REPORT

Geotechnical Design Report (GDR)

Hogan Road Bridge 5823 and Bridge 6787, 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 investigation, settlement analyses, global stability analyses, evaluation of the foundations of the existing Hogan Road I-95 Bridge 5823 (Bridge 5823), foundation design for the foundations for Bridge 5823 that will carry Hogan Road eastbound over I-95 and Hogan Road No. 2 Bridge 6787 (Bridge 6787) that will carry Hogan Road westbound over I-95, and geotechnical recommendations for the rehabilitation of Bridge 5823 over I-95 in Bangor, Maine at Exit 187 (WIN 018595.11, formerly identified as WIN 018595.10). The site location is presented in Figure 1 (attached). Bridge 5823 rehabilitation and the construction of the proposed new Bridge 6787 is part of a larger project that will reconfigure the existing interchange to a diverging diamond interchange. The purpose of this report is to present soils and bedrock information for the proposed Bridge 6787 site obtained from subsurface investigations and laboratory tests; present recommended geotechnical parameters for design and construction; and provide settlement analyses, global stability analyses, and foundation design for the proposed bridge and rehabilitated bridge foundations.

WSP reviewed the MaineDOT Preliminary Design Report[1] (PDR) for this existing bridge renovation and new bridge construction project. There are presently two bridges at the location: 1) The original two-lane bridge was constructed in 1960, is currently used for Hogan Road westbound travel, and is slated for demolition and reconstruction. 2) The second bridge was constructed in 1983, is currently used for Hogan Road eastbound travel, and will be rehabilitated for an additional 50 years of service. The PDR shows the 1983 Bridge 5823 is proposed to be used for Hogan Road eastbound travel for the new diverging diamond interchange, and a proposed new Bridge 6787 is proposed west of this bridge for Hogan Road westbound travel. MaineDOT chose a five-span steel girder bridge with pile supported semi-integral abutments with four (4) pile supported piers for the proposed Bridge 6787. For the rehabilitation of Bridge 5823, MaineDOT has elected to replace the existing abutments with new pile supported semi-integral abutments based on WSP's geotechnical evaluation of the existing foundation capacity, which is discussed in Section 7.0.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The project site lies in south-central Maine within the Seaboard Lowlands Physiographic Province which is characterized by an eroded glacial terrain with occasional low, elongated hills and ridges shaped by glacial ice flowing to the southeast and south.[2] Regional surficial geologic mapping indicates the surficial soils consist of Pleistocene Presumpscot Formation glaciomarine deposits which overlie Pleistocene glacial till deposits.[3] Granular 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 I-95 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,

[1] Maine Department of Transportation Bridge Program, December 23, 2020, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95, Exit 187 Diverging Diamond Interchange, Bangor, Maine, Federal Project #1859510, WIN 018595.10.

[2] Fenneman, N.M. and Johnson, D.W., 1946. Physical Divisions of the United States. U.S. Geological Survey, 1 sheet, scale 1:7,000,000.

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

non-sorted to poorly sorted mixture of clay, silt, sand, pebbles, cobbles, and boulders, with the upper few feet weathered.[4]

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.[5] 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 20 cm to 30 cm in thickness.[5] The metagreywacke 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 slate are also present with clasts of slate up to a few centimeters wide. These clasts are interpreted as rip-up clasts, are common near the bases of metagreywacke beds, and can also occur as isolated clasts with the metagreywacke beds.[5]

3.0 SUBSURFACE INVESTIGATIONS

WSP performed two series of bridge borings at the site: 100-series bridge borings (BB-HBR-1XX) during the 2022 preliminary geotechnical subsurface investigation, 200-series bridge borings (BB-HBR-2XX) during the 2023 supplemental geotechnical subsurface investigation, and 300-series hand-auger borings during a 2023 extended 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).

The borings were planned based on WSP's review of the 1980 Soils Report[6] from the Hogan Road bridge widening and Hogan Road Geotechnical Record Plans[7], 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[7]. 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".[6] Bedrock depth was found to vary from 41 feet below ground surface (bgs) near the abutment adjacent to northbound I-95 to 16 feet bgs near the abutment adjacent to southbound I-95.

[4] 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.

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

[6] 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.

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

3.1 2022 Preliminary Subsurface Exploration

To plan the 100-series bridge boring locations and depths, WSP reviewed the 2021 Progress Print Plans[8], 2022 Progress Print Plans[9], CADD drawings[10] received from MaineDOT on May 18, 2022, a technical presentation[11], a public meeting record[12], and preliminary bridge plans and concepts contained within the 2020 PDR[1]. The 2020 PDR[1] contained both a 3-span bridge option with abutments supported on spread footings atop MSE walls and a 5-span option with pile supported abutments for the proposed new Bridge 6787.

