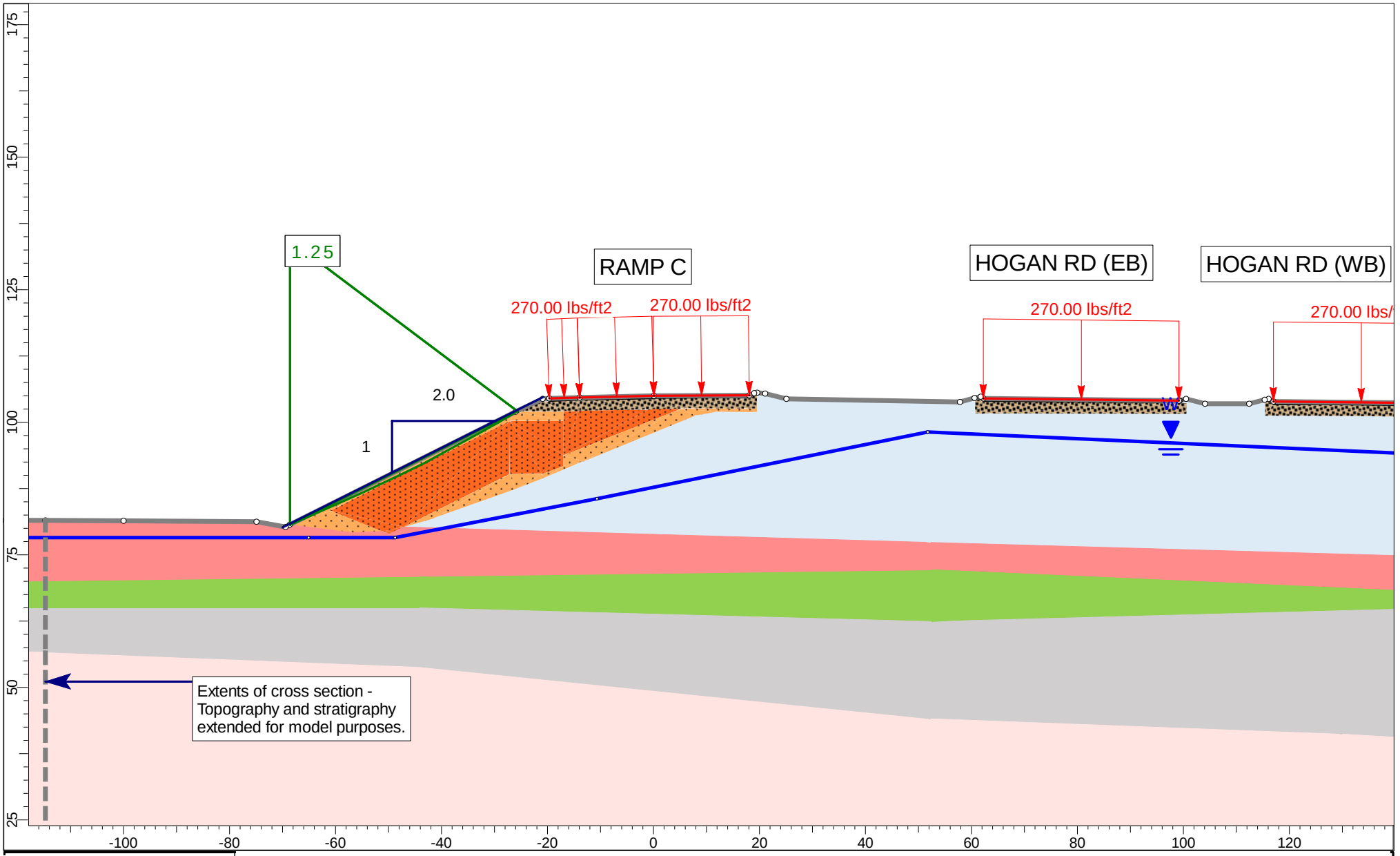
Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Sta. 808+00 - Noncircular Deep Surfaces - Silt Drained - Left Slope			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:200	Figure C.8
Date 6/24/2025	File Name 808+00.slmd			

SLIDEINTERPRET 9.038



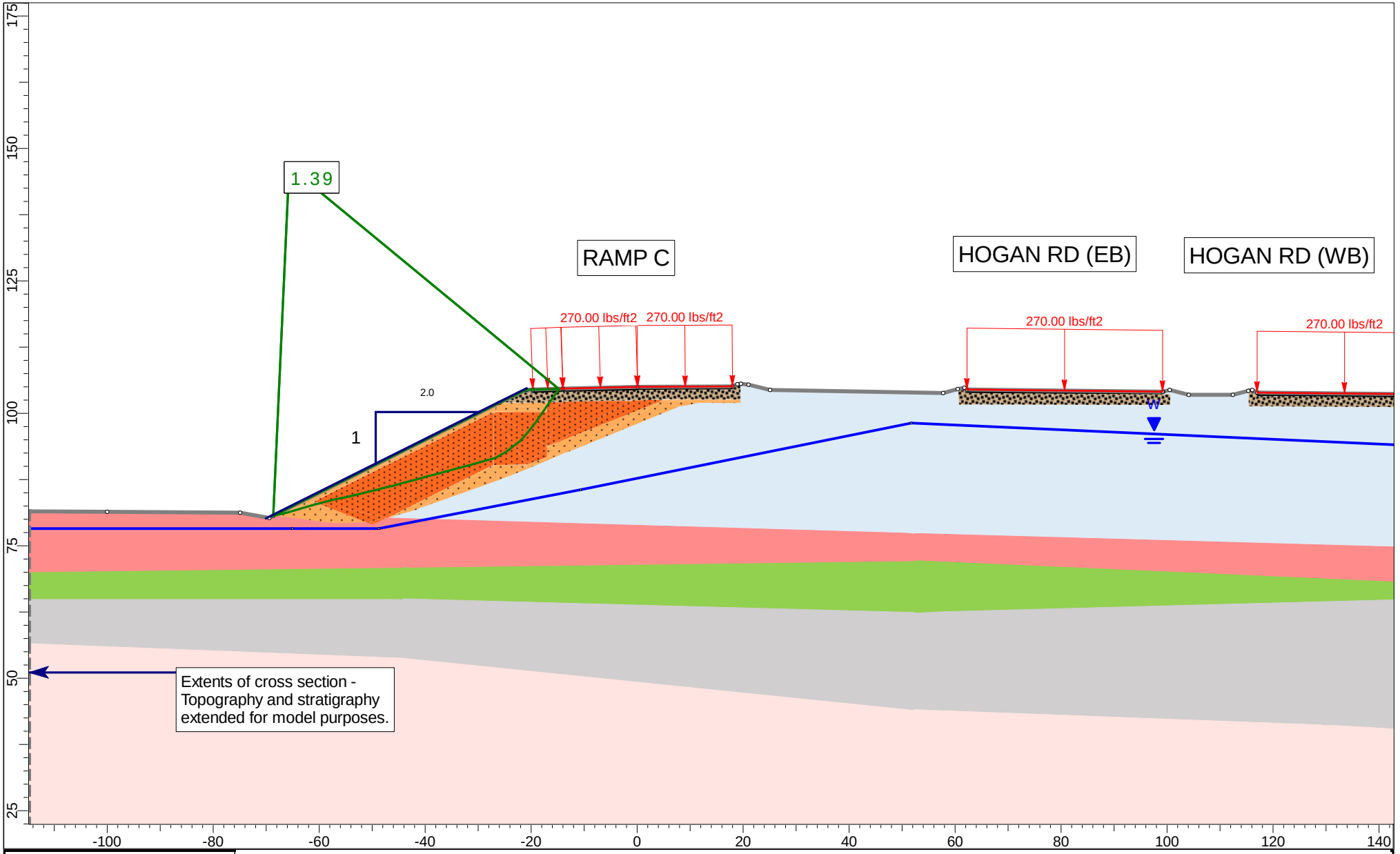
Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Proposed Geomerty			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:611	Figure D.0
Date 6/20/2025	File Name 1007+50.slmd			

SLIDE 9.038



Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Sta. 1007+50 - Noncircular Surficial Surfaces			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:300	Figure D.1
Date 6/17/2025	File Name 1007+50.slmd			

SLIDEINTERPRET 9.038



Project	Hogan Road Bridge Replacement, Bangor Maine			
Analysis Description	Sta. 1007+50 - Noncircular Deep Surfaces			
Drawn By TJ/LMP/JKW	Checked By LMP	Reviewed By CCB	Scale 1:300	Figure D.2
Date 6/23/2025	File Name 1007+50.slmd			

SLIDEINTERPRET 9.038

APPENDIX E

Settlement Calculations

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Highway	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Estimate the anticipated total and differential settlement due to the proposed embankment fills at Hogan Road Eastbound and Westbound, Ramp A, Ramp B, Ramp C, Spur B, and Spur C for the US I-95 Exit 187 / Hogan Road Diverging Diamond Interchange (DDI). This analysis uses embankment geometry from the Progress Print plans dated March 10, 2025 and the 60% bridge plans dated August 30, 2024. Soil stratigraphy and soil parameters are based on the Phase 1 subsurface investigation performed in May 2022 and the Phase 2 subsurface investigation performed in September 2023.

METHOD

Use the Rocscience software *Settle3* (Ref. 1) to create three-dimensional models of the proposed embankment fills and interpreted subsurface soils. The interchange (consisting of Hogan Road Westbound, Ramps A and C, and Spurs B and C) is modeled separately from Ramp B in order to model unique interpreted subsurface stratigraphy at each location. The Hogan Road Eastbound grade raise is modeled separately from the interchange in order to model a unique existing grade at each location.

REFERENCES

1. Rocscience Inc. Settle3 software package, version 5.025, build date February 13, 2025.

2. WSP Interpretive Subsurface Profiles A-A' through E-E' for the highway interchange and ramps, dated March 25,

3. MaineDOT CADD file containing proposed highway geometry, titled "Highway.dgn", received by WSP on April 4, 2024.

4. VHB for State of Maine Department of Transportation. Progress Print: Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Dated March 10, 2025.

5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with June 2018 updates

6. VHB for State of Maine Department of Transportation. 60% Westbound Bridge Plans: Bangor, Penobscot County, Hogan Road Bridge over I-95, WIN 018595.10 / FED 1859510. Dated August 30, 2024.

7. Das, Braja M. 2011. Principles of Foundation Engineering, 7th ed.

8. WSP boring logs for the Bridge and Highway programs, included as Appendix A of the Highway Geotechnical Design Report.

9. WSP table "Summary of In-Situ Vane Shear Testing Results", included as Table 2 of the Highway Geotechnical Design Report.

10. GeoTesting Express laboratory testing results on soil samples for the Bridge and Highway programs, included as Appendix C and summarized in Tables 4 and 5 of the Highway Geotechnical Design Report.

11. GeoTesting Express laboratory testing results on rock core for the Bridge program, included as Appendix C and summarized in Table 6 of the Bridge Geotechnical Design Report.

12. Email from <Kate.Maguire@maine.gov> to <melissa.landon@wsp.com>, subject "RE: Bangor Hogan Road Interchange Settlement Criteria", dated March 7, 2024, 3:49 PM.

13. Meeting among WSP, VHB, and MaineDOT, August 6, 2024, 10 AM - 11 AM.

14. Email from <Derrick.Carleton@maine.gov> to WSP, VHB, and MaineDOT, subject "RE: 18595.10 Bangor DDI-Constructability Discussion", dated August 21, 2024, 9:32 AM.

15. WSP Golder. Preliminary Geotechnical Design Report: Hogan Road Diverging Diamond Interchange, Bangor, Maine, MaineDOT WIN 18595.10, GAUI 31404817.001. Dated December 29, 2022.

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Highway	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

ATTACHMENT FIGURES

1. Plan view of loading in Settle3 model (Interchange)
2. 3D view of soil stratigraphy and loading in Settle3 model (Interchange)
3. Plan view of loading in Settle3 model (Ramp B)
4. 3D view of soil stratigraphy and loading in Settle3 model (Ramp B)
5. Plan view of loading in Settle3 model (Hogan Road Eastbound)
6. 3D view of soil stratigraphy and loading in Settle3 model (Hogan Road Eastbound)
7. Total settlement along Ramp A at 75 years after construction with conventional fill
8. Total settlement along Ramp B at 75 years after construction with conventional fill
9. Total settlement along Spur B at 75 years after construction with lightweight fill
10. Total settlement along Ramp C at 75 years after construction with lightweight fill
11. Total settlement along Spur C at 8 months after conventional fill placement (end of preloading period)
12. Total settlement along Spur C at 75 years after conventional fill placement
13. Graph of anticipated time rate of total settlement over 75 years after construction
14. Graph of anticipated time rate of total settlement over initial two years after construction
15. Graph of anticipated time rate of primary consolidation over 75 years after construction
16. Graph of anticipated time rate of primary consolidation over initial two years after construction

ASSUMPTIONS

1. Settlement results were compared against the limiting settlement criteria provided by MaineDOT (Refs. 12 and 13), which are listed below:

- Limiting Settlement for Highway Embankments more than 200 feet from Bridge
  - Less than 4 inches of settlement from 0 years after construction to 20 years after construction
  - Less than an additional 4 inches of settlement from 20 years after construction to 75 years after construction (i.e., less than 8 inches total at 75 years)
- Limiting Settlement for Highway Embankments within 200 feet of Bridge
  - Less than 2 inches of settlement from 0 years after construction to 5 years after construction
  - Less than an additional 2 inches of settlement from 5 years after construction to 20 years after construction (i.e., less than 4 inches total at 20 years)
  - Less than an additional 2 inches of settlement from 20 years after construction to 75 years after construction (i.e., less than 6 inches total at 75 years)
- Limiting Settlement for Approach Slab
  - Less than 1 inch of settlement from 0 years after construction to 5 years after construction

2. The proposed staged construction of the ramp embankments and bridge abutments provided by MaineDOT (Ref. 14) was incorporated into the Settle3 models to analyze relative time-rates of settlement throughout the interchange. Settlement was evaluated up to 75 years (the anticipated design life of the bridges) after construction of the final ramp. Intermediate evaluation stages at 5 years and 20 years after construction of each ramp or abutment were also modeled in order to compare settlement results for each ramp or abutment against MaineDOT's limiting settlement criteria.

CALCULATIONS

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Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. WSP initially evaluated settlement assuming the embankments would be constructed with conventional fill consisting of granular borrow. This resulted in estimated settlement amounts of up to 7 inches in the first 5 years after construction (Ref. 15), exceeding the limiting settlement criteria. After discussion with MaineDOT, WSP revised the models to incorporate settlement mitigation. The analyzed mitigation methods consisted of preloading with conventional fill where construction staging permitted, and embankment construction with lightweight fill (foamed glass aggregate) where construction staging prevented the use of preloading.

CALCULATION

1. Establish existing and proposed construction geometry to build the loading models in Settle3.

a. WSP's Interpretive Subsurface Profiles A-A', B-B', and C-C' along the Hogan Road Westbound proposed construction centerline and Interpretive Subsurface Profiles D-D' and E-E' along the Ramp B proposed construction centerline (Ref. 2) were imported into the interchange and ramp Settle3 models, respectively, and extended east and west of each centerline to create the three-dimensional soil layering in each model. The interpreted soil stratigraphy along the westbound centerline, where a larger thickness of soft clay was encountered in the borings at site, was conservatively used for the Hogan Road Eastbound model.

b. The existing grade provided by MaineDOT along the Hogan Road and Ramp B proposed construction centerlines (Ref. 3) was imported into the Settle3 models and extended east and west of the centerlines to create the three-dimensional existing ground surface.

c. To model the proposed embankment fills, the proposed grade provided in the project plan cross-sections (Ref. 4) was imported into the Settle3 models in 50-foot-long segments centered around the X+00 and X+50 stations on the Hogan Road Westbound and Eastbound, Ramp A, Ramp B, Ramp C, Spur B, and Spur C centerlines. Each cross-section was extended north and south of the X+00 or X+50 station in order to model the proposed fill thickness as a continuous embankment. An embankment fill unit weight of 125 pcf was assumed to represent the loading from embankment construction along Hogan Road Westbound and the ramps, based on the granular borrow unit weight recommended in the MaineDOT Bridge Design Guide (Ref. 5, Table 3-3). An aggregate subbase unit weight of 135 pcf was assumed to represent the loading from the proposed grade raise along Hogan Road Eastbound, based on the gravel borrow unit weight recommended in the MaineDOT Bridge Design Guide (Ref. 5, Table 3-3). For embankment areas proposed to be constructed using lightweight fill for settlement mitigation, a foamed glass aggregate unit weight of 20 pcf was assumed.

d. Proposed abutment foreslope angles were modeled for Hogan Road Westbound Abutments 1 and 2 based on the proposed grading contours shown in Ref. 6, Sheet B2. A fill unit weight of 125 pcf was assumed to represent the loading from foreslope construction, based on the granular borrow unit weight recommended in the MaineDOT Bridge Design Guide (Ref. 5, Table 3-3).

e. Loading in the models was defined as flexible, in which stress distribution on the ground immediately beneath the load is equal to the loading stress distribution, rather than rigid, in which displacement immediately beneath the load remains planar.

CALCULATIONS

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Subject:	Settlement Analysis with Lightweight Fill - Highway	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

2. Determine material parameters to build the soil models in Settle3.

a. For this analysis, it was assumed that the topsoil layer will be removed prior to construction and replaced with fill material having soil properties similar to granular borrow in the MaineDOT Bridge Design Guide (Ref. 5, Table 3-3). This replacement layer was modeled as part of the existing fill in this analysis.

b. The fill, sand & gravel, glaciofluvial, and glacial till layers were assumed to undergo immediate settlement only. The clay layers were assumed to undergo immediate, primary consolidation, and secondary consolidation settlement.

c. The silt layer was analyzed for immediate settlement only using drained material properties. The silt material was encountered near existing roadway fill areas and is assumed to be fill material from the construction of the existing roadways having undergone compaction from the construction process which will limit consolidation settlement amounts.

d. The void ratio, compression index, recompression index, overconsolidation ratio, and coefficient of consolidation for the clay layers are based on consolidation lab testing results from undisturbed thin-wall tube samples collected in the borings at site (Ref. 10). Only the Incremental Loading Consolidation (ILC) test data was used from the highway borings (HB-series). The Rate of Strain Consolidation (CRS) test data for the highway borings was found to be not in compliance with ASTM standards, and thus the data was deemed to be unreliable and was not used in the determination of the clay parameters.

Boring	Sample	Test Type	e_0	C_{c1}	C_{c2}	C_r
BB-BHR-101	1U	ILC	0.842	0.25	-	0.04
BB-BHR-101	1U	CRS	0.819	0.20	-	0.01
BB-BHR-103	1U	CRS	0.789	0.20	0.16	0.01
HB-BHR-101	1U	ILC	0.855	0.15	0.24	0.02
HB-BHR-102	1U	ILC	0.762	0.08*	0.13	0.01
HB-BHR-108	1U	ILC	0.792	0.09*	0.18	0.01
		Standard Deviation	0.035	0.04		0.01
		Average	**0.810**	**0.19**		**0.02**

* Values deemed to be outliers and therefore not used in the calculation of averages or standard deviations.

Boring	Sample	Test Type	C_a	C_{ar}	c_v (in^2/s)
BB-BHR-101	1U	ILC	0.0066	0.0037	0.0003
BB-BHR-101	1U	CRS	-	-	0.0090
BB-BHR-103	1U	CRS	-	-	0.0070
HB-BHR-101	1U	ILC	0.0012	0.0002	0.0006
HB-BHR-102	1U	ILC	0.0011	0.0003	0.0013
HB-BHR-108	1U	ILC	0.0014	0.0004	0.0007
		Std Dev.	0.0027	0.0017	0.0038
		Average	**0.0026**	**0.0012**	**0.0032**

An OCR value of 2.6 was used for the clay layers based on CRS test data from the bridge borings (BB-series).

CALCULATIONS

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Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

e. The elastic soil moduli E_s for the fill, silt, sand & gravel, glaciofluvial, and glacial till layers are based on empirical correlation (Ref. 7, Eqn. 2.29) to an N_{60}-value back-calculated using Ref. 7, Eqn. 2.26 from the design friction angle for each layer, which in turn is based on the individual N_{60} values encountered in borings HB-BHR-101 through HB-BHR-217 (Ref. 8).

The elastic soil moduli for the clay layers are based on empirical correlation (Ref. 7, Eqn. 5.31) to the undrained shear strength (based on field vane shear testing summarized in Ref. 9, laboratory testing summarized in Ref. 10, and correlation from SPT and plasticity values summarized in Refs. 8 and 10), the plasticity index (based on index lab test results, Ref. 10), and the overconsolidation ratio (based on consolidation lab test results, Ref. 10).

$$\frac{E_s}{p_a} = \alpha N_{60} \qquad \text{(Ref. 7, Eqn. 2.29)}$$

where:

p_a = atmospheric pressure = 2.12 ksf

α = 5 for sands with fines; 10 for clean normally consolidated sand; 15 for clean overconsolidated sand

Soil Layer	N_{60}	Layer Description	α	E_s (ksf)
Fill	24	Granular Borrow (assumed)	5	254
Sand & Gravel	39	Dense to very dense silty/gravelly fine to coarse sand	5	413
Silt (drained)	17	Medium stiff to very stiff silt with trace to some sand	5	180
Glaciofluvial	64	Very dense silty sand and gravel	5	677
Glacial Till	39	Dense gravel with some sand/silt Hard gravelly/sandy silt	5	413

$$E_s = \beta s_u \qquad \text{(Ref. 7, Eqn. 5.31)}$$

where β is a function of the plasticity index and the overconsolidation ratio as per Ref. 7, Table 5.7.

Soil Layer	Plasticity Index	OCR	β	s_u (ksf)	E_s (ksf)
Stiff to Very Stiff Clay	16	2.6	920	1.343	1236
Soft to Medium Stiff Clay	14	2.6	920	0.913	840

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Highway	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

f. The at-rest earth pressure coefficient for each soil layer is based on empirical correlation (Ref. 7, Eqn. 7.3 and 7.4) to the friction angle or the OCR determined for each soil layer. The friction angle is based on empirical correlation (Ref. 7, Eqn. 2.26) to the average of the N_{60}-values encountered in borings HB-BHR-101 through HB-BHR-217 for each layer (Ref. 8), and the OCR is based on consolidation lab testing results (Ref. 10).

For normally consolidated soil:

$$K_0 = 1 - \sin\phi' \qquad \text{(Ref. 7, Eqn. 7.3)}$$

Soil Layer	Design effective friction angle, ϕ' (°)	At-rest earth pressure coefficient, K_0
Fill	34	0.44
Sand & Gravel	38	0.38
Silt (drained)	32	0.47
Glaciofluvial	44	0.31
Glacial Till	38	0.38

For overconsolidated soil:

$$K_0 = (1 - \sin\phi')OCR^{\sin\phi'} \qquad \text{(Ref. 7, Eqn. 7.4)}$$

Soil Layer	Design effective friction angle, ϕ' (°)	OCR	At-rest earth pressure coefficient, K_0
Stiff to Very Stiff Clay	32	2.6	0.78
Soft to Medium Stiff Clay	29	2.6	0.82

g. The unit weight parameter for the fill (granular borrow) was selected from Table 3-3 of Ref. 5. The unit weight parameters for the silt, sand & gravel, glaciofluvial, and glacial till layers were selected based on local engineering experience. The unit weight parameter for the clay layers was based on consolidation lab testing results (Ref. 10).

Soil Layer	Unit Weight (pcf)
Fill	125
Sand & Gravel	125
Silt	125
Stiff to Very Stiff Clay	122
Soft to Medium Stiff Clay	121
Glaciofluvial	130
Glacial Till	130

h. The unit weight parameter for the metawacke bedrock encountered at site was selected as 171 pcf based on laboratory testing conducted on rock core samples from nearby Hogan Road Bridge borings BB-BHR-201 and -206 and probes BRP-BHR-202 and -203 (Ref. 11).

i. The clay layers were assumed to exhibit excess pore pressure; thus, the Skempton pore pressure coefficient (B-bar) was set equal to 1 for both materials.

j. The fill and sand & gravel layers, which overlie the silt and glacial till layers, respectively, were assumed to be drained at the bottom. The silt and glacial till layers, which overlie the clay and bedrock layers, respectively, were assumed to not be drained at the bottom.

RESULTS

The total settlements anticipated along the centerlines of Ramp A, Ramp B, Ramp C, Spur B, and Spur C are summarized in Table 1 below. Settlement values are provided at 75 years after construction of each ramp or spur, which is the anticipated life of the bridge. Model output is provided in Attachment Figures 1 - 16.

- Images of the model geometry, soil stratigraphy, and loading are shown in Attachment Figures 1 - 6.
- Images of the model results for the highway embankments are shown in Attachment Figure 7 (Ramp A), Attachment Figure 8 (Ramp B), Attachment Figure 9 (Spur B), Attachment Figure 10 (Ramp C), and Attachment Figures 11 and 12 (Spur C).
- Plots of the time rate of settlement at point of maximum settlement for each modeled feature are shown in Attachment Figures 13 - 16.

Ramps A and B were modeled with granular borrow conventional fill, since these areas had smaller proposed embankment fills and were further from the bridge abutments.

- At the centerline of Ramp A with conventional fill the total settlement ranges from 0.3 to 2.5 inches.
- The total settlement at the centerline of Ramp B with conventional fill in the area of the largest Ramp B embankment fills (Station 806+00 to 810+50) ranges from 1.5 to 2.0 inches.

Spur B and Ramp C had larger proposed embankment fills and were closer to the bridge abutments. In order to satisfy MaineDOT's limiting settlement criteria (Assumption 1), the analysis indicated that Spur B and Ramp C would require settlement mitigation. The proposed construction staging and proposed location of temporary traffic detour ramps meant that there would be insufficient time available for preloading during construction. Instead, the embankments in these areas were proposed to be constructed of lightweight fill consisting of foamed glass aggregate. The proposed thickness of lightweight fill in each embankment was varied in the models to reduce estimated total settlement below the limiting settlement criteria.

- The total settlement at the centerline of Spur B with lightweight fill ranges from 0.0 to 1.9 inches.
- At the centerline of Ramp C with lightweight fill, the total settlement ranges from 0.3 to 1.5 inches.

At Spur C, no temporary traffic detour ramps are proposed in the construction staging, and a period of 8 months would be available for preloading at Spur C while construction progressed on the northern and eastern sides of the interchange. Since the model predicted that the estimated settlement at Spur C would occur quickly after embankment construction, with 95% primary consolidation in less than one month, the proposed settlement mitigation consists of preloading the area with conventional fill for the 8-month period. The proposed embankments at Spur C were assumed in the model to be constructed of granular borrow conventional fill.

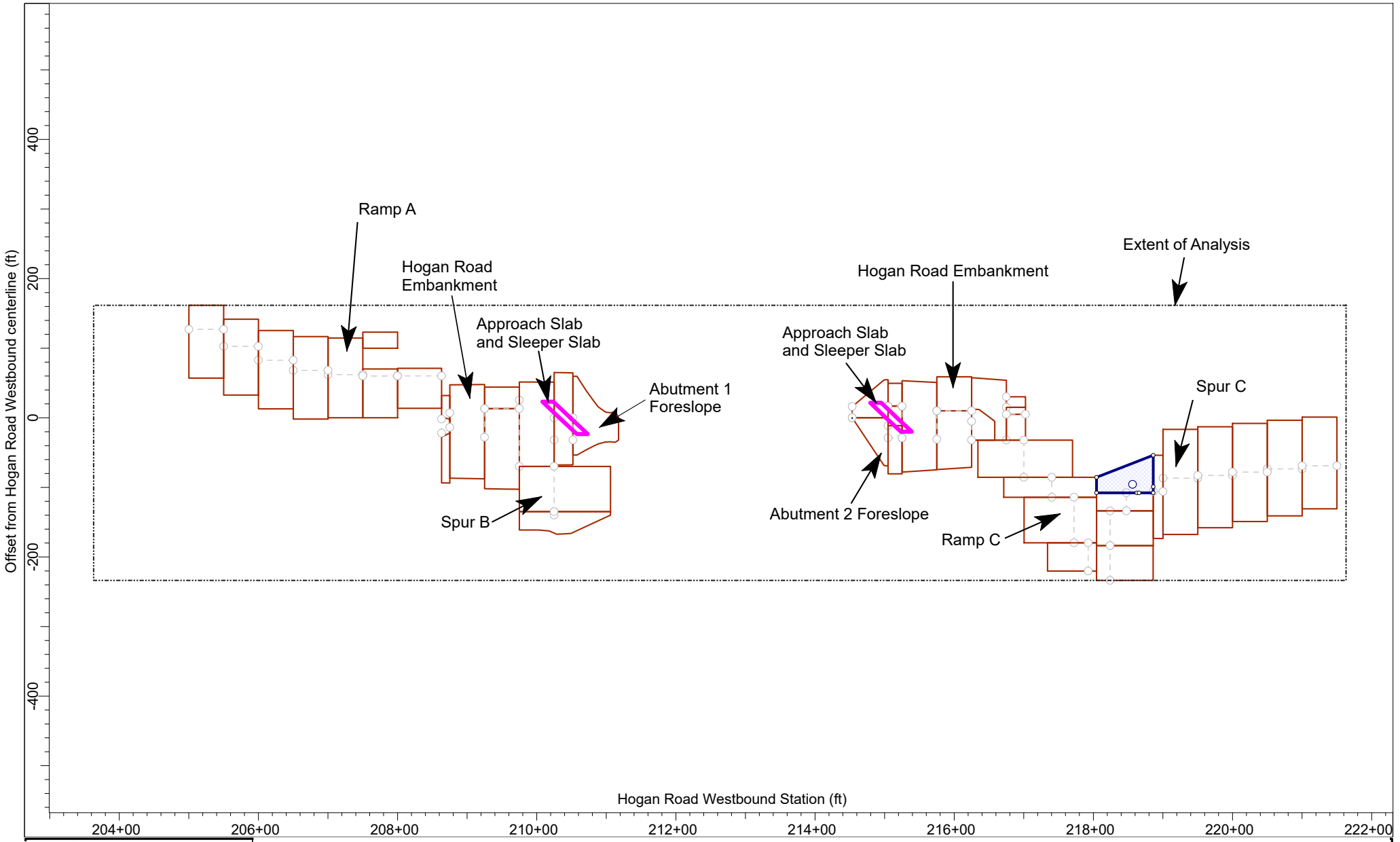
- At the centerline of Spur C with conventional fill preloading, the total settlement ranges from 0.3 to 6.6 inches.

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.003	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Highway	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

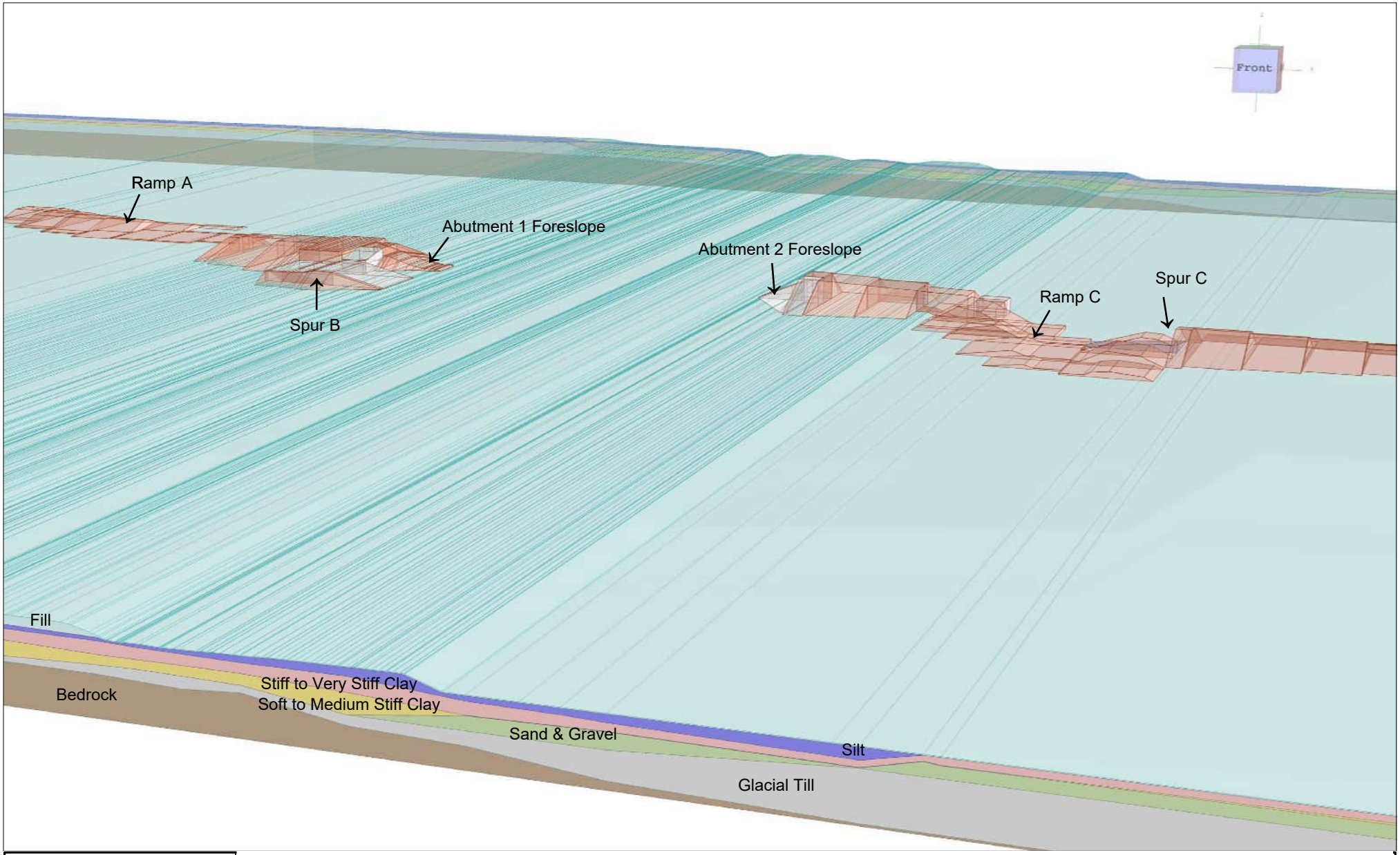
Table 1: Summary of Settlement Analysis Results along Centerlines of Ramps A, B, and C and Spurs B and C

Structure		Ramp A	Ramp B	Spur B	Ramp C	Spur C
Elevation (ft)		105.4	98.9	99.3	82.3	82.3
Feature Length (Modeled) (ft)		373	1475	103	211	435
Total Settlement (in.)	Maximum point	2.5	2.0	1.9	1.5	6.6
	Minimum point	0.3	1.5	0.0	0.3	0.3
	Attachment number	#7	#8	#9	#10	#12
Immediate Settlement (in.)	Maximum point	1.0	1.8	0.4	0.7	2.9
	Minimum point	0.0	1.0	0.0	0.1	0.2
Primary Consolidation Settlement (in.)	Maximum point	1.4	0.3	1.3	0.7	4.4
	Minimum point	0.1	0.1	0.0	0.0	0.1
Secondary Consolidation Settlement (in.)	Maximum point	0.2	0.2	0.3	0.2	0.2
	Minimum point	0.0	0.0	0.0	0.1	0.1



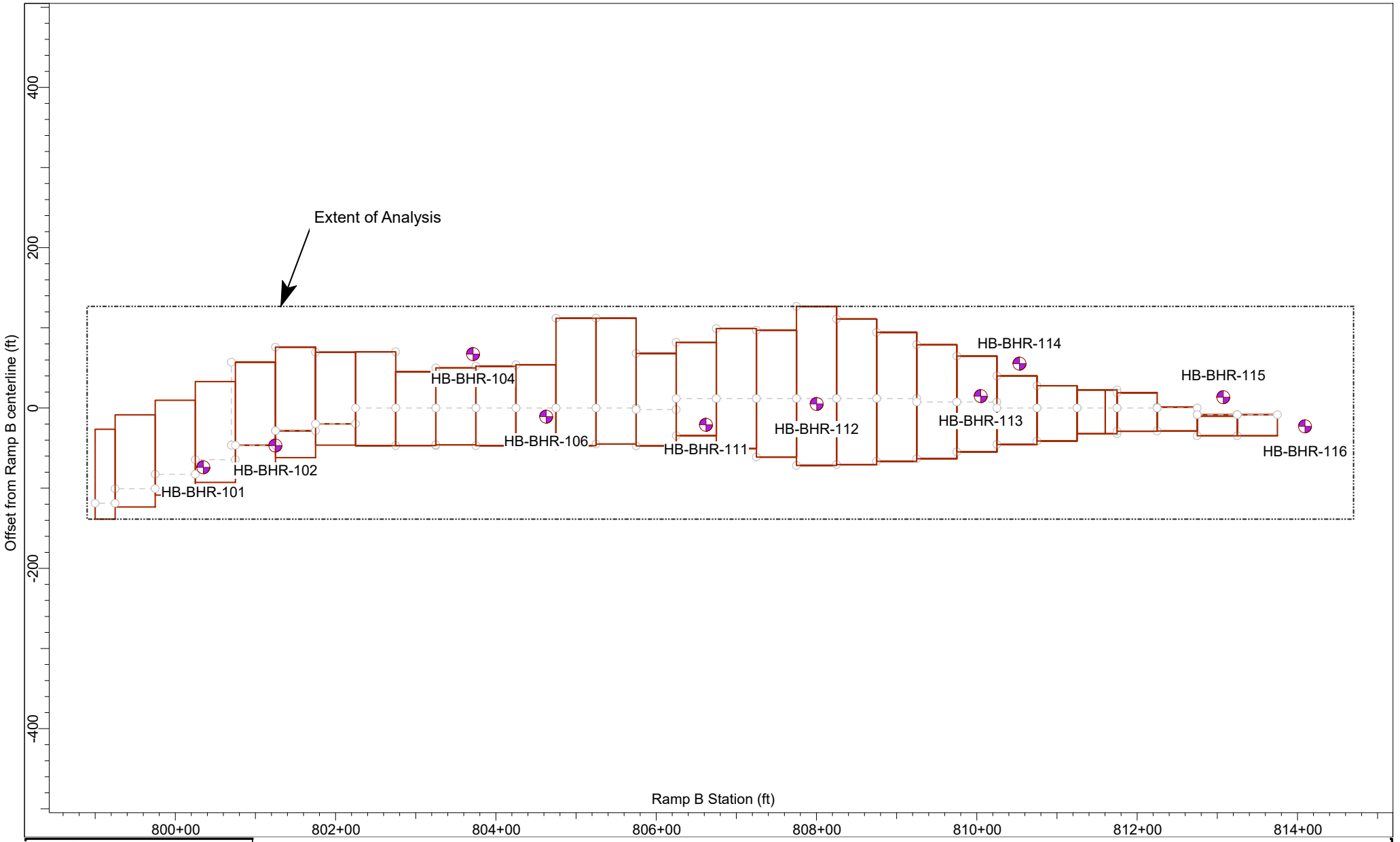
Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Plan View of Loading in Settle3 Model (Highway Interchange)			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 1**
Date: 5/29/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading.s3z			

SETTLE3 5.025



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	3D View of Soil Stratigraphy and Loading in Settle3 Model (Highway Interchange)			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 2**
Date: 6/10/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

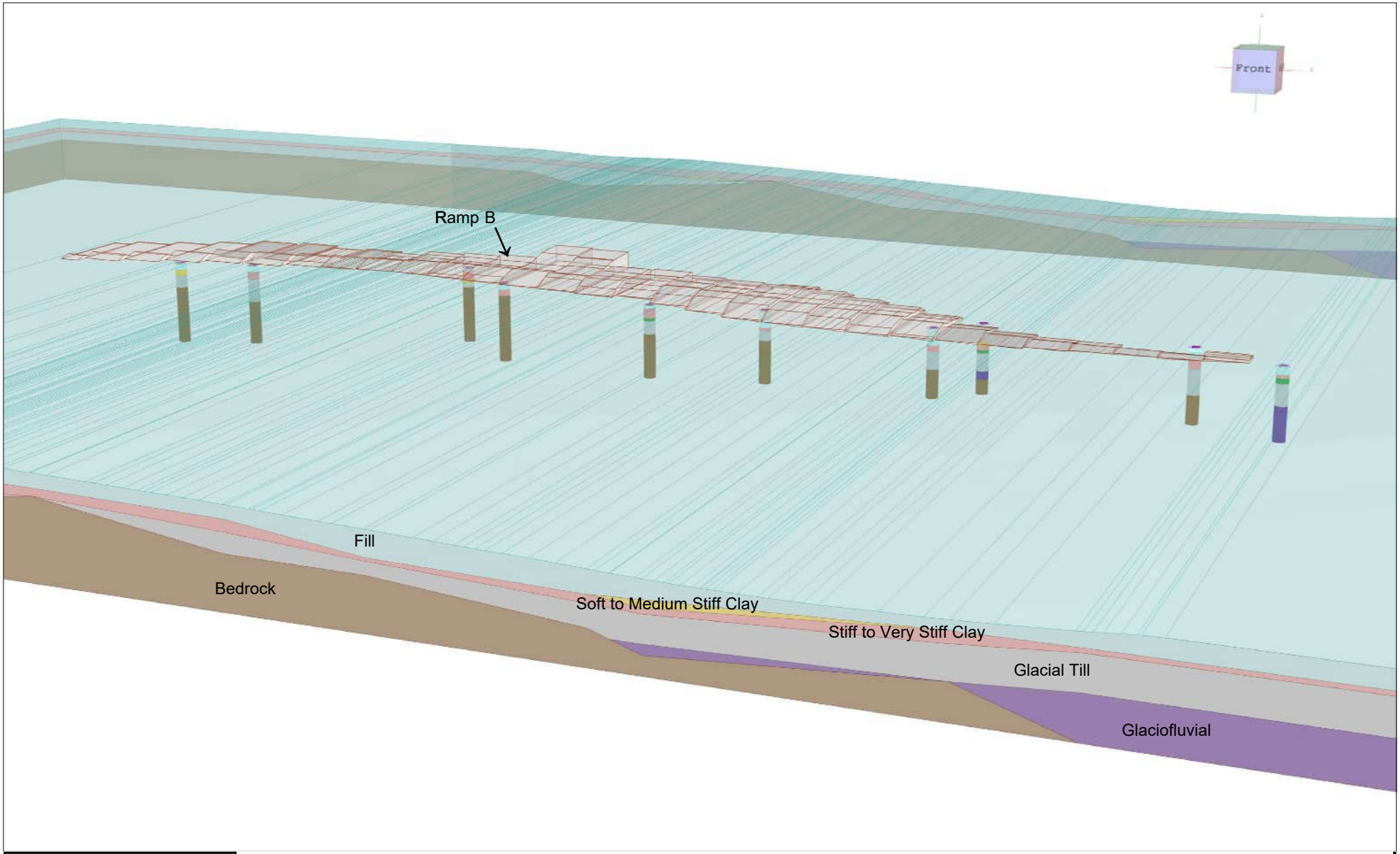
SETTLE3 5.025



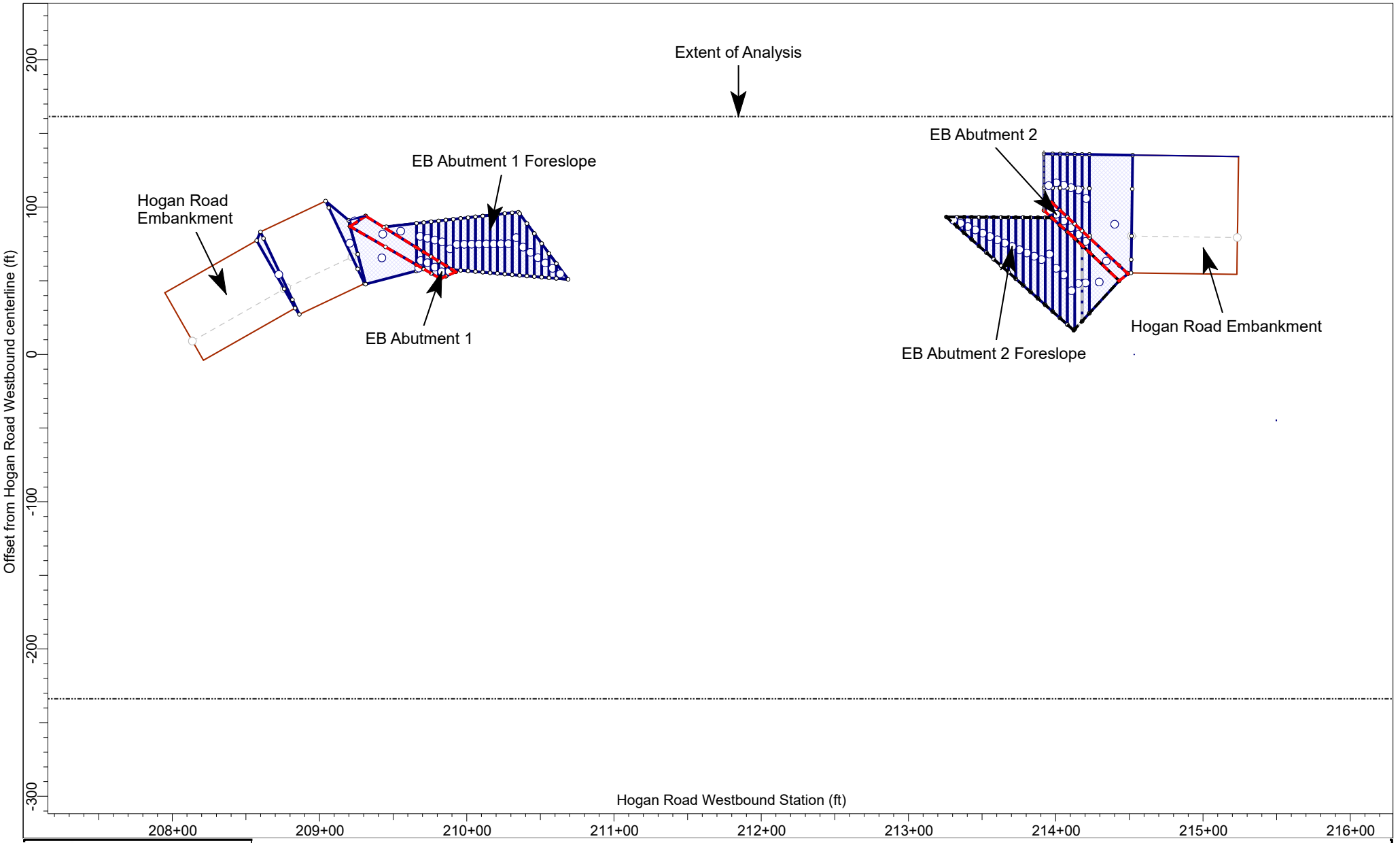
wsp

Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Plan View of Loading in Settle3 Model (Ramp B)			
Drawn By KAR	*Checked By* LMP	*Reviewed By* CCB	*Scale* as shown	**Figure 3**
Date 3/20/2025	*File Name* Hogan Rd Highway Ramp B Phase2 Jan2024.s3z			