WSP completed the geotechnical subsurface investigation 100-series borings comprised of three (3) bridge borings labelled BB-BHR-101 through -103 completed between April 26, 2022 and April 29, 2022. MaineDOT surveyed the as-drilled boring locations following the completion of the drilling program. Seaboard drilled all the bridge borings to refusal to between 14.5 feet and 24.5 feet bgs. A summary of the testing and rock coring performed is summarized below.

- Standard Penetration Testing (SPT) was performed in all borings.
- *In situ* field vane shear tests were performed in boring BB-BHR-101 and attempted without penetration into soil in borings BB-BHR-102 and BB-BHR-103.
- Undisturbed Shelby tube samples were collected in borings BB-BHR-101 and BB-BHR-103.
- Seaboard collected 10.4 feet to 15.5 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 Supplemental Subsurface Exploration

To plan the 200-series bridge boring locations, WSP reviewed the 2023 Preliminary Design Report (PDR)[13], the preliminary plan and profile included within the 2023 PDR, and the results of WSP's Preliminary Geotechnical Design Report (PGDR).[14] This review identified that MaineDOT elected to abandon the 3-span option with abutments on MSE walls and instead selected to use the 5-span bridge alternative. The selection was based on WSP's PGDR,[14] which summarized the 100-series boring program, included comparative settlement evaluations for both the 3-span and 5-span alternatives, and which discussed the need to mitigate settlement which would have been more complex for the 3-span alternative. WSP reviewed the selected 5-span alternative to identify the abutment and pier locations for proposed Bridge 6787 such that each structure had a 100-series or 200-series associated boring or probe for use in foundation design.

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[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 February 9, 2021.

[9] 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.

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

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

[12] 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.

[13] VHB for Maine Department of Transportation Preliminary Design Report, Hogan Road/ I95 Bridge #5823 WIN 018595.10, marked draft dated 2-3-2023.

[14] WSP USA, Inc. Preliminary Geotechnical Design Report (PGDR) Hogan Road Bridge, Bangor, Maine WIN 018595.10, dated September 1, 2022.

WSP completed the supplemental geotechnical subsurface investigation 200-series borings comprised of six (6) bridge borings labelled BB-BHR-201 through -206, and four (4) bridge probes labelled BP-BHR-207 through -210 between September 25, 2023 and October 2, 2023. MaineDOT surveyed the as-drilled boring locations following the completion of the drilling program.

Seaboard drilled all the bridge borings and probes to refusal between 13.5 feet and 29.6 feet bgs. 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 borings BB-BHR-201, BB-BHR-204, and BB-BHR-206.
- Undisturbed Shelby tube samples were collected in boring BB-BHR-201 and attempted without recovery in BB-BHR-206.
- Seaboard collected 5 feet to 12 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.

3.3 2023 Extended Subsurface Exploration

WSP completed an extended geotechnical investigation on December 1, 2023 where WSP performed three (3) hand auger borings BP-BHR-301 through BP-BHR-303 to depths ranging from 2.6 feet to 4.6 feet bgs to collect soil samples at the existing eastbound Bridge 5823 abutment and pier substructures for use in an evaluation of the potential steel pile section loss of the existing in-place foundation piles for the proposed rehabilitation design. These locations were not surveyed by MaineDOT and are not included in Table 1 (attached). Boring BP-BHR-301 is located adjacent to Abutment 2, boring BP-BHR-302 is located adjacent to Pier 4, and boring BP-BHR-303 is located adjacent to Abutment 1. Soil samples were collected and stored in sealed glass jars for later evaluation and laboratory testing.

3.4 Subsurface Investigation Methods

WSP's boring subcontractor Seaboard completed the 100- and 200-series borings using a Diedrich D-50 track-mounted drill rig and the cased wash 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.

In situ field vane shear tests were performed for cohesive soils at select depths in select 100-series and 200-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 in 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 and 200-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 in Appendix A.

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 of both 100-series and 200-series 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 and Table 4-2. Geotechnical laboratory tests were performed on SPT split spoon samples representative of each soil type collected from the borings to assist in soil classification and representative rock core samples to assist in rock strength evaluation. GTX performed X-rays of undisturbed Shelby tube samples collected from 100-series and 200-series borings, and we used the images to identify potential disturbance zones and select suitable sample intervals for subsequent laboratory consolidation and

[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.

strength testing. Measured index and classification test results from soil samples are summarized in Table 4 (attached). Measured consolidation and strength properties from advanced geotechnical laboratory tests on Shelby tube samples are summarized in Table 5 (attached). Measured compressive strength and elastic modulus of rock are summarized in Table 6 (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 Completed on Soils