SETTLE3 5.025

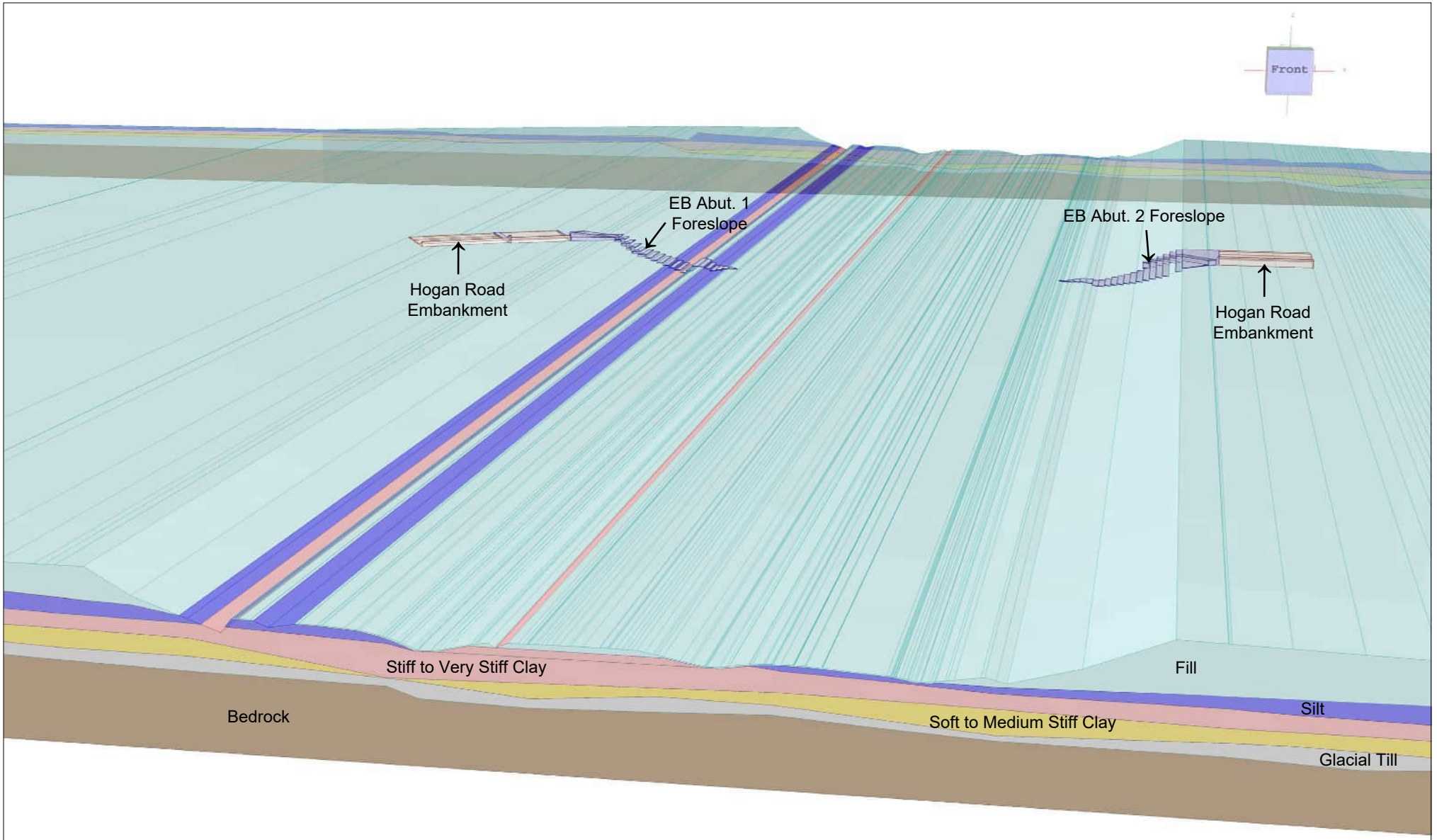


wsp	Project: MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
	Analysis Description: 3D View of Soil Stratigraphy and Loading in Settle3 Model (Ramp B)			
	Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale:
SETTLE3 5.025	Date: 3/20/2025	File Name: Hogan Rd Highway Ramp B Phase2 Jan2024.s3z		**Figure 4**

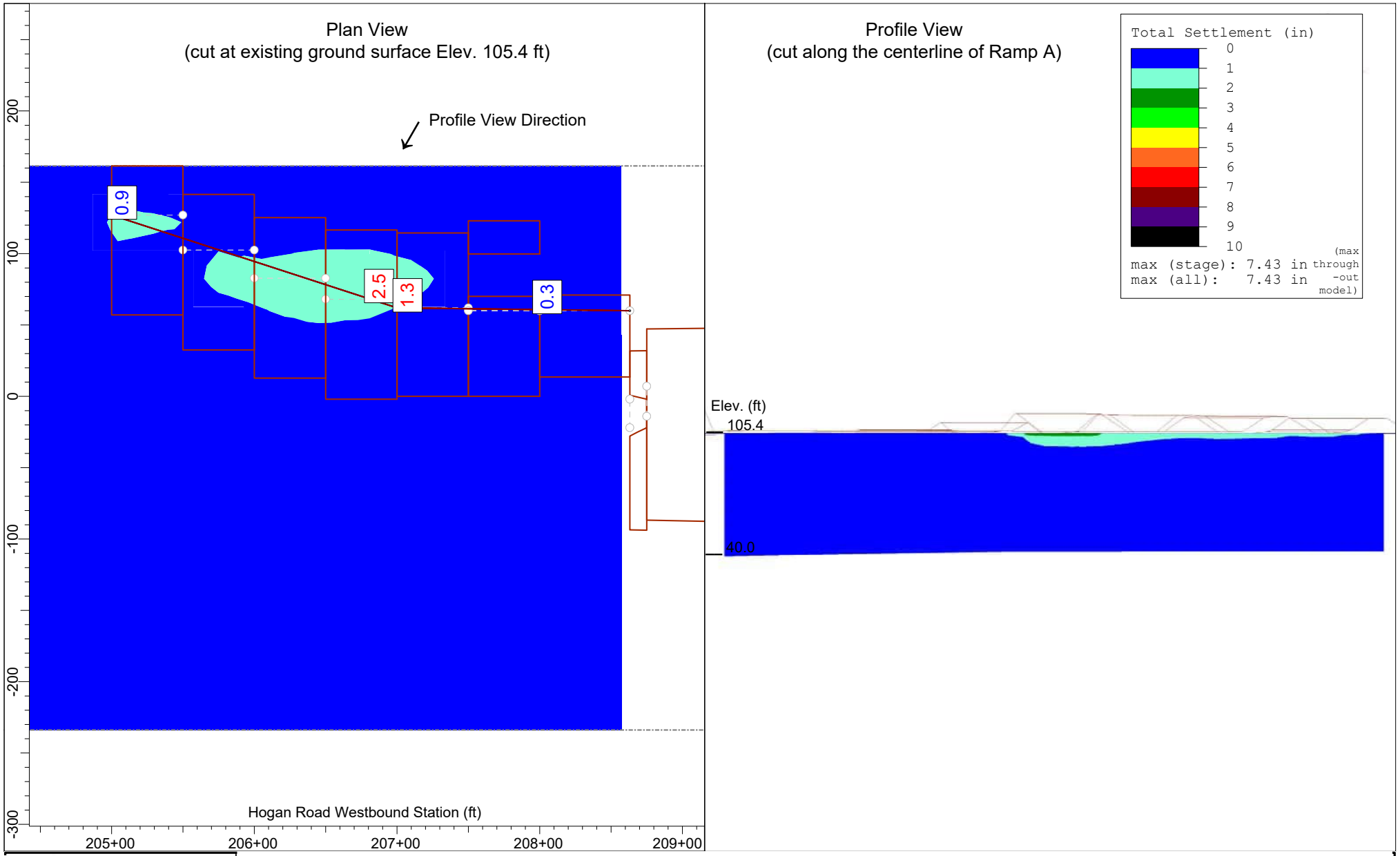


Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Plan View of Loading in Settle3 Model (Hogan Road Eastbound)			
Drawn By: LMP	Checked By: KAR	Reviewed By: CCB	Scale: as shown	**Figure 5**
Date: 3/20/2025	File Name: Hogan Rd Highway Interchange Phase2 EB V1 - Foreslope - with query points.s3z			

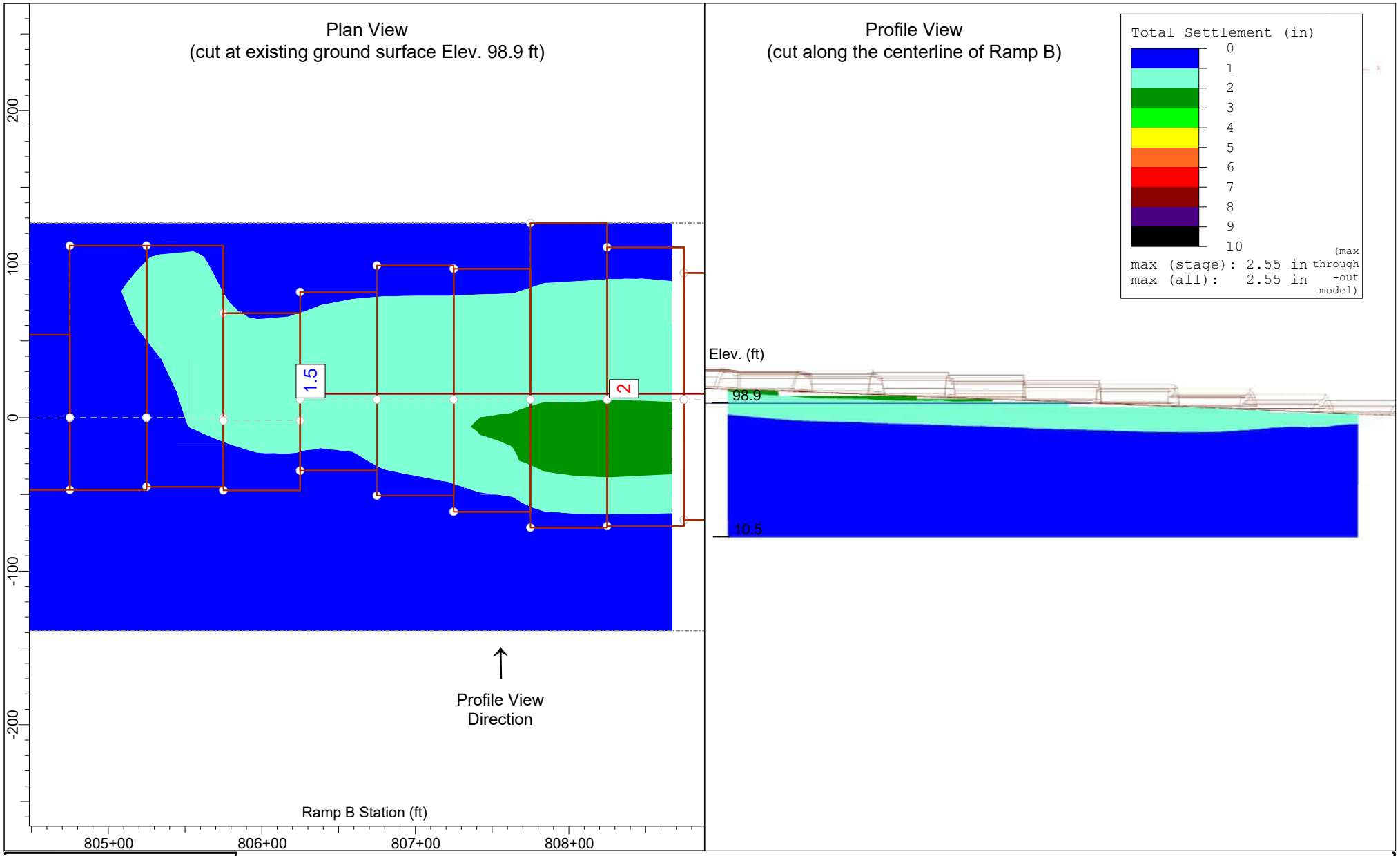
SETTLE3 5.025



wsp	Project: MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
	Analysis Description: 3D View of Soil Stratigraphy and Loading in Settle3 Model (Hogan Road Eastbound)			
	Drawn By: LMP	Checked By: KAR	Reviewed By: CCB	Scale:
SETTLE3 5.025	Date: 3/20/2025	File Name: Hogan Rd Highway Interchange Phase2 EB V1 - Foreslope - with query points.s3z		**Figure 6**

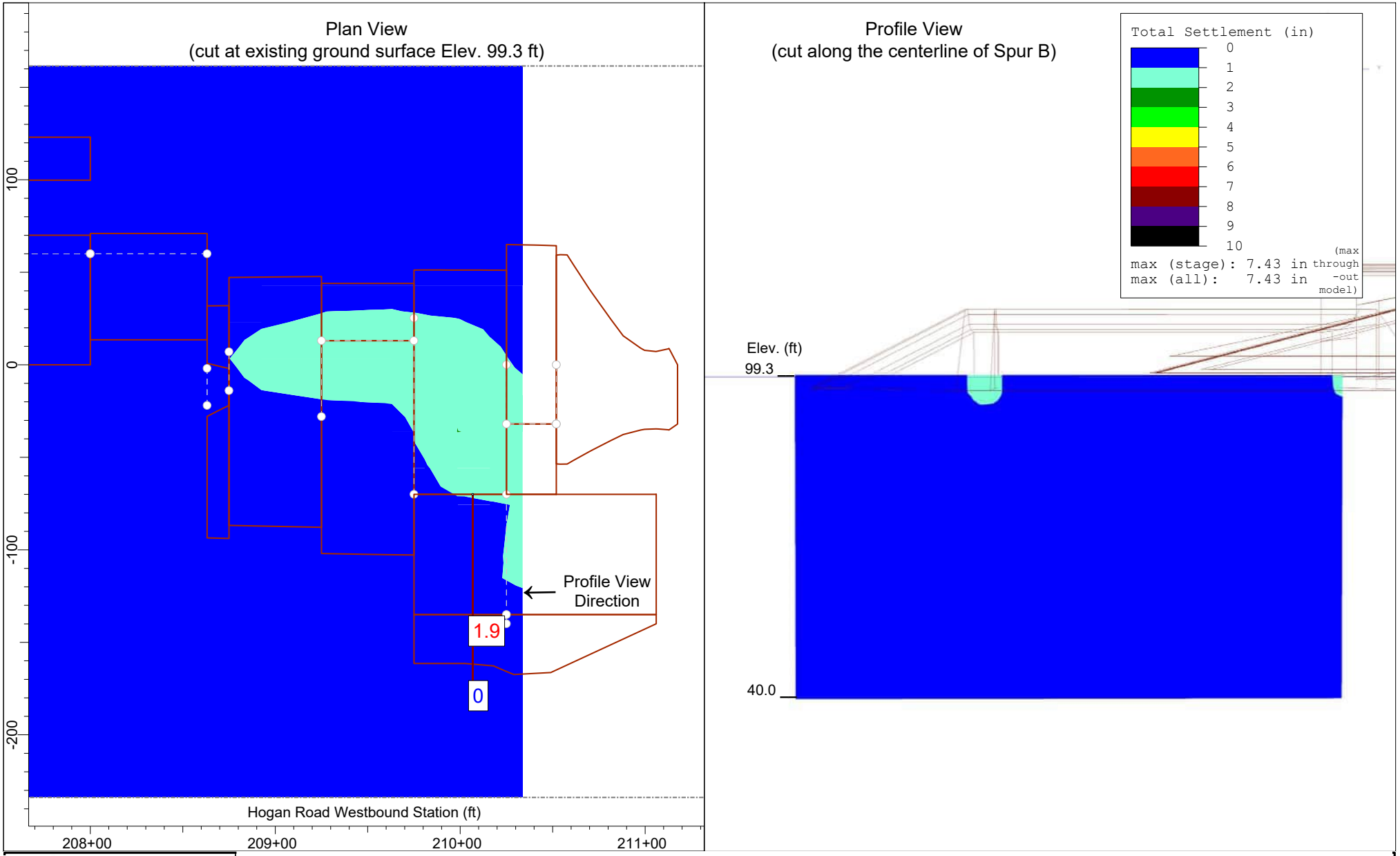


	Project: MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
	Analysis Description: Total Settlement along Ramp A at 75 Years after Conventional Fill Placement			
	Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown
SETTLE3 5.025	Date: 3/18/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF Feb2025 w slab & staging & grading.s3z		**Figure 7**



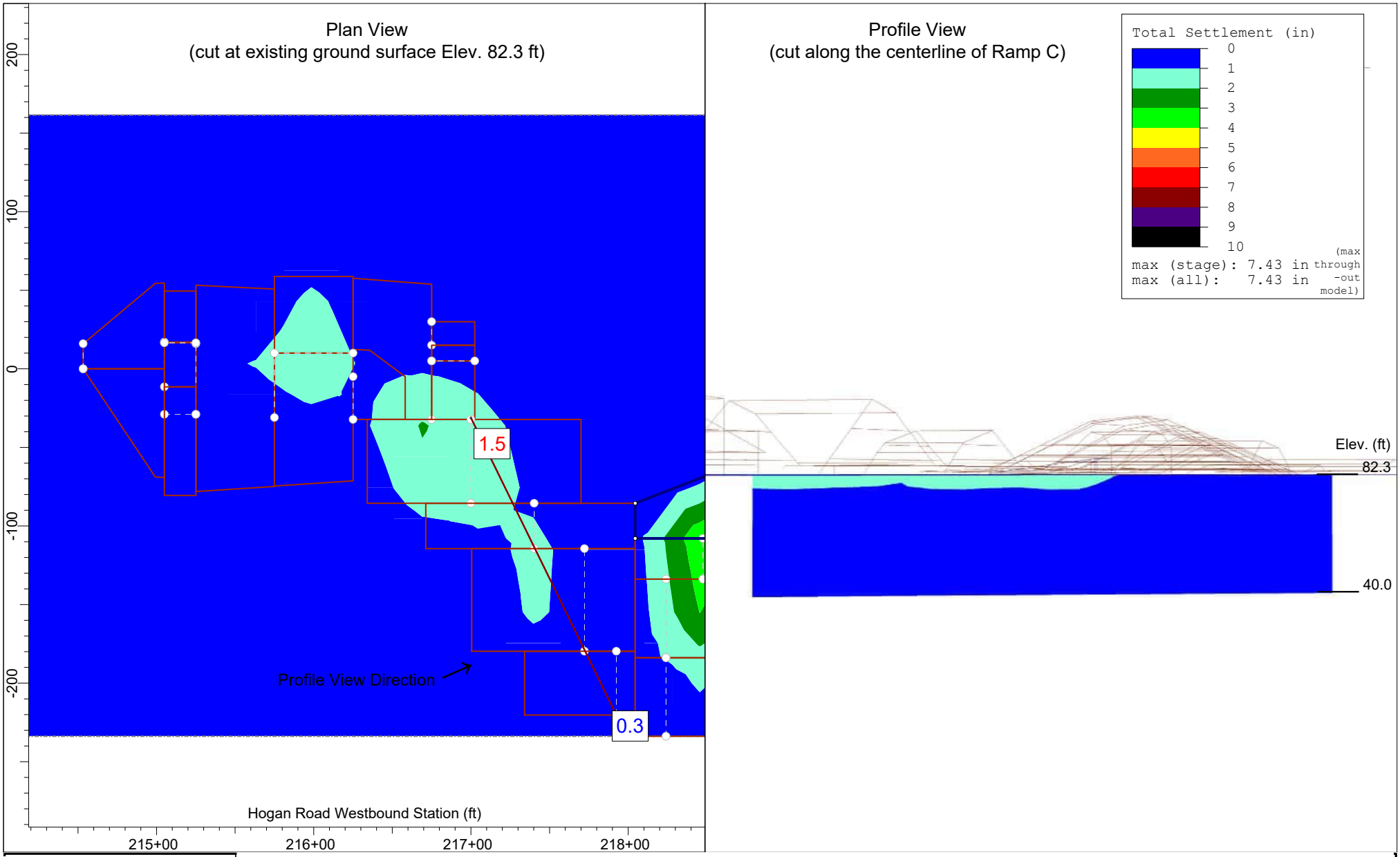
Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Total Settlement along Ramp B at 75 Years after Conventional Fill Placement			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 8**
Date: 3/19/2025	File Name: Hogan Rd Highway Ramp B Phase2 Jan2024.s3z			

SETTLE3 5.025



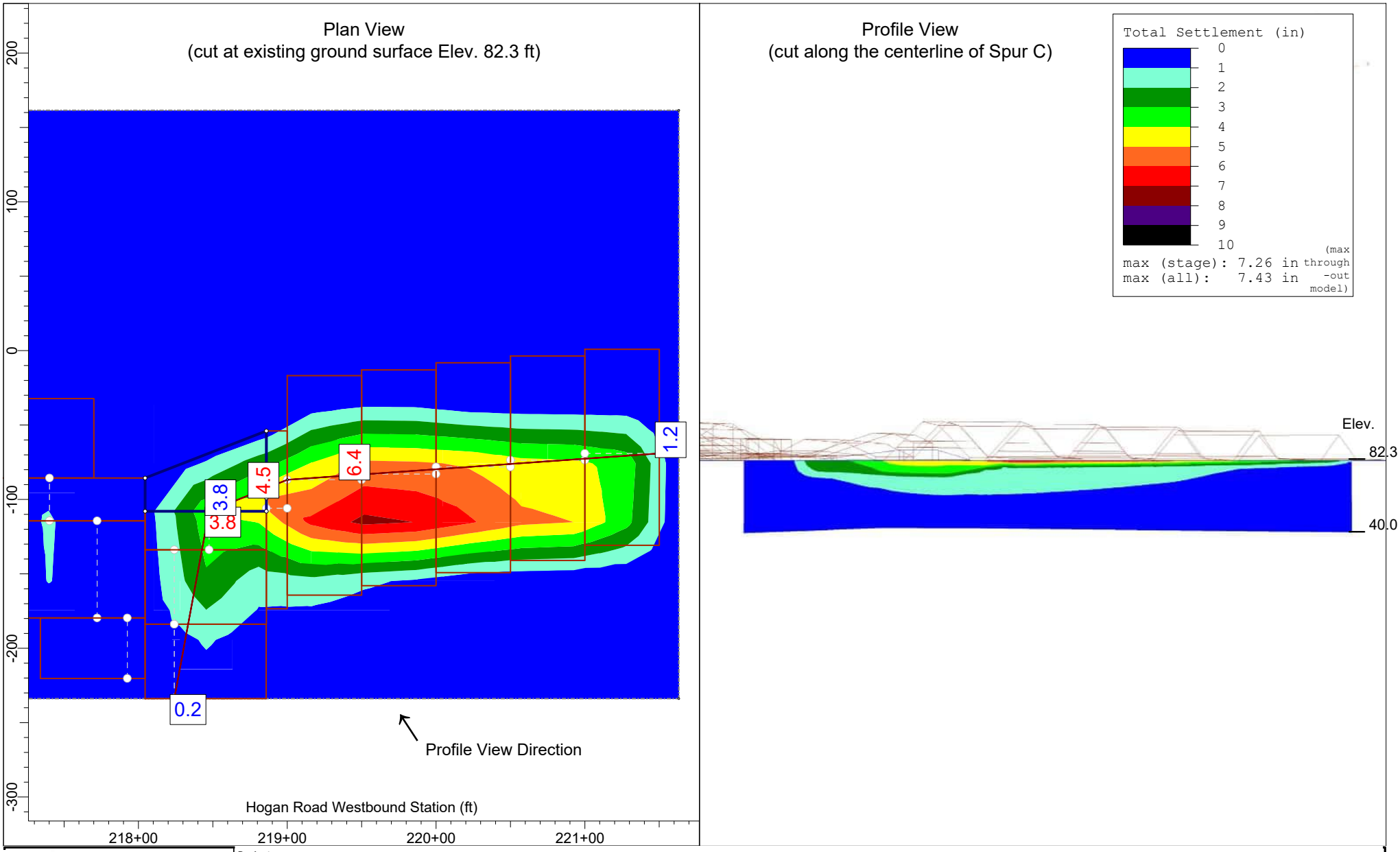
Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Total Settlement along Spur B at 75 Years after Lightweight Fill Placement			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 9**
Date: 6/10/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

SETTLE3 5.025



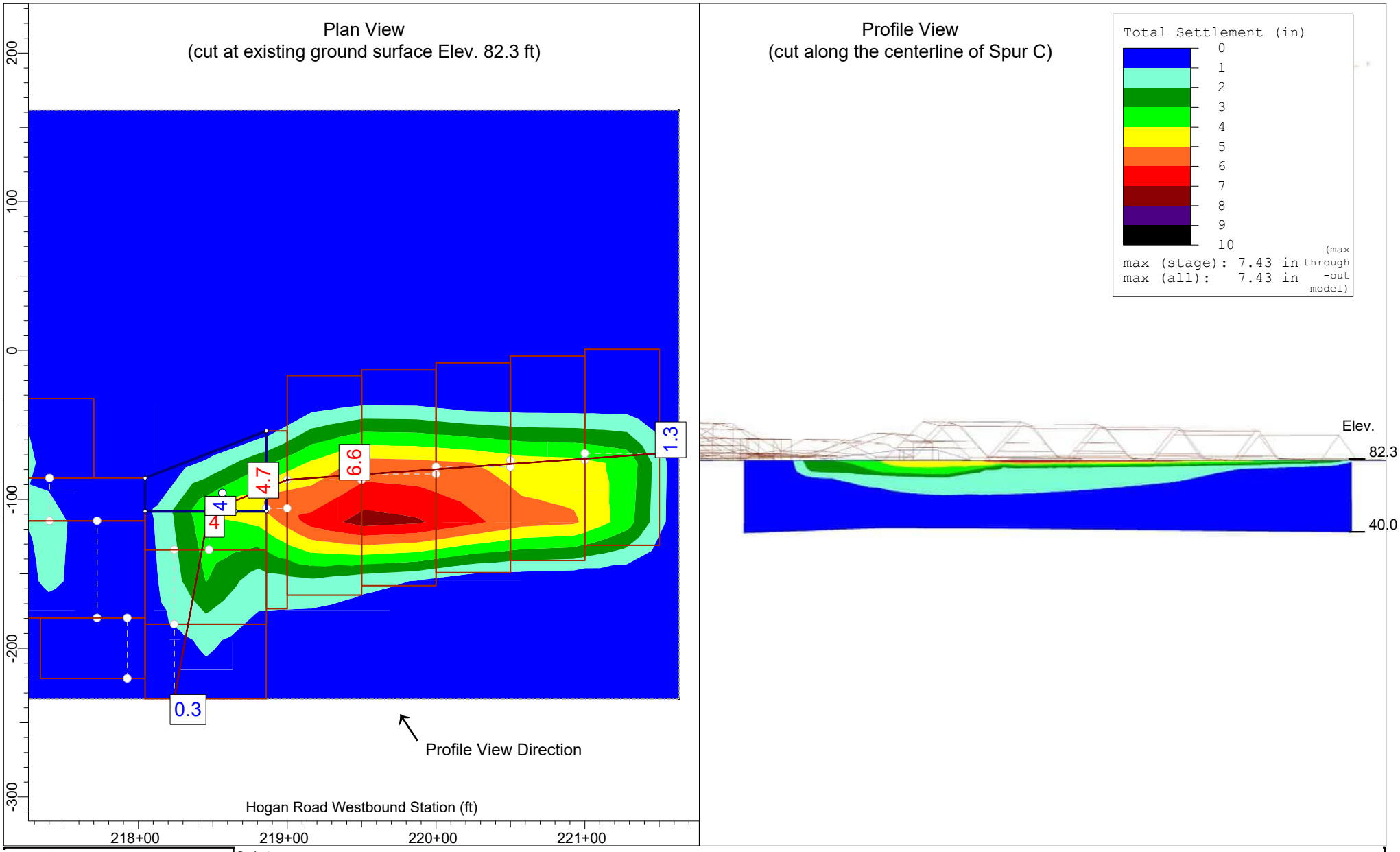
Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Total Settlement along Ramp C at 75 Years after Lightweight Fill Placement			
Drawn By KAR	*Checked By* LMP	*Reviewed By* CCB	*Scale* as shown	**Figure 10**
Date 5/29/2025	*File Name* Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading.s3z			

SETTLE3 5.025



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Total Settlement along Spur C at 8 Months after Conventional Fill Placement (End of Preloading Period)			
Drawn By KAR	*Checked By* LMP	*Reviewed By* CCB	*Scale* as shown	**Figure 11**
Date 6/10/2025	*File Name* Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

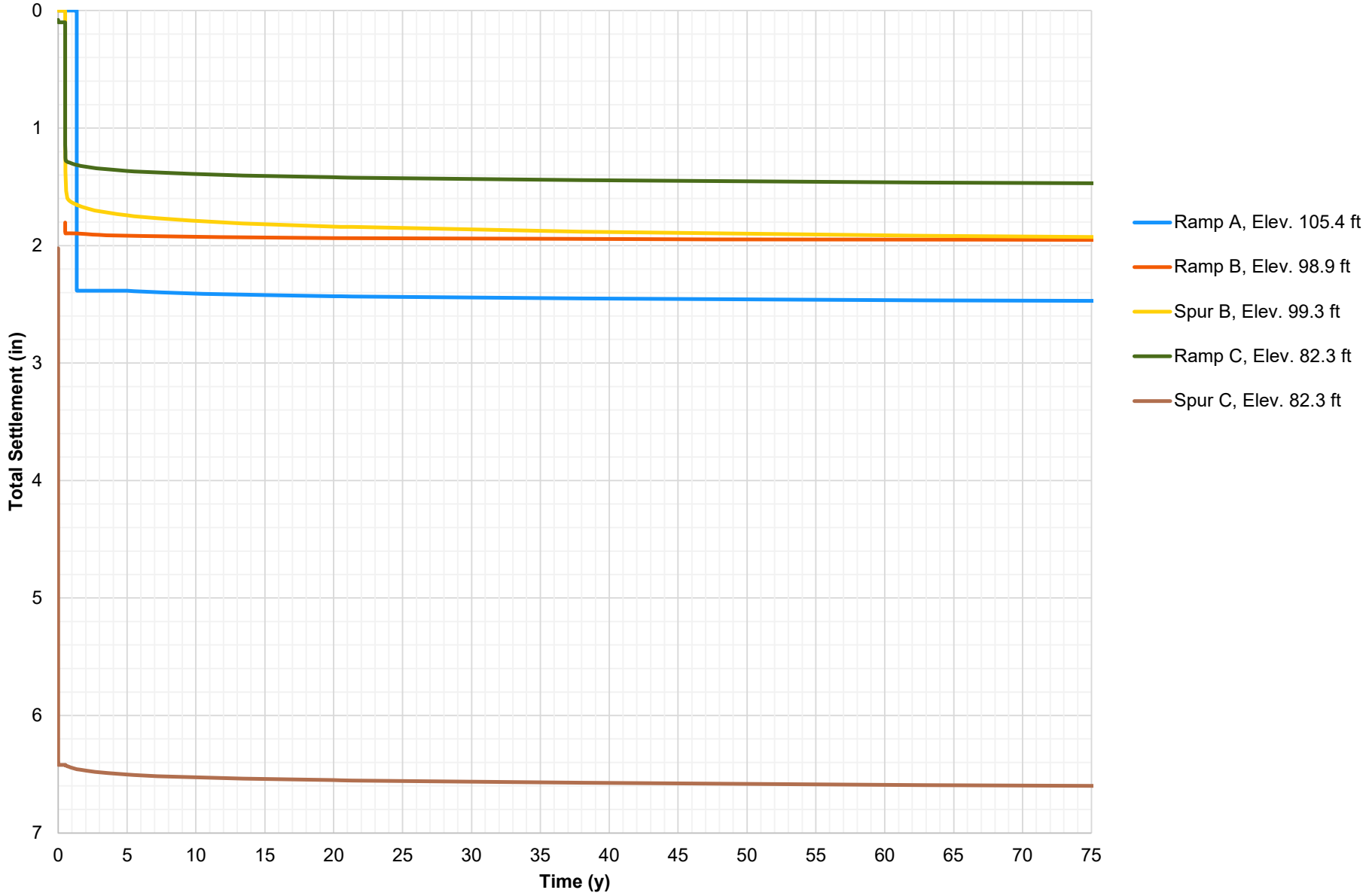
SETTLE3 5.025



Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Total Settlement along Spur C at 75 Years after Conventional Fill Placement			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 12**
Date: 6/10/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

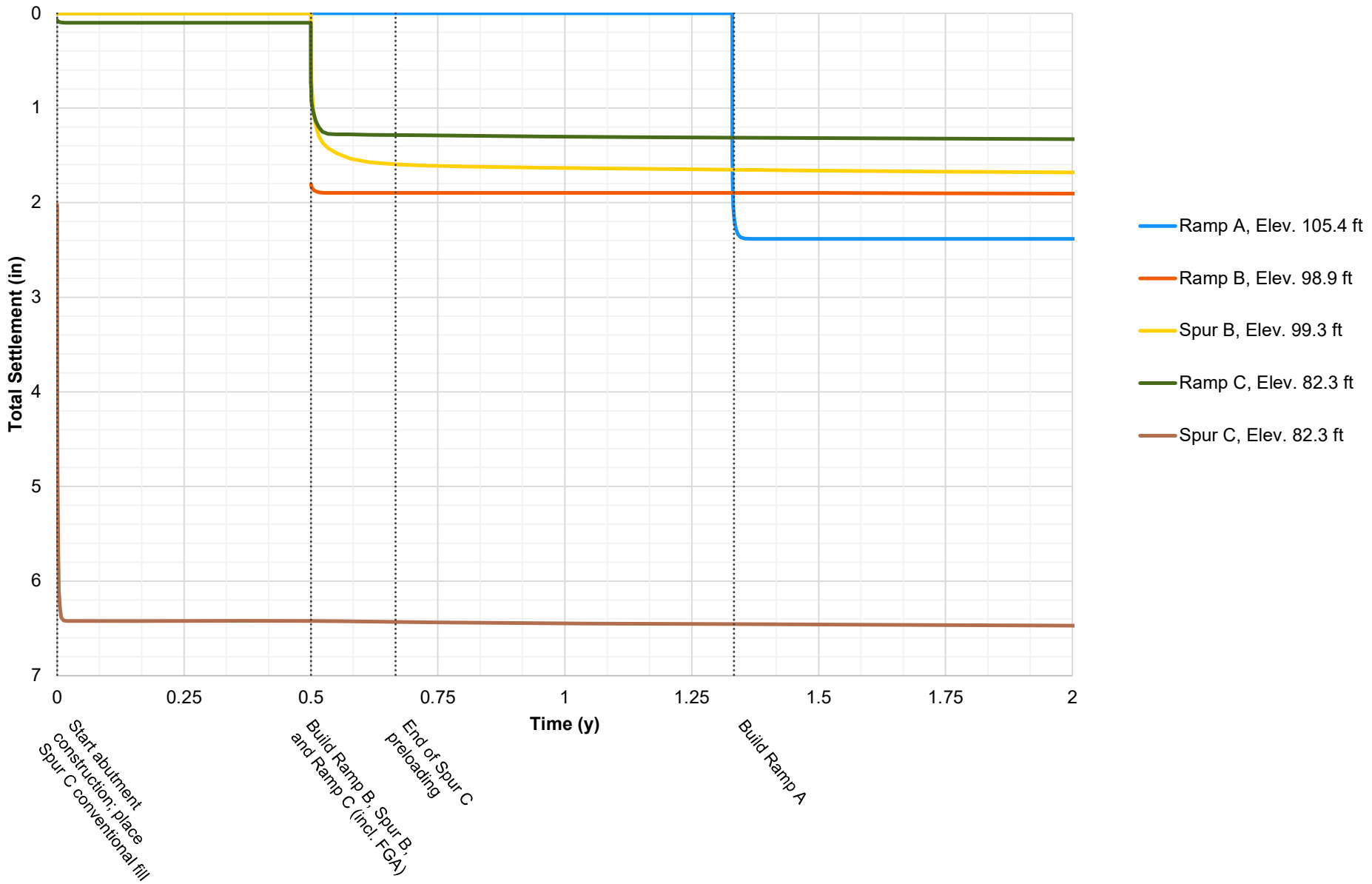
SETTLE3 5.025

Figure 13: Time Rate of Total Settlement over 75 Years



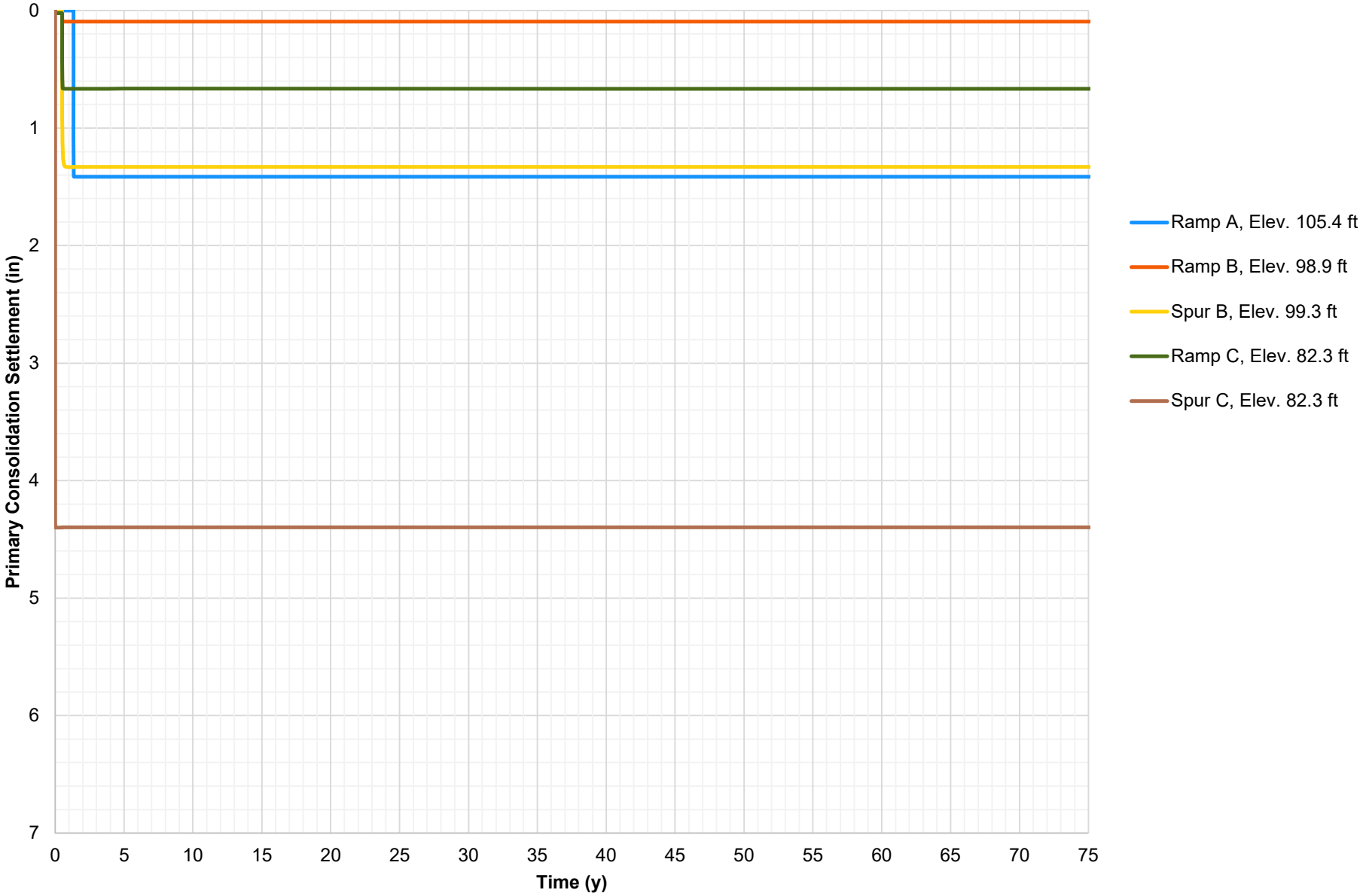
Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 14: Time Rate of Total Settlement over Initial Two Years



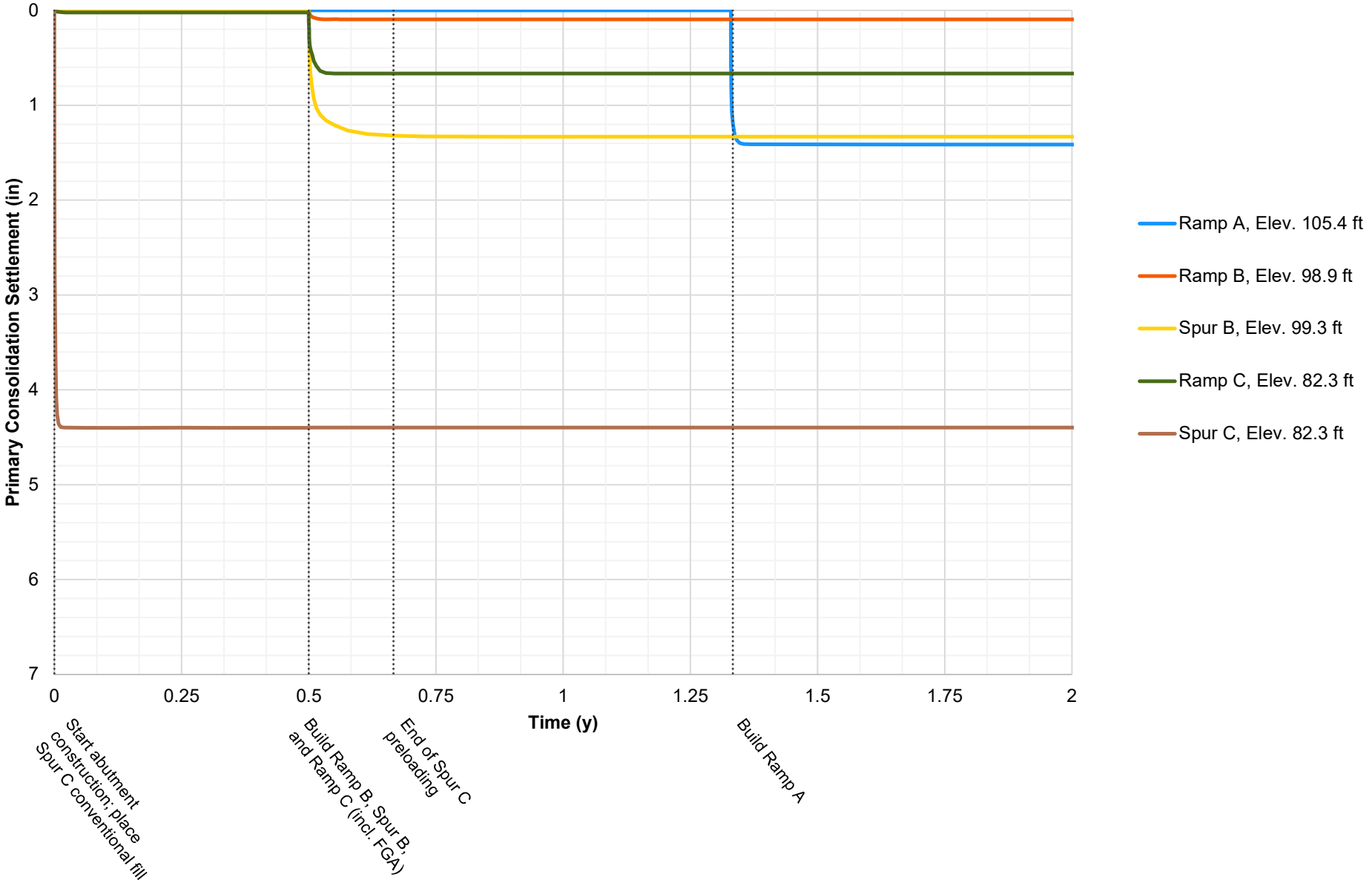
Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 15: Time Rate of Primary Consolidation over 75 Years



Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 16: Time Rate of Primary Consolidation over Initial Two Years



Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

APPENDIX F

Geotechnical Foundation Calculations

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for Mast Arm Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering														
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness	Lateral Model[1]	Below GW table[2]	Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
	Start	End	Start	End	(ft)									
Pole A-M1 at Hogan Road EB & Bangor Mall Blvd., 55' mast arm loading scenario, Station 12+33, Offset 50' RT, Layering from borehole HB-BHR-203														
Granular Borrow Replacement[6]	0.0	1.0	115.0	114.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	6.0	114.0	109.0	5.0	Sand (Reese)	N	125	-	34	127	-	-	
Stiff Clay	6.0	8.8	109.0	106.2	2.8	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Glacial Till	8.8	9.5	106.2	105.5	0.7	Sand (Reese)	Y	68	-	38	124	-	-	
Bedrock	9.5	-	105.5	-	-	Strong Rock	Y	96	-	-	-	-	2117	
Pole B-M1 at Bangor Mall Blvd. & Hogan Road EB, 50' mast arm loading scenario, Station 11+41, Offset 72' RT, Layering from borehole HB-BHR-204														
Granular Borrow Replacement[6]	0.0	1.0	116.0	115.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	5.0	115.0	111.0	4.0	Sand (Reese)	N	125	-	34	127	-	-	
Stiff Clay	5.0	10.0	111.0	106.0	5.0	Stiff Clay w/o Free Water (Reese)	N	122	1343	-	200	0.005	-	
Glacial Till	10.0	19.5	106.0	96.5	9.5	Sand (Reese)	Y	68	-	38	124	-	-	
Bedrock	19.5	-	96.5	-	-	Strong Rock	Y	96	-	-	-	-	2117	
Pole C-M1 at Hogan Road WB & Springer Drive, 60' mast arm loading scenario, Station 11+37, Offset 62' LT, Layering from borehole HB-BHR-201, 201A, & 201B														
Granular Borrow Replacement[6]	0.0	1.0	118.0	117.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	5.3	117.0	112.7	4.3	Sand (Reese)	N	125	-	34	127	-	-	
Bedrock	5.3	-	112.7	-	-	Strong Rock	Y	96	-	-	-	-	2117	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for Mast Arm Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering														
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness	Lateral Model[1]	Below GW table[2]	Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
	Start	End	Start	End	(ft)									
Pole D-M1 at Springer Drive, 45' mast arm loading scenario, Station 12+29, Offset 116.5' LT, Layering from borehole HB-BHR-202														
Granular Borrow Replacement[6]	0.0	1.0	118.7	117.7	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	4.3	117.7	114.4	3.3	Sand (Reese)	N	125	-	34	127	-	-	
Bedrock	4.3	-	114.4	-	-	Strong Rock	Y	96	-	-	-	-	2117	
Pole C-M1 at Hogan Road North Diamond Interchange, 35' mast arm loading scenario, Station 107+90 EB, Layering from borehole HB-BHR-206														
Fill	0.0	2.5	118.2	115.8	2.2	Sand (Reese)	N	125	-	34	127	-	-	Drilled Shaft
Fill	2.5	10.8	115.8	107.4	8.3	Sand (Reese)	Y	63	-	34	77	-	-	
Stiff Clay	10.8	14.8	107.4	103.4	4.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Silt	14.8	19.8	103.4	98.4	5.0	Sand (Reese)	Y	63	-	32	53	-	-	
Glacial Till	19.8	-	98.4	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
Pole A-M1 at Hogan Road South Diamond Interchange, 40' mast arm loading scenario, Station 116+44 EB, Layering from borehole HB-BHR-209														
Granular Borrow Replacement[6]	0.0	1.5	107.0	105.5	1.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.5	12.3	105.5	94.7	10.8	Sand (Reese)	N	125	-	34	127	-	-	
Fill	12.3	21.9	94.7	85.1	9.6	Sand (Reese)	Y	63	-	34	77	-	-	
Silt	21.9	26.9	85.1	80.1	5.0	Sand (Reese)	Y	63	-	32	53	-	-	
Sand and Gravel	26.9	-	80.1	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for Mast Arm Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering														
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness	Lateral Model[1]	Below GW table[2]	Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
	Start	End	Start	End	(ft)									
Pole D-M1 at Ramp C & Hogan Road WB, 15' mast arm loading scenario, Station 215+87 WB, Layering from borehole HB-BHR-208														
Granular Borrow Replacement[6]	0.0	10.2	110.6	100.4	10.2	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	10.2	24.8	100.4	85.8	14.6	Sand (Reese)	N	125	-	34	127	-	-	
Stiff Clay	24.8	29.8	85.8	80.8	5.0	Stiff Clay w/o Free Water (Reese)	N	122	1343	-	200	0.005	-	
Stiff to Medium Stiff Clay	29.8	39.8	80.8	70.8	10.0	Stiff Clay w/ Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Glacial Till	39.8	-	70.8	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
Pole D-M2 at Hogan Road Ramp C, 15' mast arm loading scenario, Station 30+87, Offset 8.0' RT Layering from borehole HB-BHR-212														
Granular Borrow Replacement[6]	0.0	7.6	101.5	93.9	7.6	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	7.6	14.0	93.9	87.5	6.4	Sand (Reese)	N	125	-	34	127	-	-	
Stiff to Medium Stiff Clay	14.0	15.6	87.5	85.9	1.6	Stiff Clay w/o Free Water (Reese)	N	121	913	-	N/A	0.010	-	
Stiff to Medium Stiff Clay	15.6	19.0	85.9	82.5	3.4	Stiff Clay w/ Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Stiff Clay	19.0	24.0	82.5	77.5	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Sand and Gravel	24.0	-	77.5	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for Mast Arm Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering						Lateral Model[1]	Below GW table[2]	Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									
Pole A-M1 at Hogan Road WB & Haskell Road, 55' mast arm loading scenario, Station 37+18, Offset 48.2' LT, Layering from borehole HB-BHR-213														
Granular Borrow Replacement[6]	0.0	1.0	81.4	80.4	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.0	7.5	80.4	73.9	6.5	Sand (Reese)	N	125	-	34	127	-	-	
Fill	7.5	14.9	73.9	66.5	7.4	Sand (Reese)	Y	63	-	34	77	-	-	Drilled Shaft
Silt	14.9	24.9	66.5	56.5	10.0	Sand (Reese)	Y	63	-	32	53	-	-	
Glacial Till	24.9	-	56.5	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
Pole B-M1 at Haskell Road & Hogan Road WB, 60' mast arm loading scenario, Station 38+65, Offset 33.75' LT, Layering from borehole HB-BHR-214														
Granular Borrow Replacement[6]	0.0	1.0	79.0	78.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	
Fill	1.0	8.3	78.0	70.7	7.3	Sand (Reese)	N	125	-	34	127	-	-	
Fill	8.3	10.2	70.7	68.8	1.9	Sand (Reese)	Y	63	-	34	77	-	-	Drilled Shaft
Sand and Gravel	10.2	15.2	68.8	63.8	5.0	Sand (Reese)	Y	68	-	38	124	-	-	
Glacial Till	15.2	-	63.8	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for Mast Arm Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering						Lateral Model[1]	Below GW table[2]	Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									
Pole C-M1 at Hogan Road EB & Sylvan Road, 55' mast arm loading scenario, Station 38+67, Offset 51.69' LT, Layering from borehole HB-BHR-215														
Granular Borrow Replacement[6]	0.0	1.0	78.6	77.6	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	5.0	77.6	73.6	4.0	Sand (Reese)	N	125	-	34	127	-	-	
Silt	5.0	10.6	73.6	68.0	5.6	Sand (Reese)	N	125	-	32	88	-	-	
Silt	10.6	20.0	68.0	58.6	9.4	Sand (Reese)	Y	63	-	32	53	-	-	
Glacial Till	20.0	-	58.6	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
Pole D-M1 at Sylvan Road & Hogan Road EB, 30' mast arm loading scenario, Station 37+77, Offset 81.5' RT, Layering from borehole HB-BHR-216														
Granular Borrow Replacement[6]	0.0	1.0	77.0	76.0	1.0	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.0	8.8	76.0	68.2	7.8	Sand (Reese)	N	125	-	34	127	-	-	
Stiff Clay	8.8	9.4	68.2	67.6	0.6	Stiff Clay w/o Free Water (Reese)	N	122	1343	-	200	0.005	-	
Stiff Clay	9.4	13.8	67.6	63.2	4.4	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Sand and Gravel	13.8	-	63.2	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Notes:

1. Lateral model: Isenhower, W.M., Wang, S.T., and Vasquez, L.G. LPILE v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Release: March 2020.
2. Water table determined based on boring log stratigraphy from WSP Series 100 and Series 200 Borehole Logs.
3. Soil properties determined based on the GDR Bridge and Highway Reports
4. Soil properties correlated from SPT N_{60} values
5. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed 7/2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
6. We propose to remove and replace the upper 1 foot of existing soil to remove organics. This layer also accounts for additional fill from grade changes per project plans.

Date: 5/13/2025
Project No.: 31404817.003
Subject: Subsurface Layering and Engineering Properties for OHSS Foundations
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10)

Made by: AMR/LDN
Checked by: JEF
Reviewed by: MEL

Layering														
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness	Lateral Model[1]	Below GW table[2]	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
	Start	End	Start	End	(ft)									
C-1, 45' mast arm, Cantilever type, Station 104+75.00 (Hogan Road EB), Offset 24.4' LT, Layering from borehole HB-BHR-207														
Fill	0.0	4.4	115.7	111.3	4.4	Sand (Reese)	N	125	-	34	127	-	-	Drilled Shaft
Fill	4.4	9.0	111.3	106.7	4.6	Sand (Reese)	Y	63	-	34	77	-	-	
Silt	9.0	14.0	106.7	101.7	5.0	Sand (Reese)	Y	63	-	32	53	-	-	
Stiff Clay	14.0	19.0	101.7	96.7	5.0	Stiff Clay with Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Stiff to Medium Stiff Clay	19.0	22.7	96.7	93.0	3.7	Stiff Clay with Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Glacial Till	22.7	-	93.0	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
C-2, 35' mast arm, Cantilever type, Station 900+00.00 (Spur B), Offset 12.9' LT, Layering from borehole BB-BHR-201														
Granular Borrow Replacement[6]	0.0	18.2	114.8	96.6	18.2	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Silt	18.2	18.8	96.6	96.0	0.6	Sand (Reese)	N	125	-	32	88	-	-	
Silt	18.8	22.4	96.0	92.4	3.6	Sand (Reese)	Y	63	-	32	53	-	-	
Stiff Clay	22.4	37.4	92.4	77.4	15.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Glacial Till	37.4	42.4	77.4	72.4	5.0	Sand (Reese)	Y	68	-	38	124	-	-	
Bedrock	42.4	-	72.4	-	-	Strong Rock	Y	96	-	-	-	-	2117	

Date: 5/13/2025
Project No.: 31404817.003
Subject: Subsurface Layering and Engineering Properties for OHSS Foundations
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10)

Made by: AMR/LDN
Checked by: JEF
Reviewed by: MEL

Soil Type	Layering: Depth (ft bgs) Start	Layering: Depth (ft bgs) End	Layering: Elevation (ft) Start	Layering: Elevation (ft) End	Layering: Thickness (ft)	Lateral Model[1]	Below GW table[2]	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
OH-1, 87' mast arm, Bridge type, Station 219+50.00 (Hogan Road WB), Offset 39.2' RT, Layering from borehole HB-BHR-212 (Left Support)														
Granular Borrow Replacement[6]	0.0	2.5	98.7	96.2	2.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	2.5	11.2	96.2	87.5	8.7	Sand (Reese)	N	125	-	34	127	-	-	
Stiff to Medium Stiff Clay	11.2	12.8	87.5	85.9	1.6	Stiff Clay w/o Free Water (Reese)	N	121	913	-	N/A	0.010	-	
Stiff to Medium Stiff Clay	12.8	16.2	85.9	82.5	3.4	Stiff Clay w/ Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Stiff Clay	16.2	21.2	82.5	77.5	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Sand and Gravel	21.2	-	77.5	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
OH-1, 87' mast arm, Bridge type, Station 401+50.00 (Ramp D), Offset 26.0'' RT, Layering from borehole HB-BHR-211 (Right Support)														
Granular Borrow Replacement[6]	0.0	3.5	96.9	93.4	3.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Silt	3.5	11.2	93.4	85.7	7.7	Sand (Reese)	N	125	-	32	88	-	-	
Silt	11.2	13.4	85.7	83.5	2.2	Sand (Reese)	Y	63	-	32	53	-	-	
Stiff Clay	13.4	18.4	83.5	78.5	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Sand and Gravel	18.4	-	78.5	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** AMR/LDN
Subject: Subsurface Layering and Engineering Properties for OHSS Foundations **Checked by:** JEF
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering						Lateral Model[1]	Below GW table[2]	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									

Notes:

1. Lateral model: Isenhower, W.M., Wang, S.T., and Vasquez, L.G. LPILE v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Release: March 2020.
2. Water table determined based on boring log stratigraphy from WSP Series 100 and Series 200 Borehole Log
3. Soil properties determined based on the GDR Bridge and Highway Reports
4. Soil properties correlated from SPT N_{60} values
5. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed 7/2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
6. We propose to remove and replace the upper 1 foot of existing soil to remove organics. This layer also accounts for additional fill from grade changes per project plans.

Date: 5/13/2025
Project No.: 31404817.003
Subject: Subsurface Layering and Engineering Properties for Span Wire Pole Foundations
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10)

Made by: LDN
Checked by: JEF/DEB
Reviewed by: MEL

Layering						Lateral Model[1]	Below GW table[2]	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness									
	Start	End	Start	End	(ft)									
Pole A at Hogan Road WB, Station 202+71, Offset 33.1' LT, Layering interpreted from borehole HB-BHR-207														
Granular Borrow Replacement[6]	0.0	3.2	115.5	112.3	3.2	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	3.2	4.2	112.3	111.3	1.0	Sand (Reese)	N	125	-	34	127	-	-	
Fill	4.2	8.8	111.3	106.7	4.6	Sand (Reese)	Y	63	-	34	77	-	-	
Silt	8.8	13.8	106.7	101.7	5.0	Sand (Reese)	Y	63	-	32	53	-	-	
Stiff Clay	13.8	18.8	101.7	96.7	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Stiff to Medium Stiff Clay	18.8	22.5	96.7	93.0	3.7	Stiff Clay w/ Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Bedrock	22.5	-	93.0	-	-	Strong Rock	Y	96	-	-	-	-	2117	
Pole B at Hogan Road WB, Station 102+71, Offset 33' RT, Layering interpreted from borehole HB-BHR-207														
Granular Borrow Replacement[6]	0.0	1.5	112.9	111.4	1.5	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	1.5	6.2	111.4	106.7	4.7	Sand (Reese)	N	125	-	34	127	-	-	
Silt	6.2	11.2	106.7	101.7	5.0	Sand (Reese)	Y	63	-	32	53	-	-	
Stiff Clay	11.2	16.2	101.7	96.7	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Stiff to Medium Stiff Clay	16.2	19.9	96.7	93.0	3.7	Stiff Clay w/ Free Water (Reese)	Y	59	913	-	N/A	0.010	-	
Bedrock	19.9	-	93.0	-	-	Strong Rock	Y	96	-	-	-	-	2217	

Date: 5/13/2025
Project No.: 31404817.003 **Made by:** LDN
Subject: Subsurface Layering and Engineering Properties for Span Wire Pole Foundations **Checked by:** JEF/DEB
Project Title: MaineDOT Bangor Hogan Road Diverging Diamond Interchange (WIN 018595.10) **Reviewed by:** MEL

Layering														
Soil Type	Depth (ft bgs)		Elevation (ft)		Thickness	Lateral Model[1]	Below GW table[2]	Effective Unit Weight (pcf)[3]	Undrained Shear Strength[4] (psf)	Friction Angle[4] (°)	Subgrade Modulus[5] (pci)	Major Principal Strain at 50%[5]	UCS[3] (ksf)	Design Note
	Start	End	Start	End	(ft)									
Pole A at Hogan Road EB, Station 122+96, Offset 28.75' RT, Layering interpreted from boreholes HB-BHR-216														
Granular Borrow Replacement[6]	0.0	6.3	85.1	78.8	6.3	Sand (Reese)	N	125	-	32	88	-	-	Drilled Shaft
Fill	6.3	10.7	78.8	74.4	4.4	Sand (Reese)	N	125	-	34	127	-	-	
Fill	10.7	16.9	74.4	68.2	6.2	Sand (Reese)	Y	63	-	34	77			
Stiff Clay	16.9	21.9	68.2	63.2	5.0	Stiff Clay w/ Free Water (Reese)	Y	60	1343	-	200	0.005	-	
Sand and Gravel	21.9	-	63.2	-	-	Sand (Reese)	Y	68	-	38	124	-	-	
Pole B at Hogan Road EB, Station 223+30, Offset 33.3' LT, Layering interpreted from borehole HB-BHR-213														
Fill	0.0	8.7	86.1	77.4	8.7	Sand (Reese)	N	125	-	34	127	-	-	Drilled Shaft
Fill	8.7	19.6	77.4	66.5	10.9	Sand (Reese)	Y	63	-	34	77	-	-	
Silt	19.6	29.6	66.5	56.5	10.0	Sand (Reese)	Y	63	-	32	53	-	-	
Glacial Till	29.6	-	56.5	-	-	Sand (Reese)	Y	68	-	38	124	-	-	

Notes:

1. Lateral model: Isenhower, W.M., Wang, S.T., and Vasquez, L.G. LPILE v2019 Technical Manual: A Program for the Analysis of Deep Foundations Under Lateral Loading. Ensoft, Inc. Release: March 2020.
2. Water table determined based on boring log stratigraphy from WSP Series 100 and Series 200 Borehole Log
3. Soil properties determined based on the GDR Bridge and Highway Reports
4. Soil properties correlated from SPT N_{60} values
5. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed 7/2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
6. We propose to remove and replace the upper 1 foot of existing soil to remove organics. This layer also accounts for additional fill from grade changes per project plans.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal A-M1 (Hogan Rd Construction Centerline Station 12+33.41, Offset 50.33' RT) at Hogan Road EB and Bangor Mall Blvd. based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
5. WSP geotechnical test boring logs for 200-series borings.
6. WSP geotechnical test boring logs for 100-series borings.
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
8. VHB. Loads to Overhead Mast Arm Signal/Sign Structures. Dated February 18, 2025.
9. GeoTesting Express laboratory testing results, dated May 2022 and November 2023.
10. VHB for State of Maine Department of Transportation. Bangor US I-95 Exit 187/Hogan Road Interchange, Hyw Plans Progress Print, dated March 10, 2025.
11. Holtz, R.D. and Kovacs, W.D. 1981. An Introduction to Geotechnical Engineering, 1st ed. Prentice Hall, Englewood Cliffs, NJ.
12. FHWA. Geotechnical Engineering Circular No. 6: Shallow Foundations. Report No. FHWA-SA-02-054. September
13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.
17. Auburn University. Torsional Resistance of Drilled Shaft Foundations. The Auburn University Highway Research Center. October 2018
18. WSP Lab Field Data Summary Plots
19. WSP Mast Arm and Light Design Basis
20. VHB. Boring Location Plan

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 53 and 57, there is no direct cross section available for Mast Arm B-M1 at Hogan Road EB and Bangor Mall Blvd. due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 24, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soil will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced due to being embedded in bedrock. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-203 is used for the analysis at Pole A-M1.

Proposed top of drilled shaft elevation =	115.0	ft	(Estimated from Ref. 20, Sheet 2)
Existing ground surface elevation =	115.0	ft	
Elevation to which remove/replace soil =	114.0	ft	(Assumption 2)
Groundwater elevation =	109.0	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2,5]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	1-6	Sand (Reese)	125	24	34.0	-	127	-	-
Stiff Clay	6-8.8	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005	-
Glacial Till	8.8-9.5	Sand (Reese)	68	42	38.0	-	124	-	-
Bedrock	9.5-?	Sand (Reese)	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

5) Assumption 6 for Bedrock

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	7
Moment	kip-ft	150	180	130
Shear	kip	0	6	3
Torsion	kip-ft	0	140	60

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road EB & Bangor Mall Blvd.	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

Calculate the unit side resistance q_s of the rock layer. Use the normal method for rock (Ref. 1, Article 10.8.3.5.4b).

$$\frac{q_s}{p_a} C = \sqrt{\frac{q_u}{p_a}} \qquad \text{(Ref. 1, Eq. 10.8.3.5.4b-1)}$$

Regression coefficient, C = 1.0 for normal conditions

Uniaxial compressive strength, q_u = 304.8 ksf

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	4.0	0.5	34.0	24	0.6	6.7	-	1.27	0.63
Stiff Clay	7.4	0.8	-	17	-	-	0.6	0.55	0.46
Glacial Till	9.2	0.9	38.0	42	0.8	19.8	-	1.96	1.84
Bedrock	-	-	-	-	-	-	-	-	25.42

CALCULATIONS

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Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Bedrock layer. Calculate the unit tip resistance q_p for the Bedrock layer. Use the rock method (Ref. 1, Article 10.8.3.5.4c). Note that per Ref. 1 Article 10.8.3.5.4c, the value of q_u should be multiplied by 2.5 if the socket is deeper than 1.5B.

$$\text{If } z > 1.5B, q_p = 2.5q_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.4c-1)}$$

where:

Uniaxial compressive strength, q_u = 304.8 ksf (From Step 1)

Bedrock q_p = 762.1 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.63	12.6	4.0	50.3	31.9	0.55	17.6
Stiff Clay	0.46		2.8	35.2	16.1	0.45	7.3
Glacial Till	1.84		0.7	8.8	16.2	0.55	8.9
Bedrock	25.42		0.5	6.3	159.7	0.55	87.9

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5
2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Bedrock	762.1	12.6	9577.1	0.50	4788.5

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 4822 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
8 kips	<	4822 kips	OK

Demand capacity ratio = 0.00

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	37.5	0.0	0.02	Yes
	2	0.50	8	75.0	0.0	0.04	Yes
	3	0.75	8	112.5	0.0	0.07	Yes
	4	1.00	8	150.0	0.0	0.10	Yes
	5	1.25	8	187.5	0.0	0.13	Yes
	6	1.50	8	225.0	0.0	0.16	Yes
Extreme I	7	0.25	7	45.0	1.5	0.03	Yes
	8	0.50	7	90.0	3.0	0.08	Yes
	9	0.75	7	135.0	4.5	0.12	Yes
	10	1.00	7	180.0	6.0	0.18	Yes
	11	1.25	7	225.0	7.5	0.24	Yes
					maximum =	0.24	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.24 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	7	130.0	3	0.11	Yes

The computed deflection of 0.11 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

wsp

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesive soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 17 Section 2.4.3. Ref. 17 recommends neglecting the top length of the pile of 1.5-feet or 1.5*D depending on method selected. For the purpose of these calculations, the depth of skin friction to be ignored for cohesive soils will be consistent with the analyzed frost depth.

$$T_n = \frac{1}{2}\pi D^2 (L - 1.5) f_s$$ (Ref. 17, Section 2.4.3)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in rock. Note, the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1 is modified with the unit side resistance of the bedrock.

$$T_n = \pi D L q_s \left(\frac{D}{2}\right)$$ (modified Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on rock. Note, the method for cohesionless soil in Ref. 17 Section 2.4.6 is used and modified with the friction angle of interface materials (Ref. 1, Table C3.11.5.3-1).

$$T_t = \frac{1}{3} D W tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4}\pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	140	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	
Angle of friction for Clean Sound Rock =	35	°	(Ref. 1, Table C3.11.5.3-1)

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	4.0	0.5	18	1.5	0.8	4.0	75.4	-
Stiff Clay	7.4	0.8	13	1.3	1.1	2.8	76.2	-
Glacial Till	9.2	0.9	25	1.5	1.4	0.7	24.8	-
Bedrock	-	-	-	-	25.4	0.5	319.5	17.6

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 8.0 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 514 kip-ft
Factored torsion resistance, T_r = 514 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
140	kip-ft	<	514	kip-ft	OK

Demand capacity ratio = 0.27

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.24 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.01 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.25 in
Shaft length = 10.0 ft (from step 2)
Total shaft rotation = deflection / length = 0.002 rad

Maximum mast arm length for C1 = 60.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 1.52 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.52	inches	OK

The estimated vertical movement of 1.52 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

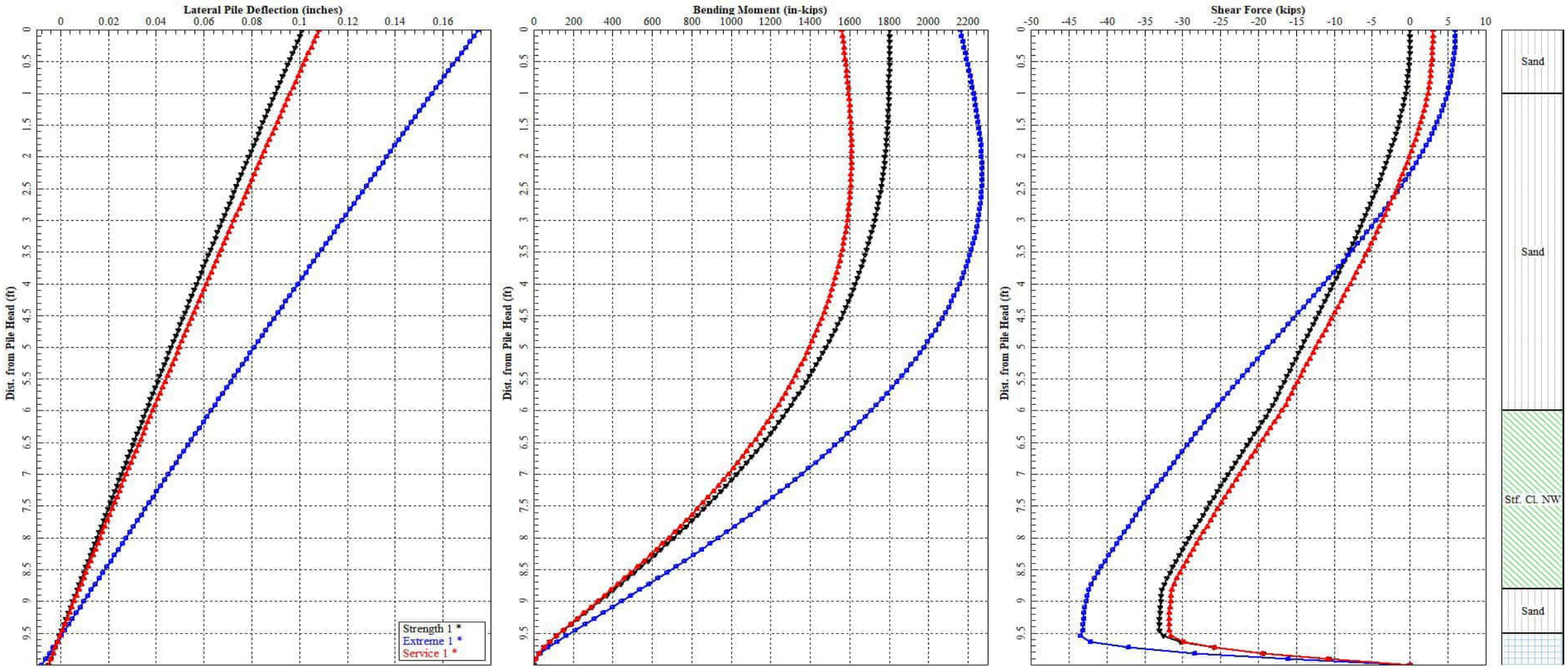
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Mast Arm A-M1 at Hogan Road EB and Bangor Mall Blvd. based on the final design loads provided by VHB. A maximum lateral deflection of 0.11 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.24 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.27 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 1.52 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal B-M1 (Hogan Rd Construction Centerline Station 11+40.93, Offset 72.27' RT) at Bangor Mall Blvd. and Hogan Road EB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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19. WSP Mast Arm and Light Design Basis
20. VHB. Boring Location Plan

CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 53 and 57, there is no direct cross section available for Mast Arm B-M1 at Bangor Mall Blvd. and Hogan Road EB due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 24, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet if existing soil will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced by the limited depth granular borrow added at the surface. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-204 is used for the analysis at Pole B-M1.

Proposed top of drilled shaft elevation =	116.0	ft	(Estimated from Ref. 20, Sheet 2)
Existing ground surface elevation =	116.0	ft	
Elevation to which remove/replace soil =	115.0	ft	(Assumption 2)
Groundwater elevation =	106.0	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	1-5	Sand (Reese)	125	24	34.0	-	127	-	-
Stiff Clay	5-10	Stiff Clay w/o Free Water (Reese)	122	17	-	1343	200	0.005	-
Glacial Till	10-19.5	Sand (Reese)	68	42	38.0	-	124	-	-
Bedrock	19.5-?	Sand (Reese)	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 12.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 11 ties for a shaft length of 12.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	11		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	6
Moment	kip-ft	110	150	100
Shear	kip	0	6	3
Torsion	kip-ft	0	110	50

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $\dfrac{\sigma'_p}{p_a} = 0.47(N_{60})^m$ (Ref. 1, Eq. 10.8.3.5.2b-4)

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $\dfrac{\sigma'_p}{p_a} = 0.15(N_{60})$ (Ref. 1, Eq. 10.8.3.5.2b-5)

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	1.5	0.2	34.0	24	0.6	6.7	-	2.20	0.41
Stiff Clay	7.5	0.9	-	17	-	-	0.6	0.55	0.51
Glacial Till	11.3	1.3	38.0	42	0.8	19.8	-	1.59	2.10

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Glacial Till layer. Calculate the unit tip resistance q_p for the Glacial Till layer. Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 42 ksf (From Step 1)

Glacial Till q_p = 50.4 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.41	12.6	3.0	37.7	15.5	0.55	8.5
Stiff Clay	0.51		5.0	62.8	32.1	0.45	14.5
Glacial Till	2.10		2.5	31.4	66.0	0.55	36.3

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Glacial Till	50.4	12.6	633.3	0.50	316.7

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 376 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 8 kips < Factored geotechnical axial compression resistance: 376 kips OK

Demand capacity ratio = 0.02

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	27.5	0.0	0.01	Yes
	2	0.50	8	55.0	0.0	0.03	Yes
	3	0.75	8	82.5	0.0	0.04	Yes
	4	1.00	8	110.0	0.0	0.06	Yes
	5	1.25	8	137.5	0.0	0.08	Yes
	6	1.50	8	165.0	0.0	0.11	Yes
Extreme I	7	0.25	7	37.5	1.5	0.02	Yes
	8	0.50	7	75.0	3.0	0.06	Yes
	9	0.75	7	112.5	4.5	0.10	Yes
	10	1.00	7	150.0	6.0	0.14	Yes
	11	1.25	7	187.5	7.5	0.18	Yes
					maximum =	0.18	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.18 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 15 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	6	100.0	3	0.08	Yes

The computed deflection of 0.08 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 15 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesive soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 17 Section 2.4.3. Ref. 17 recommends neglecting the top length of the pile of 1.5-feet or 1.5*D depending on method selected. For the purpose of these calculations, the depth of skin friction to be ignored for cohesive soils will be consistent with the analyzed frost depth.

$$T_n = \frac{1}{2}\pi D^2 (L - 1.5) f_s$$ (Ref. 17, Section 2.4.3)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} D W tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4}\pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	110	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	23.6	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	1.5	0.2	18	1.5	0.3	3.0	21.2	-
Stiff Clay	7.5	0.9	13	1.3	1.2	5.0	151.9	-
Glacial Till	11.3	1.3	25	1.5	2.0	2.5	124.4	24.5

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 10.5 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 322 kip-ft
Factored torsion resistance, T_r = 322 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
110	kip-ft	<	322	kip-ft	OK

Demand capacity ratio = 0.34

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.18 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.08 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.26 in
Shaft length = 12.5 ft (from step 2)
Total shaft rotation = deflection / length = 0.002 rad

Maximum mast arm length for C1 = 50.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 1.04 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.04	inches	OK

The estimated vertical movement of 0.86 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

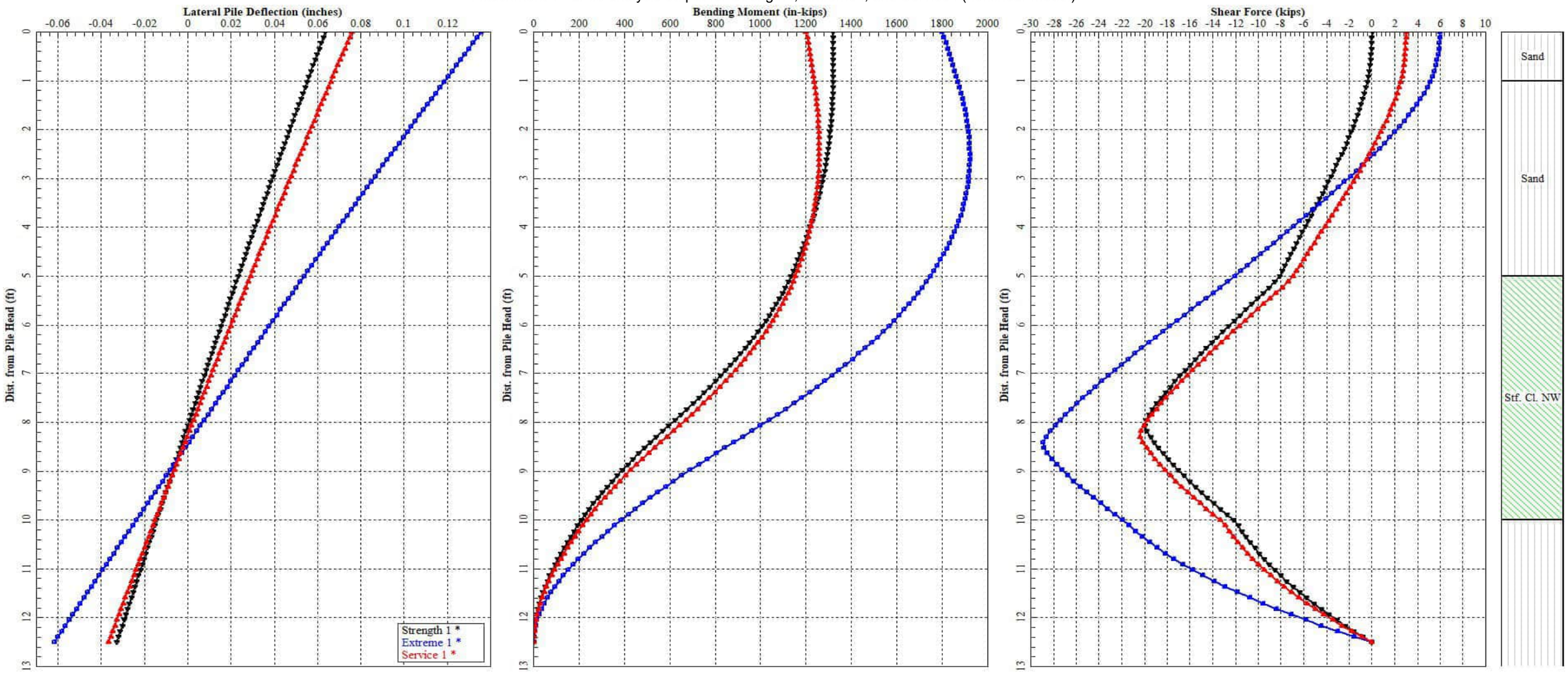
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 12.5 feet will provide adequate support for Mast Arm B-M1 at Bangor Mall Blvd. and Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.08 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.18 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.34 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 1.04 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Subject:	Drilled Shaft Design for Pole C-M1 at Hogan Road WB & Springer Drive	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal C-M1 (Hogan Rd Construction Centerline Station 11+36.46, Offset 61.75' LT) at Hogan Road WB and Springer Drive based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 53 and 57, there is no direct cross section available for Mast Arm C-M1 at Hogan Road WB and Springer Drive due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 24, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soil will be replaced by granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is not expected to result in downdrag loading as a portion of the shaft length is embedded in bedrock. The underlying bedrock is assumed to exhibit negligible settlement following shaft installation. As a result, additional loading due to downdrag is not considered in this analysis.

6. Due to no ground water elevation encountered, top of bedrock is assumed to be ground water table elevation.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-201A is used for the analysis at Pole C-M1.

Proposed top of drilled shaft elevation =	118.0	ft	(Estimated from Ref. 20, Sheet 2)
Existing ground surface elevation =	118.0	ft	
Elevation to which remove/replace soil =	117.0	ft	(Assumption 2)
Groundwater elevation =	112.7	ft	(Assumption 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2,5]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	1-5.3	Sand (Reese)	125	24	34.0	-	127	-	-
Bedrock	5.3-?	Strong Rock	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

5) Assumption 6 for Bedrock

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 7.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 6 ties for a shaft length of 7.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	6		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	7
Moment	kip-ft	150	180	130
Shear	kip	0	6	3
Torsion	kip-ft	0	130	60

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole C-M1 at Hogan Road WB & Springer Drive	**Reviewed by:**	CCB
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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s of the rock layer. Use the normal method for rock (Ref. 1, Article 10.8.3.5.4b).

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}} \qquad \text{(Ref. 1, Eq. 10.8.3.5.4b-1)}$$

Regression coefficient, C = 1.0 for normal conditions

Uniaxial compressive strength, q_u = 304.8 ksf

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	3.7	0.5	34.0	24	0.6	6.7	-	1.34	0.61
Bedrock	-	-	-	-	-	-	-	-	25.42

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Bedrock layer. Calculate the unit tip resistance qp for the Bedrock layer. Use the rock method (Ref. 1, Article 10.8.3.5.4c). Note that per Ref. 1 Article 10.8.3.5.4c, the value of qu should be multiplied by 2.5 if the socket is deeper than 1.5B.

$$\text{If } z > 1.5B, q_p = 2.5q_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.4c-1)}$$

where:

Uniaxial compressive strength, q_u = 304.8 ksf (From Step 1)
Socket depth, z = 2.2 ft

Bedrock q_p = 304.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.61	12.6	3.3	41.5	25.3	0.55	13.9
Bedrock	25.42		2.2	27.6	702.8	0.55	386.5

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Bedrock	304.8	12.6	3830.8	0.50	1915.4

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 2316 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 8 kips < Factored geotechnical axial compression resistance: 2316 kips OK

Demand capacity ratio = 0.00

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	37.5	0.0	0.01	Yes
	2	0.50	8	75.0	0.0	0.02	Yes
	3	0.75	8	112.5	0.0	0.03	Yes
	4	1.00	8	150.0	0.0	0.04	Yes
	5	1.25	8	187.5	0.0	0.05	Yes
	6	1.50	8	225.0	0.0	0.06	Yes
Extreme I	7	0.25	7	45.0	1.5	0.02	Yes
	8	0.50	7	90.0	3.0	0.03	Yes
	9	0.75	7	135.0	4.5	0.05	Yes
	10	1.00	7	180.0	6.0	0.06	Yes
	11	1.25	7	225.0	7.5	0.08	Yes
					maximum =	0.08	

The trial shaft length of 7.5 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.8 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 7.5 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	7	130.0	3	0.04	Yes

The computed deflection of 0.04 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 7.5 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in rock. Note, the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1 is modified with the unit side resistance of the bedrock.

$$T_n = \pi D L q_s \left(\frac{D}{2}\right)$$ (modified Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on rock. Note, the method for cohesionless soil in Ref. 17 Section 2.4.6 is used and modified with the friction angle of interface materials (Ref. 1, Table C3.11.5.3-1).

$$T_t = \frac{1}{3} D W tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	130	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	14.1	kip	
Unit Weight of concrete, γ_c =	150	pcf	
Angle of friction for Clean Sound Rock =	35	°	(Ref. 1, Table C3.11.5.3-1)

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	3.7	0.5	33	1.5	0.7	3.3	56.8	-
Bedrock	-	-	-	-	25.4	2.2	1405.6	13.2

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 5.5 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 1476 kip-ft
Factored torsion resistance, T_r = 1476 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
130	kip-ft	<	1476	kip-ft	OK

Demand capacity ratio = 0.09

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.08 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.01 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.09 in
Shaft length = 7.5 ft (from step 2)
Total shaft rotation = deflection / length = 0.001 rad

Maximum mast arm length for C1 = 60.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 0.70 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	0.70	inches	OK

The estimated vertical movement of 1.76 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

wsp

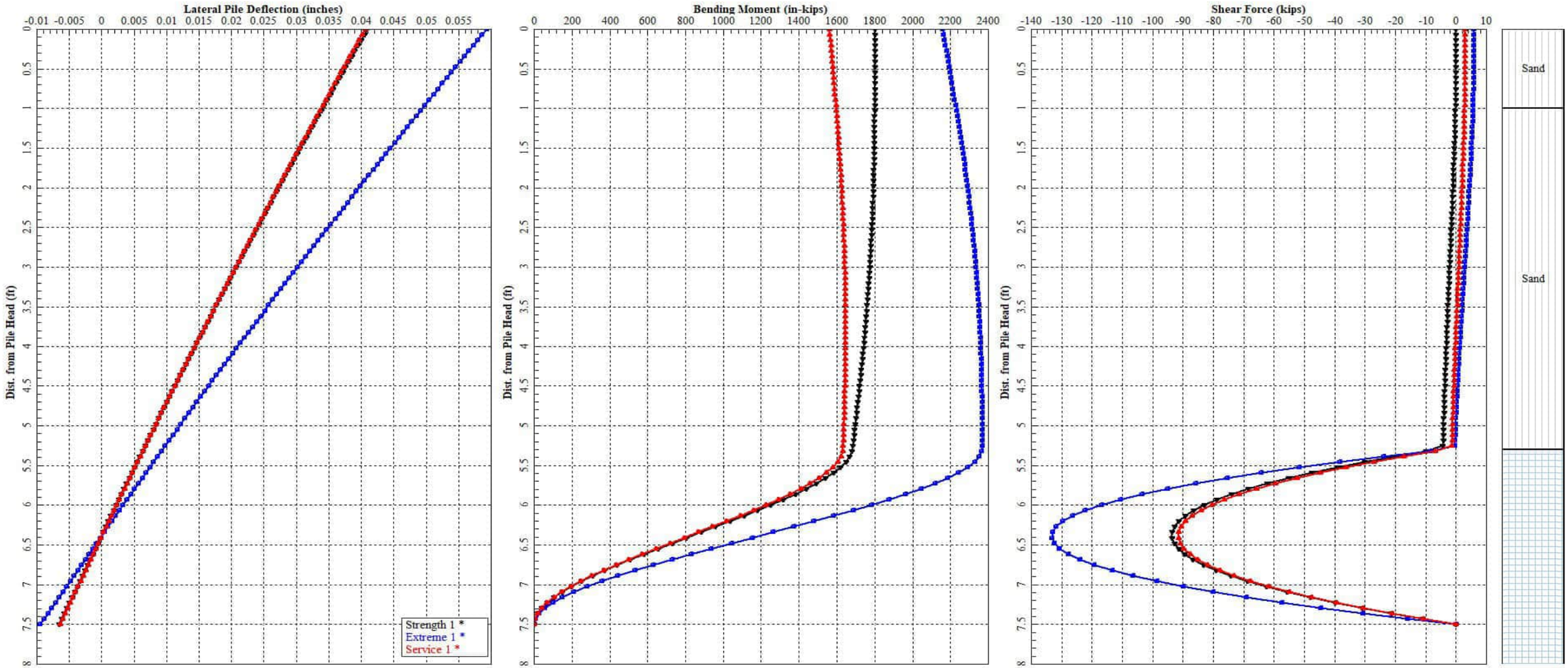
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 7.5 feet will provide adequate support for Pole C-M1 at Hogan Road WB and Springer Drive based on the final design loads provided by VHB. A maximum lateral deflection of 0.04 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.08 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.09 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 0.70 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal D-M1 (Hogan Rd Construction Centerline Station 12+31.57, Offset 116.73' LT) at Springer Drive based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 53 and 57, there is no direct cross section available for Mast Arm D-M1 at Springer Drive due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 24, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soil will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is not expected to result in downdrag loading as a portion of the shaft length is embedded in bedrock. The underlying bedrock is assumed to exhibit negligible settlement following shaft installation. As a result, additional loading due to downdrag is not considered in this analysis.

6. Due to no ground water elevation encountered, top of bedrock is assumed to be ground water table elevation.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-202 is used for the analysis at Pole D-M1.

Proposed top of drilled shaft elevation =	118.7	ft	(Ref. 19)
Existing ground surface elevation =	118.7	ft	
Elevation to which remove/replace soil =	117.7	ft	(Assumption 2)
Groundwater elevation =	114.4	ft	(Assumption 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2,5]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	1-4.3	Sand (Reese)	125	24	34.0	-	127	-	-
Bedrock	4.3-?	Strong Rock	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

5) Assumption 6 for Bedrock

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 7.5 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 6 ties for a shaft length of 7.5 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	6		(Ref. 13, pages 626(01) & 626(02))

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CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	7	6	6
Moment	kip-ft	90	130	80
Shear	kip	0	5	2
Torsion	kip-ft	0	80	40

As per Assumption 5, no downdrag loading will be added.