Laboratory Test	Standard(s)	100-series No. of Tests	200-series No. of Tests	300-series No. of Tests	Total No. of Tests[1]	Purpose
Grain Size (sieve)	ASTM D6913 AASHTO T 88	-	6	-	6	Index & Classification
Grain Size (sieve and hydrometer)	ASTM D6913 ASTM D7928 AASHTO T 88	8	2	-	10	Index & Classification
Atterberg Limits	ASTM D4318 AASHTO T 89, AASHTO T 90	6	11	-	17	Index & Classification
Moisture Content	ASTM D2216 AASHTO T 267	9	12	-	21	Index & Classification
Organic Content	ASTM D2974 AASHTO T 267	2	-	-	2	Index & Classification
X-Ray Shelby Tube	ASTM D4452	2	1	-	3	Selection for advanced laboratory testing
1D Incremental Load (IL) Consolidation	ASTM D2435	1	1	-	2	Consolidation and settlement soil properties
1D Constant Rate of Strain (CRS) Consolidation	ASTM D4186	2	-	-	2	Consolidation and settlement soil properties
Consolidated Undrained Triaxial Shear	ASTM D4767	-	1	-	1	Undrained shear strength
Direct Simple Shear	ASTM D6528	1	-	-	1	Undrained shear strength
pH	ASTM D4972 AASHTO T 289	3	6		9	Corrosion Evaluation
Electrical Resistivity	ASTM G572	3	6	2	11	Corrosion Evaluation
Chloride Ion in Water	ASTM D512 AASHTO T 291	3	7		10	Corrosion Evaluation

Laboratory Test	Standard(s)	100-series No. of Tests	200-series No. of Tests	300-series No. of Tests	Total No. of Tests[1]	Purpose
Sulfate Ion in Water	ASTM D516 AASHTO T 290	3	7		10	Corrosion Evaluation

Notes: [1]Testing quantities include laboratory tests from Highway borings HB-BHR-101, HB-BHR-102 and HB-BHR-108. These borings are discussed in the corresponding Geotechnical Design Report (GDR)[17] for Hogan Road Diverging Diamond Interchange. These data are presented in Table 5 of this report, and results are presented in Appendix C of this report. [2]WSP specified AASHTO T 288 testing, however GTX indicated that not enough soil was delivered to perform the tests, and we instead elected to have the test performed per ASTM G57.

Table 4-2: Summary of Laboratory Testing Completed on Rock Core

Laboratory Test	Standard(s)	100-series No. of Tests	200-series No. of Tests	Total No. of Tests	Purpose
Unconfined Compressive Strength	ASTM D7012 Method D	-	4	4	Compressive Strength Elastic Modulus

5.0 SUBSURFACE CONDITIONS

Soils encountered in the borings were found to generally include topsoil, fill, naturally occurring silt and clay deposits interpreted to be glaciomarine sediments, and gravel with sand and silt interpreted as glacial till over bedrock. Detailed descriptions of the soil and bedrock conditions encountered in the borings are provided in the boring logs in Appendix A.

WSP created one Interpretive Subsurface Profile (ISP) per bridge, where Figure 3 shows the ISP for Bridge 6787 and Figure 7 shows the ISP for Bridge 5823. WSP additionally created Interpretive Subsurface Cross-sections (ISCs) for each of the bridge substructures, where Bridge 6787 abutment and pier ISCs are shown on Figure 4 through Figure 6 and Bridge 5823 ISCs are shown on Figure 8. The following descriptions summarize these six stratigraphic units from the existing ground surface to depth illustrated in Figure 3 through Figure 8, which are attached to this report.

Topsoil: Topsoil thickness observed to be 0.2 feet to 0.8 feet thick in borings BB-BHR-101 through -103, BB-BHR-201 through -206, and BP-BHR-207 through -210.

Silt: A layer of fill was encountered in borings BB-BHR-201 to -204, -206, BP-BHR-207 to -210, at thickness ranging from 2.7 feet to 10 feet. The silt generally consisted of olive to brown, clayey to sandy silt with trace fine to medium sand and trace fine gravel No laboratory testing was done for this layer. SPT N_{60} values ranged from 5 to 17 (medium stiff to very stiff).

Clay: Clay was encountered in borings BB-BHR-101 through -103, BB-BHR-201 through -206 and BP-BHR-207 through -210 at thickness ranging from 5.0 feet to 18.6 feet. The clay generally consisted of olive to grey, low to moderately plastic, clay with trace to some silt, trace fine to coarse sand, and trace fine gravel Laboratory

[17] WSP USA, Inc. Geotechnical Design Report (GDR), Hogan Road Diverging Diamond Interchange, Bangor, Maine (WIN 018595.11), Submitted to MaineDOT on December 5, 2025.

classifications generally described the layer as CL (USCS classification) and A-4 or A-6 (AASHTO classifications). Field vane shear test data yielded undrained shear strength (s_u) values ranging from 659 psf to 1,339 psf. Where field vane tests were not conducted, SPT N_{60} values ranged from less than 1 to 72 (very soft to hard).