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CALCULATIONS

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Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M1 at Springer Drive	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s of the rock layer. Use the normal method for rock (Ref. 1, Article 10.8.3.5.4b).

$$\frac{q_s}{p_a} = C\sqrt{\frac{q_u}{p_a}} \qquad \text{(Ref. 1, Eq. 10.8.3.5.4b-1)}$$

Regression coefficient, C = 1.0 for normal conditions

Uniaxial compressive strength, q_u = 304.8 ksf

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	3.2	0.4	34.0	24	0.6	6.7	-	1.45	0.57
Bedrock	-	-	-	-	-	-	-	-	25.42

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Bedrock layer. Calculate the unit tip resistance qp for the Bedrock layer. Use the rock method (Ref. 1, Article 10.8.3.5.4c). Note that per Ref. 1 Article 10.8.3.5.4c, the value of qu should be multiplied by 2.5 if the socket is deeper than 1.5B.

$$\text{If } z > 1.5B, q_p = 2.5q_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.4c-1)}$$

where:

Uniaxial compressive strength, q_u = 304.8 ksf (From Step 1)
Socket depth, z = 3.2 ft

Bedrock q_p = 304.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.57	12.6	2.3	28.9	16.5	0.55	9.1
Bedrock	25.42		3.2	40.2	1022.3	0.55	562.3

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Bedrock	304.8	12.6	3830.8	0.50	1915.4

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 2487 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 7 kips < Factored geotechnical axial compression resistance: 2487 kips OK

Demand capacity ratio = 0.00

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	7	22.5	0.0	0.00	Yes
	2	0.50	7	45.0	0.0	0.00	Yes
	3	0.75	7	67.5	0.0	0.01	Yes
	4	1.00	7	90.0	0.0	0.01	Yes
	5	1.25	7	112.5	0.0	0.01	Yes
	6	1.50	7	135.0	0.0	0.01	Yes
Extreme I	7	0.25	6	32.5	1.3	0.00	Yes
	8	0.50	6	65.0	2.5	0.01	Yes
	9	0.75	6	97.5	3.8	0.01	Yes
	10	1.00	6	130.0	5.0	0.02	Yes
	11	1.25	6	162.5	6.3	0.02	Yes
					maximum =	0.02	

The trial shaft length of 7.5 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.02 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 7.5 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	6	80.0	2	0.01	Yes

The computed deflection of 0.01 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 7.5 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in rock. Note, the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1 is modified with the unit side resistance of the bedrock.

$$T_n = \pi D L q_s \left(\frac{D}{2}\right)$$ (modified Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on rock. Note, the method for cohesionless soil in Ref. 17 Section 2.4.6 is used and modified with the friction angle of interface materials (Ref. 1, Table C3.11.5.3-1).

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	80	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	14.1	kip	
Unit Weight of concrete, γ_c =	150	pcf	
Angle of friction for Clean Sound Rock =	35	°	(Ref. 1, Table C3.11.5.3-1)

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	3.2	0.4	17	1.5	0.6	2.3	34.1	-
Bedrock	-	-	-	-	25.4	3.2	2044.6	13.2

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 5.5 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 2092 kip-ft
Factored torsion resistance, T_r = 2092 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
80	kip-ft	<	2092	kip-ft	OK

Demand capacity ratio = 0.04

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.02 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.00 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.03 in
Shaft length = 7.5 ft (from step 2)
Total shaft rotation = deflection / length = 0.000 rad

Maximum mast arm length for C1 = 45.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 0.16 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	0.16	inches	OK

The estimated vertical movement of 0.16 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

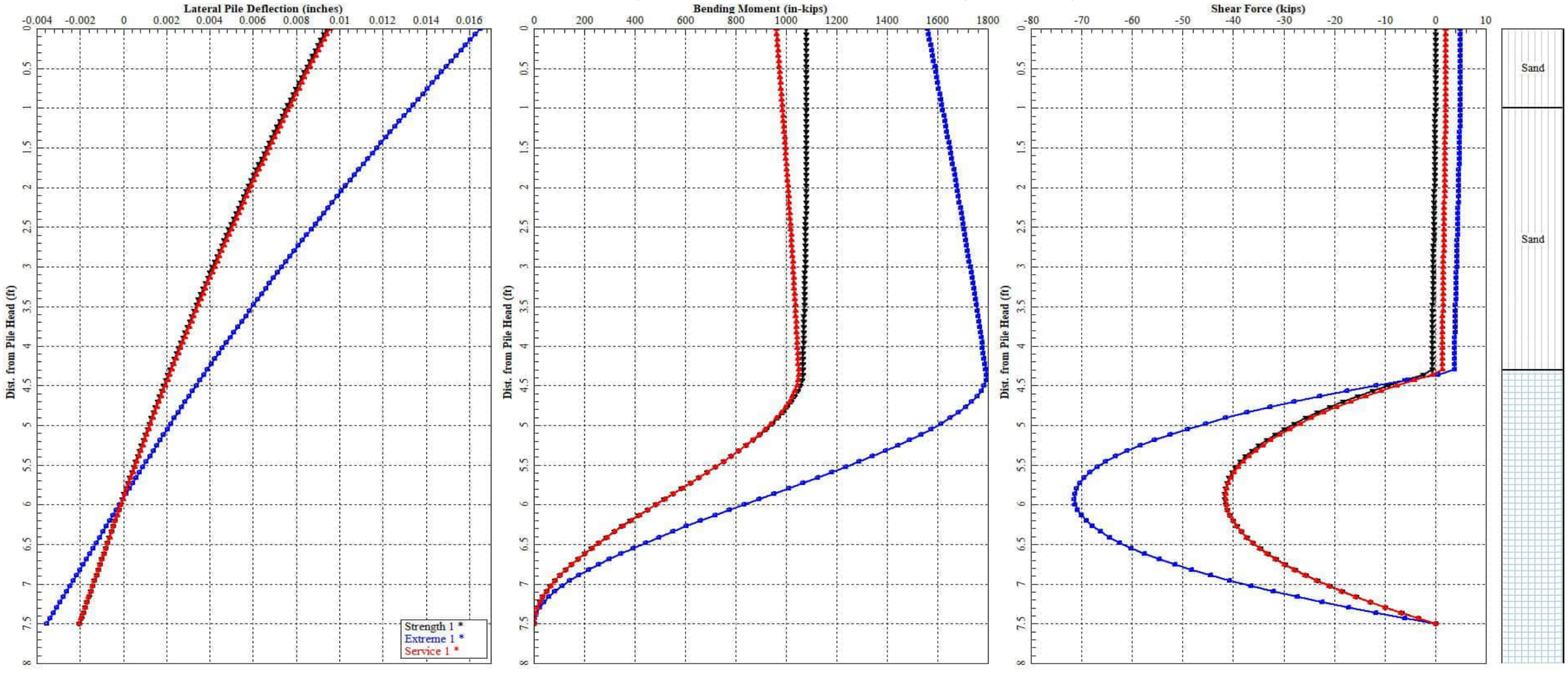
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 7.5 feet will provide adequate support for Pole D-M1 at Springer Drive based on the final design loads provided by VHB. A maximum lateral deflection of 0.01 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.02 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.04 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 0.16 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal C-M1 (Construction Centerline Station 20+47.11, Offset 24.71' LT) at Hogan Road North Diamond Interchange based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 87, a cut of 0.3 feet is proposed at the mast arm location. The analysis assumes that granular borrow will not be placed over the existing soils, and no existing soils will be removed.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the natural cohesionless soils. Settlement of the underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-206 is used for the analysis at Pole C-M1.

Proposed top of drilled shaft elevation =	118.2	ft	(Ref. 19)
Existing ground surface elevation =	118.5	ft	
Elevation to which remove/replace soil =	118.2	ft	(Assumption 2)
Groundwater elevation =	115.7	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Existing Fill	0-2.5	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	2.5-10.8	Sand (Reese)	63	24	34.0	-	77	-
Stiff Clay	10.8-14.8	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Silt	14.8-19.8	Sand (Reese)	63	20	32.0	-	53	-
Glacial Till	19.8-?	Sand (Reese)	68	42	38.0	-	124	-

1) Granular borrow assumed N60 value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): φ = 27.1 + 0.3N60 - 0.00054[N60]2, friction design angle of 32.0 (Ref. 2, Table 3-3). The N60 values for the in-situ soils are based on the average of N60 values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.
This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	4	4	4
Moment	kip-ft	50	120	60
Shear	kip	0	5	2
Torsion	kip-ft	0	80	40

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

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Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole C-M1 at Hogan Road North Diamond Interchange **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	2.3	0.3	34.0	24	0.6	6.7	-	1.75	0.49
Existing Fill	6.3	0.5	34.0	24	0.6	6.7	-	1.21	0.66

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance qp for the Existing Fill layer. Because the tip of the shaft is overlying a weaker layer and it is less than 1.5 the diameter deeper than the tip, the base resistance will need to be reduced (Ref. 1, Article 10.8.3.5.3). Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c) to find the tip resistance 1.5B above the top of the weaker layer; the value of qp should be limited to 60 ksf unless greater values can be justified through load test data. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c) to fine the tip resistance at the top of the weaker layer. Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

Average N_{60} value for stronger layer = 17

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

Existing Fill q_p = 20.4 ksf

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$q_p = N_c S_u \leq 80\ ksf$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

penetration of shaft z = 0 ft

Bearing Capacity factor for Cohesive soils N_C = 6.0

Top of Stiff Clay Layer q_p = 8.1 ksf

where:

Δq_p = 2.1 ksf / ft

Tip of Drilled Shaft q_p = 9.7 ksf

CALCULATIONS

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Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.49	12.6	0.5	6.3	3.1	0.55	1.7
Existing Fill	0.66		7.5	94.2	62.3	0.55	34.3

1) Neglecting top 2 feet of shaft for Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	9.7	12.6	121.9	0.50	61.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$$R_R = R_{R,s} + R_{R,p} = 97 \text{ kips}$$

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
4 kips	<	97 kips	OK

Demand capacity ratio = 0.04

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	4	12.5	0.0	0.02	Yes
	2	0.50	4	25.0	0.0	0.05	Yes
	3	0.75	4	37.5	0.0	0.07	Yes
	4	1.00	4	50.0	0.0	0.09	Yes
	5	1.25	4	62.5	0.0	0.11	Yes
	6	1.50	4	75.0	0.0	0.14	Yes
Extreme I	7	0.25	4	30.0	1.3	0.07	Yes
	8	0.50	4	60.0	2.5	0.14	Yes
	9	0.75	4	90.0	3.8	0.23	Yes
	10	1.00	4	120.0	5.0	0.34	Yes
	11	1.25	4	150.0	6.3	0.48	Yes
					maximum =	0.48	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.48 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	4	60.0	2	0.14	Yes

The computed deflection of 0.14 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4}\pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	80	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	2.3	0.3	25	1.5	0.4	0.5	5.3	-
Existing Fill	6.3	0.5	25	1.5	0.8	7.5	154.7	17.0

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 8.0 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 177 kip-ft
Factored torsion resistance, T_r = 177 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
80	kip-ft	<	177	kip-ft	OK

Demand capacity ratio = 0.45

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.48 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.23 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.71 in
Shaft length = 10.0 ft (from step 2)
Total shaft rotation = deflection / length = 0.006 rad

Maximum mast arm length for C1 = 35.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 2.47 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	2.47	inches	OK

The estimated vertical movement of 2.47 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

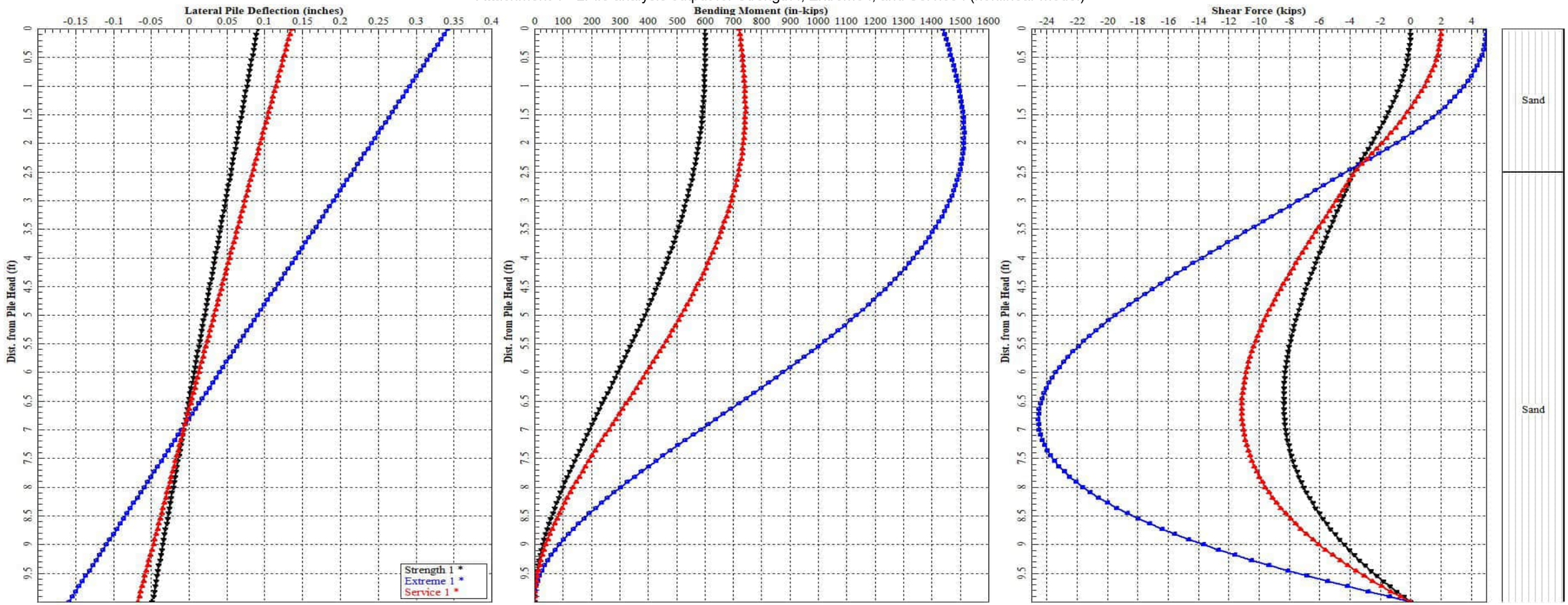
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole C-M1 at Hogan Road North Diamond Interchange based on the final design loads provided by VHB. A maximum lateral deflection of 0.14 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.48 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.45 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 2.47 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road South Diamond Interchange	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal A-M1 (Hogan Rd Construction Centerline Station 29+43.37, Offset 35.42' LT) at Hogan Road South Diamond Interchange based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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19. WSP Mast Arm and Light Design Basis

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road South Diamond Interchange	**Reviewed by:**	CCB
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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 95, a grade raise of 1.5 feet is proposed at the mast arm location. The analysis assumes that 1.5 feet of granular borrow will be placed over the existing soils, and no existing soils will be removed.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-209 is used for the analysis at Pole A-M1.

Proposed top of drilled shaft elevation =	107.0	ft	(Ref. 19)
Existing ground surface elevation =	105.5	ft	
Elevation to which remove/replace soil =	105.5	ft	(Assumption 2)
Groundwater elevation =	94.7	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-1.5	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	1.5-12.3	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	12.3-21.9	Sand (Reese)	63	24	34.0	-	77	-
Silt	21.9-26.9	Sand (Reese)	63	20	32.0	-	53	-
Sand and Gravel	26.9-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.
This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	5	4	4
Moment	kip-ft	60	120	70
Shear	kip	0	5	2
Torsion	kip-ft	0	110	50

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	6.0	0.8	34.0	24	0.6	6.7	-	1.01	0.76

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, q_p = 1.2N_{60} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Existing Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.76	12.6	8.0	100.5	76.3	0.55	42.0

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 223 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 5 kips < Factored geotechnical axial compression resistance: 223 kips OK

Demand capacity ratio = 0.02

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)
Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	5	15.0	0.0	0.02	Yes
	2	0.50	5	30.0	0.0	0.04	Yes
	3	0.75	5	45.0	0.0	0.06	Yes
	4	1.00	5	60.0	0.0	0.08	Yes
	5	1.25	5	75.0	0.0	0.10	Yes
	6	1.50	5	90.0	0.0	0.13	Yes
Extreme I	7	0.25	4	30.0	1.3	0.06	Yes
	8	0.50	4	60.0	2.5	0.11	Yes
	9	0.75	4	90.0	3.8	0.17	Yes
	10	1.00	4	120.0	5.0	0.23	Yes
	11	1.25	4	150.0	6.3	0.31	Yes
					maximum =	0.31	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.31 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	4	70.0	2	0.12	Yes

The computed deflection of 0.12 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4}\pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	110	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	6.0	0.8	29	1.5	1.1	8.0	226.2	17.0

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 8.0 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 243 kip-ft
Factored torsion resistance, T_r = 243 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
110	kip-ft	<	243	kip-ft	OK

Demand capacity ratio = 0.45

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.31 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.13 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.44 in
Shaft length = 10.0 ft (from step 2)
Total shaft rotation = deflection / length = 0.004 rad

Maximum mast arm length for C1 = 40.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 1.76 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.76	inches	OK

The estimated vertical movement of 1.76 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

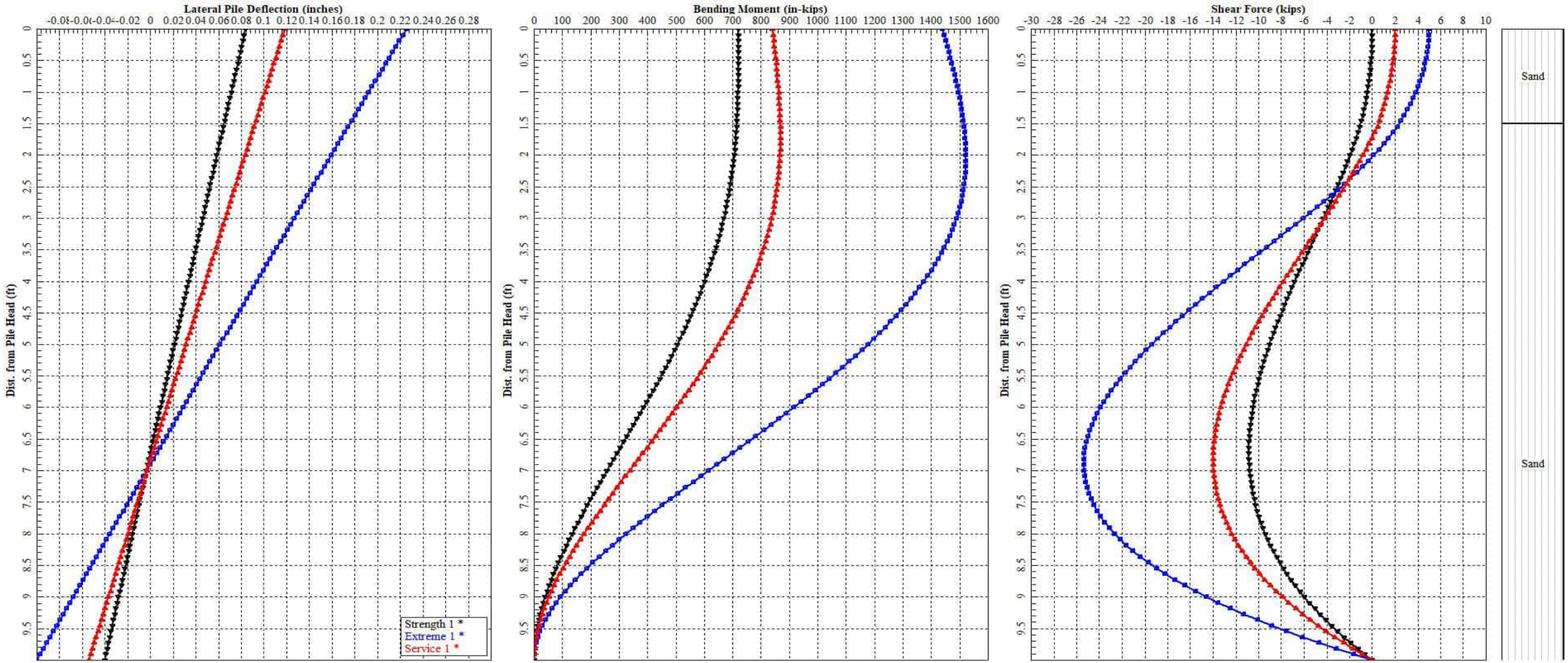
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole A-M1 at Hogan Road South Diamond Interchange based on the final design loads provided by VHB. A maximum lateral deflection of 0.12 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.31 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.45 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 1.76 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal D-M1 (Hogan Rd Construction Centerline Station 28+20.45, Offset 3.10' RT) at Ramp C and Hogan Road WB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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19. WSP Mast Arm and Light Design Basis

CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 121, a grade raise of 9.2 feet is proposed at the mast arm location. Per Ref. 10 Sheet 26, the mast arm is located in a grass area. Therefore, the analysis assumes that 9.2 feet of granular borrow will be placed over the existing soils, and 1.0 feet of existing soils will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 4.8 inches (step 8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow. The granular borrow is assumed to be properly compacted and settlement of the granular borrow following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

6. Ground water level was interpreted based on borehole log descriptions indicating "wet" conditions below 10 ft bgs.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-208 is used for the analysis at Pole D-M1.

Proposed top of drilled shaft elevation =	110.6	ft	(Ref. 19)
Existing ground surface elevation =	101.4	ft	
Elevation to which remove/replace soil =	100.4	ft	(Assumption 2)
Groundwater elevation =	80.8	ft	(Assumption 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-10.2	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	10.2-24.8	Sand (Reese)	125	24	34.0	-	127	-
Stiff Clay	24.8-29.8	Stiff Clay w/o Free Water (Reese)	122	17	-	1343	200	0.005
Stiff to Medium Stiff Clay	29.8-39.8	Stiff Clay w/ Free Water (Reese)	59	6	-	913	-	0.010
Glacial Till	39.8-?	Sand (Reese)	68	42	38.0	-	124	-

1) Granular borrow assumed N60 value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): φ = 27.1 + 0.3N60 - 0.00054[N60]2, friction design angle of 32.0 (Ref. 2, Table 3-3). The N60 values for the in-situ soils are based on the average of N60 values encountered in all highway and bridge borings for each soil layer.

2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

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CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by VHB (Reference 8):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for LPile to calculate results for all loading scenarios.

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 6 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	6	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Standard Lighting)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	3	2	2
Moment	kip-ft	10	70	30
Shear	kip	0	3	2
Torsion	kip-ft	0	20	10

As per Assumption 5, no downdrag loading will be added.

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CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

wsp

CALCULATIONS

Date: 6/17/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole D-M1 at Ramp C & Hogan Road WB **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	6.0	0.8	32.0	17	0.6	5.5	-	0.84	0.63

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Granular Borrow layer. Calculate the unit tip resistance q_p for the Granular Borrow layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of qp should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, q_p = 1.2N_{60} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 17 ksf (From Step 1)

Existing Fill q_p = 20.4 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.63	12.6	8.0	100.5	63.4	0.55	34.9

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Granular Borrow	20.4	12.6	256.4	0.50	128.2

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 163 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 3 kips < Factored geotechnical axial compression resistance: 163 kips OK

Demand capacity ratio = 0.02

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3	2.5	0.0	0.00	Yes
	2	0.50	3	5.0	0.0	0.01	Yes
	3	0.75	3	7.5	0.0	0.01	Yes
	4	1.00	3	10.0	0.0	0.02	Yes
	5	1.25	3	12.5	0.0	0.02	Yes
	6	1.50	3	15.0	0.0	0.03	Yes
Extreme I	7	0.25	2	17.5	0.8	0.04	Yes
	8	0.50	2	35.0	1.5	0.09	Yes
	9	0.75	2	52.5	2.3	0.13	Yes
	10	1.00	2	70.0	3.0	0.18	Yes
	11	1.25	2	87.5	3.8	0.22	Yes
					maximum =	0.22	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.22 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
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Subject:	Drilled Shaft Design for Pole D-M1 at Ramp C & Hogan Road WB	**Reviewed by:**	CCB
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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	2	30.0	2	0.09	Yes

The computed deflection of 0.09 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

wsp

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M1 at Ramp C & Hogan Road WB	**Reviewed by:**	CCB
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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_s = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Section 2.4.6)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	20	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

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Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_s	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Granular Borrow	6.0	0.8	11.0	1.1	0.8	8.0	165.9	15.7

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 8.0 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 182 kip-ft
Factored torsion resistance, T_r = 182 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
20	kip-ft	<	182	kip-ft	OK

Demand capacity ratio = 0.11

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.22 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.10 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.32 in
Shaft length = 10.0 ft (from step 2)
Total shaft rotation = deflection + length = 0.003 rad

Maximum mast arm length for C1 = 15.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 0.48 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
4.8	inches	>	0.48	inches	OK

The estimated vertical movement of 0.48 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 4.8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

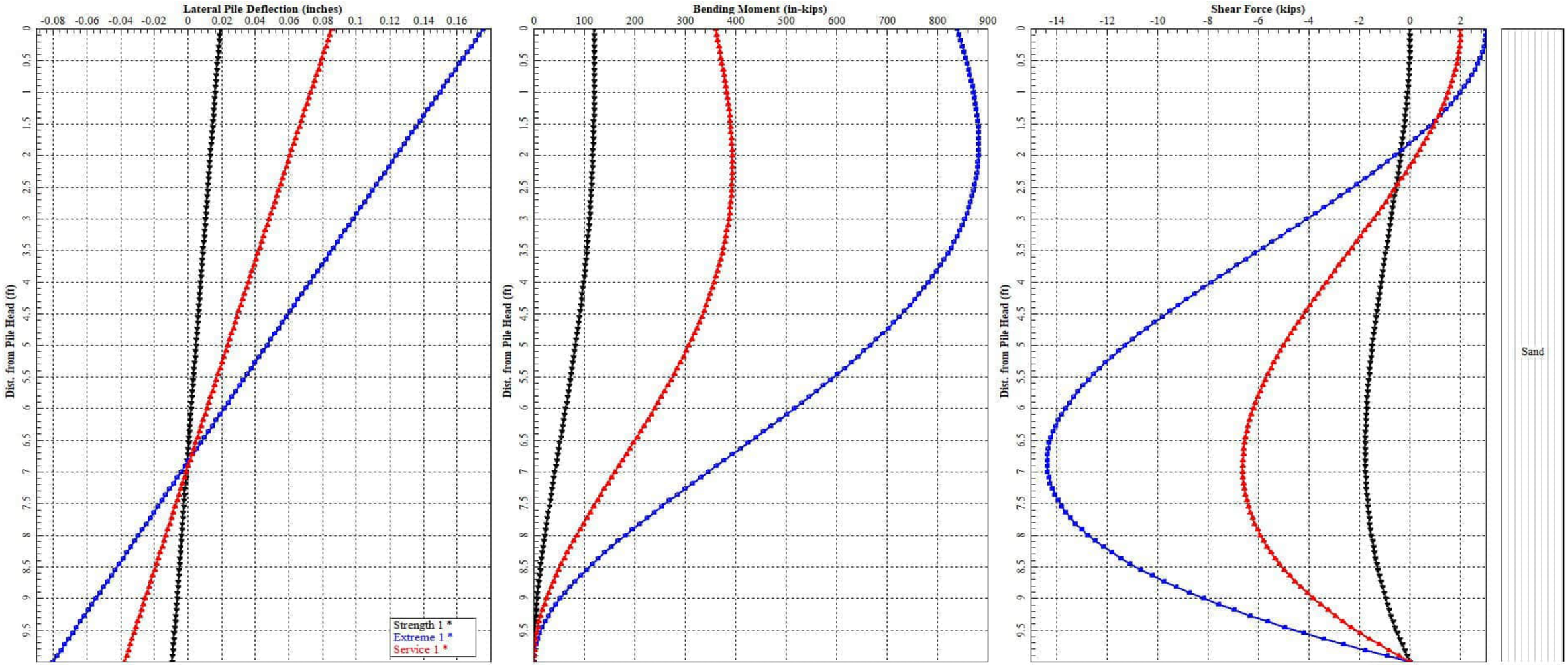
CALCULATIONS

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Subject:	Drilled Shaft Design for Pole D-M1 at Ramp C & Hogan Road WB	**Reviewed by:**	CCB
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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole D-M1 at Ramp C and Hogan Road WB based on the final design loads provided by VHB. A maximum lateral deflection of 0.09 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.22 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.11 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 0.48 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 4.8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal D-M2 (Hogan Rd Construction Centerline Station 30+86.82, Offset 6.94' RT) at Ramp C based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 98, a grade raise of 6.6 feet is proposed at the mast arm location. Per Ref. 10 Sheet 27, the mast arm is located in a grass area. Therefore, the analysis assumes that 6.6 feet of granular borrow will be placed over the existing soils, and 1.0 feet of existing soils will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 4.8 inches (step 8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-212 is used for the analysis at Pole D-M2.

Proposed top of drilled shaft elevation =	101.5	ft	(Ref. 19)
Existing ground surface elevation =	94.9	ft	
Elevation to which remove/replace soil =	93.9	ft	(Assumption 2)
Groundwater elevation =	85.9	ft	(Assumption 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-7.6	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	7.6-14	Sand (Reese)	125	24	34.0	-	127	-
Stiff to Medium Stiff Clay	14-15.6	Stiff Clay w/o Free Water (Reese)	121	6	-	913	-	0.010
Stiff to Medium Stiff Clay	15.6-19	Stiff Clay w/ Free Water (Reese)	59	6	-	913	-	0.010
Stiff Clay	19-24	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Sand and Gravel	24-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N60 value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): φ = 27.1 + 0.3N60 - 0.00054[N60]2, friction design angle of 32.0 (Ref. 2, Table 3-3). The N60 values for the in-situ soils are based on the average of N60 values encountered in all highway and bridge borings for each soil layer.

2) Proposed soils: Ref. 2, Table 3-3. Existing soils: correlation to average N_{60} value for each layer, Ref. 5.

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the minimum shaft diameter to accommodate poles and anchorages as provided by VHB (Reference 8):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for LPile to calculate results for all loading scenarios.

Shaft length =	10.0	feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 6 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	6	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Standard Lighting)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	2	2	2
Moment	kip-ft	10	40	20
Shear	kip	0	2	1
Torsion	kip-ft	0	20	10

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	4.8	0.6	32.0	17	0.6	5.5	-	0.95	0.57
Existing Fill	8.8	1.1	34.0	24	0.6	6.7	-	0.82	0.90

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of qp should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, q_p = 1.2N_{60} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Existing Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.57	12.6	5.6	70.4	39.9	0.55	22.0
Existing Fill	0.90		2.4	30.2	27.1	0.55	14.9

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 218 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 2 kips < Factored geotechnical axial compression resistance: 218 kips OK

Demand capacity ratio = 0.01

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	2	2.5	0.0	0.00	Yes
	2	0.50	2	5.0	0.0	0.01	Yes
	3	0.75	2	7.5	0.0	0.01	Yes
	4	1.00	2	10.0	0.0	0.02	Yes
	5	1.25	2	12.5	0.0	0.02	Yes
	6	1.50	2	15.0	0.0	0.03	Yes
Extreme I	7	0.25	2	10.0	0.5	0.02	Yes
	8	0.50	2	20.0	1.0	0.05	Yes
	9	0.75	2	30.0	1.5	0.07	Yes
	10	1.00	2	40.0	2.0	0.10	Yes
	11	1.25	2	50.0	2.5	0.12	Yes
					maximum =	0.12	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.12 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	2	20.0	1	0.05	Yes

The computed deflection of 0.05 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$T_s = \pi D L F_s \left(\frac{D}{2}\right) \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$F_s = \sigma'_v \omega_{fdot} \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15 \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15 \quad \text{(Ref. 16, Section 13.6.1.1)}$$

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_t = \frac{1}{3} DW tan\phi \quad \text{(Ref. 17, Section 2.4.6)}$$

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	20	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

Date: 6/17/2025 **Made by:** LDN
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Subject: Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_s	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Granular Borrow	4.8	0.6	11.0	1.1	0.7	5.6	92.9	-
Existing Fill	8.8	1.1	19.0	1.5	1.7	2.4	99.5	15.7

Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	8.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	208	kip-ft	
Factored torsion resistance, T_r =	208	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
20	kip-ft	<	208	kip-ft	OK

Demand capacity ratio = 0.10

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.12	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.05	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.17	in	
Shaft length =	10.0	ft	(from step 2)
Total shaft rotation = deflection + length =	0.001	rad	
Maximum mast arm length for C1 =	15.00	ft	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	0.25	in	

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
4.8	inches	>	0.25	inches	OK

The estimated vertical movement of 0.25 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 4.8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

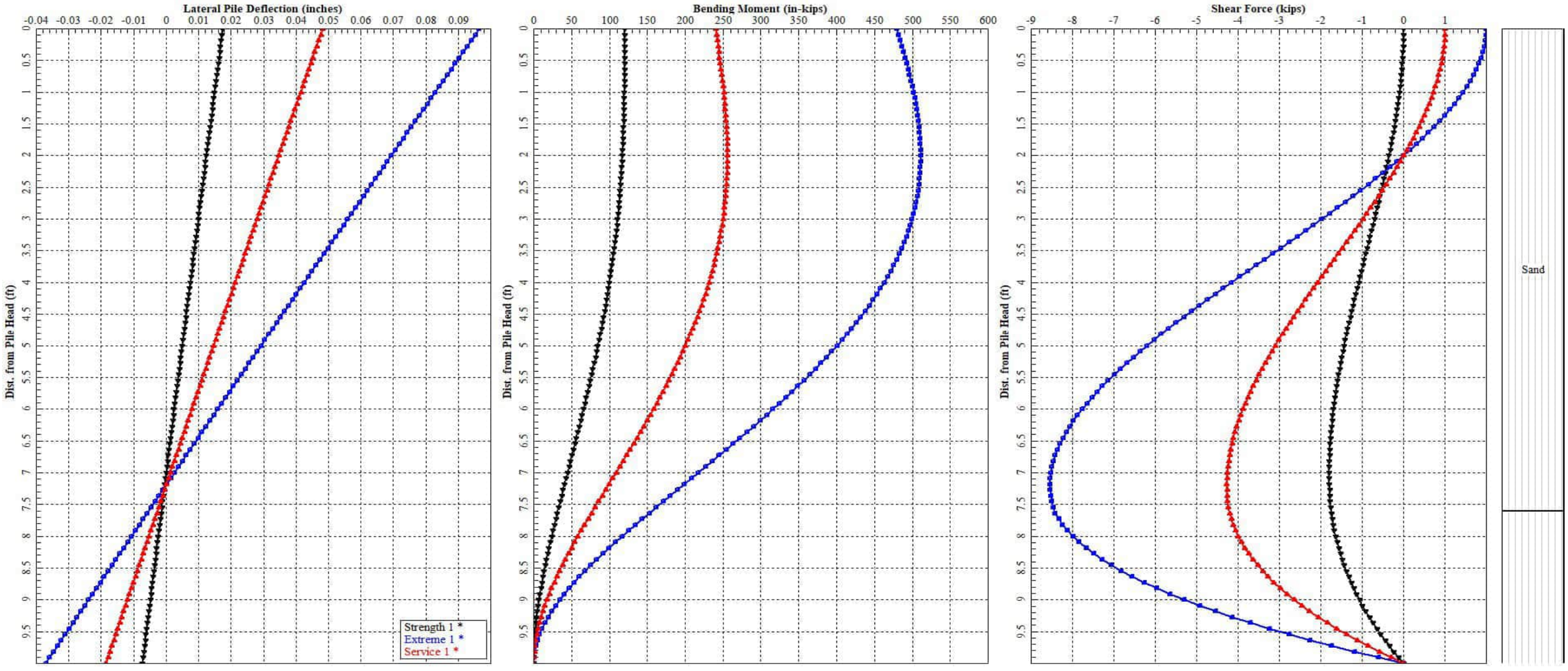
CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole D-M2 at Hogan Road Ramp C	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole D-M2 at Ramp C based on the final design loads provided by VHB. A maximum lateral deflection of 0.05 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.12 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.10 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 0.25 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 4.8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal A-M1 (Hogan Rd Construction Centerline Station 37+17.84, Offset 48.05' LT) at Hogan Road WB and Haskell Road based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-section (Ref. 10 Sheets 131), a grade raise of 0.0 feet is proposed at the mast arm location. Per Ref. 10 Sheet 28, the mast arm is located in a grass area. Therefore, the analysis assumes that 1.0 feet of existing soil will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-213 is used for the analysis at Pole A-M1.

Proposed top of drilled shaft elevation =	81.4	ft	(Ref. 19)
Existing ground surface elevation =	81.4	ft	
Elevation to which remove/replace soil =	81.4	ft	(Assumption 2)
Groundwater elevation =	73.9	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	1-7.5	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	7.5-14.9	Sand (Reese)	63	24	34.0	-	77	-
Silt	14.9-24.9	Sand (Reese)	63	20	32.0	-	53	-
Sand and Gravel	24.9-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
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Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	6
Moment	kip-ft	130	160	110
Shear	kip	0	5	2
Torsion	kip-ft	0	120	50

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road	**Reviewed by:**	CCB
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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

Date: 6/17/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole A-M1 at Hogan Road WB & Haskell Road **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	4.8	0.6	34.0	24	0.6	6.7	-	1.15	0.68
Existing Fill	8.8	1.0	34.0	24	0.6	6.7	-	0.85	0.87

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

If $N_{60} \leq 50, q_p = 1.2N_{60}$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Existing Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.68	12.6	5.5	69.1	47.3	0.55	26.0
Existing Fill	0.87		2.5	31.4	27.3	0.55	15.0

1) Neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 222 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 8 kips < Factored geotechnical axial compression resistance: 222 kips OK

Demand capacity ratio = 0.04

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	32.5	0.0	0.05	Yes
	2	0.50	8	65.0	0.0	0.10	Yes
	3	0.75	8	97.5	0.0	0.15	Yes
	4	1.00	8	130.0	0.0	0.21	Yes
	5	1.25	8	162.5	0.0	0.28	Yes
	6	1.50	8	195.0	0.0	0.36	Yes
Extreme I	7	0.25	7	40.0	1.3	0.08	Yes
	8	0.50	7	80.0	2.5	0.15	Yes
	9	0.75	7	120.0	3.8	0.24	Yes
	10	1.00	7	160.0	5.0	0.37	Yes
	11	1.25	7	200.0	6.3	0.55	Yes
					maximum =	0.55	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.55 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	6	110.0	2	0.20	Yes

The computed deflection of 0.20 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	120	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	4.8	0.6	27	1.5	0.9	5.5	123.1	-
Existing Fill	8.8	1.0	27	1.5	1.5	2.5	95.7	17.0

Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	8.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	236	kip-ft	
Factored torsion resistance, T_r =	236	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
120	kip-ft	<	236	kip-ft	OK

Demand capacity ratio = 0.51

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.55	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.27	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.81	in	
Shaft length =	10.0	ft	(from step 2)
Total shaft rotation = deflection / length =	0.007	rad	
Maximum mast arm length for C1 =	55.00	ft	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	4.47	in	

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	4.47	inches	OK

The estimated vertical movement of 4.47 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

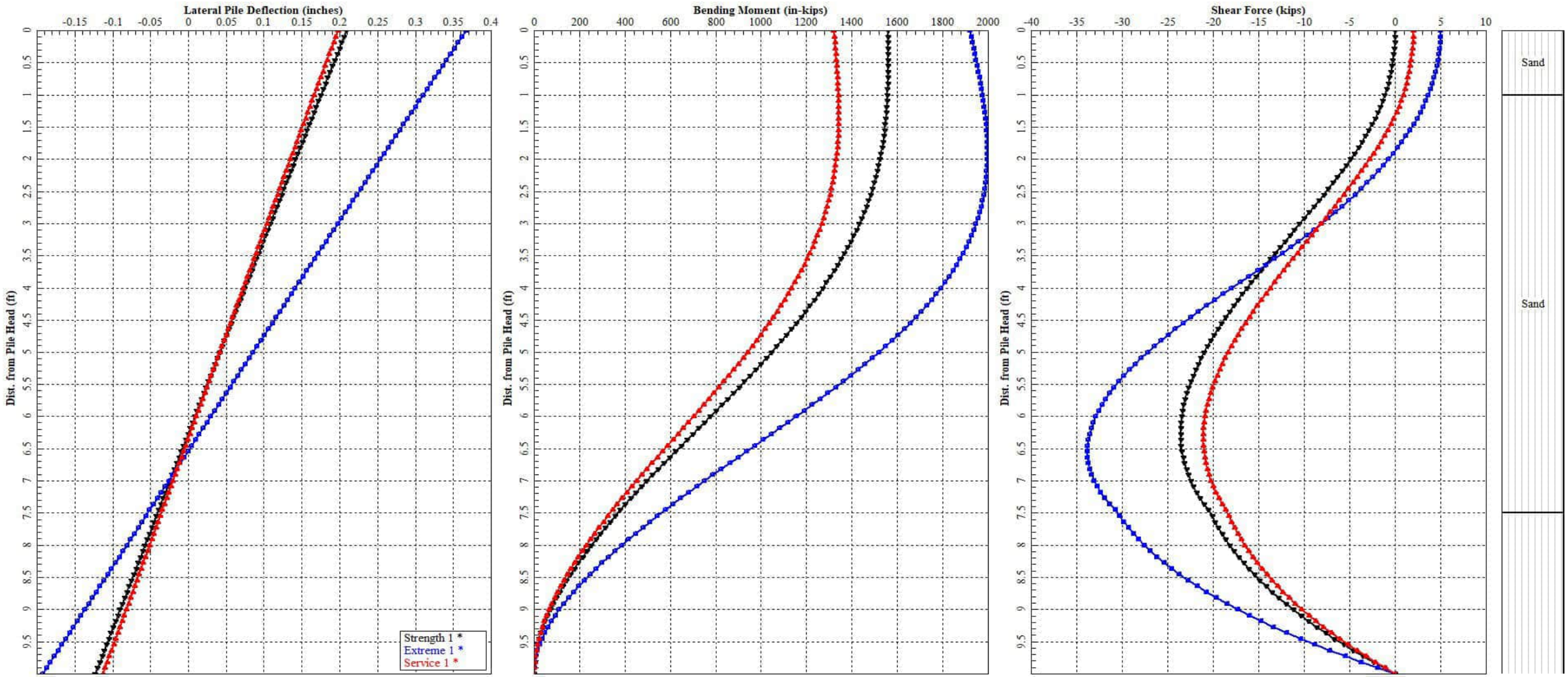
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole A-M1 at Hogan Road WB and Haskell Road based on the final design loads provided by VHB. A maximum lateral deflection of 0.20 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.55 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.51 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 4.47 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Subject:	Drilled Shaft Design for Pole B-M1 at Haskell Road & Hogan Road WB	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal B-M1 (Hogan Rd Construction Centerline Station 38+66.09, Offset 33.55' LT) at Haskell Road based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 56 and 60, there is no direct cross section available for Mast Arm B-M1 at Haskell Road due to its placement beyond the limits of other earthwork. Hence, there is no raise grade proposed at the mast arm location. Per Ref. 10 Sheet 28, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soil will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-214 is used for the analysis at Pole B-M1.

Proposed top of drilled shaft elevation =	79.0	ft	(Ref. 19)
Existing ground surface elevation =	79.0	ft	
Elevation to which remove/replace soil =	78.0	ft	(Assumption 2)
Groundwater elevation =	70.7	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	1-8.3	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	8.3-10.2	Sand (Reese)	63	24	34.0	-	77	-
Sand and Gravel	10.2-15.2	Sand (Reese)	68	52	38.0	-	124	-
Glacial Till	15.2-?	Sand (Reese)	68	42	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

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CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	7
Moment	kip-ft	150	170	130
Shear	kip	0	6	3
Torsion	kip-ft	0	110	50

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	4.7	0.6	34.0	24	0.6	6.7	-	1.17	0.68
Existing Fill	9.2	1.0	34.0	24	0.6	6.7	-	0.88	0.85

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

If $N_{60} \leq 50, q_p = 1.2N_{60}$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Existing Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.68	12.6	6.3	79.2	53.7	0.55	29.5
Existing Fill	0.85		1.7	21.4	18.1	0.55	10.0

1) neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 220 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 8 kips < Factored geotechnical axial compression resistance: 220 kips OK

Demand capacity ratio = 0.04

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	37.5	0.0	0.06	Yes
	2	0.50	8	75.0	0.0	0.12	Yes
	3	0.75	8	112.5	0.0	0.17	Yes
	4	1.00	8	150.0	0.0	0.25	Yes
	5	1.25	8	187.5	0.0	0.34	Yes
	6	1.50	8	225.0	0.0	0.45	Yes
Extreme I	7	0.25	7	42.5	1.5	0.08	Yes
	8	0.50	7	85.0	3.0	0.16	Yes
	9	0.75	7	127.5	4.5	0.27	Yes
	10	1.00	7	170.0	6.0	0.42	Yes
	11	1.25	7	212.5	7.5	0.63	Yes
					maximum =	0.63	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.63 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

wsp

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	7	130.0	3	0.25	Yes

The computed deflection of 0.25 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	110	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	4.7	0.6	18	1.5	0.9	6.3	138.0	-
Existing Fill	9.2	1.0	18	1.5	1.4	1.7	61.9	17.0

Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	8.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	217	kip-ft	
Factored torsion resistance, T_r =	217	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
110	kip-ft	<	217	kip-ft	OK

Demand capacity ratio = 0.51

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.63	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.29	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.92	in	
Shaft length =	10.0	ft	(from step 2)
Total shaft rotation = deflection / length =	0.008	rad	
Maximum mast arm length for C1 =	60.00	ft	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	5.50	in	

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	5.50	inches	OK

The estimated vertical movement of 5.50 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

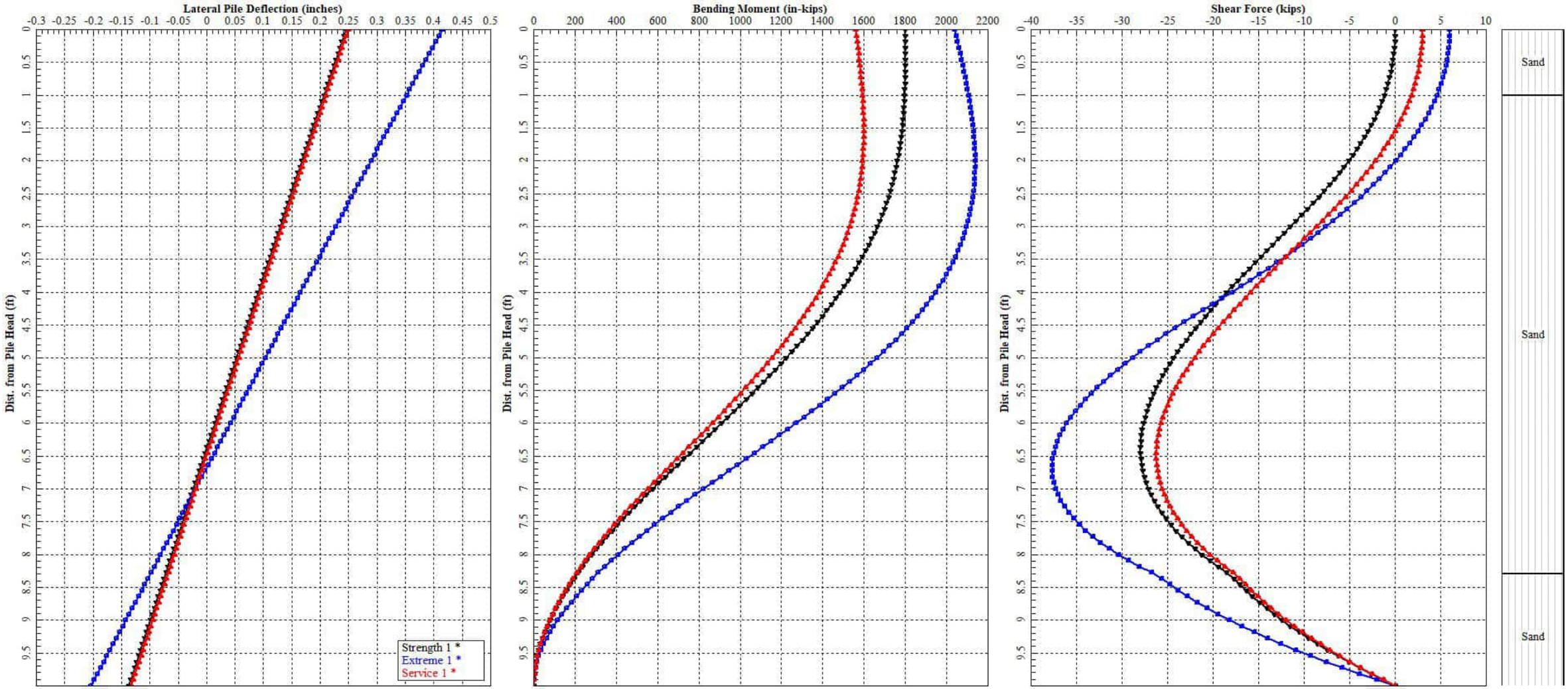
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole A-M1 at Hogan Road South Diamond Interchange based on the final design loads provided by VHB. A maximum lateral deflection of 0.25 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.63 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.51 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 5.50 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole C-M1 at Hogan Road EB & Sylvan Road	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal C-M1 (Hogan Rd Construction Centerline Station 38+66.80, Offset 51.69' RT) at Hogan Road EB and Sylvan Road based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 56 and 60, there is no direct cross section available for Mast Arm C-M1 at Hogan Road EB and Sylvan Road due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 28, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soils will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-215 is used for the analysis at Pole C-M1.

Proposed top of drilled shaft elevation =	78.6	ft	(Ref. 19)
Existing ground surface elevation =	78.6	ft	
Elevation to which remove/replace soil =	77.6	ft	(Assumption 2)
Groundwater elevation =	68.0	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	1-5	Sand (Reese)	125	24	34.0	-	127	-
Silt	5-10.6	Sand (Reese)	125	20	32.0	-	88	-
Silt	10.6-20	Sand (Reese)	63	20	32.0	-	53	-
Glacial Till	20-?	Sand (Reese)	68	-	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.
This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	8	7	7
Moment	kip-ft	130	170	120
Shear	kip	0	6	3
Torsion	kip-ft	0	130	50

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $\dfrac{\sigma'_p}{p_a} = 0.47(N_{60})^m$ (Ref. 1, Eq. 10.8.3.5.2b-4)

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)

Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $\dfrac{\sigma'_p}{p_a} = 0.15(N_{60})$ (Ref. 1, Eq. 10.8.3.5.2b-5)

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	3.5	0.4	34.0	24	0.6	6.7	-	1.37	0.60
Silt	7.5	0.8	32.0	20	0.6	6.0	-	0.85	0.69

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Silt layer. Calculate the unit tip resistance q_p for the Silt layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, q_p = 1.2N_{60} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 20 ksf (From Step 1)

Silt q_p = 24.0 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.60	12.6	3.0	37.7	22.6	0.55	12.4
Silt	0.69		5.0	62.8	43.3	0.55	23.8

1) neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

CALCULATIONS

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	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp} [1]	$R_{R,p}$ (kips)
Silt	24.0	12.6	301.6	0.50	150.8

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$$R_R = R_{R,s} + R_{R,p} = 187 \text{ kips}$$

Check against Strength I factored axial design load:

Strength I factored axial design load: 8 kips < Factored geotechnical axial compression resistance: 187 kips OK

Demand capacity ratio = 0.04

CALCULATIONS

Date:	6/17/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole C-M1 at Hogan Road EB & Sylvan Road	**Reviewed by:**	CCB
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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	8	32.5	0.0	0.05	Yes
	2	0.50	8	65.0	0.0	0.10	Yes
	3	0.75	8	97.5	0.0	0.15	Yes
	4	1.00	8	130.0	0.0	0.20	Yes
	5	1.25	8	162.5	0.0	0.27	Yes
	6	1.50	8	195.0	0.0	0.35	Yes
Extreme I	7	0.25	7	42.5	1.5	0.08	Yes
	8	0.50	7	85.0	3.0	0.16	Yes
	9	0.75	7	127.5	4.5	0.26	Yes
	10	1.00	7	170.0	6.0	0.42	Yes
	11	1.25	7	212.5	7.5	0.68	Yes
					maximum =	0.68	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.68 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	7	120.0	3	0.22	Yes

The computed deflection of 0.22 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi$$ (Ref. 17, Eq. 2.30)

$$W = \frac{1}{4} \pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	130	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	18.8	kip	
Unit Weight of concrete, γ_c =	150	pcf	

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	3.5	0.4	29	1.5	0.7	3.0	49.5	-
Silt	7.5	0.8	22	1.5	1.2	5.0	153.2	15.7

Resistance factor for torsion, ϕ_{tor} = 1.0 (Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L = 8.0 ft (neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n = 218 kip-ft
Factored torsion resistance, T_r = 218 kip-ft

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
130	kip-ft	<	218	kip-ft	OK

Demand capacity ratio = 0.60

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection = 0.68 in (from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection = 0.30 in (from Lpile load case 11, from step 5)
Total shaft deflection = 0.99 in
Shaft length = 10.0 ft (from step 2)
Total shaft rotation = deflection / length = 0.008 rad

Maximum mast arm length for C1 = 55.00 ft (Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length = 5.42 in

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	5.42	inches	OK

The estimated vertical movement of 5.42 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

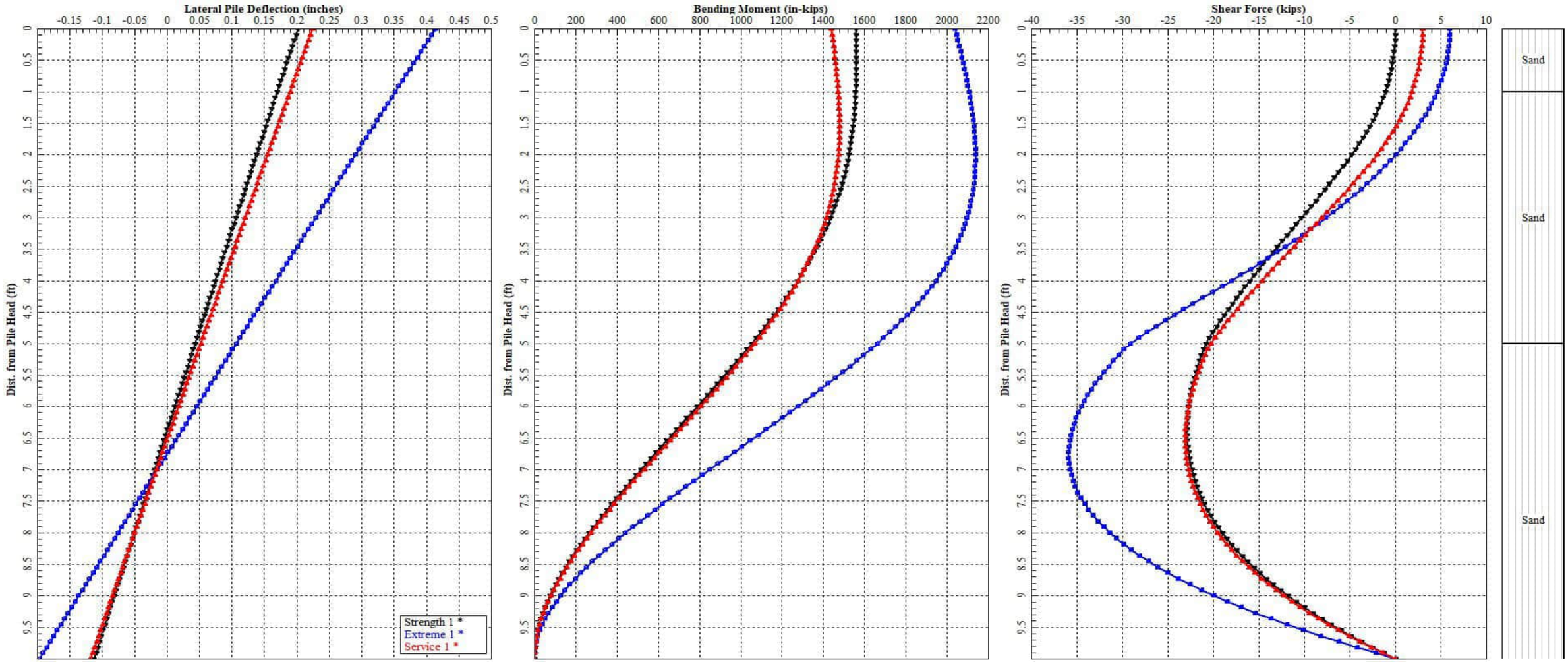
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole C-M1 at Hogan Road EB and Sylvan Road based on the final design loads provided by VHB. A maximum lateral deflection of 0.22 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.68 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.60 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 5.42 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Subject:	Drilled Shaft Design for Pole D-M1 at Sylvan Road & Hogan Road EB	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Traffic Signal D-M1 (Hogan Rd Construction Centerline Station 37+76.27, Offset 82.99' RT) at Sylvan Road and Hogan Road EB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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20. VHB. Boring Location Plan

CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheets 56 and 60, there is no direct cross section available for Mast Arm D-M1 at Sylvan Road and Hogan Road EB due to its placement beyond the limits of other earthwork. Hence, there is no proposed grade change at the mast arm location. Per Ref. 10 Sheet 28, the mast arm is located in a grass area. Therefore, the analysis assumes 1.0 feet of existing soils will be replaced with granular borrow.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced by the limited depth granular borrow added at the surface. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-216 is used for the analysis at Pole D-M1.

Proposed top of drilled shaft elevation =	77.0	ft	(Ref. 19)
Existing ground surface elevation =	77.0	ft	
Elevation to which remove/replace soil =	76.0	ft	(Assumption 2)
Groundwater elevation =	67.6	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-1	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	1-8.8	Sand (Reese)	125	24	34.0	-	127	-
Stiff Clay	8.8-9.4	Stiff Clay w/o Free Water (Reese)	122	17	-	1343	200	0.005
Stiff Clay	9.4-13.8	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Sand and Gravel	13.8-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

wsp

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	4	4	3
Moment	kip-ft	30	100	50
Shear	kip	0	5	2
Torsion	kip-ft	0	40	20

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \quad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \quad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \quad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5 \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	5.4	0.7	34.0	24	0.6	6.7	-	1.07	0.72
Stiff Clay	9.1	1.1	-	17	-	-	0.6	0.55	0.63
Stiff Clay	9.7	1.2	-	17	-	-	0.6	0.55	0.66

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Stiff Clay layer. Calculate the unit tip resistance q_p for the Stiff Clay layer. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

$$q_p = N_c S_u \leq 80\ ksf$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

penetration of shaft z = 1.2 ft

Bearing Capacity factor for Cohesive soils N_C = 6.1

Top of Stiff Clay Layer q_p = 8.2 ksf

CALCULATIONS

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Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.72	12.6	6.8	85.5	61.9	0.55	34.1
Stiff Clay	0.63		0.6	7.5	4.7	0.45	2.1
Stiff Clay	0.66		0.6	7.5	4.9	0.45	2.2

1) neglecting top 2 feet of shaft for Granular Borrow and Existing Fill length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Stiff Clay	8.2	12.6	103.2	0.40	41.3

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 77 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
4 kips	<	77 kips	OK

Demand capacity ratio = 0.05

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	4	7.5	0.0	0.01	Yes
	2	0.50	4	15.0	0.0	0.02	Yes
	3	0.75	4	22.5	0.0	0.02	Yes
	4	1.00	4	30.0	0.0	0.03	Yes
	5	1.25	4	37.5	0.0	0.04	Yes
	6	1.50	4	45.0	0.0	0.05	Yes
Extreme I	7	0.25	4	25.0	1.3	0.04	Yes
	8	0.50	4	50.0	2.5	0.08	Yes
	9	0.75	4	75.0	3.8	0.13	Yes
	10	1.00	4	100.0	5.0	0.19	Yes
	11	1.25	4	125.0	6.3	0.26	Yes
					maximum =	0.26	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.26 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	50.0	2	0.08	Yes

The computed deflection of 0.08 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

wsp

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesive soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 17 Section 2.4.3. Ref. 17 recommends neglecting the top length of the pile of 1.5-feet or 1.5*D depending on method selected. For the purpose of these calculations, the depth of skin friction to be ignored for cohesive soils will be consistent with the analyzed frost depth.

$$T_n = \frac{1}{2}\pi D^2 (L - 1.5) f_s$$ (Ref. 17, Section 2.4.3)

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesive soil. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_t = \frac{1}{12}\pi D^3 f_s$$ (Ref. 17, Section 2.4.3)

$$f_s = 0.55 S_u$$ (Ref. 17, Section 2.4.3)

where:

Factored torsional loading demand, T_u = 40 kip-ft (From Step 3 - Extreme I)

wsp

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	5.4	0.7	17	1.5	1.0	6.8	173.0	-
Stiff Clay	9.1	1.1	6.0	-	0.7	0.6	11.1	-
Stiff Clay	9.7	1.2	6.0	-	0.7	0.6	11.1	12.4

Resistance factor for torsion, ϕ_{tor} =	1.0		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	8.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	208	kip-ft	
Factored torsion resistance, T_r =	208	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
40	kip-ft	<	208	kip-ft	OK

Demand capacity ratio = 0.19

8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.26	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.12	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.38	in	
Shaft length =	10.0	ft	(from step 2)
Total shaft rotation = deflection / length =	0.003	rad	
Maximum mast arm length for C1 =	30.00	ft	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	1.13	in	

Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.13	inches	OK

The estimated vertical movement of 1.13 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

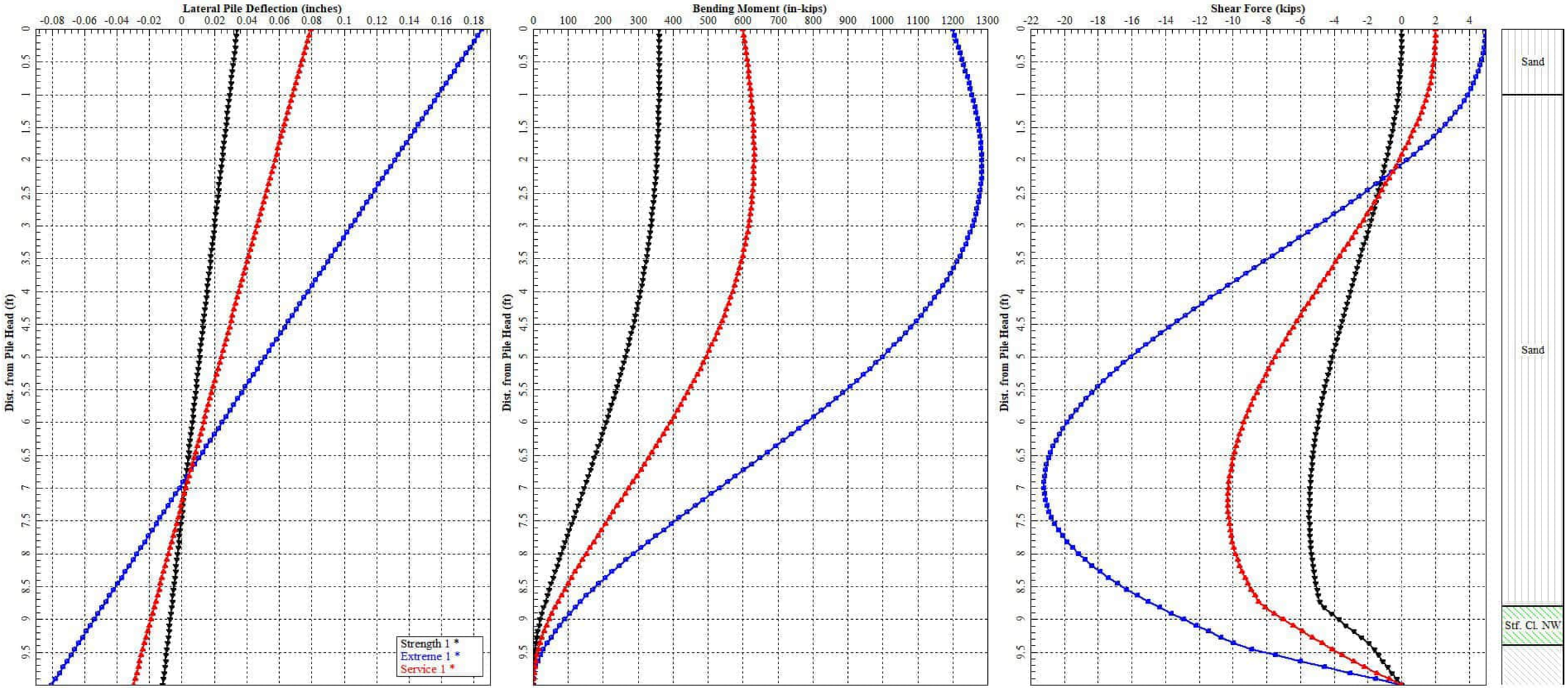
CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Mast Arm D-M1 at Sylvan Road and Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.08 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.26 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.29 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the mast arm of 1.13 inches was calculated under the Extreme I factored load times 1.25, satisfying the limiting requirements of 8 inches. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in this modeling, it is understood that WSP will perform the final structural check in a separate calculation and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for OHSS C-1 at Station 104+75	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for OHSS C-1 (Hogan Rd EB alignment Station 104+75, Offset 24.37' LT) at Hogan Road EB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 6 ft.

2. Per the cross-sections in Ref. 10 Sheet 83, a cut of 0.9 feet is proposed at the OHSS location. Therefore, the analysis assumes that no granular borrow will be placed over the exsisting soils, and 0.9 feet of exsisting soils will be removed.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced by the limited depth granular borrow added at the surface. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-207 is used for the analysis at OHSS C-1.

Proposed top of drilled shaft elevation =	115.7	ft	(Ref. 19)
Existing ground surface elevation =	116.6	ft	
Elevation to which remove/replace soil =	115.7	ft	(Assumption 2)
Groundwater elevation =	111.3	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Existing Fill	0-4.4	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	4.4-9	Sand (Reese)	63	24	34.0	-	77	-
Silt	9-14	Sand (Reese)	63	20	32.0	-	53	-
Stiff Clay	14-19	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Stiff to Medium Stiff Clay	19-22.7	Stiff Clay w/ Free Water (Reese)	59	6	-	913	N/A	0.010
Glacial Till	22.7-?	Sand (Reese)	68	42	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Overhead Sign Supports =	72	inches	=	6.0	feet
which corresponds to shaft circumference =	226	inches	=	18.8	feet
and shaft base area =	4072	in^2	=	28.3	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 20.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with thirty-six Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	36				
Steel reinforcement area per shaft =	45.72	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 19 ties for a shaft length of 20 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	19		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to OHSS were selected.

Factored Design Forces (Traffic Signal Structures)					
Load	Units	Strength I	Extreme I	Service I	Extreme II
Axial	kip	12	12	10	7
Moment	kip-ft	180	582	244	582
Shear	kip	0	25	10	25
Torsion	kip-ft	0	650	225	650

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	3.2	0.4	34.0	24	0.6	6.7	-	1.44	0.58
Existing Fill	6.7	0.7	34.0	24	0.6	6.7	-	1.06	0.73
Silt	11.5	1.0	32.0	20	0.8	10.9	-	1.05	1.04
Stiff Clay	16.5	1.3	-	17	-	-	0.6	0.55	0.74
Stiff to Medium Stiff Clay	19.5	1.5	-	6	-	-	0.4	0.55	0.50

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Stiff to Medium Stiff Clay layer. Calculate the unit tip resistance qp for the Stiff to Medium Stiff Clay layer. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$q_p = N_c S_u \leq 80\ ksf$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

penetration of shaft z = 1 ft

Bearing Capacity factor for Cohesive soils N_C = 6.0

Unit Tip Resistance, q_p = 5.5 ksf

CALCULATIONS

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Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for OHSS C-1 at Station 104+75	**Reviewed by:**	CCB
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Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.58	18.8	2.4	45.2	26.0	0.55	14.3
Existing Fill	0.73		4.6	86.7	63.6	0.55	35.0
Silt	1.04		5.0	94.2	98.1	0.55	54.0
Stiff Clay	0.74		5.0	94.2	69.6	0.45	31.3
Stiff to Medium Stiff Clay	0.50		1.0	18.8	9.5	0.45	4.3

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Stiff Clay	5.5	28.3	156.0	0.40	62.4

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$$R_R = R_{R,s} + R_{R,p} = 201 \text{ kips}$$

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
12 kips	<	201 kips	OK

Demand capacity ratio = 0.06

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 1,319,167 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	12	45.0	0.0	0.01	Yes
	2	0.50	12	90.0	0.0	0.02	Yes
	3	0.75	12	135.0	0.0	0.03	Yes
	4	1.00	12	180.0	0.0	0.04	Yes
	5	1.25	12	225.0	0.0	0.05	Yes
	6	1.50	12	270.0	0.0	0.06	Yes
Extreme I	7	0.25	12	145.5	6.3	0.06	Yes
	8	0.50	12	291.0	12.5	0.11	Yes
	9	0.75	12	436.5	18.8	0.18	Yes
	10	1.00	12	582.0	25.0	0.25	Yes
	11	1.25	12	727.5	31.3	0.35	Yes
					maximum =	0.35	

The trial shaft length of 20 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.35 inches is less than 10% of the shaft diameter (10% of 72 inches = 7.2 inches). Thus, based on the pushover analysis, the trial shaft length of 20 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	10	244.0	10	0.09	Yes

The computed deflection of 0.09 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 20 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribuition of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \leq \phi_{tor} \cdot T_n$$ (Ref. 16, Section 13.6.1.1)

$$T_n = \pi D L F_s \left(\frac{D}{2}\right)$$ (Ref. 16, Section 13.6.1.1)

$$F_s = \sigma'_v \omega_{fdot}$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15$$ (Ref. 16, Section 13.6.1.1)

$$\omega_{fdot} = 1.5\ for\ N_{value} \geq 15$$ (Ref. 16, Section 13.6.1.1)

Compute nominal and factored torsion resistance contribution of the drilled shaft sides in cohesive soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 17 Section 2.4.3. Ref. 17 recommends neglecting the top length of the pile of 1.5-feet or 1.5*D depending on method selected. For the purpose of these calculations, the depth of skin friction to be ignored for cohesive soils will be consistent with the analyzed frost depth.

$$T_n = \frac{1}{2}\pi D^2 (L - 1.5) f_s$$ (Ref. 17, Section 2.4.3)

$$f_s = 0.55 S_u$$ (Ref. 17, Section 2.4.3)

Compute nominal and factored torsion resistance contribuition of the drilled shaft toe on cohesive soil. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_t = \frac{1}{12}\pi D^3 f_s$$ (Ref. 17, Section 2.4.3)

where:

Factored torsional loading demand, T_u = 650 kip-ft (From Step 3 - Extreme I)

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Existing Fill	3.2	0.4	38	1.5	0.6	2.4	81.4	-
Existing Fill	6.7	0.7	38	1.5	1.0	4.6	270.8	-
Silt	11.5	1.0	9	0.9	0.9	5.0	253.1	-
Stiff Clay	16.5	1.3	10	-	0.7	5.0	208.8	-
Stiff to Medium Stiff Clay	19.5	1.5	5	-	0.5	1.0	28.4	28.4

Resistance factor for torsion, ϕ_{tor} =	0.9		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	18.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	871	kip-ft	
Factored torsion resistance, T_r =	784	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
650	kip-ft	<	784	kip-ft	OK

Demand capacity ratio = 0.83

CALCULATIONS

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8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

Computed maximum shaft head deflection =	0.35	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.11	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.46	in	
Shaft length =	20.0	ft	(from step 2)
Total shaft rotation = deflection / length =	0.002	rad	
Maximum mast arm length for C1 =	45.00	ft	(Ref. 8)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	1.05	in	

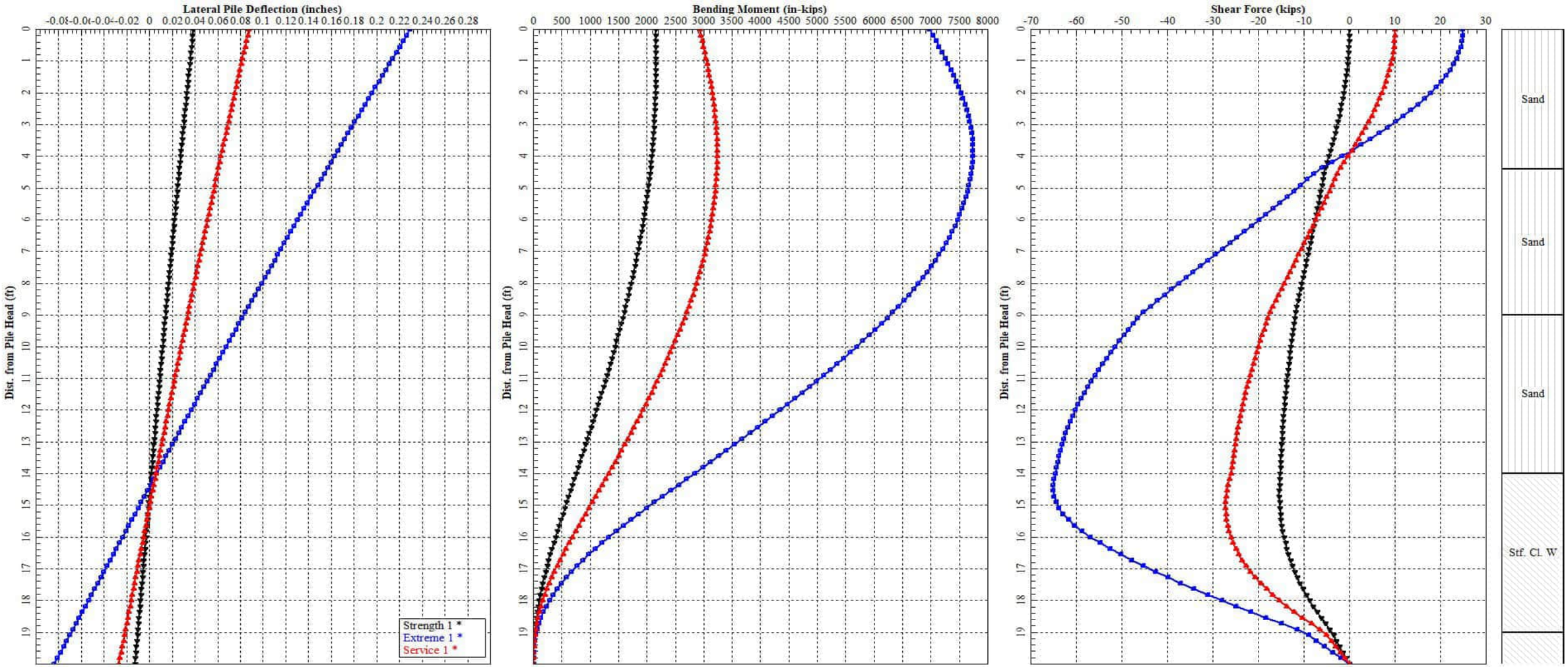
Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.05	inches	OK

The estimated vertical movement of 0.93 inches at the tip of the mast arm during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 72 inches and a shaft length of 20 feet will provide adequate support for OHSS C-1 at Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.09 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.35 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 7.2 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.83 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the cantilevering arm of 0.93 inches was calculated under the Extreme I factored load times 1.25, satisfying the limitting requirements of 8 inches. Although reinforcement consisting of thiry-six Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for OHSS C-2 at Station 900+00	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for OHSS C-2 (Hogan Rd Spur B baseline Station 900+00, Offset 12.32' LT) at Spur B based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 5 ft.

2. Per the cross-sections in Ref. 20 Sheet 168, a grade change of 17.2 feet is proposed at the OHSS location. Per Ref. 10 Sheet 25, the mast arm is located in a grass area. Therefore, the analysis assumes that 17.2 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the end of the pushover analysis is limited so that the estimated vertical movement at the tip of the mast arm will not exceed 8 inches (step 8) (Ref. 3, Section 11.8).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow. The granular borrow is assumed to be properly compacted and settlement of the granular borrow following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.
7. Check embedment length to resist torsion loading demand at the Extreme I limit state.
8. Check estimated vertical movement at the tip of the mast arm during the pushover analysis.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-201 is used for the analysis at OHSS C-2.

Proposed top of drilled shaft elevation =	114.8	ft	(Ref. 19)
Existing ground surface elevation =	97.6	ft	
Elevation to which remove/replace soil =	96.6	ft	(Assumption 2)
Groundwater elevation =	96.0	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-18.2	Sand (Reese)	125	17	32.0	-	88	-	-
Silt	18.2-18.8	Sand (Reese)	125	20	32.0	-	88	-	-
Silt	18.8-22.4	Sand (Reese)	63	20	32.0	-	53	-	-
Stiff Clay	22.4-37.4	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005	-
Glacial Till	37.4-42.4	Sand (Reese)	68	42	38.0	-	124	-	-
Bedrock	42.4-?	Strong Rock	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Overhead Sign Supports =	60	inches	=	5.0	feet
which corresponds to shaft circumference =	188	inches	=	15.7	feet
and shaft base area =	2827	in^2	=	19.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 15.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twenty-two Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	22				
Steel reinforcement area per shaft =	27.94	in^2	=	0.99%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 14 ties for a shaft length of 15 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	14		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to OHSS were selected.

Factored Design Forces (Traffic Signal Structures)					
Load	Units	Strength I	Extreme I	Service I	Extreme II
Axial	kip	10	10	8	6
Moment	kip-ft	100	381	153	381
Shear	kip	0	15	5	15
Torsion	kip-ft	0	350	125	350

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	8.5	1.1	32.0	17	0.6	5.5	-	0.70	0.74

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Granular Borrow layer. Calculate the unit tip resistance q_p for the Granular Borrow layer. Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 17 ksf (From Step 1)

Exsisting Fill q_p = 20.4 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.74	15.7	13.0	204.2	151.6	0.55	83.4

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Granular Borrow	20.4	19.6	400.6	0.50	200.3

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 284 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 10 kips < Factored geotechnical axial compression resistance: 284 kips OK

Demand capacity ratio = 0.04

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model =	4,000,000	psi	(Ref. 4, page 9-21)
Moment of inertia for linear model, I =	636,173	in^4	(Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	10	25.0	0.0	0.01	Yes
	2	0.50	10	50.0	0.0	0.03	Yes
	3	0.75	10	75.0	0.0	0.04	Yes
	4	1.00	10	100.0	0.0	0.06	Yes
	5	1.25	10	125.0	0.0	0.07	Yes
	6	1.50	10	150.0	0.0	0.09	Yes
Extreme I	7	0.25	10	95.3	3.8	0.08	Yes
	8	0.50	10	190.5	7.5	0.16	Yes
	9	0.75	10	285.8	11.3	0.24	Yes
	10	1.00	10	381.0	15.0	0.32	Yes
	11	1.25	10	476.3	18.8	0.42	Yes
					maximum =	0.42	

The trial shaft length of 15 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.42 inches is less than 10% of the shaft diameter (10% of 60 inches = 6.0 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	8	153.0	5	0.12	Yes

The computed deflection of 0.12 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 15 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CALCULATIONS

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7. Check embedment length to resist torsion loading demand at the Extreme I limit state.

Compute nominal and factored torsion resistance contribuition of the drilled shaft sides in cohesionless soils. Use the method in Ref. 15 Section 4.2.2 and Ref. 16 Section 13.6.1.1.

$$T_u \le \phi_{tor} \cdot T_n \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$T_n = \pi D L F_s \left(\frac{D}{2}\right) \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$F_s = \sigma'_v \omega_{fdot} \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$\omega_{fdot} = 1.5 * \left(\frac{N_{value}}{15}\right) for\ N_{value} < 15 \quad \text{(Ref. 16, Section 13.6.1.1)}$$

$$\omega_{fdot} = 1.5\ for\ N_{value} \ge 15 \quad \text{(Ref. 16, Section 13.6.1.1)}$$

Compute nominal and factored torsion resistance contribution of the drilled shaft toe on cohesionless soil. Use the method in Ref. 17 Section 2.4.6.

$$T_t = \frac{1}{3} DW tan\phi \quad \text{(Ref. 17, Eq. 2.30)}$$

$$W = \frac{1}{4}\pi D^2 h * \gamma_c$$

where:

Factored torsional loading demand, T_u =	350	kip-ft	(From Step 3 - Extreme I)
Pile Weight, W =	44.2	kip	
Unit Weight of concrete, γ_c =	150	pcf	

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Uncorrected N-Value	Load Transfer Ratio ω_{fdot}	Unit Skin Friction F_s	Length of Shaft L	Side Res. T_n	Toe Res. T_t
	(ft)	(ksf)			(ksf)	(ft)	(kip-ft)	(kip-ft)
Granular Borrow	8.5	1.1	11	1.1	1.2	13.0	596.7	46.0

Resistance factor for torsion, ϕ_{tor} =	0.9		(Ref. 16, Section 13.6.1.1)
Shaft length contributing to resistance, L =	13.0	ft	(neglecting top 2 feet as per Ref. 14)
Nominal torsion resistance, T_n =	643	kip-ft	
Factored torsion resistance, T_r =	578	kip-ft	

Check against Extreme I factored torsion design load:

Extreme I factored torsion design load:			Factored torsion resistance:		
350	kip-ft	<	578	kip-ft	OK

Demand capacity ratio = 0.61

CALCULATIONS

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8. Check estimated vertical movement at the tip of the OHSS during the pushover analysis.

Computed maximum shaft head deflection =	0.42	in	(from Lpile load case 11, from step 5)
Computed maximum shaft tip deflection =	0.17	in	(from Lpile load case 11, from step 5)
Total shaft deflection =	0.59	in	
Shaft length =	15.0	ft	(from step 2)
Total shaft rotation = deflection / length =	0.003	rad	
Maximum mast arm length for C-2 =	35.00	ft	(Ref. 19)
Vertical movement at tip of mast arm = shaft rotation x mast arm length =	1.38	in	

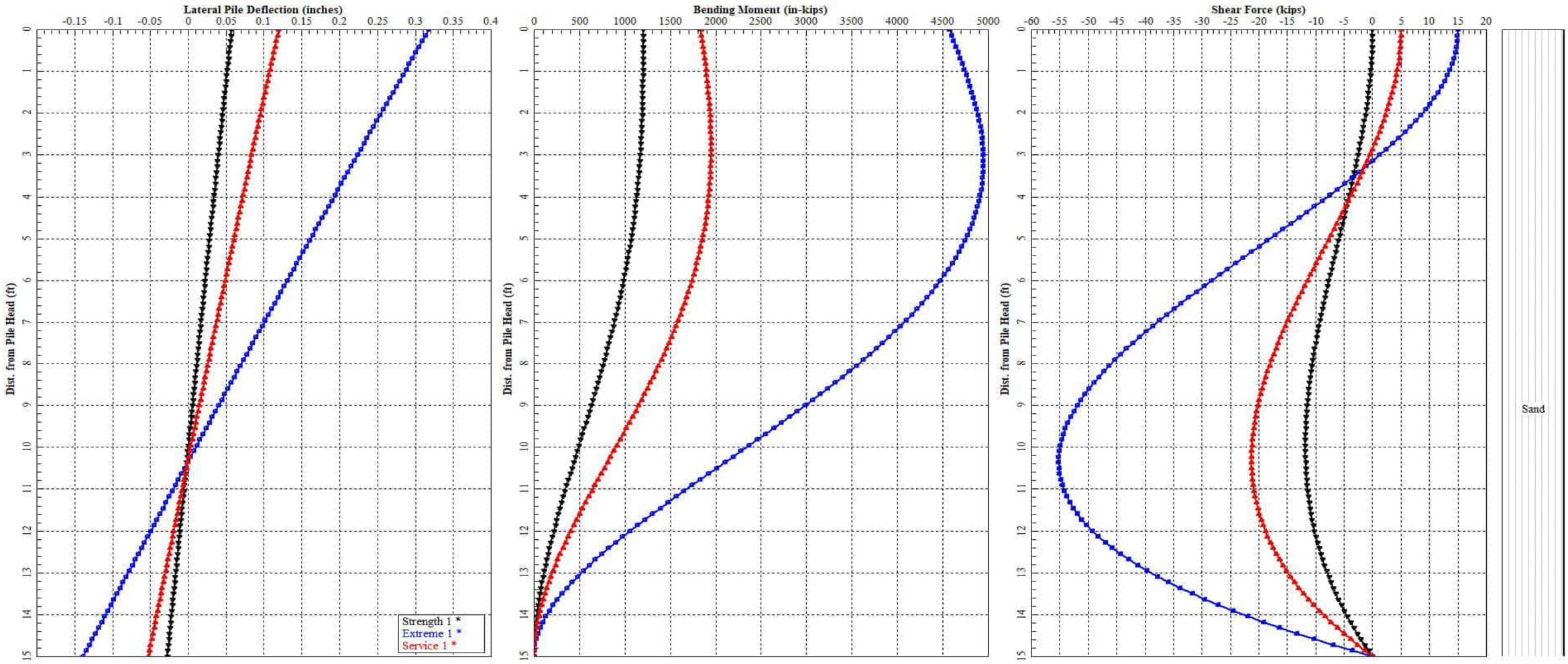
Allowable vertical movement at the tip of mast arm			Estimated vertical movement at the tip of the mast arm		
8	inches	>	1.38	inches	OK

The estimated vertical movement of 1.36 inches at the tip of the OHSS during the pushover analysis is less than the required limit of 8 inches (Assumption 4). Thus the total shaft rotation will be considered sufficiently small.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 60 inches and a shaft length of 15 feet will provide adequate support for OHSS C-2 at Spur B based on the final design loads provided by VHB. A maximum lateral deflection of 0.12 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.42 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 6.0 inches, 10% of the shaft diameter. The shaft length is controlled by the torsional resistance, and a torsion demand capacity ratio of 0.41 was calculated under the Extreme I load case. A maximum vertical movement at the tip of the cantilevering arm of 1.36 inches was calculated under the Extreme I factored load times 1.25, satisfying the limitting requirements of 8 inches. Although reinforcement consisting of twenty-two Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

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Subject:	Drilled Shaft Design for OHSS OH-1 Right Support at Station 219+50	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for OHSS OH-1 Right Support (Hogan Rd WB alignment Station 219+50, Offset 39.19' RT) at Hogan Road WB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 5 ft.

2. Per the cross-sections in Ref. 10 Sheet 125, a grade change of 2.5 feet is proposed at the OHSS location. Per Ref. 10 Sheet 27, the mast arm is located in a grass area. Therefore, the analysis assumes that 2.5 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the pushover analysis is not required, as a double-membered horizontal sign structure is not required to have vertical deflections checked due to their inherent stiffness (Ref. 3, Section C11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced by the limited depth granular borrow added at the surface. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

wsp

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-211 is used for the analysis at OHSS C-1 (Right).

Proposed top of drilled shaft elevation =	96.9	ft	(Ref. 19)
Existing ground surface elevation =	94.4	ft	
Elevation to which remove/replace soil =	93.4	ft	(Assumption 2)
Groundwater elevation =	83.5	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-3.5	Sand (Reese)	125	17	32.0	-	88	-
Silt	3.5-11.2	Sand (Reese)	125	20	32.0	-	88	-
Silt	11.2-13.4	Sand (Reese)	63	20	32.0	-	53	-
Stiff Clay	13.4-18.4	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Sand and Gravel	18.4-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

wsp

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Overhead Sign Supports =	60	inches	=	5.0	feet
which corresponds to shaft circumference =	188	inches	=	15.7	feet
and shaft base area =	2827	in^2	=	19.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 15.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twenty-two Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	22				
Steel reinforcement area per shaft =	27.94	in^2	=	0.99%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 14 ties for a shaft length of 15 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	14		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to OHSS were selected.