Glacial Till: A layer of sandy and silty gravel interpreted as glacial till was encountered in BB-BHR-101 to -103, BB-BHR-201 to -205, and BP-BHR-207, -208, and -210, with a thickness ranging from 3.5 feet and 9.6 feet. The glacial till generally consisted of gravel with fine to coarse sand, little to some silt and trace clay. Laboratory classifications generally described the layer as GM or SM (USCS classification) and A-1-b, A-2-4, or A-4 (AASHTO classification). SPT N_{60} values in the layer ranged from 18 to 95 (medium dense to very dense).

Bedrock: The bedrock surface was encountered in all borings between 13.5 feet to 29.6 feet below ground surface (bgs). Between 10 feet and 15 feet of rock core was collected in each boring in up to 5-foot runs. The predominant bedrock lithology encountered was grey, very fine grained to fine grained, medium weak (R3)[18] to very strong (R5)[18], fresh to moderately weathered 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 = 76%) and the estimated RMR[19] (rock mass rating) ranged from 38 to 66. Table 3 (attached) provides detailed information about recovery, RQD, RMR, and descriptions of lithology, rock mass, and discontinuities.

Groundwater: Groundwater level measurements were made in all borings upon completion of the boreholes and before the removal of the casing. Measured groundwater levels ranged from 0 feet bgs to 9.9 feet bgs, measured elevations ranged from between EL 78.0 feet and EL 96.0 feet. Groundwater levels shown on the ISP's and ISCs (attached Figure 3 through Figure 8) were interpreted based on these water level measurements and moisture contents (e.g., dry, moist, wet) observed in the soil samples collected from the borings

In general, groundwater measurements made during drilling are influenced by the addition of water used during the drilling process and the measurements may not be representative of a stabilized water level Groundwater levels also fluctuate due to other natural variations, such as seasonal/climatic influences, precipitation, barometric pressure, and temperature, and due to other variations, such as construction and groundwater pumping.

6.0 GEOTECHNICAL 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 pertaining to site conditions. Evaluations were performed in accordance with AASHTO LRFD[20] (2020).

6.1 Engineering Design Parameters

WSP evaluated the geotechnical data collected and developed design parameters for the bridge extents. Based on our evaluation, the soils and their measured and correlated engineering properties varied between the bridge abutments and piers. Therefore, we developed engineering design properties for each soil layer encountered at

[18] Wyllie, D.C., 2018. Rock Slope Engineering, 5th ed., CRC Press; see p. 501, Appendix II, Table II.3 for relative description of rock strengths based on field identification methods, which is based on the International Society for Rock Mechanics (ISRM) scale.

[19] Bieniawski, Z.T., 1989. Engineering rock Mass Classifications, John Wiley & Sons, NY.

[20] AASHTO LRFD: American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 9th Ed. 2020.

the bridge area which are different from the properties developed for the highway area. Table 6-1 provides a summary of design parameters for each *in-situ* soil layer encountered, and Table 6-2 provides a summary of design parameters for construction fills based on recommendations from MaineDOT Standard Specifications[21] for typical fill materials and WSP's interpretation of supplier information, published literature, and ULFGA laboratory data from past projects for Ultra-Lightweight Foamed Glass Aggregate (ULFGA) These properties were used for the analysis we performed.

Table 6-1: Recommended 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	30	-	-	180
Soft to Medium Stiff Clay	121	-	2.6	888	840
Stiff to Very Stiff Clay	122	-	2.6	1343	1236
Glacial Till	130	35	-	-	413
Bedrock	158	-	-	-	-

Note: Additional geotechnical design parameters for *in situ* soils used in the settlement modeling are provided in the corresponding Geotechnical Design Report (GDR)[17] for Hogan Road Diverging Diamond Interchange.

Table 6-2: Recommended Geotechnical Design Parameters for Fill Materials

Soil Layer	Unit weight (pcf)	Friction Angle (degrees)	Lateral Earth Pressure at Rest, K_0[1]
Asphalt	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
Crushed Stone	130	35	0.44

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. [4]The remaining material properties are based on recommendations in MaineDOT Standard Specifications and WSP's experience.