Factored Design Forces (Traffic Signal Structures)					
Load	Units	Strength I	Extreme I	Service I	Extreme II
Axial	kip	16	14	12	9
Moment	kip-ft	0	450	175	450
Shear	kip	0	20	10	20
Torsion	kip-ft	0	0	0	0

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \quad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \quad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \quad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \quad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \leq \frac{S_u}{p_a} \leq 2.5 \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \quad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	2.8	0.3	32.0	17	0.6	5.5	-	1.27	0.44
Silt	7.4	0.9	32.0	20	0.6	6.0	-	0.79	0.73
Silt	12.3	1.5	32.0	20	0.6	6.0	-	0.62	0.91
Stiff Clay	14.2	1.6	-	17	-	-	0.6	0.55	0.74

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Stiff Clay layer. Calculate the unit tip resistance qp for the Stiff Clay layer. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1c-1)}$$

$$q_p = N_c S_u \leq 80\ ksf \qquad \text{(Ref. 1, Eq. 10.8.3.5.1c-2)}$$

where:

penetration of shaft z = 1.6 ft

Bearing Capacity factor for Cohesive soils N_C = 6.4

Unit Tip Resistance, q_p = 8.6 ksf

CALCULATIONS

Date: 9/5/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
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Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.44	15.7	1.5	23.6	10.3	0.55	5.7
Silt	0.73		7.7	121.0	88.3	0.55	48.6
Silt	0.91		2.2	34.6	31.5	0.55	17.3
Stiff Clay	0.74		1.6	25.1	18.6	0.45	8.4

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Stiff Clay	8.6	19.6	168.3	0.40	67.3

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$$R_R = R_{R,s} + R_{R,p} = \quad 147 \quad \text{kips}$$

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
16 kips	<	147 kips	OK

Demand capacity ratio = 0.11

CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for OHSS OH-1 Right Support at Station 219+50	**Reviewed by:**	CCB
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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 636,173 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	1.00	16	0.0	0.0	0.00	Yes
Extreme I	2	0.25	14	112.5	5.0	0.08	Yes
	3	0.50	14	225.0	10.0	0.18	Yes
	4	0.75	14	337.5	15.0	0.29	Yes
	5	1.00	14	450.0	20.0	0.43	Yes
	6	1.25	14	562.5	25.0	0.64	Yes
					maximum =	0.64	

The trial shaft length of 15 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.64 inches is less than 10% of the shaft diameter (10% of 60 inches = 6.0 inches). Thus, based on the pushover analysis, the trial shaft length of 15 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

wsp

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

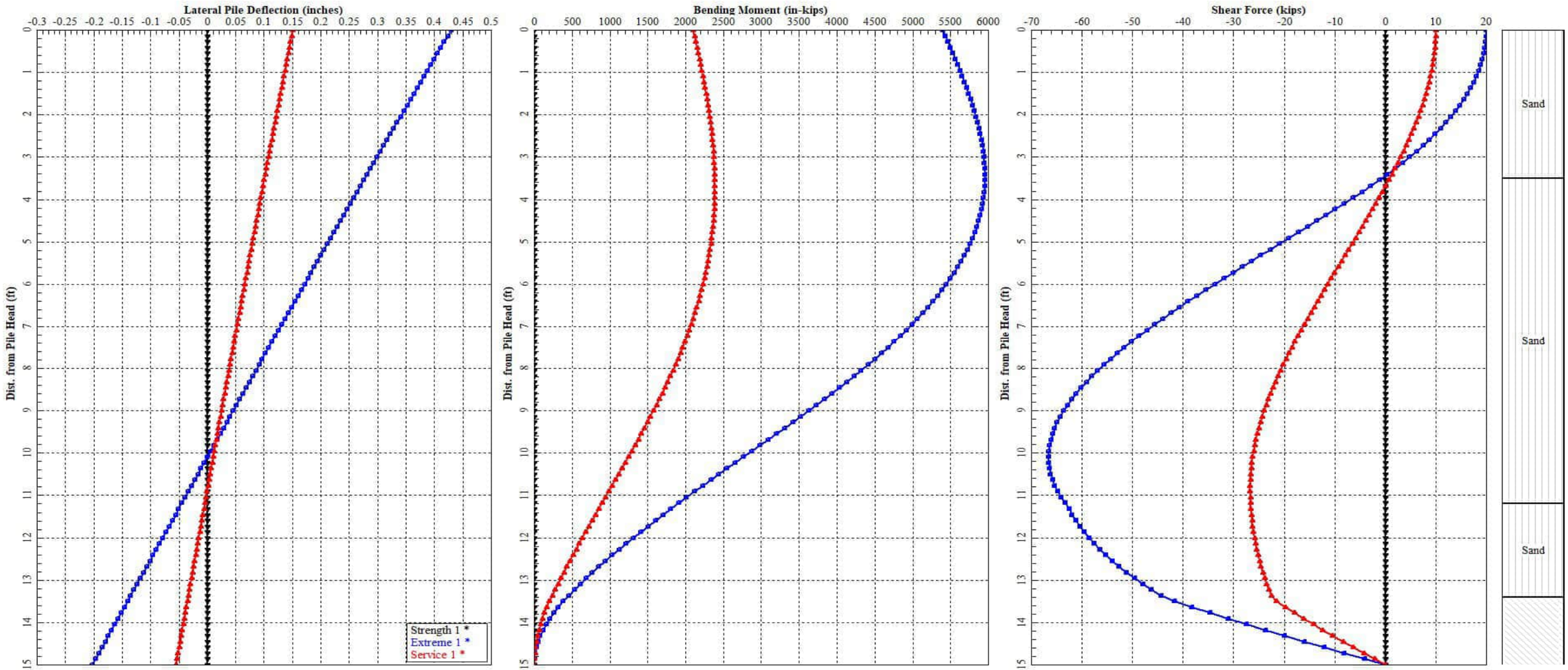
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	7	1.00	12	175.0	10	0.15	Yes

The computed deflection of 0.15 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 60 inches and a shaft length of 15 feet will provide adequate support for OH-1 Right Support at Hogan Road WB based on the final design loads provided by VHB. A maximum lateral deflection of 0.15 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.64 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 6.0 inches, 10% of the shaft diameter. Although reinforcement consisting of twenty-two Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
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Subject:	Drilled Shaft Design for OHSS OH-1 Left Support at Station 401+50	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for OHSS OH-1 Left Support (Hogan Rd Ramp D alignment Station 401+50, Offset 27.00' RT) at Hogan Road WB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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CALCULATIONS

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ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 5 ft.

2. Per the cross-sections in Ref. 10 Sheet 125, a grade change of 1.5 feet is proposed at the OHSS location. Per Ref. 10 Sheet 27, the mast arm is located in a grass area. Therefore, the analysis assumes that 1.5 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1). Furthermore, total shaft rotation at the pushover analysis is not required, as a double-membered horizontal sign structure is not required to have vertical deflections checked due to their inherent stiffness (Ref. 3, Section C11.8).

5. Although a portion of the shaft length is embedded within cohesive soils, its overall settlement behavior is not anticipated to be influenced by the limited depth granular borrow added at the surface. Settlement of the drilled shaft foundation is not expected to result in downdrag loading, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

CALCULATIONS

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1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-212 is used for the analysis at OHSS C-1 (Left).

Proposed top of drilled shaft elevation =	98.7	ft	(Ref. 19)
Existing ground surface elevation =	97.2	ft	
Elevation to which remove/replace soil =	96.2	ft	(Assumption 2)
Groundwater elevation =	85.9	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-2.5	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	2.5-11.2	Sand (Reese)	125	24	34.0	-	127	-
Stiff to Medium Stiff Clay	11.2-12.8	Stiff Clay w/o Free Water (Reese)	121	6	-	913	N/A	0.010
Stiff to Medium Stiff Clay	12.8-16.2	Stiff Clay w/ Free Water (Reese)	59	6	-	913	N/A	0.010
Stiff Clay	16.2-21.2	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Sand and Gravel	21.2-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

wsp

CALCULATIONS

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2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Overhead Sign Supports =	60	inches	=	5.0	feet
which corresponds to shaft circumference =	188	inches	=	15.7	feet
and shaft base area =	2827	in^2	=	19.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length =	15.0	feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with twenty-two Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	22				
Steel reinforcement area per shaft =	27.94	in^2	=	0.99%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 14 ties for a shaft length of 15 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	14		(Ref. 13, pages 626(01) & 626(02))

wsp

CALCULATIONS

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3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to OHSS were selected.

Factored Design Forces (Traffic Signal Structures)					
Load	Units	Strength I	Extreme I	Service I	Extreme II
Axial	kip	14	14	12	9
Moment	kip-ft	0	450	150	450
Shear	kip	0	20	10	20
Torsion	kip-ft	0	0	0	0

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

Calculate the unit side resistance q_s at the midpoint of cohesive soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \propto S_u \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-1)}$$

$$\propto = 0.55 - 0.1\left(\frac{S_u}{p_a} - 1.5\right) \text{for } 1.5 \le \frac{S_u}{p_a} \le 2.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-2)}$$

$$\propto = 0.55 \text{ for } \frac{S_u}{p_a} < 1.5 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1b-3)}$$

CALCULATIONS

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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	2.3	0.3	32.0	17	0.6	5.5	-	1.41	0.40
Existing Fill	6.9	0.9	34.0	24	0.6	6.7	-	0.94	0.80
Stiff to Medium Stiff Clay	12.0	1.5	-	6	-	-	0.4	0.55	0.50
Stiff to Medium Stiff Clay	13.9	1.7	-	6	-	-	0.4	0.55	0.50

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Stiff to Medium Stiff Clay layer. Calculate the unit tip resistance qp for the Stiff to Medium Stiff Clay layer. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c). Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9 \qquad \text{(Ref. 1, Eq. 10.8.3.5.1c-1)}$$

$$q_p = N_c S_u \leq 80\ ksf \qquad \text{(Ref. 1, Eq. 10.8.3.5.1c-2)}$$

where:

penetration of shaft z = 2.2 ft

Bearing Capacity factor for Cohesive soils N_C = 6.5

Unit Tip Resistance, q_p = 6.0 ksf

wsp

CALCULATIONS

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Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.40	15.7	0.5	7.9	3.1	0.55	1.7
Existing Fill	0.80		8.7	136.7	110.0	0.55	60.5
Stiff to Medium Stiff Clay	0.50		1.6	25.1	12.6	0.45	5.7
Stiff to Medium Stiff Clay	0.50		2.2	34.6	17.4	0.45	7.8

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Stiff to M. Stiff Clay	6.0	19.6	117.0	0.40	46.8

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 123 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
14 kips	<	123 kips	OK

Demand capacity ratio = 0.11

wsp

CALCULATIONS

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Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 636,173 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	1.00	14	0.0	0.0	0.00	Yes
Extreme I	2	0.25	14	112.5	5.0	0.07	Yes
	3	0.50	14	225.0	10.0	0.17	Yes
	4	0.75	14	337.5	15.0	0.28	Yes
	5	1.00	14	450.0	20.0	0.47	Yes
	6	1.25	14	562.5	25.0	0.77	Yes
					maximum =	0.77	

The trial shaft length of 15 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.77 inches is less than 10% of the shaft diameter (10% of 60 inches = 6.0 inches). Thus, based on the pushover analysis, the trial shaft length of 15 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

wsp

CALCULATIONS

Date:	9/5/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for OHSS OH-1 Left Support at Station 401+50	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

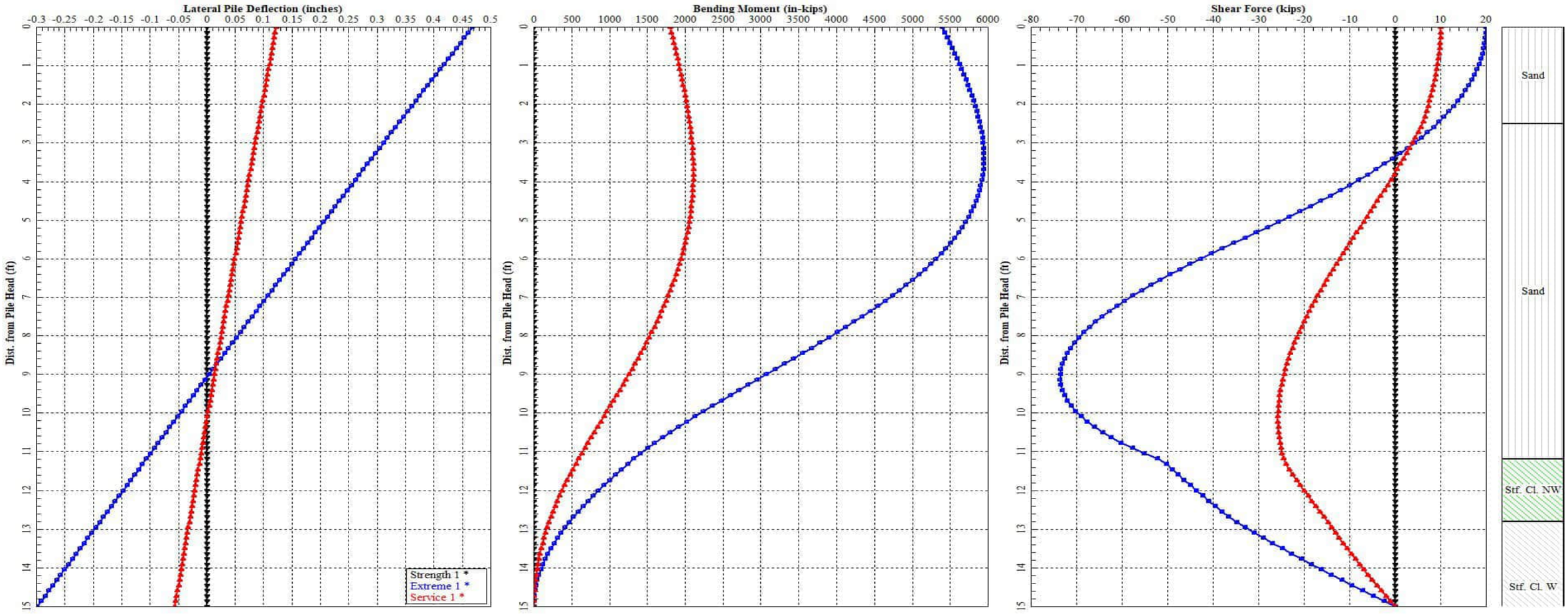
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	7	1.00	12	150.0	10	0.12	Yes

The computed deflection of 0.12 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 15 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 60 inches and a shaft length of 15 feet will provide adequate support for OHSS OH-1 Left Support at Ramp D based on the final design loads provided by VHB. A maximum lateral deflection of 0.12 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.77 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 6.0 inches, 10% of the shaft diameter. Although reinforcement consisting of twenty-two Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Span Wire Pole A (Hogan Rd Construction Centerline Station 15+36.44, Offset 71.95' LT) at Hogan Road WB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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5. WSP geotechnical test boring logs for 200-series borings.
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18. WSP Lab Field Data Summary Plots
19. WSP Mast Arm and Light Design Basis

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 109, a grade raise of 2.2 feet is proposed at the span wire pole location. Per Ref. 10 Sheet 24, the pole is located in a grass area. Therefore, the analysis assumes that 2.2 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-207 is used for the analysis at Pole A.

Proposed top of drilled shaft elevation =	115.5	ft	(Ref. 19)
Existing ground surface elevation =	113.3	ft	
Elevation to which remove/replace soil =	112.3	ft	(Assumption 2)
Groundwater elevation =	111.3	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-3.2	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	3.2-4.2	Sand (Reese)	125	24	34.0	-	127	-	-
Existing Fill	4.2-8.8	Sand (Reese)	63	24	34.0	-	77	-	-
Silt	8.8-13.8	Sand (Reese)	63	20	32.0	-	53	-	-
Stiff Clay	13.8-18.8	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005	-
Stiff to Medium Stiff Clay	18.8-22.5	Stiff Clay w/ Free Water (Reese)	59	6	-	913	N/A	0.010	-
Bedrock	22.5-?	Sand (Reese)	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	3	3	3
Moment	kip-ft	40	230	110
Shear	kip	2	8	4
Torsion	kip-ft	0	0	0

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

Date: 6/6/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole A at Hogan Road WB, Bangor **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	2.6	0.3	32.0	17	0.6	5.5	-	1.31	0.43
Existing Fill	3.7	0.5	34.0	24	0.6	6.7	-	1.33	0.61
Existing Fill	6.5	0.7	34.0	24	0.6	6.7	-	1.08	0.72
Silt	9.4	0.9	32.0	20	0.8	10.9	-	1.14	0.97

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Silt layer. Calculate the unit tip resistance q_p for the Silt layer. Because the tip of the shaft is overlying a weaker layer and it is less than 1.5 the diameter deeper than the shaft tip, the base resistance will need to be reduced (Ref. 1, Article 10.8.3.5.3). Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c) to find the tip resistance 1.5B above the top of the weaker layer; the value of qp should be limited to 60 ksf unless greater values can be justified through load test data. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c) to find the tip resistance at the top of the weaker layer. Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

Average N_{60} value for stronger layer = 20

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

Silt q_p = 24.0 ksf

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$q_p = N_c S_u \leq 80\ ksf$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

penetration of shaft z = 0 ft

Bearing Capacity factor for Cohesive soils N_C = 6.0

Top of Stiff Clay Layer q_p = 8.1 ksf

where:

Δq_p = 2.7 ksf / ft

Tip of Drilled Shaft q_p = 18.2 ksf

CALCULATIONS

Date: 6/6/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole A at Hogan Road WB, Bangor **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.43	12.6	1.2	15.1	6.4	0.55	3.5
Existing Fill	0.61		1.0	12.6	7.7	0.55	4.2
Existing Fill	0.72		4.6	57.8	41.7	0.55	23.0
Silt	0.97		1.2	15.1	14.6	0.55	8.0

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Silt	18.2	12.6	228.1	0.50	114.1

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 153 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
3 kips	<	153 kips	OK

Demand capacity ratio = 0.02

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3	10.0	0.5	0.03	Yes
	2	0.50	3	20.0	1.0	0.06	Yes
	3	0.75	3	30.0	1.5	0.08	Yes
	4	1.00	3	40.0	2.0	0.11	Yes
	5	1.25	3	50.0	2.5	0.14	Yes
	6	1.50	3	60.0	3.0	0.17	Yes
Extreme I	7	0.25	3	57.5	2.0	0.15	Yes
	8	0.50	3	115.0	4.0	0.31	Yes
	9	0.75	3	172.5	6.0	0.61	Yes
	10	1.00	3	230.0	8.0	1.16	Yes
	11	1.25	3	287.5	10.0	2.17	Yes
					maximum =	2.17	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 2.17 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

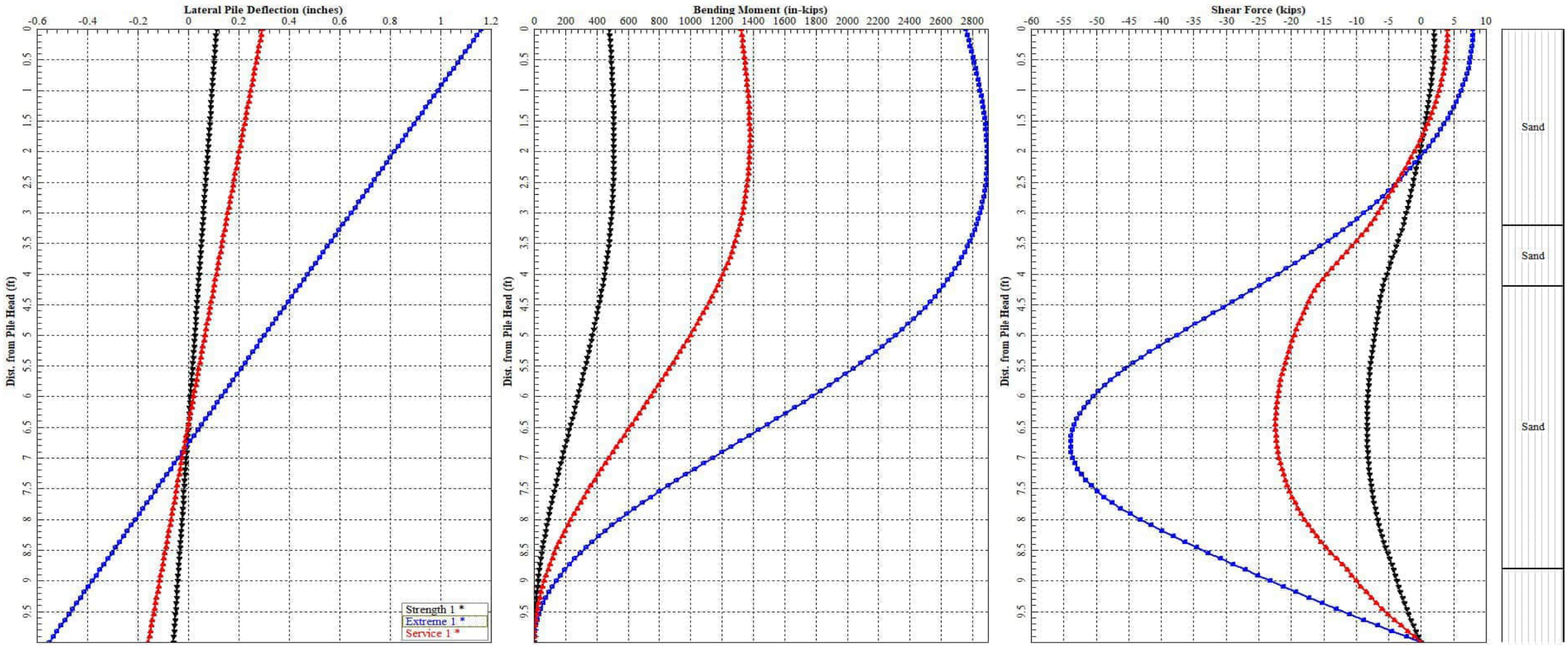
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	110.0	4	0.29	Yes

The computed deflection of 0.29 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Span Wire Pole A at Hogan Road WB based on the final design loads provided by VHB. A maximum lateral deflection of 0.29 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 2.17 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Span Wire Pole B (Hogan Rd Construction Centerline Station 15+36.44, Offset 52.92' RT) at Hogan Road WB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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4. FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018.
5. WSP geotechnical test boring logs for 200-series borings.
6. WSP geotechnical test boring logs for 100-series borings.
7. Bridge Software Institute. FB-MultiPier Soil Parameter Table (US Customary Units). Accessed July 2021. https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf
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13. MaineDOT. Standard Details. March 2020.
14. VTrans. Materials & Research Engineering Instructions MREI 10-01. March 9, 2010.
15. Rodriguez, C.M., et al. Final Report: State of Practice and Literature Review on Foundations for Coastal Traffic Signal Mast Arm Structures. State of North Carolina Department of Transportation Research & Development, Report No. FHWA/NC/2018-17. May 14, 2020.
16. Florida Department of Transportation. FDOT Modifications to LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (LRFDLTS-1). Structures Manual, Volume 3. January 2021.
17. Auburn University. Torsional Resistance of Drilled Shaft Foundations. The Auburn University Highway Research Center. October 2018
18. WSP Lab Field Data Summary Plots
19. WSP Mast Arm and Light Design Basis

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 109, a grade raise of 2.2 feet is proposed at the span wire pole location. Per Ref. 10 Sheet 24, the pole is located in a grass area. Therefore, the analysis assumes that 2.2 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

CALCULATIONS

Date: 6/6/2025 **Made by:** LDN
Project No.: 31404817.003 **Checked by:** JKW
Subject: Drilled Shaft Design for Pole B at Hogan Road WB, Bangor **Reviewed by:** CCB
Project Title: MaineDOT Bangor I-95 Exit 187 DDI

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-207 is used for the analysis at Pole B.

Proposed top of drilled shaft elevation =	112.9	ft	(Ref. 19)
Existing ground surface elevation =	112.4	ft	
Elevation to which remove/replace soil =	111.4	ft	(Assumption 2)
Groundwater elevation =	106.7	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]	UCS (psi)[4]
Granular Borrow	0-1.5	Sand (Reese)	125	17	32.0	-	88	-	-
Existing Fill	1.5-6.2	Sand (Reese)	125	24	34.0	-	127	-	-
Silt	6.2-11.2	Stiff Clay w/ Free Water (Reese)	63	20	32.0	-	53	-	-
Stiff Clay	11.2-16.2	Sand (Reese)	60	17	-	1343	200	0.005	-
Stiff to Medium Stiff Clay	16.2-19.9	Sand (Reese)	59	6	-	913	N/A	0.01	-
Bedrock	19.9-?	Sand (Reese)	96	-	-	-	-	-	2117

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

4) Ref. 5, 6, and 9

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.
This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2		(Ref. 4, Table 6-2, #10-size)	
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi		(Ref. 4, Section 12.2.2)	
Steel elastic modulus, E =	29,000	ksi		(Ref. 4, Section 12.2.2)	

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
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Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	3	3	3
Moment	kip-ft	40	230	100
Shear	kip	2	8	4
Torsion	kip-ft	0	0	0

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

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CALCULATIONS

Date: 6/6/2025 **Made by:** LDN
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Subject: Drilled Shaft Design for Pole B at Hogan Road WB, Bangor **Reviewed by:** CCB
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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	4.1	0.5	34.0	24	0.6	6.7	-	1.25	0.64
Silt	8.1	0.9	32.0	20	0.8	10.9	-	1.11	0.99

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Silt layer. Calculate the unit tip resistance qp for the Silt layer. Because the tip of the shaft is overlying a weaker layer and it is less than 1.5 the diameter deeper than the tip, the base resistance will need to be reduced (Ref. 1, Article 10.8.3.5.3). Use the cohesionless soil method (Ref. 1, Article 10.8.3.5.2c) to find the tip ressitance 1.5B above the top of the weaker layer; the value of qp should be limited to 60 ksf unless greater values can be justified through load test data. Use the cohesive method (Ref. 1, Article 10.8.3.5.1c) to find the tip resistance at the top of the weaker layer. Note that as per Ref. 1 Article 10.8.3.5.1c, the value of qp should be limited to 80 ksf unless greater values can be justified through load test data.

where:

Average N_{60} value for stronger layer = 20

$$\text{If } N_{60} \leq 50, \text{then } q_p = 1.2N_{60} \leq 60 \text{ ksf}$$ (Ref. 1, Eq. 10.8.3.5.2c-1)

Silt q_p = 24.0 ksf

where:

$$N_c = 6\left[1 + 0.2\left(\frac{Z}{D}\right)\right] \leq 9$$ (Ref. 1, Eq. 10.8.3.5.1c-1)

$$q_p = N_c S_u \leq 80\ ksf$$ (Ref. 1, Eq. 10.8.3.5.1c-2)

where:

penetration of shaft z = 0 ft

Bearing Capacity factor for Cohesive soils N_C = 6.0

Top of Stiff Clay Layer q_p = 8.1 ksf

where:

Δq_p = 2.7 ksf / ft

Tip of Drilled Shaft q_p = 11.2 ksf

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CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.64	12.6	4.2	52.8	33.9	0.55	18.6
Silt	0.99		3.8	47.8	47.3	0.55	26.0

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	11.2	12.6	141.3	0.50	70.7

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p} =$ 115 kips

Check against Strength I factored axial design load:

Strength I factored axial design load:		Factored geotechnical axial compression resistance:	
3 kips	<	115 kips	OK

Demand capacity ratio = 0.03

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3	10.0	0.5	0.02	Yes
	2	0.50	3	20.0	1.0	0.05	Yes
	3	0.75	3	30.0	1.5	0.07	Yes
	4	1.00	3	40.0	2.0	0.10	Yes
	5	1.25	3	50.0	2.5	0.12	Yes
	6	1.50	3	60.0	3.0	0.14	Yes
Extreme I	7	0.25	3	57.5	2.0	0.13	Yes
	8	0.50	3	115.0	4.0	0.27	Yes
	9	0.75	3	172.5	6.0	0.51	Yes
	10	1.00	3	230.0	8.0	0.97	Yes
	11	1.25	3	287.5	10.0	1.81	Yes
					maximum =	1.81	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 1.81 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road WB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

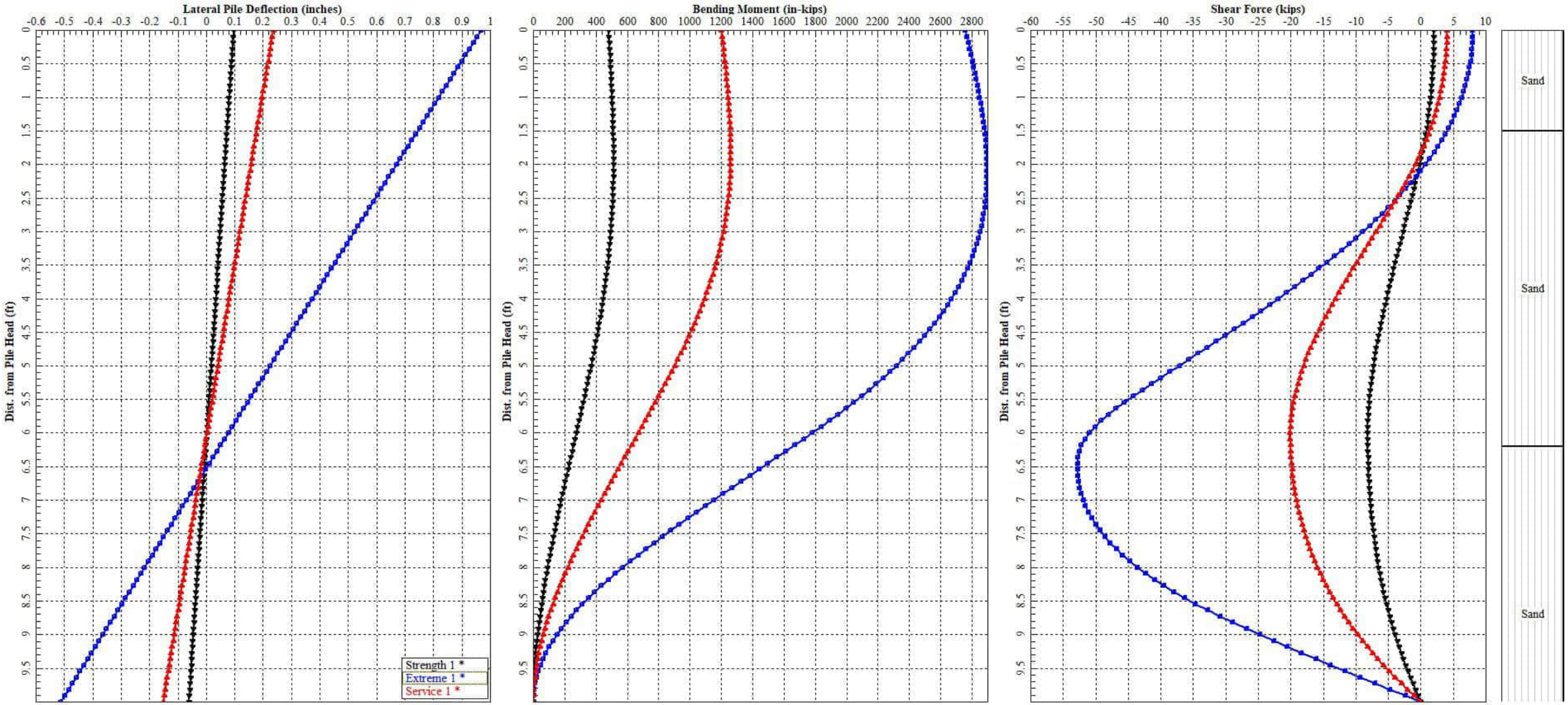
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	100.0	4	0.24	Yes

The computed deflection of 0.24 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Pole B at Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.24 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 1.81 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Span Wire Pole A (Hogan Rd Construction Centerline Station 35+57.60, Offset 64.62' RT) at Hogan Road EB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
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17. Auburn University. Torsional Resistance of Drilled Shaft Foundations. The Auburn University Highway Research Center. October 2018
18. WSP Lab Field Data Summary Plots
19. WSP Mast Arm and Light Design Basis

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 104, a grade raise of 5.3 feet is proposed at the span wire pole location. Per Ref. 10 Sheet 27, the pole is located in a grass area. Therefore, the analysis assumes that 5.3 feet of granular borrow will be placed over the exsisting soils, and 1.0 feet of exsisting soils will be replaced with granular borrow for grubbing.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1).

5. Settlement of the drilled shaft foundation is expected to mimic the surrounding embankment as the full shaft length is embedded within the compacted granular borrow and natural cohesionless soils. The granular borrow is assumed to be properly compacted and settlement of the granular borrow and underlying cohesionless soils following the installation of the drilled shaft foundation will be negligible, hence additional loading due to downdrag is not considered in this analysis.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-216 is used for the analysis at Pole A.

Proposed top of drilled shaft elevation =	85.1	ft	(Ref. 19)
Existing ground surface elevation =	79.8	ft	
Elevation to which remove/replace soil =	78.8	ft	(Assumption 2)
Groundwater elevation =	74.4	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Granular Borrow	0-6.3	Sand (Reese)	125	17	32.0	-	88	-
Existing Fill	6.3-10.7	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	10.7-16.9	Sand (Reese)	63	24	34.0	-	77	-
Stiff Clay	16.9-21.9	Stiff Clay w/ Free Water (Reese)	60	17	-	1343	200	0.005
Sand and Gravel	21.9-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Mast Arms were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	3	3	3
Moment	kip-ft	40	210	90
Shear	kip	2	7	3
Torsion	kip-ft	0	0	0

As per Assumption 5, no downdrag loading will be added.

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $$\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-4)}$$

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $$\frac{\sigma'_p}{p_a} = 0.15(N_{60}) \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-5)}$$

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer $\varphi'f$	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Granular Borrow	4.2	0.5	32.0	17	0.6	5.5	-	1.02	0.53
Existing Fill	8.2	1.0	34.0	24	0.6	6.7	-	0.85	0.87

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Although the tip of the shaft is overlying a weaker layer, it is more than 1.5 the diameter deeper than the shaft tip meaning the base resistance will not need to be reduced (Ref. 1, Article 10.8.3.5.3). Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

If $N_{60} \leq 50, q_p = 1.2N_{60}$ (Ref. 1, Eq. 10.8.3.5.2c-1)

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Exsisting Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Granular Borrow	0.53	12.6	4.3	54.0	28.6	0.55	15.8
Existing Fill	0.87		3.7	46.5	40.4	0.55	22.2

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 219 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 3 kips < Factored geotechnical axial compression resistance: 219 kips OK

Demand capacity ratio = 0.01

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3	10.0	0.5	0.02	Yes
	2	0.50	3	20.0	1.0	0.05	Yes
	3	0.75	3	30.0	1.5	0.07	Yes
	4	1.00	3	40.0	2.0	0.10	Yes
	5	1.25	3	50.0	2.5	0.12	Yes
	6	1.50	3	60.0	3.0	0.14	Yes
Extreme I	7	0.25	3	52.5	1.8	0.12	Yes
	8	0.50	3	105.0	3.5	0.23	Yes
	9	0.75	3	157.5	5.3	0.38	Yes
	10	1.00	3	210.0	7.0	0.67	Yes
	11	1.25	3	262.5	8.8	1.23	Yes
					maximum =	1.23	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 1.23 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole A at Hogen Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

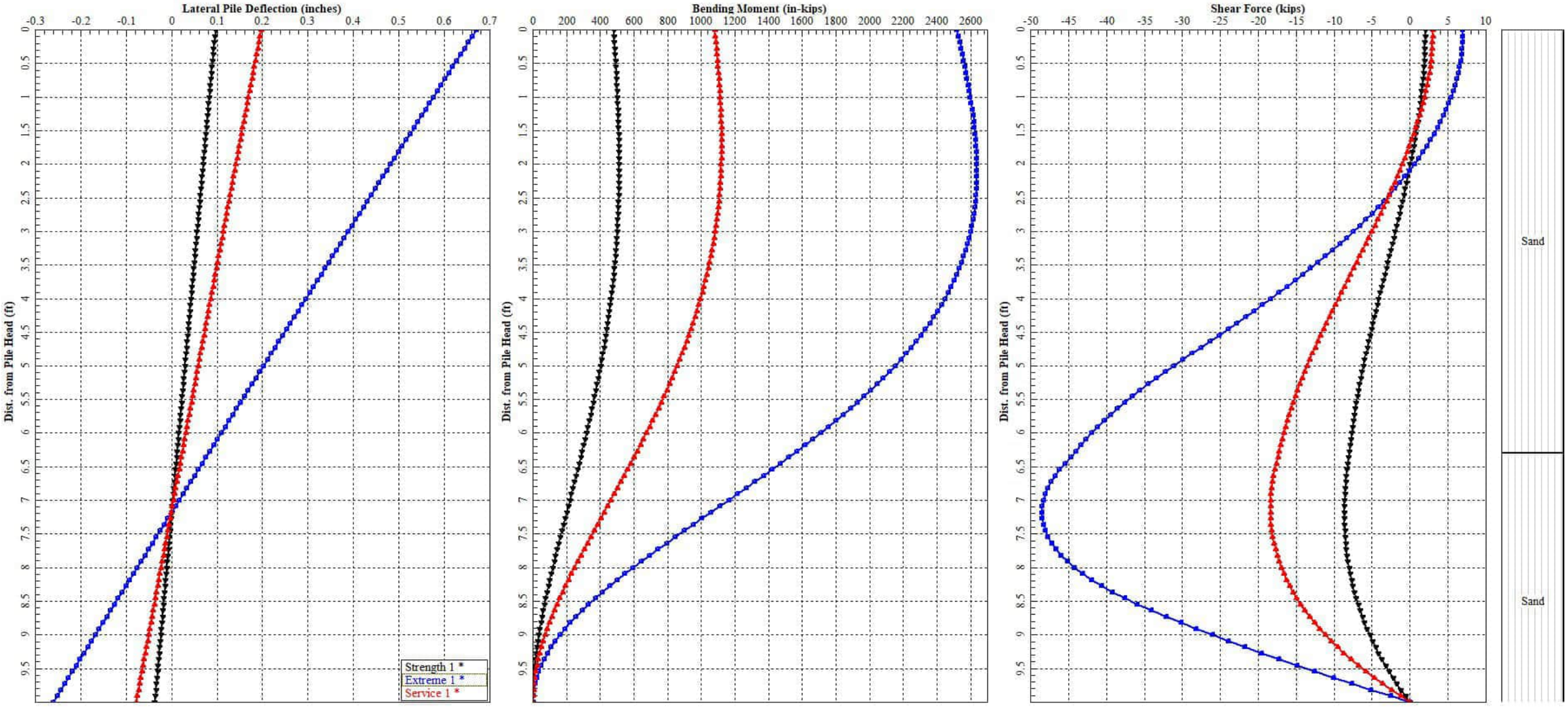
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	90.0	3	0.20	Yes

The computed deflection of 0.20 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Span Wire Pole A at Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.20 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 1.23 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

OBJECTIVE

Determine if the proposed drilled shaft foundation will provide adequate support for Span Wire Pole B (Hogan Rd Construction Centerline Station 35+57.60, Offset 48.35' LT) at Hogan Road EB based on the assumed design loads provided by VHB.

METHOD

As per the MaineDOT Bridge Design Guide Section 5.8 (Ref. 2), use the procedure outlined in AASHTO LRFD Article 10.8 (Ref. 1) and FHWA GEC-10 (Ref. 4).

REFERENCES

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2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 with 2017, 2018, and 2020 Interim Revisions.
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19. WSP Mast Arm and Light Design Basis

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

ASSUMPTIONS

1. The drilled shafts are assumed to be installed either open hole or with temporary casing that is removed after construction. It is assumed that permanent casing will not be used, and the effect of permanent casing is not included in the analysis of nominal side resistance. The shaft diameter is assumed to be 4 ft.

2. Per the cross-sections in Ref. 10 Sheet 128, a cut of 1.4 feet is proposed at the span wire pole location. The analysis assumes that granular borrow will not be placed over the existing soils, and 1.4 ft of existing soils will be removed.

3. Typical steel reinforcement details are assumed for the purpose of the serviceability limit state lateral geotechnical analysis. Full steel reinforcement design and structural analysis will be performed by a non-WSP structural engineer.

4. Shaft head deflection is limited to 0.5 inches under Service loads (Ref. 4, Section 9.3.3.3.3) and less than 10% of the shaft diameter under a "pushover" analysis (step 5) (Ref. 4, Section 9.3.3.3.1).

5. The immediate settlement of the cohesionless soils is assumed to be sufficiently small that downdrag loading will be negligible.

ATTACHMENTS

1. LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)

CALCULATION

Summary of calculation steps:

1. Define subsurface profile for analysis.
2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.
3. Define the factored design loads for the analysis.
4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.
5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.
6. Check horizontal movement at the top of the shaft at the Service I limit state.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

1. Define subsurface profile for analysis.

The soil stratigraphy encountered in nearby boring HB-BHR-213 is used for the analysis at Pole B.

Proposed top of drilled shaft elevation =	86.1	ft	(Ref. 19)
Existing ground surface elevation =	87.5	ft	
Elevation to which remove/replace soil =	86.1	ft	(Assumption 2)
Groundwater elevation =	66.5	ft	(Ref. 5 and 6)

Layer	Depth below top of shaft (ft)	Lateral Model	Effective Unit Weight (pcf)[2]	Average N_{60} Value[1]	Design Friction Angle (°)[2]	Undrained Shear Strength (psf)[2]	Subgrade Modulus (pci)[3]	Major Principal Strain at 50%[3]
Existing Fill	0-8.7	Sand (Reese)	125	24	34.0	-	127	-
Existing Fill	8.7-19.6	Sand (Reese)	63	24	34.0	-	77	-
Silt	19.6-29.6	Sand (Reese)	63	20	32.0	-	53	-
Sand and Gravel	29.6-?	Sand (Reese)	68	52	38.0	-	124	-

1) Granular borrow assumed N_{60} value correlated from design friction angle using Peck, Hanson, and Thornburn (1974): $\phi = 27.1 + 0.3N_{60} - 0.00054[N_{60}]^2$, friction design angle of 32.0 (Ref. 2, Table 3-3). The N_{60} values for the in-situ soils are based on the average of N_{60} values encountered in all highway and bridge borings for each soil layer.

2) Existing soils correlated to average N_{60} value for each layer, based on all borings (Ref. 5, calculated in Ref. 18)

3) Ref. 7. Interpolation based on friction angle for cohesionless layers and on undrained shear strength for cohesive layers.

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

2. Select trial shaft diameter, shaft length, and shaft reinforcement for evaluation.

The analysis will be evaluated with the assumed shaft diameter (Assumption 1):

Shaft diameter for Standard Lighting =	48	inches	=	4.0	feet
which corresponds to shaft circumference =	151	inches	=	12.6	feet
and shaft base area =	1810	in^2	=	12.6	ft^2

The analysis will be evaluated with the minimum shaft length required for the LPile analysis to converge, since non-convergence indicates an unstable shaft condition (Ref. 4, Section 9.3.3.3.1).

Shaft length = 10.0 feet

Ref. 3 Article 13.6.2.1 requires a minimum concrete cover of 3 inches over steel reinforcement (to protect against corrosion). Ref. 4 Section 12.4 recommends a minimum concrete cover of 4 inches for shaft diameters greater than 3 feet and smaller than 5 feet.

Concrete cover to edge of bar =	3	in	(Ref. 3, Article 13.6.2.1)
Concrete compressive strength, f'_c =	5	ksi	(Ref. 1, Article C5.6.4.2)

Ref. 1 Article 5.6.4.2 and Ref. 4 Section 12.3.1 recommend a steel longitudinal reinforcement area of 1% to 1.5% of the gross area of the section and require a minimum of six bars in a circular arrangement plus a minimum #5 bar size. The MaineDOT Standard Details (Ref. 13, pg. 626(01) and 626(02)) specify six #6-size bars for an 18-inch diameter shaft and eight #6-size bars for a 24-inch diameter shaft, corresponding to 1.04% and 0.78% steel area, respectively.

This analysis will be evaluated with sixteen Grade 60 #10-size bars, arranged in a symmetrical circular pattern with single-bar bundles, for the purpose of geotechnical design only.

Steel reinforcement area per bar =	1.27	in^2			(Ref. 4, Table 6-2, #10-size)
Number of bars =	16				
Steel reinforcement area per shaft =	20.32	in^2	=	1.12%	of total shaft area
Steel yield strength, F_y =	60	ksi			(Ref. 4, Section 12.2.2)
Steel elastic modulus, E =	29,000	ksi			(Ref. 4, Section 12.2.2)

As per Assumption 3, steel transverse reinforcement for structural design purposes is not analyzed. Tie hoop reinforcement will be assumed for constructability purposes, as per Ref. 4 Section 6.1. Assume #3-size ties (based on Ref. 13 page 626(02)) spaced a maximum of 12 inches apart (based on Ref. 1 Article 5.7.2.6) and need 6 inches of space from the ends of the drilled shaft (based on Ref. 13 page 626(01) & 626 (02)), resulting in 9 ties for a shaft length of 10 feet.

Steel reinforcement area per tie, A_v =	0.11	in^2	(Ref. 4, Table 6-2, #3-size)
Steel yield strength for ties =	60	ksi	(Ref. 4, Section 12.2.2)
Spacing of the ties, s =	12	in	(Ref. 1, Article 5.7.2.6)
Number of ties =	9		(Ref. 13, pages 626(01) & 626(02))

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
Project No.:	31404817.003	**Checked by:**	JKW
Subject:	Drilled Shaft Design for Pole B at Hogan Road EB, Bangor	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI		

3. Define the factored design loads for the analysis.

The factored design loads at the top of the shaft were provided by VHB (Ref. 8). The loads corresponding to Span Wire Poles were selected.

Factored Design Forces (Traffic Signal Structures)				
Load	Units	Strength I	Extreme I	Service I
Axial	kip	3	3	3
Moment	kip-ft	40	200	90
Shear	kip	2	7	3
Torsion	kip-ft	0	0	0

As per Assumption 5, no downdrag loading will be added.

CALCULATIONS

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4. Check geotechnical axial compression resistance of the shaft at the Strength I limit state.

Compute nominal side resistance for all layers through which the shaft extends and nominal tip resistance for the layer at the tip elevation. Select appropriate resistance factors and calculate factored resistances.

Note that as per Ref. 14 page 5, a minimum of the upper 2 feet of soil should be neglected for contributions to skin friction resistance. As per Ref. 1 Article C10.8.3.5.1b, for cohesive soils at least the top 5 feet of any shaft should not be taken to contribute to the development of resistance through skin friction, to account for the effects of seasonal moisture changes, disturbance during construction, cyclic lateral loading, and low lateral stresses from freshly placed concrete.