[21] Maine Department of Transportation, 2020, Standard Specifications. https://www.maine.gov/dot/sites/maine.gov.dot/files/docs/contractors/publications/standardspec/docs/2020/2020%20Standard%20Specification.pdf

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), laboratory-measured consolidation and strength properties presented in Table 5 (attached), and laboratory-measured rock core properties presented in Table 6 (attached). While these parameters served as a basis for analyses, adjustments or assumptions may differ for the specific analysis performed.

6.2 Frost Penetration Depth

WSP estimated an average frost penetration depth for roadway and bridge abutment and pier infrastructure fills using the MaineDOT Bridge Design Guide[22] (BDG). The site has an air design freezing index of approximately 1,726 °F-degree days. For roadway and bridge abutment and pier coarse-grained fills with 11% water content representative embankment fills, we estimate a frost penetration depth of approximately 87.1 inches (7.3 feet) at the site. We recommend structures be located deeper than the frost penetration depth of 7.3 feet. Refer to the full methodology of the analysis, calculations, and locations in Appendix D

6.3 Seismic Site Classification

WSP analyzed the seismic site classification following the criteria in AASHTO LRFD Article 3.10.3.1 for abutments at both bridges based on SPT N_{60} values. WSP determined the site's Seismic Zone as per AASHTO LRFD Article 3.10.6. Table 6-3 provides additional seismic parameters for each bridge. Refer to the full methodology of the analysis, calculations, and locations in Appendix D.

Table 6-3: Seismic Site Classification and Parameters

Location	PGA	A_s (1/s)	S_{DS} (1/s)	S_{D1} (1/s)	Site Class	Seismic Zone
Bridge 5823	0.090	0.104	0.224	0.106	D	Seismic Zone 1
Bridge 6787	0.089	0.104	0.224	0.106	D	Seismic Zone 1

Notes: See calculation in Appendix D for definitions of the parameters. PGA is peak ground acceleration, g; A_s is the zero period response seismic coefficient; S_{DS} is the short period response seismic coefficient (0.2 seconds); S_{D1} is the long period response seismic coefficient (1.0 seconds).

6.4 Site Soil Corrosivity

We tested representative samples for indicators of corrosivity (i.e., pH and electrical resistivity) at the ground surface to 17 feet bgs as shown in (Table 4, attached). The tested pH values ranged from 6.4 to 8.2, indicating that the soil is slightly acidic to moderately basic. Electrical resistivity values ranged from 1,038 ohm-centimeters (ohm-cm) to 7,666 ohm-cm. Measured sulfate contents ranged from <10 parts per million (ppm) to 30 ppm, and the measured chloride contents ranged from < 10 ppm to 120 ppm. Based on AASHTO LRFD Article 10.7.5, the following soil or site conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Resistivity less than 2,000 ohm-cm
- pH less than 5.5, and pH between 5.5 and 8.5 in soils with high organic content

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

- Sulfate concentrations greater than 1,000 ppm
- Chloride concentrations greater than 500 ppm
- Areas with a mixture of high resistivity soils and low resistivity alkaline soils

As such, the site soils tested may be characterized as noncorrosive based on measured pH, sulfate, and chloride, and corrosive based on median measured resistivity. Based on the corrosivity laboratory results, AASHTO (2023)[23] recommends using a corrosion probe to monitor microcell corrosion. Since a corrosion probe was not installed at the site, approximate corrosion rates from FHWA GEC 012[24] and NCHRP[25] were used to estimate section loss of the proposed piles.

We performed our pile foundation analyses for the proposed abutments for Bridge 5823 (Section 10.0) and the proposed abutments and piers for Bridge 6787 (discussed subsequently in Section 11.0 and Section 12.0) assuming a sacrificial thickness of piling steel will be lost to corrosion. Based on FHWA GEC 012 Section 6.12.1.2, we used an average corrosion rate for buried steel piles buried in fill or disturbed natural soils of 0.003 inches per year. Extrapolating this rate over the anticipated 100-year design life of the bridge, we assumed an end-of-life pile thickness loss of 0.08 inches per exposed steel face. We analyzed the factored axial and flexural resistance of the piles using a pile section reduced in size by this thickness section loss. Refer to the full methodology of the analysis, calculations, and locations in Appendix E.

WSP provided MaineDOT with a summary of our evaluation of Bridge 5823 HP 10x42 pile section loss as a Memorandum[26] dated August 13, 2024, which is provided under separate cover.

7.0 BRIDGE 5823 FOUNDATION REUSE EVALUATION

WSP evaluated the geotechnical and structural resistance of the existing Bridge 5823 HP 10x42 pile foundations at Abutment 1, Abutment 2, and Pier 5. The pile foundations were evaluated at current conditions assuming no section loss, and with section loss to the desired future design life of 50 years. The existing pile foundations were evaluated for the proposed loading as a result of the bridge rehabilitation. The analysis concluded that the existing HP 10x42 piles supporting Abutment 1 and Abutment 2 have sufficient geotechnical resistance but are undersized structurally when considering section loss, the proposed loading, and design life. The existing HP 10x42 piles supporting Pier 5 were sufficient structurally when considering section loss and the proposed loading and design life of 50 years.

WSP also evaluated the strength and service limit state bearing resistance of the existing Bridge 5823 Pier 1, 2, 3, and 4 spread footing foundations. WSP performed a bearing check for each of the four (4) pier spread footings bearing on bedrock and additionally evaluated sliding, and overturning resistance for Pier 4 spread footings for the

[23] AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R 27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023.

[24] Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016.

[25] National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011.

[26] WSP USA, Inc., Technical Memorandum: Geotechnical Soil Corrosivity Findings – REV01 Hogan Road Bridge Replacement, Bangor, Maine WIN 018595.10, dated August 13, 2024, 25 pages.

proposed loading provided by VHB for the bridge rehabilitation. The analysis concluded the pier foundations bearing on rock were adequate for the proposed loading and design life of 50 years.

WSP provided MaineDOT with additional details of the analyses performed, calculations, and results of the Bridge 5823 evaluation of existing foundations can be found in Technical Memorandum[27] dated November 6, 2024, which is provided under separate cover.

8.0 GEOTECHNICAL ANALYSES

This section describes the geotechnical analyses we performed for the proposed embankment fills associated with the proposed realignment of Hogan Road and the offramps and onramps from and to I-95 from Hogan Road provided in the 60% Bridge Plans[28], which VHB confirmed were only negligibly updated prior to submission of the Draft PS&E plans[29] The analyses described in the subsections below were conducted using the 9th edition of AASHTO LRFD[20], as it was the current edition when the design contract was awarded. These analyses include settlement and global stability of embankments, including approach embankments for Bridge 6787 and Bridge 5823 as described in the subsections below.

8.1 Global Stability Analysis

WSP evaluated global stability longitudinal to Hogan Road eastbound between Sta. 108+00 and Sta. 115+00 and Hogan Road westbound between Sta. 209+00 and Sta. 216+00 that includes the proposed abutments and abutment foreslopes. WSP also evaluated global stability transverse to Hogan Road westbound proposed Abutment 2 at Sta. 215+15 and Sta. 215+65. We obtained the geometry for the embankments from the 60% Bridge Plans[28] and the design material properties discussed in Section 6.1.

We evaluated global stability for both Bridge 5823 and Bridge 6787 longitudinal to both abutments considering both potential failure surfaces that extend below the abutment and that are contained within the abutment foreslopes under both static and pseudo-static seismic conditions. We additionally evaluated global stability transverse to Hogan Road Bridge 6787 Abutment 2 approach embankment at two locations where new embankment fills are proposed: just past the edge of the approach slab at Sta. 215+15 and 50 feet behind the edge of the approach slab at Sta. 215+65.

Global stability evaluations transverse to Bridge 6787 Abutment 1 were not conducted, as the slopes transverse to Abutment 1 and before the approach slab at Abutment 1 were relatively flat and were deemed stable by inspection based on the grading of the embankment for the abutment and Spur B.

8.1.1 Methodology

We performed global stability analyses using the two-dimensional limit equilibrium modeling software *Slide2* by Rocscience[30] for post-construction static load conditions longitudinal to the bridge centers for abutments and abutment foreslopes and transverse to the roadway centerline for embankment fills. Cross-sections were analyzed for potential failure surfaces and their factors of safety (FS) using: 1) the Spencer simplified method with

[27] WSP USA, Inc., Technical Memorandum: Geotechnical Evaluation of Existing Bridge Piles – Rev02, Hogan Road Bridge Replacement, Bangor, Maine WIN 018595.10, dated November 6, 2024, 270 pages.

[28] State of Maine Department of Transportation Bangor Penobscot County Hogan Road Bridge over I-95 Federal Aid Project No. 1859510, 60% Bridge Plans, Dated 8/31/2024, Received September 10, 2024.

[29] VHB for State of Maine Department of Transportation. Bangor, Penobscot County, US I-95 Exit 187 / Hogan Road Interchange, WIN 018595.10 / FED 1859510. Draft PS&E Plans dated 10/23/2025.

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

an auto refine search for circular surfaces, and 2) the Spencer method Cuckoo search with surface altering optimization for noncircular surfaces.