The factored resistance of the drilled shaft, R_R, shall be taken as:

$$R_R = \varphi R_n = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-1)}$$

consisting of the nominal shaft tip resistance, R_p:

$$R_p = q_p A_p \qquad \text{(Ref. 1, Eq. 10.8.3.5-2)}$$

and the nominal shaft side resistance, R_s:

$$R_s = q_s A_s \qquad \text{(Ref. 1, Eq. 10.8.3.5-3)}$$

Calculate the unit side resistance q_s at the midpoint of cohesionless soil layers. Use the beta method for cohesionless soils (Ref. 1, Article 10.8.3.5.2b).

$$q_s = \beta \sigma'_v \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-1)}$$

$$\beta = (1 - \sin\phi'_f)\left(\frac{\sigma'_p}{\sigma'_v}\right)^{\sin\phi'_f} \tan\phi'_f \qquad \text{(Ref. 1, Eq. 10.8.3.5.2b-2)}$$

For sands: $\frac{\sigma'_p}{p_a} = 0.47(N_{60})^m$ (Ref. 1, Eq. 10.8.3.5.2b-4)

Sand constant, m = 0.6 for clean quartzitic sands (Ref. 1, Article 10.8.3.5.2b)
Sand constant, m = 0.8 for silty sand to sandy silts (Ref. 1, Article 10.8.3.5.2b)

For gravelly soils: $\frac{\sigma'_p}{p_a} = 0.15(N_{60})$ (Ref. 1, Eq. 10.8.3.5.2b-5)

Atmospheric pressure, p_a = 2.12 ksf (Ref. 1, Article 10.8.3.5.2b)

wsp

CALCULATIONS

Date:	6/6/2025	**Made by:**	LDN
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	Depth to Midpoint of Layer	Vertical Effective Stress at Midpoint σ'_v	Friction Angle of Layer φ'f	Average N_{60} of Layer	Sand Constant m	Precon. Stress σ'_p	S_u/p_a	β or ∝	q_s
	(ft)	(ksf)	(deg)			(ksf)			(ksf)
Existing Fill	5.4	0.7	34.0	24	0.6	6.7	-	1.08	0.72
Existing Fill	9.4	1.1	34.0	24	0.6	6.7	-	0.81	0.91

Based on the shaft length selected in Step 2, the drilled shaft will terminate in the Existing Fill layer. Calculate the unit tip resistance q_p for the Existing Fill layer. Use the cohesionless method (Ref. 1, Article 10.8.3.5.2c). Note that as per Ref. 1 Article 10.8.3.5.2c, the value of q_p should be limited to 60 ksf unless greater values can be justified through load test data.

$$\text{If } N_{60} \leq 50, q_p = 1.2N_{60} \qquad \text{(Ref. 1, Eq. 10.8.3.5.2c-1)}$$

where:

Average N_{60} value for layer = 24 ksf (From Step 1)

Exsisting Fill q_p = 28.8 ksf

Compute nominal and factored side and tip resistances for all layers.

	q_s (ksf)	Shaft circumference (ft)	Length of shaft[1] (ft)	A_s (ft^2)	R_s (kips)	φ_{qs}[2]	$R_{R,s}$ (kips)
Existing Fill	0.72	12.6	6.7	84.2	60.8	0.55	33.4
Existing Fill	0.91		1.3	16.3	14.8	0.55	8.2

1) neglecting top 2 feet of shaft for Granular Borrow length as per Ref. 14, page 5

2) Ref. 1, Table 10.5.5.2.4-1

	q_p (ksf)	A_p (ft^2)	R_p (kips)	φ_{qp}[1]	$R_{R,p}$ (kips)
Existing Fill	28.8	12.6	361.9	0.50	181.0

1) Ref. 1, Table 10.5.5.2.4-1

Factored geotechnical axial compression resistance of the drilled shaft:

$R_R = R_{R,s} + R_{R,p}$ = 223 kips

Check against Strength I factored axial design load:

Strength I factored axial design load: 3 kips < Factored geotechnical axial compression resistance: 223 kips OK

Demand capacity ratio = 0.01

wsp

CALCULATIONS

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5. Check lateral geotechnical resistance of the shaft at the Strength I and Extreme I limit states.

Perform a pushover analysis using LPile to compute shaft head deflection at various multiples of the factored shear and moment design loads, up to 1/φ times the factored shear and moment design loads. As per Ref. 4 Section 9.3.3.3.1, for a stable condition the analyses should each converge to a solution with a computed deflection no larger than 10% of the shaft diameter. For the pushover analysis the shaft should be modeled as a simple linear elastic beam rather than a nonlinear stress-strain model (Ref. 4, Section 9.3.3.3.1).

Elastic modulus for linear model = 4,000,000 psi (Ref. 4, page 9-21)

Moment of inertia for linear model, I = 260,576 in^4 (Ref. 4, page 9-21: $I = \pi D^4/64$)

Recommended resistance factor φ for lateral geotechnical resistance (Ref. 4, Table 9-1):

Limit State	φ	1/φ
Strength I	0.67	1.5
Extreme I	0.8	1.25

Computed shaft deflection at each load multiple analyzed with LPile:

Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 10% of Shaft Diameter?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Strength I	1	0.25	3	10.0	0.5	0.02	Yes
	2	0.50	3	20.0	1.0	0.04	Yes
	3	0.75	3	30.0	1.5	0.06	Yes
	4	1.00	3	40.0	2.0	0.08	Yes
	5	1.25	3	50.0	2.5	0.10	Yes
	6	1.50	3	60.0	3.0	0.12	Yes
Extreme I	7	0.25	3	50.0	1.8	0.09	Yes
	8	0.50	3	100.0	3.5	0.19	Yes
	9	0.75	3	150.0	5.3	0.32	Yes
	10	1.00	3	200.0	7.0	0.52	Yes
	11	1.25	3	250.0	8.8	0.86	Yes
					maximum =	0.86	

The trial shaft length of 10 feet exhibits stable behavior up through 1/φ times the factored design loads, and the maximum computed deflection of 0.86 inches is less than 10% of the shaft diameter (10% of 48 inches = 4.8 inches). Thus, based on the pushover analysis, the trial shaft length of 10 feet satisfies the lateral geotechnical criterion at the Strength I and Extreme I limit states.

CALCULATIONS

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6. Check horizontal movement at the top of the shaft at the Service I limit state.

Use LPile to compute shaft head deflection at the Service I factored design loads. As per Ref. 4 page 9-27 and Ref. 14 page 3, deflection due to combined loading on the structure should be limited to 0.5 inches at the top of the shaft. For the serviceability analysis the shaft should be modeled as a nonlinear reinforced concrete shaft in flexure (Ref. 4, Section 9.3.3.3.3).

Computed shaft deflection analyzed with LPile:

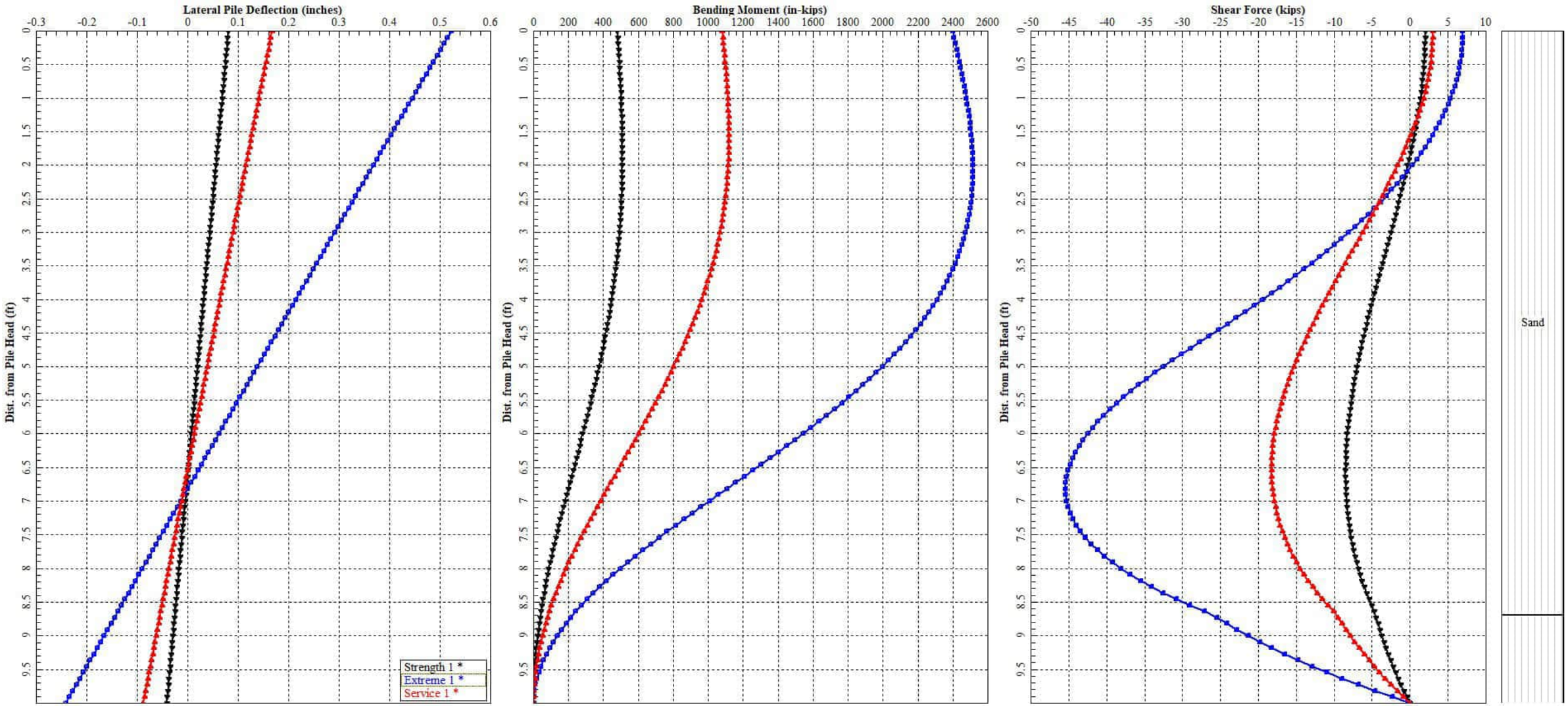
Limit State	LPile Load Case	Multiple of Factored Design Loads	Loads Applied in LPile			Computed Lateral Head Deflection from LPile (in)	Deflection < 0.5 Inches?
			Axial (kips)	Moment (kip-ft)	Shear (kips)		
Service I	12	1.00	3	90.0	3	0.17	Yes

The computed deflection of 0.17 inches is less than the required limit of 0.5 inches. Thus the trial shaft length of 10 feet satisfies the lateral criterion at the Service I limit state. The LPile analysis output for factored design loads at the Service I limit state is included as Attachment 1.

CONCLUSIONS

The results of the analysis indicate that the proposed drilled shaft foundation with a shaft diameter of 48 inches and a shaft length of 10 feet will provide adequate support for Span Wire Pole B at Hogan Road EB based on the final design loads provided by VHB. A maximum lateral deflection of 0.17 inches occurs at the top of the shaft under the Service I load case, satisfying the limiting requirement of 0.5 inches. A maximum lateral deflection of 0.86 inches occurs at the top of the shaft under the Extreme I load case, satisfying the limiting requirement of 4.8 inches, 10% of the shaft diameter. Although reinforcement consisting of sixteen Grade 60 #10 bars arranged in a circular pattern was used in WSP's modeling, it is understood that VHB will perform the final structural check and generate the required reinforcement pattern.

Attachment 1 - LPile analysis output for Strength I, Extreme I, and Service I (nonlinear model)



APPENDIX G

Structural Foundation Calculations

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 4ft Dia. Drilled Shaft

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

General:

- The following calculations cover the design for the drilled shaft foundation that supports the Spanwire poles or Mast Arms.

References:

- AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020 [BDS]
- AASHTO Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 w/ 2017-2020 Interims [AASHTO Signs]
- FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018. [FHWA]
- ACI 318-14, Building Code Requirements for Structural Concrete and Commentary [ACI]
- AISC 360-10, Steel Construction Manual 14th Ed. [SCM]
- MaineDOT Bridge Design Guide, August 2003 with 2018 additions [MaineDOT BDG]
- MaineDOT Standard Details and Specifications [MaineDOT Specs]
- Summary of LPile Results - Drilled Shafts 6.3.25 Spreadsheet provided by Geotech https://wsponlinenam.sharepoint.com/:x:/r/sites/GLD-157336/Project%20Files/5%20Technical%20Work/GDR%20Geotech%20Mast%20Arms%20%26%20OHSS/Summary%20Table%20of%20LPile%20Results%20-%20Drilled%20Shafts%206.3.25.xlsx?d=wd4aecb6215c24738a71d3498e30f9ca3&csf=1&web=1&e=LvfUnZ

Assumptions:

- Foundation diameter, lengths and loads were provided by the WSP geotechnical team.
- AASHTO LRFD governs design unless a more conservative provision is included in the MaineDOT Bridge Design Guide.
- Per MaineDOT BDG Table 6-2 Class A Concrete shall be used for the drilled shafts.
- The highest value for shear, axial, moment and torsion demands will be used for the calculations.

Drilled Shaft Parameters:

$D_{shaft} := 4\ ft$ — Drilled shaft diameter, provided by Geotechnical team.

$L_{shaft} := 12.5\ ft$ — Drilled shaft length

$cover := 3\ in$ — Rebar cover, BDS Table 5.10.1-1

$A_g := \frac{D_{shaft}^{2} \cdot \pi}{4} = 1809.557\ in^2$ — Shaft cross sectional area

$I_{yy} := \frac{D_{shaft}^{4} \cdot \pi}{64} = 260576.3\ in^4$ — Moment of inertia of the shaft

$I_{xx} := I_{yy} = 260576.3\ in^4$ — Moment of inertia

$r_{xx} := \sqrt{\frac{I_{xx}}{A_g}} = 12\ in$ — Radius of gyration

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 4ft Dia. Drilled Shaft

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

Drilled Shaft Reinforcement Properties:

$cover = 3\ in$ — Rebar cover

$d_{bh.st} := 0.625\ in$ — Stirrup rebar diameter, #5

$N_b := 21$ — Total number vertical bars

$A_b := 0.79\ in^2$ — Area of long. rebar, #8

$d_b := 1\ in$ — Long. rebar dia #8

$f_y := 60\ ksi$ — Yielding strength of steel reinforcement

Drilled Shaft Material Properties:

$f_c := 4\ ksi$ — Compressive strength Class A Conc, MaineDOT Spec Section 502 Table 1

$\beta_1 := .85$ — For $f_c = 4\ ksi$, BDS, Section 5.6.2.2

$\gamma_c := 150\ pcf$ — Concrete unit weight w/ reinforcement, BDS Table 3.5.1-1 + BDS C3.5.1

$E_s := 29000\ ksi$ — Steel Elastic Modulus

$K_1 := 1.0$ — Aggregate correction factor, BDS Section 5.4.2.4

$$E_c := 120000 \cdot K_1 \cdot \left(\frac{\gamma_c}{\frac{kip}{ft^3}} \right)^{2.0} \left(\frac{f_c}{ksi} \right)^{0.33} \cdot ksi = 4266.2\ ksi$$

Concrete Elastic Modulus, BDS Eqn. 5.4.2.4-1

Maximum Demands on Drilled Shaft from LPile Analysis:

$\gamma_p := 1.25$ — DC load factor Strength I, BDS Table 3.4.1-2

$P_{geo\&sw} := 8\ kip + \gamma_p \cdot A_g \cdot L_{shaft} \cdot \gamma_c = 37.5\ kip$ — Axial loading from geotech design basis + self weight

$M_{geo} := 2900\ kip \cdot in = 241.7\ kip \cdot ft$ — Maximum moment from LPile results

$V_{geo} := 135\ kip$ — Maximum shear from LPile results

$T_{geo} := 140\ kip \cdot ft$ — Maximum torsions from geotech design basis

SP-Column Analysis for Drilled Shaft:

SP-Column input:

Applied axial load and maximum moment demand as calculated by geotechnical team.

SP-Column output:

No.	Demand		Capacity		Parameters at Capacity			Capacity
	Pu	Mux	ϕPn	ϕMnx	NA Depth	εt	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	40.00	250.00	0.00	1405.29	8.77	0.01200	0.900	0.54

Figure 1: Factored loads and moments with corresponding capacity ratios

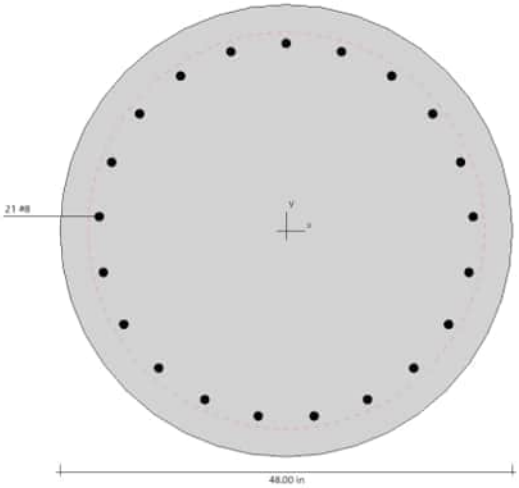


Figure 2: Cross-section and reinforcement layout

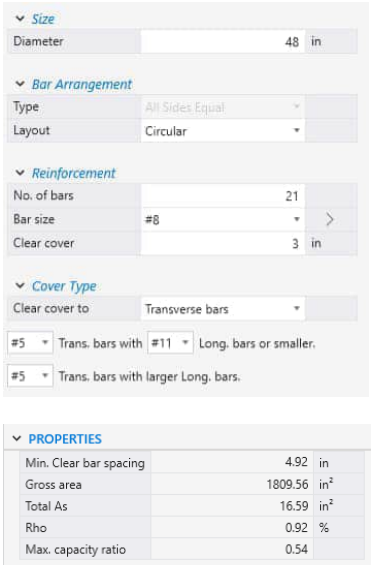


Figure 3: Reinforcement detail

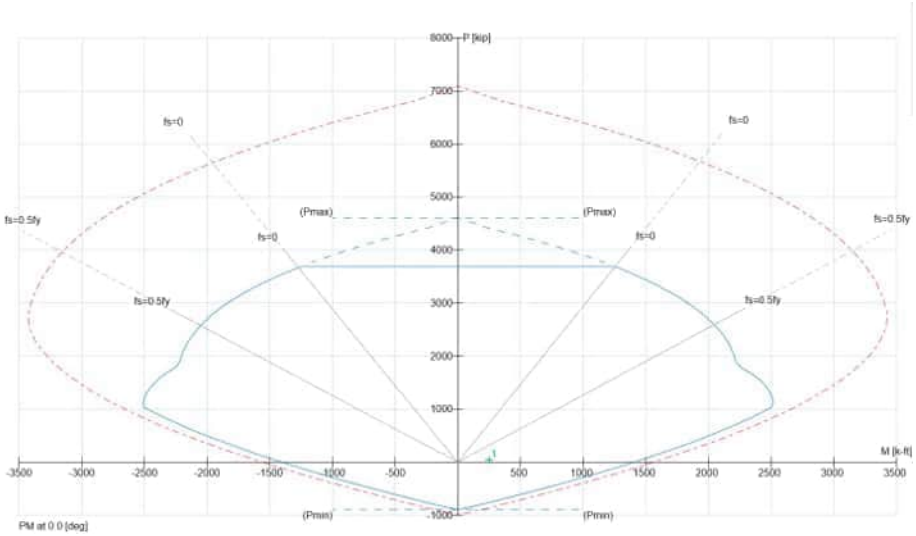


Figure 4: P&M interaction diagram

Reinforcement Limits:

Longitudinal reinforcement limit:

$\rho_{min} := 0.135 \cdot \left(\frac{f_c}{f_y}\right) = 0.009$ — BDS Eqn. 5.6.4.2-3

$A_{smin} := \rho_{min} \cdot A_g = 16.3\ in^2$ — Minimum area of steel

$\rho_{max} := 0.08$ — BDS Eqn. 5.6.4.2-1

$A_{smax} := \rho_{max} \cdot A_g = 144.8\ in^2$ — Maximum area of steel

$n_{bar} := N_b = 21$ — Assumed no. of bars

$A_b = 0.79\ in^2$

$A_s := n_{bar} \cdot A_b = 16.6\ in^2$ — Assumed area of steel

$A_{smin} \le A_s \le A_{smax} = 1$ $\quad \rho := \frac{A_s}{A_g} = 0.92\%$ — Check, OK

Minimum spacing check:

$s_{min} := min\left(1.5 \cdot d_b, 1.5\ in\right) = 1.5\ in$ — BDS 5.10.3.1.1

$s_{actual} := 4.92\ in$ — Actual clear spacing as reported in SP-Column.

$s_{min} < s_{actual} = 1$ — Check, OK

Design for Shear and Torsion:

- General procedure for non-prestressed section, used for the shear and torsion reinforcement design per AASHTO LRFD Bridge Design, Section 5.7

$\phi := 0.9$ — Strength reduction factor, shear (BDS 5.5.4)

$b_{v.shaft} := D_{shaft} = 48\ in$ — Effective web width for circular section, the diameter of the section (BDS Eq. 5.7.2.8-1)

$D_r := D_{shaft} - 2 \cdot cover - 2 \cdot d_{bh.st} - d_b = 39.8\ in$ — Diameter of the circle passing through the centers of the longitudinal reinforcement

$d_e := \frac{D_{shaft}}{2} + \frac{D_r}{\pi} = 36.7\ in$ — BDS Eq. C5.7.2.8-1

$d_{v.shaft} := 0.9 \cdot d_e = 33\ in$ — Effective shear depth, BDS Section 5.7.2.8

$A_{cp} := \pi \frac{D_{shaft}^2}{4} = 1809.56\ in^2$ — Area enclosed by outside perimeter of concrete section

$p_c := \pi\ D_{shaft} = 150.8\ in$ — Length of outside perimeter of the concrete section

$P_h := \left(D_r + d_b + d_{bh.st}\right) \cdot \pi = 130\ in$ — Perimeter enclosed by centerline of outermost shear reinf.

$A_{oh} := \pi\ \frac{\left(D_r + d_b + d_{bh.st}\right)^2}{4} = 1344.5\ in^2$ — Area enclosed by centerline of outermost shear reinf.

$A_o := 0.85 \cdot A_{oh} = 1142.8\ in^2$ — Area enclosed by the shear flow path, ACI Section 22.7.6.1.1

Check shear capacity (BDS 5.7.3.3):

$v_u := \frac{V_{geo}}{\phi \cdot b_{v.shaft} \cdot d_{v.shaft}} = 94.7\ psi$ — Shear stress on the section, BDS 5.7.2.8-1

$v_u < 0.125 \cdot f_c = 1$ — BDS 5.7.2.6 to determine adjustments to maximum spacing

Try BDS 5.7.3.4.2 to calculate β and θ for use in the shear/torsion equations:

$V_{eff} := \sqrt{{V_{geo}}^2 + \left(\frac{0.9 \cdot P_h \cdot T_{geo}}{2 \cdot A_o}\right)^2} = 160.1\ kip$ — Effective factored shear forces to be used in BDS Equation 5.7.3.4.2-4, BDS 5.7.3.4.2-5

$\varepsilon_s := \max\left(\frac{\left(\frac{M_{geo}}{d_{v.shaft}} - .5 \cdot P_{geo\&sw} + \left|V_{eff}\right|\right)}{E_s \cdot A_s}\right) = 4.8 \cdot 10^{-4}$ — Net longitudinal tensile strain in the section at the centroid of the reinforcement, BDS 5.7.3.4.2-4

$\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} = 3.5$ — Factor indicating ability of diagonally cracked concrete to transmit tension and shear, for sections containing at least minimum amount of transverse reinforcement: BDS 5.7.3.4.2-1

$\theta := \left(29 + 3500 \cdot \varepsilon_s\right)\ deg = 30.7\ deg$ — Angle of inclination of diagonal compressive stresses, BDS 5.7.3.4.2-3

$\alpha := 90\ deg$ — Angle of inclination of transverse reinforcement to longitudinal axis

$\lambda := 1$ — Concrete density modification factor, BDS Section 5.4.2.8

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_{v.shaft} \cdot d_{v.shaft} = 353.9\ kip$ — Concrete shear resistance, BDS 5.7.3.3-3

$\phi \cdot V_c > V_{geo} = 1$ — Provide shear reinforcement to satisfy minimum requirement (Required due the consideration of torsion being required, BDS 5.7.2.3)

$$s_{max} := \begin{cases} \min\left(0.8 \cdot d_{v.shaft}, 24\ in\right) & \text{if } v_u < 0.125 \cdot f_c \\ \min\left(0.4 \cdot d_{v.shaft}, 12\ in\right) & \text{else} \end{cases} = 24\ in$$

Maximum shear reinforcement spacing, BDS Eq. 5.7.2.6-1 & 5.7.2.6-2

$s := 12\ in$ $\quad s < s_{max} = 1$

Transverse reinforcement spacing

$$A_{v.min} := 0.0316 \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot \frac{b_{v.shaft} \cdot s}{f_y} = 0.61\ in^2$$

Minimum transverse reinforcement area required within distances s, BDS 5.7.2.5-1

$A_v := 2 \cdot .31\ in^2 = 0.62\ in^2$ $\quad A_v > A_{v.min} = 1$

2 legs (one circular stirrup) Transverse reinforcement bar area (#5)

$$V_s := \frac{A_v \cdot f_y \cdot d_{v.shaft} \cdot \cot(\theta)}{s} = 172.4\ kip$$

Steel shear resistance, BDS C5.7.3.3-1

$$V_n := \min\left(V_c + V_s, 0.25 \cdot f_c \cdot b_{v.shaft} \cdot d_{v.shaft}\right) = 526.3\ kip$$

Nominal shear resistance for the section, BDS 5.7.3.3-1

$\phi V_n := \phi \cdot V_n = 473.7\ kip$

$\phi V_n > V_{geo} = 1$ $\quad \frac{V_{geo}}{\phi V_n} = 28.5\%$

Check Torsion (BDS 5.7.3.6.2):

$T_{geo} = 140\ kip \cdot ft$

Applied torsional moment (from Geotech basis of design)

$$f_{pc} := \frac{\frac{P_{geo\&sw}}{\gamma_p}}{A_g} = 16.56\ psi$$

Un-factored compressive stress in concrete

$$K := \min\left(\sqrt{1 + \frac{f_{pc}}{.126 \cdot \lambda \cdot \sqrt{f_c \cdot ksi}}}, 2.0\right) = 1.03$$

BDS Eq. 5.7.2.1-6

$$T_{cr} := 0.126 \cdot K \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot \frac{{A_{cp}}^2}{p_c} = 470.8\ kip \cdot ft$$

Torsional cracking moment, BDS 5.7.2.1-4

if $\max(T_{geo}) > 0.25 \cdot \phi \cdot T_{cr}$
  "Torsional effects shall be investigated"
else
  "Torsional effects shall not be investigated"

= "Torsional effects shall be investigated"

$A_t := .5 \cdot A_v = 0.31$ ***in***^{2} — Area of one leg of transverse reinforcement

$\theta = 30.7$ ***deg*** — Angle of inclination of diagonal compressive stresses

$T_n := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{s} \cdot 1 = 497.9$ ***kip*** $\cdot$ ***ft*** — Nominal torsional resistance of the section, BDS 5.7.3.6.2-1

$\phi T_n := \phi \cdot T_n = 448.1$ ***kip*** $\cdot$ ***ft***

$\phi T_n > T_{geo} = 1$

$\frac{T_{geo}}{\phi T_n} = 31.24\%$

Check combined shear and torsion (BDS 5.7.3.6):

$A_s = 16.6$ ***in***^{2} — Total area of longitudinal reinforcement resisting torsion

$M_{geo} = 241.7$ ***kip*** $\cdot$ ***ft*** — Moment demands in the shaft, by envelope

$N_u := -P_{geo\&sw} = -37.5$ ***kip*** — Factored axial load, (use negative if compression and positive if tension), BDS 5.7.3

$$F_{combined} := \max\left(\frac{M_{geo}}{\phi \cdot d_{v.shaft}} + \frac{0.5 \cdot N_u}{\phi} \hookleftarrow \right.$$
$$\left. + \cot(\theta) \cdot \sqrt{\left(\frac{V_{geo}}{\phi} - 0.5\ V_s\right)^2 + \left(\frac{0.45 \cdot P_h \cdot T_{geo}}{2 \cdot A_o \cdot \phi}\right)^2}\right) = 211.2\ \textbf{kip}$$

Combined force, BDS 5.7.3.6.3-1

$A_s \cdot f_y > F_{combined} = 1$

$U := \frac{F_{combined}}{A_s \cdot f_y} \cdot 100\% = 21.22\%$

Torsion check, AASHTO 5.7.3.6.3-1

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 4ft Dia. Drilled Shaft

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

Drilled Shaft Design Summary:

- Loads used for design are taken from L-pile analysis output summary spreadsheet provided by geotechnical team.
- This drilled shaft design is valid for max shaft length of 12.5' and the demand loads provided herein.
- Minimum concrete compressive shall be 4 ksi and minimum yield strength of reinforcement rebars shall be 60 ksi.
- Provide 21#8 vertical bars with equal spacing and #5@12" on center transverse bars for shear.

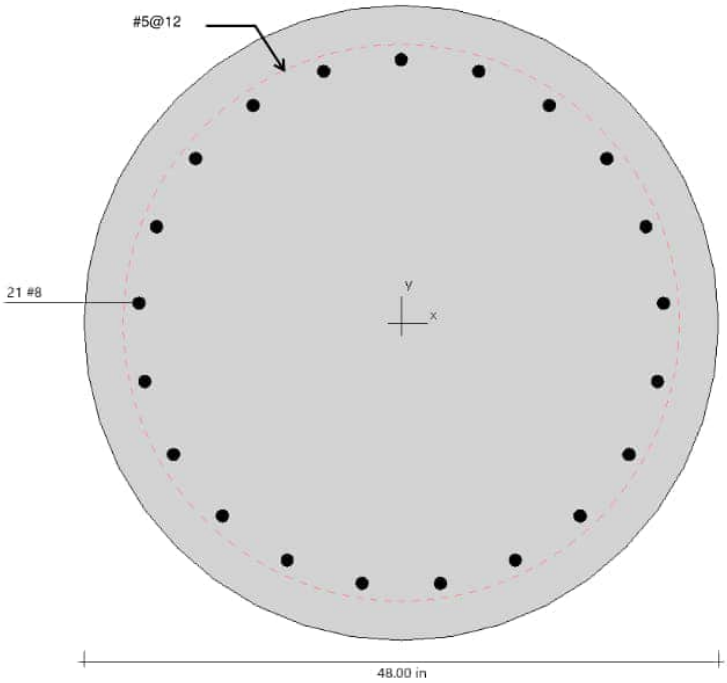


Figure 5: Reinforcement section

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

General:

- The following calculations cover the design for the drilled shaft foundation that supports the OHSS bridge and cantilevers. The hammerhead design is only applicable for the OHSS bridge drilled shafts.

References:

- AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020 [BDS]
- AASHTO Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 w/ 2017-2020 Interims [AASHTO Signs]
- FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018. [FHWA]
- ACI 318-14, Building Code Requirements for Structural Concrete and Commentary [ACI]
- AISC 360-10, Steel Construction Manual 14th Ed. [SCM]
- MaineDOT Bridge Design Guide, August 2003 with 2018 additions [MaineDOT BDG]
- MaineDOT Standard Details and Specifications [MaineDOT Specs]
- Summary of LPile Results - Drilled Shafts 6.3.25 Spreadsheet provided by Geotech https://wsponlinenam.sharepoint.com/:x:/r/sites/GLD-157336/Project%20Files/5%20Technical%20Work/GDR%20Geotech%20Mast%20Arms%20%26%20OHSS/Summary%20Table%20of%20LPile%20Results%20-%20Drilled%20Shafts%206.3.25.xlsx?d=wd4aecb6215c24738a71d3498e30f9ca3&csf=1&web=1&e=LvfUnZ

Assumptions:

- Foundation diameter, lengths and loads were provided by the WSP geotechnical team.
- AASHTO LRFD governs design unless a more conservative provision is included in the MaineDOT Bridge Design Guide.
- Per MaineDOT BDG Table 6-2 Class A Concrete shall be used for the drilled shafts.
- The highest value for shear, axial, moment and torsion demands will be used for the calculations. The addition weight of the hammerhead will be considered for all the drilled shafts.

Drilled Shaft Parameters:

$D_{shaft} := 5\ ft$ — Drilled shaft diameter, provided by Geotechnical team.

$L_{shaft} := 15\ ft$ — Drilled shaft length

$cover := 3\ in$ — Rebar cover, BDS Table 5.10.1-1

$A_g := \frac{D_{shaft}^{2} \cdot \pi}{4} = 2827.433\ in^2$ — Shaft cross sectional area

$I_{yy} := \frac{D_{shaft}^{4} \cdot \pi}{64} = 636172.5\ in^4$ — Moment of inertia of the shaft

$I_{xx} := I_{yy} = 636172.5\ in^4$ — Moment of inertia

$r_{xx} := \sqrt{\frac{I_{xx}}{A_g}} = 15\ in$ — Radius of gyration

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

Drilled Shaft Reinforcement Properties:

$cover = 3\ in$ — Rebar cover

$d_{bh.st} := 0.625\ in$ — Stirrup rebar dia. (#5)

$N_b := 20$ — Total number vertical bars

$A_b := 1.56\ in^2$ — Area of long. rebar, #11

$d_b := 1.41\ in$ — Long. rebar dia #11

$f_y := 60\ ksi$ — Yielding strength of steel reinforcement

Drilled Shaft Material Properties:

$f_c := 4\ ksi$ — Compressive strength Class A Conc, MaineDOT Spec Section 502 Table 1

$\beta_1 := .85$ — For $f_c = 4\ ksi$, BDS, Section 5.6.2.2

$\gamma_c := 150\ pcf$ — Concrete unit weight w/ reinforcement, BDS Table 3.5.1-1 + BDS C3.5.1

$E_s := 29000\ ksi$ — Steel Elastic Modulus

$K_1 := 1.0$ — Aggregate correction factor, BDS Section 5.4.2.4

$$E_c := 120000 \cdot K_1 \cdot \left(\frac{\gamma_c}{\frac{kip}{ft^3}}\right)^{2.0} \left(\frac{f_c}{ksi}\right)^{0.33} \cdot ksi = 4266.2\ ksi$$

Concrete Elastic Modulus, BDS Eqn. 5.4.2.4-1

Hammerhead Parameters:

$d_{hh} := 6\ ft$ — Hammerhead depth (plan)

$b_{hh} := 12\ ft$ — Hammerhead width (plan)

$cover = 3\ in$ — Hammerhead cover

$T_{hh} := 5\ ft = 60\ in$ — Hammerhead thickness

$A_{g.hh} := d_{hh} \cdot b_{hh} = 10368\ in^2$ — Column cross sectional area

Maximum Demands on Drilled Shaft from LPile Analysis:

$\gamma_p := 1.25$ — DC load factor Strength I, BDS Table 3.4.1-2

$P_{geo\&sw} := 16\ kip + \gamma_p \cdot A_g \cdot L_{shaft} \cdot \gamma_c + kip + \gamma_p \cdot A_{g.hh} \cdot T_{hh} \cdot \gamma_c = 139.7\ kip$ — Axial loading from geotech design basis + self weight

$M_{geo} := 5955\ kip \cdot in = 496.3\ kip \cdot ft$ — Maximum moment from LPile results

$V_{geo} := 75\ kip$ — Maximum shear from LPile results

$T_{geo} := 350\ kip \cdot ft$ — Maximum torsions from geotech design basis

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

SP-Column Analysis for Drilled Shaft:

SP-Column input:

Applied axial load and maximum moment demand as calculated by geotechnical team.

SP-Column output:

Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	Pu	Mux	ϕPn	ϕMnx	NA Depth	εt	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	150.00	500.00	0.00	3280.40	11.64	0.01135	0.900	0.48

Figure 1: Factored loads and moments with corresponding capacity ratios

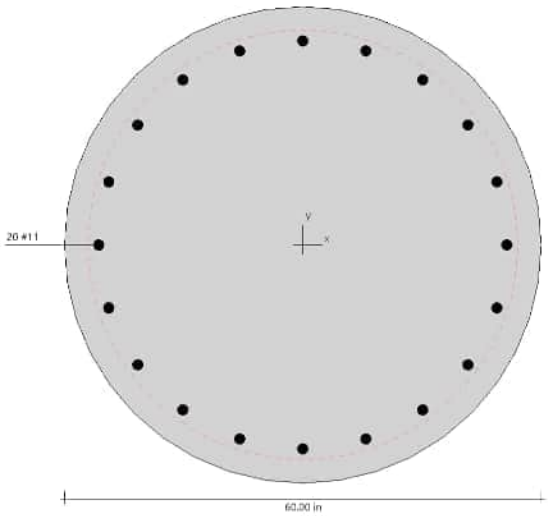


Figure 2: Cross-section and reinforcement layout

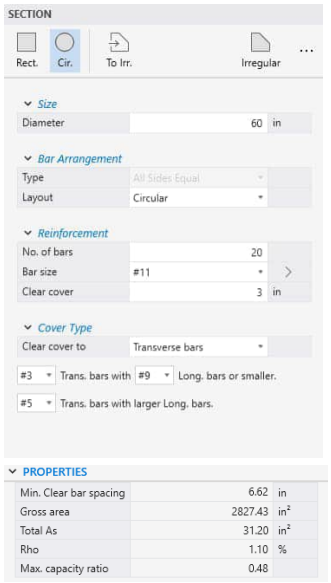


Figure 3: Reinforcement detail

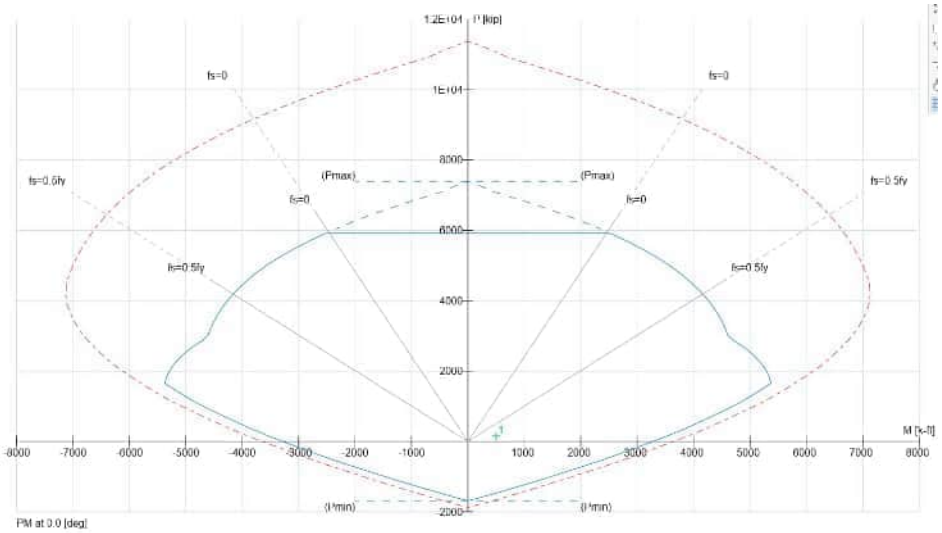


Figure 4: P&M interaction diagram

Reinforcement Limits:

Longitudinal reinforcement limit:

$\rho_{min} := 0.135 \cdot \left(\frac{f_c}{f_y}\right) = 0.009$ — BDS Eqn. 5.6.4.2-3

$A_{smin} := \rho_{min} \cdot A_g = 25.4\ in^2$ — Minimum area of steel

$\rho_{max} := 0.08$ — BDS Eqn. 5.6.4.2-1

$A_{smax} := \rho_{max} \cdot A_g = 226.2\ in^2$ — Maximum area of steel

$n_{bar} := N_b = 20$ — Assumed no. of bars

$A_b = 1.56\ in^2$

$A_s := n_{bar} \cdot A_b = 31.2\ in^2$ — Assumed area of steel

$A_{smin} \le A_s \le A_{smax} = 1$ $\quad \rho := \frac{A_s}{A_g} = 1.1\%$ — Check, OK

Minimum spacing check:

$s_{min} := min\left(1.5 \cdot d_b, 1.5\ in\right) = 1.5\ in$ — BDS 5.10.3.1.1

$s_{actual} := 6.62\ in$ — Actual clear spacing as reported in SP-Column.

$s_{min} < s_{actual} = 1$ — Check, OK

Design for Shear and Torsion:

- General procedure for non-prestressed section, used for the shear and torsion reinforcement design per AASHTO LRFD Bridge Design, Section 5.7

$\phi := 0.9$ — Strength reduction factor, shear (BDS 5.5.4)

$b_{v.shaft} := D_{shaft} = 60\ in$ — Effective web width for circular section, the diameter of the section (BDS Eq. 5.7.2.8-1)

$D_r := D_{shaft} - 2 \cdot cover - 2 \cdot d_{bh.st} - d_b = 51.3\ in$ — Diameter of the circle passing through the centers of the longitudinal reinforcement

$d_e := \frac{D_{shaft}}{2} + \frac{D_r}{\pi} = 46.3\ in$ — BDS Eq. C5.7.2.8-1

Effective shear depth, BDS Section 5.7.2.8

$d_{v.shaft} := 0.9 \cdot d_e = 41.7\ in$

$A_{cp} := \pi \frac{D_{shaft}^2}{4} = 2827.43\ in^2$ — Area enclosed by outside perimeter of concrete section

$p_c := \pi \ D_{shaft} = 188.5 \ in$ Length of outside perimeter of the concrete section

$P_h := \left(D_r + d_b + d_{bh.st}\right) \cdot \pi = 167.7 \ in$ Perimeter enclosed by centerline of outermost shear reinf.

$A_{oh} := \pi \ \frac{\left(D_r + d_b + d_{bh.st}\right)^2}{4} = 2237.5 \ in^2$ Area enclosed by centerline of outermost shear reinf.

$A_o := 0.85 \cdot A_{oh} = 1901.9 \ in^2$ Area enclosed by the shear flow path, ACI Section 22.7.6.1.1

Check shear capacity (BDS 5.7.3.3):

$v_u := \frac{V_{geo}}{\phi \cdot b_{v.shaft} \cdot d_{v.shaft}} = 33.3 \ psi$ Shear stress on the section, BDS 5.7.2.8-1

$v_u < 0.125 \cdot f_c = 1$ BDS 5.7.2.6 to determine adjustments to maximum spacing

Try BDS 5.7.3.4.2 to calculate β and θ for use in the shear/torsion equations:

$V_{eff} := \sqrt{V_{geo}{}^2 + \left(\frac{0.9 \cdot P_h \cdot T_{geo}}{2 \cdot A_o}\right)^2} = 182.7 \ kip$ Effective factored shear forces to be used in BDS Equation 5.7.3.4.2-4, BDS 5.7.3.4.2-5

$\varepsilon_s := \max\left(\frac{\left(\frac{M_{geo}}{d_{v.shaft}} - .5 \cdot P_{geo\&sw} + \left|V_{eff}\right|\right)}{E_s \cdot A_s}\right) = 2.8 \cdot 10^{-4}$ Net longitudinal tensile strain in the section at the centroid of the reinforcement, BDS 5.7.3.4.2-4

$\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} = 4$ Factor indicating ability of diagonally cracked concrete to transmit tension and shear, for sections containing at least minimum amount of transverse reinforcement: BDS 5.7.3.4.2-1

$\theta := \left(29 + 3500 \cdot \varepsilon_s\right) \ deg = 30 \ deg$ Angle of inclination of diagonal compressive stresses, BDS 5.7.3.4.2-3

$\alpha := 90 \ deg$ Angle of inclination of transverse reinforcement to longitudinal axis

$\lambda := 1$ Concrete density modification factor, BDS Section 5.4.2.8

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_{v.shaft} \cdot d_{v.shaft} = 626.4 \ kip$ Concrete shear resistance, BDS 5.7.3.3-3

$\phi \cdot V_c > V_{geo} = 1$ Provide shear reinforcement to satisfy minimum requirement (Required due the consideration of torsion being required, BDS 5.7.2.3)

$$s_{max} := \begin{cases} \text{if } v_u < 0.125 \cdot f_c \\ \quad \left\| \min\left(0.8 \cdot d_{v.shaft}, 24\ \textit{in}\right) \right. \\ \text{else} \\ \quad \left\| \min\left(0.4 \cdot d_{v.shaft}, 12\ \textit{in}\right) \right. \end{cases} = 24\ \textit{in}$$

Maximum shear reinforcement spacing, BDS Eq. 5.7.2.6-1 & 5.7.2.6-2

$s := 8\ \textit{in}$ $\quad s < s_{max} = 1$

Transverse reinforcement spacing

$$A_{v.min} := 0.0316 \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}} \cdot \frac{b_{v.shaft} \cdot s}{f_y} = 0.51\ \textit{in}^2$$

Minimum transverse reinforcement area required within distances s, BDS 5.7.2.5-1

$A_v := 2 \cdot .31\ \textit{in}^2 = 0.62\ \textit{in}^2$ $\quad A_v > A_{v.min} = 1$

2 legs (one circular stirrup) Transverse reinforcement bar area (#5)

$$V_s := \frac{A_v \cdot f_y \cdot d_{v.shaft} \cdot \cot(\theta)}{s} = 336.1\ \textit{kip}$$

Steel shear resistance, BDS C5.7.3.3-1

$$V_n := \min\left(V_c + V_s, 0.25 \cdot f_c \cdot b_{v.shaft} \cdot d_{v.shaft}\right) = 962.5\ \textit{kip}$$

Nominal shear resistance for the section, BDS 5.7.3.3-1

$\phi V_n := \phi \cdot V_n = 866.2\ \textit{kip}$

$\phi V_n > V_{geo} = 1$ $\quad \frac{V_{geo}}{\phi V_n} = 8.66\%$

Check Torsion (BDS 5.7.3.6.2):

$T_{geo} = 350\ \textit{kip} \cdot \textit{ft}$

Applied torsional moment (from Geotech basis of design)

$$f_{pc} := \frac{\frac{P_{geo\&sw}}{\gamma_p}}{A_g} = 39.53\ \textit{psi}$$

Un-factored compressive stress in concrete

$$K := \min\left(\sqrt{1 + \frac{f_{pc}}{.126 \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}}}}, 2.0\right) = 1.08$$

BDS Eq. 5.7.2.1-6

$$T_{cr} := 0.126 \cdot K \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}} \cdot \frac{{A_{cp}}^2}{p_c} = 958\ \textit{kip} \cdot \textit{ft}$$

Torsional cracking moment, BDS 5.7.2.1-4

if $\max(T_{geo}) > 0.25 \cdot \phi \cdot T_{cr}$
  "Torsional effects shall be investigated"
else
  "Torsional effects shall not be investigated"

= "Torsional effects shall be investigated"

$A_t := .5 \cdot A_v = 0.31\ in^2$ Area of one leg of transverse reinforcement

$\theta = 30\ deg$ Angle of inclination of diagonal compressive stresses

$T_n := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{s} \cdot 1 = 1277.1\ kip \cdot ft$ Nominal torsional resistance of the section, BDS 5.7.3.6.2-1

$\phi T_n := \phi \cdot T_n = 1149.4\ kip \cdot ft$

$\phi T_n > T_{geo} = 1$

$\frac{T_{geo}}{\phi T_n} = 30.45\%$

Check combined shear and torsion (BDS 5.7.3.6):

$A_s = 31.2\ in^2$ Total area of longitudinal reinforcement resisting torsion

$M_{geo} = 496.3\ kip \cdot ft$ Moment demands in the shaft, by envelope

$N_u := -P_{geo\&sw} = -139.7\ kip$ Factored axial load, (use negative if compression and positive if tension), BDS 5.7.3

$$F_{combined} := \max\left(\frac{M_{geo}}{\phi \cdot d_{v.shaft}} + \frac{0.5 \cdot N_u}{\phi},\ \cot(\theta) \cdot \sqrt{\left(\frac{V_{geo}}{\phi} - 0.5\ V_s\right)^2 + \left(\frac{0.45 \cdot P_h \cdot T_{geo}}{2 \cdot A_o \cdot \phi}\right)^2}\right) = 298.4\ kip$$

Combined force, BDS 5.7.3.6.3-1

$A_s \cdot f_y > F_{combined} = 1$

$U := \frac{F_{combined}}{A_s \cdot f_y} \cdot 100\% = 15.94\%$

Torsion check, AASHTO 5.7.3.6.3-1

Drilled Shaft Design Summary:

- Loads used for design are taken from L-pile analysis output summary spreadsheet provided by geotechnical team.