AASHTO LRFD Article 11.6.3.7 indicates that global stability of earth slopes with and without foundations should use an appropriate resistance factor, where ϕ = 0.75 (FS = 1.3) may be used where geotechnical parameters (e.g., stratigraphy and strength) are well defined and ϕ = 0.65 (FS = 1.5) should be used where geotechnical parameters are based on limited information or the subsurface conditions are highly variable. We recommend, and have used, the following factors of safety in our static global stability analyses:

- FS = 1.3 for potential failure surfaces at the abutment foreslopes and the transverse embankments under static conditions.
- FS = 1.3 for Bridge 5823 and embankments at both bridges for potential failure surfaces including *in situ* soils and extending below abutments and embankment fills. This lower FS is based on our understanding that the existing bridge structure and embankment fills having a similar geometry as proposed have been in place without slope stability issues for decades. A lower FS is further justified as the existing piles that will be cut off and remain in-place will contribute to the overall stability of the abutments but were excluded from the stability analysis.
- FS = 1.5 for Bridge 6787 for potential failure surfaces including proposed fills and extend below the abutments.

FHWA (2011)[31] recommends a minimum FS of 1.1 for pseudo-static seismic conditions, and our analyses use FS = 1.1 for all scenarios. The pseudo-static seismic analyses are based on FHWA (2011), which recommends using 50% of the peak average acceleration of the potential failure mass, k_{max}, determined for the site from the seismic site classification (see Section 6.3). We propose, and have used, FS = 1.1 for pseudo-static seismic global stability analyses

8.1.2 Global Stability Results – Longitudinal to Abutments

Results are summarized in Table 8-1 and Table 8-2 for static conditions for the longitudinal stability of both Bridge 6787 and Bridge 5823. Results are summarized in Table 8-3 for static conditions for potential failure surfaces for two the transverse stability analyses at Abutment 2. The full methodology of the analysis, calculations, and locations of the lowest factor of safety potential failure surfaces are presented in Appendix F.

Table 8-1 and Table 8-2 summarize the results for static conditions (targets FS ≥ 1.5 and FS ≥ 1.3) and pseudo-static seismic conditions (target FS ≥ 1.1) for the lowest FS for a potential failure surface at the bridge abutments and their foreslopes. Bridge 5823 was analyzed from Sta. 108+00 to Sta. 115+00. Bridge 6787 was analyzed from Sta. 209+00 to Sta. 216+00.

[31] FHWA. 2011. Geotechnical Engineering Circular No. 3 - LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual, Publication No. FHWA-NHI-11-032.

Table 8-1: Hogan Road Bridge 5823 Longitudinal Global Stability Factors of Safety

Structure	Loading Condition	Factor of Safety
Abutment 1 – Deep Surface	Static[1]	1.42
	Pseudo-static seismic[2]	1.27
Abutment 2 – Deep Surface	Static[1]	1.32
	Pseudo-static seismic[2]	1.23
Abutment 1 – Foreslope	Static[1]	1.53
	Pseudo-static seismjusic[2]	1.38
Abutment 2 – Foreslope	Static[1]	1.45
	Pseudo-static seismic[2]	1.30

Notes: [1]Recommended minimum FS = 1.3. [2]Recommended minimum FS = 1.1.

Table 8-2: Hogan Road Bridge 6787 Longitudinal Global Stability Factors of Safety

Structure	Loading Condition	Factor of Safety
Abutment 1 – Deep Surface	Static[2]	1.68
	Pseudo-static seismic[3]	1.54
Abutment 2 – Deep Surface	Static[2]	1.67
	Pseudo-static seismic[3]	1.47
Abutment 1 – Foreslope	Static[1]	1.39
	Pseudo-static seismic[3]	1.26
Abutment 2 – Foreslope	Static[1]	1.33
	Pseudo-static seismic[3]	1.19

Notes: [1]Recommended minimum FS = 1.3. [2]Recommended minimum FS = 1.5.
[3]Recommended minimum FS = 1.1.

8.1.3 Global Stability Results – Transverse at the Approach Embankments

Table 8-3 summarizes the analyses results for static conditions (targets FS ≥ 1.3 and FS ≥ 1.1) for the lowest FS for a potential failure surface at the embankments at abutment 2. The two transverse sections analyzed were at the back edge of the approach slab (Sta. 215+15) and 50 feet behind the edge of the approach slab (Sta. 215+65).

Table 8-3: Bridge 6787 Abutment 2 Approach Transverse Global Stability Factors of Safety

Structure	Loading Condition	Factor of Safety
Abutment 2 Approach Sta. 215+15	Static[1]	1.30
	Pseudo-static seismic[2]	1.18
Abutment 2 Approach Sta. 215+65	Static[1]	1.32
	Pseudo-static seismic[2]	1.19

Notes: [1]Recommended minimum FS = 1.3. [2]Recommended minimum FS = 1.1.