- This drilled shaft design is valid for max shaft length of 15' and the demand loads provided herein.
- Minimum concrete compressive shall be 4 ksi and minimum yield strength of reinforcement rebars shall be 60 ksi.

- Provide 20#11 vertical bars with equal spacing and #5@8" on center transverse bars for shear.

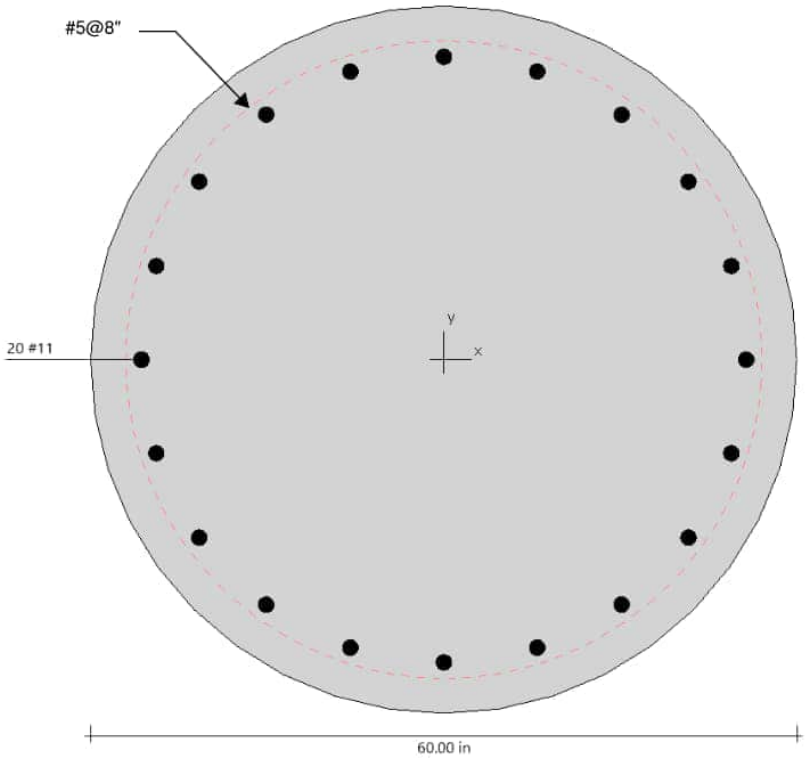


Figure 5: Reinforcement section

Hammerhead Parameters:

$d_{hh} = 6\ ft$ — Hammerhead depth

$b_{hh} = 12\ ft$ — Hammerhead width

$cover = 3\ in$ — Hammerhead cover

$T_{hh} = 5\ ft$ — Hammerhead thickness

$A_{g.hh} = 10368\ in^2$ — Column cross sectional area

$I_{yy.hh} := b_{hh}^{\ 3} \cdot \frac{d_{hh}}{12} = 17915904\ in^4$ — Moment of inertia about strong axis

$I_{xx.hh} := b_{hh} \cdot \frac{d_{hh}^{\ 3}}{12} = 4478976\ in^4$ — Moment of inertia about weak axis

$r_{xx.hh} := \sqrt{\frac{I_{xx.hh}}{A_{g.hh}}} = 20.8\ in$ — Radius of gyration about weak axis

$r_{yy.dow} := \sqrt{\frac{I_{yy.hh}}{A_{g.hh}}} = 41.6\ in$ — Radius of gyration about strong axis

$S_c := \frac{I_{xx.hh}}{(0.5 \cdot d_{hh})} = 124416\ in^3$ — Section modulus for the extreme fiber - assumed moment is about weak axis based on sign geometry

Hammerhead Reinforcement Properties:

$d_{bh.st.hh} := 0.625\ in$ — Stirrup rebar dia. (#5)

$d_{b.hh} := 1\ in$ — Long. rebar dia #8

$A_{b.hh} := 0.79\ in^2$ — Area of long. rebar, #8

$N_{b.hh} := 46$ — Total number vertical bars

Material Properties:

$f_c = 4\ ksi$ — Assumed compressive strength concrete (drilled shaft)

$f_y = 60\ ksi$ — Yielding strength of steel reinforcement

$f_r := 0.24\ \sqrt[2]{f_c \cdot ksi} = 0.5\ ksi$ — Modulus of rupture

$\beta_1 = 0.85$ — For $f_c = 4\ ksi$, BDS, Section 5.6.2.2

$\gamma_c = 150\ pcf$ — Concrete unit weight, BDS, Table 3.5.1-1

$E_s = 29000\ ksi$ — Steel Elastic Modulus

$K_1 = 1$ — Aggregate correction factor, BDS Section 5.4.2.4

$E_c = 4266.2\ ksi$ — Concrete Elastic Modulus, BDS Eqn. 5.4.2.4-1

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

SP-Column Analysis for Drilled Shaft Hammerhead Axial and Moment Demand:

SP-Column input:

Applied axial load and maximum moment demand as listed in the geotechnical team basis of design.

SP-Column output:

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	Pu	Mux	ϕPn	ϕMnx	NA Depth	εt	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	85.00	450.00	0.00	5507.63	3.71	0.05186	0.900	0.79

Figure 6: Factored loads and moments with corresponding capacity ratios

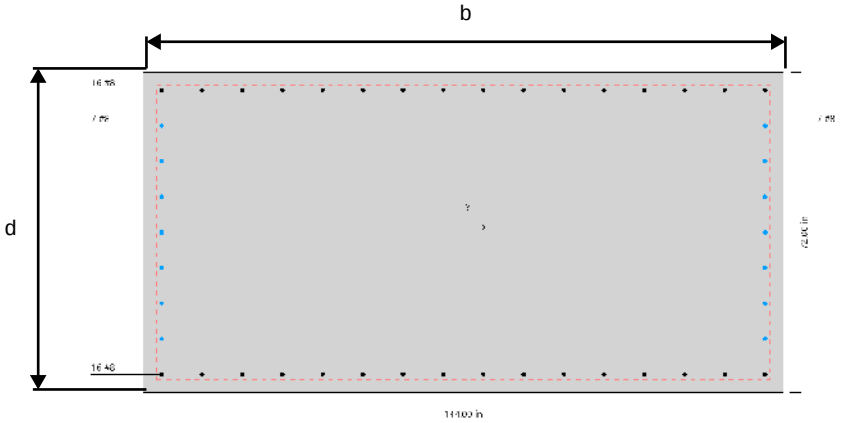


Figure 7: Cross-section and reinforcement layout

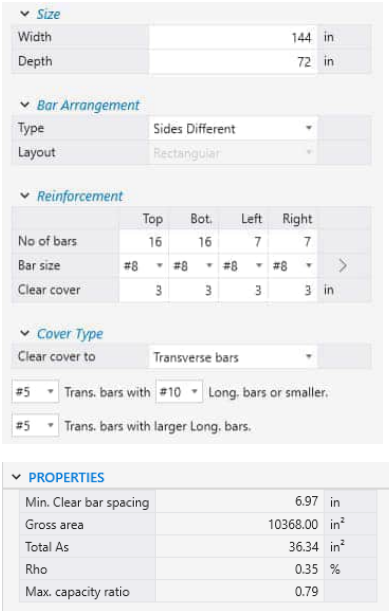


Figure 8: Reinforcement detail

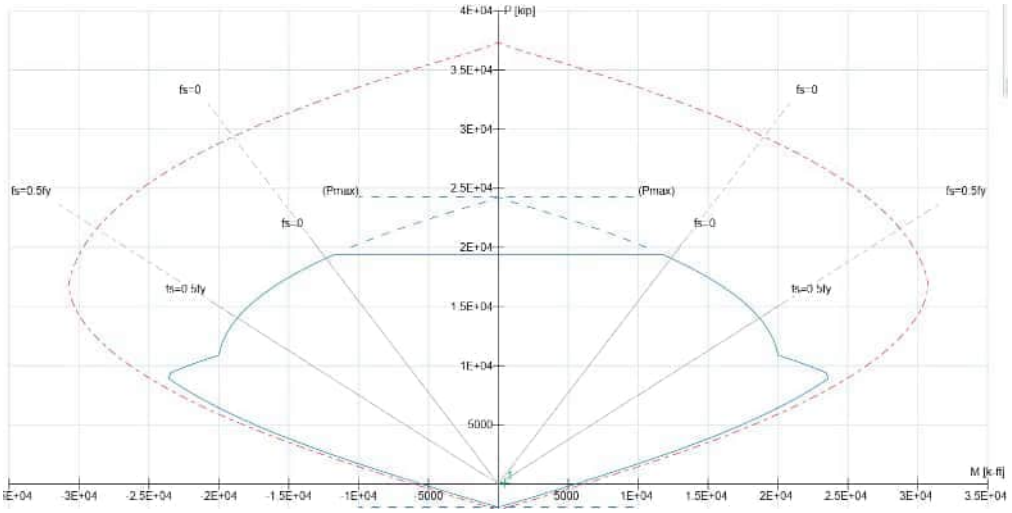


Figure 9: P&M interaction diagrams

Maximum Demands on Drilled Shaft from Geotech Design Basis:

$\gamma_p = 1.25$	DC load factor Strength I, BDS Table 3.4.1-2
$P_{u.hh} := 16\ kip + \gamma_p \cdot A_{g.hh} \cdot T_{hh} \cdot \gamma_c = 83.5\ kip$	Axial loading from geotech design basis
$M_{u.hh} := 450\ kip \cdot ft$	Maximum moment from geotech design basis
$V_{u.hh} := 20\ kip$	Maximum shear from geotech design basis
$T_{u.hh} := 0\ kip \cdot ft$	Maximum torsions geotech design basis

Reinforcement Limits:

Longitudinal reinforcement limit:

The hammerhead will act as beam with one-way and two-way shear and flexure. The hammerhead will be checked for axial (gravity load) and bending, but the minimum reinforcement requirement will be determined by flexure and shear requirements.

$n_{bar.hh} := N_{b.hh} = 46$	Assumed no. of bars
$n_{bar.hh.d} := 7$	Assumed no. of bars along d (excluding corner bars)
$n_{bar.hh.b} := 16$	Assumed no. of bars along b (including corner bars)
$A_{b.hh} = 0.79\ in^2$	Area of long. rebar, #8
$A_{s.hh} := n_{bar.hh} \cdot A_{b.hh} = 36.3\ in^2$	Assumed area of steel
$M_r := 5507\ kip \cdot ft$	Moment resistance from spColumn
$\gamma_1 := 1.6$	Flexural cracking variability factor, BDS 5.6.3.3
$\gamma_3 := 0.67$	Ratio of specified minimum yield, BDS 5.6.3.3
$M_{cr} := \gamma_3 \cdot (\gamma_1 \cdot f_r) \cdot S_c = 5335\ kip \cdot ft$	Cracking Moment, BDS Eq. 5.6.3.3-1
$1.33 \cdot M_{u.hh} = 598.5\ kip \cdot ft$	Limit due to demand, BDS 5.6.3.3
$M_r \geq min(1.33 \cdot M_{u.hh}, M_{cr}) = 1$	Flexural limit for reinforcement, BDS 5.6.3.3

Minimum spacing check:

$s_{min.hh} := 1.5 \cdot d_{b.hh} = 1.5\ in$	BDS 5.10.3.1.1
$s_{actual.hh} := 6.97\ in$	Actual min. clear spacing as reported in SP-Column.
$s_{min.hh} < s_{actual.hh} = 1$	OK

$\phi = 0.9$ Strength reduction factor flexure, shear, and torsion (BDS 5.5.4)

$d_s := d_{hh} - cover - d_{bh.st.hh} - d_{b.hh} \cdot 0.5 = 67.9\ in$ Distance from extreme compression fiber to the centroid of the extreme tensions and steel element

$d_{e.hh} := \frac{A_{s.hh} \cdot f_y \cdot d_s}{A_{s.hh} \cdot f_y} = 67.9\ in$ BDS Eq. 5.7.2.8-2

$b_{v.hh} := b_{hh} - cover \cdot 2 - d_{bh.st.hh} = 137.4\ in$ Distance between centerline of ties, width

$d_o := d_{hh} - cover \cdot 2 - d_{bh.st.hh} = 65.4\ in$ Distance between centerline of ties, depth

$d_{v.hh} := \max\left(d_{hh} - cover \cdot 2 - 2 \cdot d_{bh.st.hh} - d_{b.hh}, 0.9 \cdot d_{e.hh}, 0.72 \cdot d_{hh}\right) = 63.8\ in$ Effective shear depth

$A_{oh.hh} := b_{v.hh} \cdot d_o = 8980.9\ in^2$ Area enclosed by centerline of outermost shear reinf.

$A_{o.hh} := 0.85 \cdot A_{oh.hh} = 7633.8\ in^2$ Area enclosed by the shear flow path, ACI Section 22.7.6.1.1

$P_{h.hh} := 2 \cdot \left(b_{v.hh} + d_o\right) = 405.5\ in$ Perimeter enclosed by centerline of outermost shear reinf.

Shear Check:

$v_{u.hh} := \left(\frac{V_{u.hh}}{\phi \cdot b_{v.hh} \cdot d_o}\right) = 2.5\ psi$ Shear stress on the section, BDS 5.7.2.8-1

$\max\left(v_{u.hh}\right) < 0.125 \cdot f_c = 1$ BDS 5.7.2.6 to determine adjustments to maximum spacing

Try BDS 5.7.3.4.2 to calculate β and θ for use in the shear/torsion equations

$N_{u.hh} := P_{u.hh}$

$\varepsilon_{s.hh} := \max\left(\frac{\left(\frac{M_{u.hh}}{d_{v.hh}} - .5 \cdot N_{u.hh} + \left|V_{u.hh}\right|\right)}{E_s \cdot A_{s.hh}}\right) = 6 \cdot 10^{-5}$

Net longitudinal tensile strain in the section at the centroid of the reinforcement, BDS 5.7.3.4.2-4

$\beta_{.hh} := \frac{4.8}{1 + 750 \cdot \varepsilon_{s.hh}} = 4.6$

Factor indicating ability of diagonally cracked concrete to transmit tension and shear, for sections containing at least minimum amount of transverse reinforcement: BDS 5.7.3.4.2-1

$\theta_{.hh} := \left(29 + 3500 \cdot \varepsilon_{s.hh}\right)\ deg = 29.2\ deg$

Angle of inclination of diagonal compressive stresses, BDS 5.7.3.4.2-2

$\lambda = 1$ Concrete density modification factor, BDS 5.4.2.8-1

$V_{c.hh} := 0.0316 \cdot \beta_{.hh} \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_{v.hh} \cdot d_o = 2607.6\ kip$ Concrete shear resistance, BDS 5.7.3.3-3

$.5 \cdot \phi \cdot V_{c.hh} > \max\left(V_{u.hh}\right) = 1$ Transverse reinforcement is not required for shear

$\phi \cdot V_{c.hh} > \max\left(V_{u.hh}\right) = 1$ Factored concrete shear resistance, BDS 5.7.2.1-1; compare against demand

$s_{max.hh} := min\left(0.8 \cdot d_o, 24\ in\right) = 24\ in$ Maximum shear reinforcement spacing, BDS 5.7.2.6-1

$s_{.hh} := 6\ in$ $s_{.hh} < s_{max.hh} = 1$ Spacing of horizontal bars in hammer head

Shear reinforcement not required for shear strength of hammer head but provided for confinement of anchorage.

$A_{v.hh} := 2 \cdot .31 \cdot in^2 = 0.62\ in^2$ 2 legs (one stirrup) transverse reinforcement bar area (#5)

$V_{s.hh} := \frac{A_{v.hh} \cdot f_y \cdot d_o \cdot \left(\cot\left(\theta_{.hh}\right)\right)}{s_{.hh}} = 725\ kip$ Steel shear resistance, BDS 5.7.3.3-1

$V_{n.hh} := min\left(V_{c.hh} + V_{s.hh}, 0.25 \cdot f_c \cdot b_{v.hh} \cdot d_o\right) = 3332.6\ kip$ Nominal shear resistance for the section, BDS 5.7.3.3-1

$\phi V_{n.hh} := \phi \cdot V_{n.hh} = 2999.3\ kip$

$\phi V_{n.hh} > V_{u.hh} = 1$ $\frac{V_{u.hh}}{\phi V_{n.hh}} = 0.67\%$

Two-Way Shear Check (Due to Axial):

$\beta_c := \frac{b_{hh}}{d_{hh}} = 2$

Ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted, BDS 5.12.8.6.3-1

$b_o := \pi \cdot \left(d_{v.hh} + D_{shaft}\right) = 32.4\ ft$

The perimeter of the critical section for shear, BDS 5.12.8.6.3-1

$V_{n.hh2} := min\left(\left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_o \cdot d_{v.hh}, 0.126 \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_o \cdot d_{v.hh}\right) = 6245.6\ kip$

Nominal two-way shear resistance BDS 5.12.8.6.3-1

$\phi V_{n.hh2} := \phi \cdot V_{n.hh2} = 5621.1\ kip$

Two-way shear strength

$\phi V_{n.hh2} > \max\left(P_{u.hh}\right) = 1$

$\frac{P_{u.hh}}{\phi V_{n.hh2}} = 1.49\%$

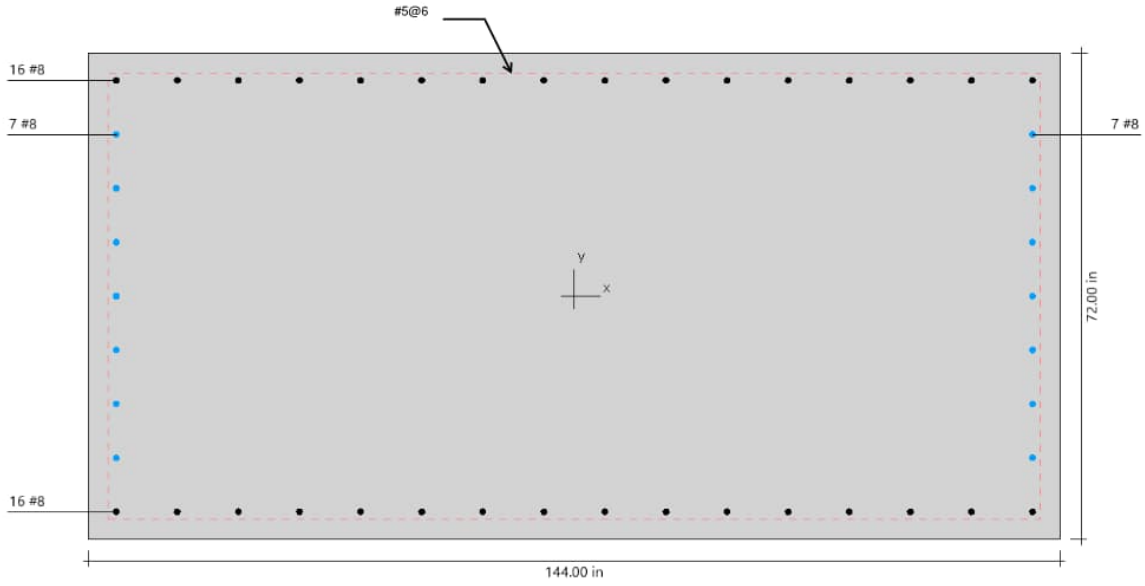


Figure 10: Reinforcement section

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

Interface Shear Transfer Check:

- Assuming normal weight concrete placed against a clean concrete surface, free of laitance with surface intentionally roughened to an amplitude of 0.25 inches.
- Check interface strength using lowest axial load since compression aids in the interface resistance.

$A_{cv.hh} := A_{o.hh} = 7633.8\ in^2$ — Area of concrete engaged in interface shear transfer BDS (5.7.4.3-6)

$V_{ui} := V_{u.hh} = 20\ kip$ — Factored interface shear force, BDS (5.7.4.5-2)

$A_{vf.hh} := A_s = 31.2\ in^2$ — Area of interface reinforcement crossing the shear plane within the area of concrete engaged.

$A_{vf.hh} > \frac{0.05 \cdot min\left(A_{cv.hh}\right) \cdot ksi}{f_y} = 1$ — Minimum area of interface shear reinforcement check, BDS (5.7.4.2-1)

$c := 0.24\ ksi$ — Cohesion factor, BDS 5.7.4.4

$\mu := 1$ — Friction factor, BDS 5.7.4.4

$K_{1.hh} := 0.25$ — Fraction of concrete strength available to resist interface shear, BDS 5.7.4.4

$K_2 := 1.5\ ksi$ — Limiting interface shear resistance, BDS 5.7.4.4

$V_{ni.hh} := min\left(K_{1.hh} \cdot f_c \cdot A_{cv.hh}, K_2 \cdot A_{cv.hh}, \left(c \cdot A_{cv.hh} + \mu \cdot \left(A_{vf.hh} \cdot f_y + P_{u.hh}\right)\right)\right) = 3787.6\ kip$

Nominal shear resistance of the interface plane, BDS (5.7.4.3-3,4, and 5)

$V_{r.hh} := \phi \cdot V_{ni.hh} = 3408.8\ kip$ — Factored interface shear resistance, BDS (5.7.4.3-1)

$\max\left(V_{ui}\right) < V_{r.hh} = 1$

$\frac{\max\left(V_{ui}\right)}{V_{r.hh}} \cdot 100\% = 0.59\%$

Calculation by: T. LeBaron
Date: 8/21/2025
Checked by / Reviewed by: B. Packard / C. Benda
Date: 8/26/2025 / 9/12/2025

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 5ft Dia. Drilled Shaft & Hammer Head

Development and Splice Lengths for Reinforcement

(20) #11 Longitudinal Bars

Developing #11 bars

$l_{db1} := \frac{0.63 \cdot d_b \cdot f_y}{\sqrt{f_c \cdot ksi}} = 26.6\ in$ — Basic development length BDS 5.10.8.2.2a-2

$l_{db2} := 0.3 \cdot d_b \cdot \frac{f_y}{ksi} = 25.4\ in$ — Basic development length BDS 5.10.8.2.2a-3

$l_{db} := \max\left(l_{db1}, l_{db2}\right) = 26.6\ in$ — Basic development length, BDS Section 5.10.8.2.2a-3

$\lambda_{er} := 1$ $\lambda_{rc} := 1$ — Modification factors which may decrease ld BDS 5.10.8.2.1c

$\lambda_{rl} := 1$ reinforcement is not horizontal

$\lambda_{cW} := 1$ reinforcement is uncoated — Modification factors which may increase ld BDS 5.10.8.2.1b

$l_{d.compression} := l_{db} \cdot \lambda_{er} \cdot \lambda_{rc} = 26.6\ in$ — Compression development length BDS (5.10.8.2.2a-1)

$l_{hb} := \frac{38 \cdot d_b}{60} \cdot \frac{f_y}{\sqrt{f_c \cdot ksi}} = 26.8\ in$ — Basic development length BDS (5.10.8.2.4a-2)

$l_{dh} := \max\left(l_{hb} \cdot \left(\lambda_{cW} \cdot \lambda_{rc} \cdot \lambda_{er}\right), 6\ in, 8 \cdot d_b\right) = 26.8\ in$ — Development length of a standard hook in tension BDS (5.10.8.2.4a-1)

$l_d := \max\left(l_{d.compression}, l_{dh}\right) = 26.8\ in$ — Controlling development length

Project: MaineDOT Hogan Road
Calc. Set: Drilled Shaft Design - 6ft Dia. Drilled Shaft

General:

- The following calculations cover the design for the drilled shaft foundation that supports the the OHSS cantilever at Station 104+75.

References:

- AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020 [BDS]
- AASHTO Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Ed. 2015 w/ 2017-2020 Interims [AASHTO Signs]
- FHWA. Drilled Shafts: Construction Procedures and Design Methods. Publication No. FHWA-NHI 18-024 and FHWA GEC 010. September 2018. [FHWA]
- ACI 318-14, Building Code Requirements for Structural Concrete and Commentary [ACI]
- AISC 360-10, Steel Construction Manual 14th Ed. [SCM]
- MaineDOT Bridge Design Guide, August 2003 with 2018 additions [MaineDOT BDG]
- MaineDOT Standard Details and Specifications [MaineDOT Specs]
- Summary of LPile Results - Drilled Shafts 6.3.25 Spreadsheet provided by Geotech https://wsponlinenam.sharepoint.com/:x:/r/sites/GLD-157336/Project%20Files/5%20Technical%20Work/GDR%20Geotech%20Mast%20Arms%20%26%20OHSS/Summary%20Table%20of%20LPile%20Results%20-%20Drilled%20Shafts%206.3.25.xlsx?d=wd4aecb6215c24738a71d3498e30f9ca3&csf=1&web=1&e=LvfUnZ

Assumptions:

- Foundation diameter, lengths and loads were provided by the WSP geotechnical team.
- AASHTO LRFD governs design unless a more conservative provision is included in the MaineDOT Bridge Design Guide.
- Per MaineDOT BDG Table 6-2 Class A Concrete shall be used for the drilled shafts.
- The highest value for shear, axial, moment and torsion demands will be used for the calculations.

Drilled Shaft Parameters:

$D_{shaft} := 6\ ft$ — Drilled shaft diameter, provided by Geotechnical team.

$L_{shaft} := 20\ ft$ — Drilled shaft length

$cover := 3\ in$ — Rebar cover, BDS Table 5.10.1-1

$A_g := \frac{D_{shaft}^{2} \cdot \pi}{4} = 4071.504\ in^2$ — Shaft cross sectional area

$I_{yy} := \frac{D_{shaft}^{4} \cdot \pi}{64} = 1319167.3\ in^4$ — Moment of inertia of the shaft

$I_{xx} := I_{yy} = 1319167.3\ in^4$ — Moment of inertia

$r_{xx} := \sqrt{\frac{I_{xx}}{A_g}} = 18\ in$ — Radius of gyration

Drilled Shaft Reinforcement Properties:

$cover = 3\ in$ — Rebar cover

$d_{bh.st} := 0.625\ in$ — Stirrup rebar dia. (#5)

$N_b := 27$ — Total number vertical bars

$A_b := 1.56\ in^2$ — Area of long. rebar, #11

$d_b := 1.41\ in$ — Long. rebar dia #11

$f_y := 60\ ksi$ — Yielding strength of steel reinforcement

Drilled Shaft Material Properties:

$f_c := 4\ ksi$ — Compressive strength Class A Conc, MaineDOT Spec Section 502 Table 1

$\beta_1 := .85$ — For $f_c = 4\ ksi$, BDS, Section 5.6.2.2

$\gamma_c := 150\ pcf$ — Concrete unit weight w/ reinforcement, BDS Table 3.5.1-1 + BDS C3.5.1

$E_s := 29000\ ksi$ — Steel Elastic Modulus

$K_1 := 1.0$ — Aggregate correction factor, BDS Section 5.4.2.4

$$E_c := 120000 \cdot K_1 \cdot \left(\frac{\gamma_c}{\frac{kip}{ft^3}} \right)^{2.0} \left(\frac{f_c}{ksi} \right)^{0.33} \cdot ksi = 4266.2\ ksi$$

Concrete Elastic Modulus, BDS Eqn. 5.4.2.4-1

Maximum Demands on Drilled Shaft from LPile Analysis:

$\gamma_p := 1.25$ — DC load factor Strength I, BDS Table 3.4.1-2

$P_{geo\&sw} := 12\ kip + \gamma_p \cdot A_g \cdot L_{shaft} \cdot \gamma_c = 118\ kip$ — Axial loading from geotech design basis + self weight

$M_{geo} := 7745\ kip \cdot in = 645.4\ kip \cdot ft$ — Maximum moment from LPile results

$V_{geo} := 70\ kip$ — Maximum shear from LPile results

$T_{geo} := 650\ kip \cdot ft$ — Maximum torsions from geotech design basis

SP-Column Analysis for Drilled Shaft:

SP-Column input:

Applied axial load and maximum moment demand as calculated by geotechnical team.

SP-Column output:

Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Critical Capacity" Method.

No.	Demand		Capacity		Parameters at Capacity			Capacity
	Pu	Mux	ϕPn	ϕMnx	NA Depth	εt	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	110.00	250.00	0.00	5427.06	13.27	0.01230	0.900	0.44

Figure 1: Factored loads and moments with corresponding capacity ratios

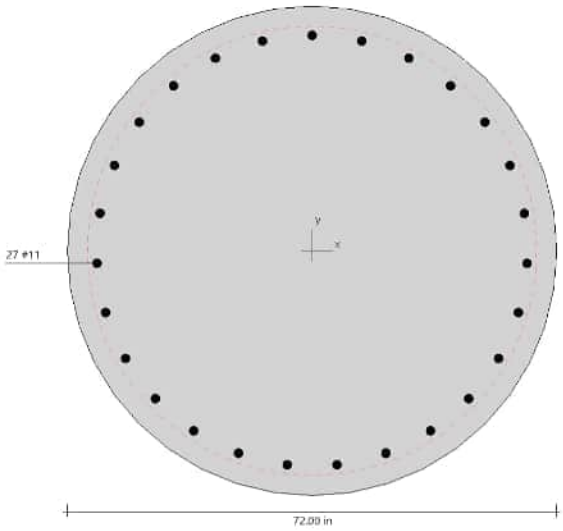


Figure 2: Cross-section and reinforcement layout

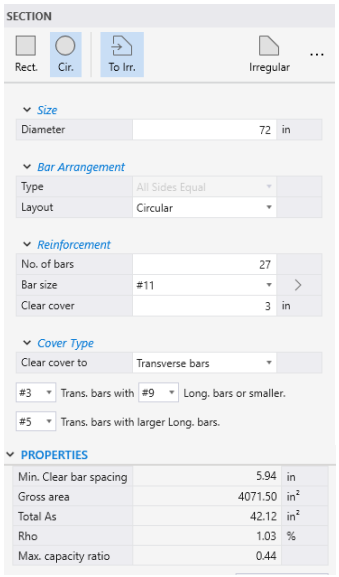


Figure 3: Reinforcement detail

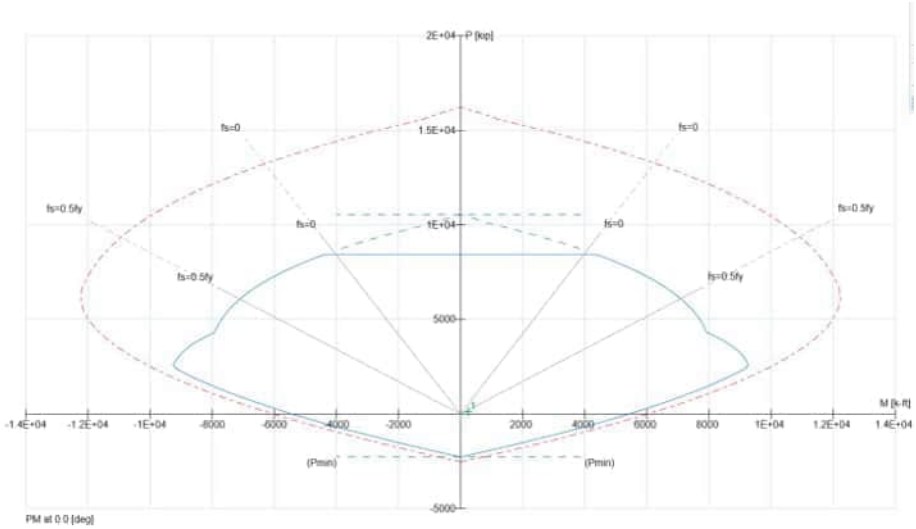


Figure 4: P&M interaction diagram

Reinforcement Limits:

Longitudinal reinforcement limit:

$\rho_{min} := 0.135 \cdot \left(\frac{f_c}{f_y}\right) = 0.009$ — BDS Eqn. 5.6.4.2-3

$A_{smin} := \rho_{min} \cdot A_g = 36.6\ in^2$ — Minimum area of steel

$\rho_{max} := 0.08$ — BDS Eqn. 5.6.4.2-1

$A_{smax} := \rho_{max} \cdot A_g = 325.7\ in^2$ — Maximum area of steel

$n_{bar} := N_b = 27$ — Assumed no. of bars

$A_b = 1.56\ in^2$

$A_s := n_{bar} \cdot A_b = 42.1\ in^2$ — Assumed area of steel

$A_{smin} \le A_s \le A_{smax} = 1$ $\quad \rho := \frac{A_s}{A_g} = 1.03\%$ — Check, OK

Minimum spacing check:

$s_{min} := min\left(1.5 \cdot d_b, 1.5\ in\right) = 1.5\ in$ — BDS 5.10.3.1.1

$s_{actual} := 5.94\ in$ — Actual clear spacing as reported in SP-Column.

$s_{min} < s_{actual} = 1$ — Check, OK

Design for Shear and Torsion:

- General procedure for non-prestressed section, used for the shear and torsion reinforcement design per AASHTO LRFD Bridge Design, Section 5.7

$\phi := 0.9$ — Strength reduction factor, shear (BDS 5.5.4)

$b_{v.shaft} := D_{shaft} = 72\ in$ — Effective web width for circular section, the diameter of the section (BDS Eq. 5.7.2.8-1)

$D_r := D_{shaft} - 2 \cdot cover - 2 \cdot d_{bh.st} - d_b = 63.3\ in$ — Diameter of the circle passing through the centers of the longitudinal reinforcement

$d_e := \frac{D_{shaft}}{2} + \frac{D_r}{\pi} = 56.2\ in$ — BDS Eq. C5.7.2.8-1

$d_{v.shaft} := 0.9 \cdot d_e = 50.5\ in$ — Effective shear depth, BDS Section 5.7.2.8

$A_{cp} := \pi \frac{D_{shaft}^2}{4} = 4071.5\ in^2$ — Area enclosed by outside perimeter of concrete section

$p_c := \pi\ D_{shaft} = 226.2\ in$ — Length of outside perimeter of the concrete section

$P_h := \left(D_r + d_b + d_{bh.st}\right) \cdot \pi = 205.4\ in$ — Perimeter enclosed by centerline of outermost shear reinf.

$A_{oh} := \pi\ \frac{\left(D_r + d_b + d_{bh.st}\right)^2}{4} = 3356.7\ in^2$ — Area enclosed by centerline of outermost shear reinf.

$A_o := 0.85 \cdot A_{oh} = 2853.2\ in^2$ — Area enclosed by the shear flow path, ACI Section 22.7.6.1.1

Check shear capacity (BDS 5.7.3.3):

$v_u := \frac{V_{geo}}{\phi \cdot b_{v.shaft} \cdot d_{v.shaft}} = 21.4\ psi$ — Shear stress on the section, BDS 5.7.2.8-1

$v_u < 0.125 \cdot f_c = 1$ — BDS 5.7.2.6 to determine adjustments to maximum spacing

Try BDS 5.7.3.4.2 to calculate β and θ for use in the shear/torsion equations:

$V_{eff} := \sqrt{V_{geo}^{\ 2} + \left(\frac{0.9 \cdot P_h \cdot T_{geo}}{2 \cdot A_o}\right)^2} = 262.2\ kip$ — Effective factored shear forces to be used in BDS Equation 5.7.3.4.2-4, BDS 5.7.3.4.2-5

$\varepsilon_s := \max\left(\frac{\left(\frac{M_{geo}}{d_{v.shaft}} - .5 \cdot P_{geo\&sw} + \left|V_{eff}\right|\right)}{E_s \cdot A_s}\right) = 2.9 \cdot 10^{-4}$ — Net longitudinal tensile strain in the section at the centroid of the reinforcement, BDS 5.7.3.4.2-4

$\beta := \frac{4.8}{1 + 750 \cdot \varepsilon_s} = 3.9$ — Factor indicating ability of diagonally cracked concrete to transmit tension and shear, for sections containing at least minimum amount of transverse reinforcement: BDS 5.7.3.4.2-1

$\theta := \left(29 + 3500 \cdot \varepsilon_s\right)\ deg = 30\ deg$ — Angle of inclination of diagonal compressive stresses, BDS 5.7.3.4.2-3

$\alpha := 90\ deg$ — Angle of inclination of transverse reinforcement to longitudinal axis

$\lambda := 1$ — Concrete density modification factor, BDS Section 5.4.2.8

$V_c := 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot b_{v.shaft} \cdot d_{v.shaft} = 905.8\ kip$ — Concrete shear resistance, BDS 5.7.3.3-3

$\phi \cdot V_c > V_{geo} = 1$ — Provide shear reinforcement to satisfy minimum requirement (Required due the consideration of torsion being required, BDS 5.7.2.3)

$$s_{max} := \begin{cases} \text{if } v_u < 0.125 \cdot f_c \\ \quad \left\| \min\left(0.8 \cdot d_{v.shaft}, 24\ \textit{in}\right) \right. \\ \text{else} \\ \quad \left\| \min\left(0.4 \cdot d_{v.shaft}, 12\ \textit{in}\right) \right. \end{cases} = 24\ \textit{in}$$

Maximum shear reinforcement spacing, BDS Eq. 5.7.2.6-1 & 5.7.2.6-2

$s := 8\ \textit{in}$ $\quad s < s_{max} = 1$

Transverse reinforcement spacing

$$A_{v.min} := 0.0316 \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}} \cdot \frac{b_{v.shaft} \cdot s}{f_y} = 0.61\ \textit{in}^2$$

Minimum transverse reinforcement area required within distances s, BDS 5.7.2.5-1

$A_v := 2 \cdot .31\ \textit{in}^2 = 0.62\ \textit{in}^2$ $\quad A_v > A_{v.min} = 1$

2 legs (one circular stirrup) Transverse reinforcement bar area (#5)

$$V_s := \frac{A_v \cdot f_y \cdot d_{v.shaft} \cdot \cot(\theta)}{s} = 406.7\ \textit{kip}$$

Steel shear resistance, BDS C5.7.3.3-1

$$V_n := \min\left(V_c + V_s, 0.25 \cdot f_c \cdot b_{v.shaft} \cdot d_{v.shaft}\right) = \left(1.3 \cdot 10^3\right)\ \textit{kip}$$

Nominal shear resistance for the section, BDS 5.7.3.3-1

$\phi V_n := \phi \cdot V_n = \left(1.2 \cdot 10^3\right)\ \textit{kip}$

$\phi V_n > V_{geo} = 1$ $\quad \frac{V_{geo}}{\phi V_n} = 5.93\%$

Check Torsion (BDS 5.7.3.6.2):

$T_{geo} = 650\ \textit{kip} \cdot \textit{ft}$

Applied torsional moment (from Geotech basis of design)

$$f_{pc} := \frac{\frac{P_{geo\&sw}}{\gamma_p}}{A_g} = 23.19\ \textit{psi}$$

Un-factored compressive stress in concrete

$$K := \min\left(\sqrt{1 + \frac{f_{pc}}{.126 \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}}}}, 2.0\right) = 1.05$$

BDS Eq. 5.7.2.1-6

$$T_{cr} := 0.126 \cdot K \cdot \lambda \cdot \sqrt{f_c \cdot \textit{ksi}} \cdot \frac{A_{cp}^{\ 2}}{p_c} = 1608.3\ \textit{kip} \cdot \textit{ft}$$

Torsional cracking moment, BDS 5.7.2.1-4

if $\max(T_{geo}) > 0.25 \cdot \phi \cdot T_{cr}$
  "Torsional effects shall be investigated"
else
  "Torsional effects shall not be investigated"

= "Torsional effects shall be investigated"

$A_t := .5 \cdot A_v = 0.31$ ***in***^{2} — Area of one leg of transverse reinforcement

$\theta = 30$ ***deg*** — Angle of inclination of diagonal compressive stresses

$T_n := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{s} \cdot 1 = 1913.3$ ***kip*** $\cdot$ ***ft*** — Nominal torsional resistance of the section, BDS 5.7.3.6.2-1

$\phi T_n := \phi \cdot T_n = 1722$ ***kip*** $\cdot$ ***ft***

$\phi T_n > T_{geo} = 1$ $\quad \frac{T_{geo}}{\phi T_n} = 37.75\%$

Check combined shear and torsion (BDS 5.7.3.6):

$A_s = 42.1$ ***in***^{2} — Total area of longitudinal reinforcement resisting torsion

$M_{geo} = 645.4$ ***kip*** $\cdot$ ***ft*** — Moment demands in the shaft, by envelope

$N_u := -P_{geo\&sw} = -118$ ***kip*** — Factored axial load, (use negative if compression and positive if tension), BDS 5.7.3

$$F_{combined} := \max\left(\frac{M_{geo}}{\phi \cdot d_{v.shaft}} + \frac{0.5 \cdot N_u}{\phi} \hookleft + \cot(\theta) \cdot \sqrt{\left(\frac{V_{geo}}{\phi} - 0.5\ V_s\right)^2 + \left(\frac{0.45 \cdot P_h \cdot T_{geo}}{2 \cdot A_o \cdot \phi}\right)^2}\right) = 430.6\ \textbf{kip}$$

Combined force, BDS 5.7.3.6.3-1

$A_s \cdot f_y > F_{combined} = 1$ $\quad U := \frac{F_{combined}}{A_s \cdot f_y} \cdot 100\% = 17.04\%$ — Torsion check, AASHTO 5.7.3.6.3-1

Drilled Shaft Design Summary:

- Loads used for design are taken from L-pile analysis output summary spreadsheet provided by geotechnical team.
- This drilled shaft design is valid for max shaft length of 20' and the demand loads provided herein.
- Minimum concrete compressive shall be 4 ksi and minimum yield strength of reinforcement rebars shall be 60 ksi.

- Provide 27#11 vertical bars with equal spacing and #5@8" on center transverse bars for shear.

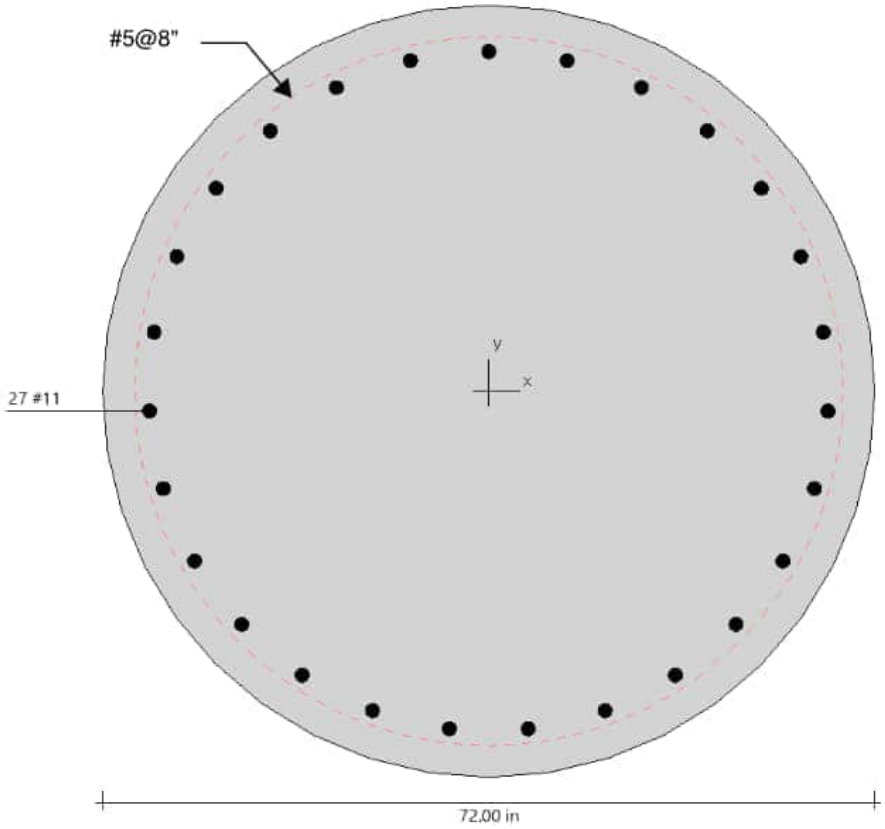


Figure 5: Reinforcement detail



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