8.2 Settlement Analyses

8.2.1 Settlement Model Setup

WSP evaluated the anticipated total and differential settlement expected at the bridge abutments and approach slabs for the proposed new Bridge 6787 and renovated Bridge 5823 using the three-dimensional (3D) analysis software *Settle3*[32] by *Rocscience*. The *Settle3* models incorporate features from the March 2025 Progress Print Plans[33] for full the diverging diamond interchange (DDI), the 60% Bridge 6787 Plans[28], and the February 2025 Bridge 5823 Plan Markups[34] provided by VHB. We created two *Settle3* models for the bridges: an Interchange model that includes the proposed Bridge 6787 and surrounding ramps, and an Eastbound model for the renovated Bridge 5823.

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)
- Proposed Bridge 6787 abutment foreslope grading

The Eastbound model includes embankment fills for:

- Hogan Road Eastbound, Sta. 108+00 to Sta. 115+00 (700-foot length)

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

[33] 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.

[34] VHB for State of Maine Department of Transportation. Comment markups on “Hogan Road Bridge over Interstate 95 in the City of Bangor, Penobscot County (1983 As Built Record Plans)”. Received by WSP on February 12, 2025.

- Proposed Bridge 5823 abutment foreslope grading

The *Settle3* models were created using the existing topographic ground surfaces from the 2022 Progress Print Plans[9], the subsurface soil stratigraphy based on the 100-series and 200-series highway and bridge borings (attached Figure 3 through Figure 8), and the proposed embankment geometry from the March 2025 Progress Print Plans[33] for the locations modeled. Loading was applied above the existing ground surface using the geometry and unit weight of the proposed fills. For the proposed new Bridge 6787, we assumed an embankment fill unit weight of 125 pounds per cubic foot (pcf) for conventional fill (granular borrow) and 20 pcf for lightweight fill (foamed glass aggregate). For the renovated Bridge 5823, we assumed an aggregate subbase unit weight of 135 pcf to represent the loading from the proposed grade raise along Hogan Road eastbound.

We populated the *Settle3* models using our interpreted subsurface soil stratigraphy reported in both WSP's Preliminary Geotechnical Design Report (PGDR)[35] for the Diverging Diamond Interchange and Geotechnical Design Report (GDR)[17] for the Diverging Diamond Interchange for the highway portion of the project, which we revised near the bridge approaches based on the results of our supplemental 200-series bridge borings. Topsoil identified in the borings was assumed to be removed before placement of site development fills.

Our interpreted engineering properties for the soil layers are described in Section 6.1. We modeled the cohesionless soil layers (existing fills including the silt layer, sand and gravel, glaciofluvial sand, and glacial till) with immediate 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 borings (see Table 5, attached). Refer to the full methodology and basis of analysis, material properties, and model development in Appendix G.

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.

8.2.2 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-

[35] WSP USA, Inc. (formerly Golder Associates USA, Inc.) Preliminary Geotechnical Design Report (PGDR), Hogan Road Diverging Diamond Interchange, Bangor, Maine (WIN 018595.10), Submitted to MaineDOT on December 29, 2022.

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 conventional fill consisting of 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, 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 ULFGA where construction staging prevented the use of preloading.

8.2.3 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 8-4 provides a summary of the settlement estimates from *Settle3* for Abutment 1 and Abutment 2 at Bridge 6787 and Bridge 5823. These settlement estimates include the proposed settlement mitigation, in which the approach embankments behind the proposed new Bridge 6787 abutments are 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 settlement values presented in Table 8-4 include estimated total, immediate, and consolidation (primary consolidation and secondary compression) settlement over the life of the project (75 years) for the bridge approach embankment fills and abutment foreslope fills. Due to the staged construction, the settlement values for each abutment are shown at 75 years after construction completion of that abutment. Table 8-4 presents both the minimum and maximum settlement estimated along the back edge of each abutment. Appendix G 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 abutment in both plan view and section view to show variation in settlement.

Table 8-4: Settlement Results: Bridge Abutments

Structure		Bridge 6787		Bridge 5823	
Abutment		Abutment 1	Abutment 2	Abutment 1	Abutment 2
Elevation (ft)		98.4	87.0	113.1	110.0
Total Settlement, S_t (in.)	Max.	2.1	1.5	0.5	0.8
	Min.	0.5	0.4	0.4	0.6
Immediate Settlement, S_i (in.)	Max.	0.7	0.4	0.2	0.5
	Min.	0.1	0.1	0.1	0.3
Primary Consolidation Settlement, S_c (in.)	Max.	1.1	0.8	0.0	0.0
	Min.	0.1	0.1	0.0	0.0
Secondary Consolidation Settlement, S_s (in.)	Max.	0.3	0.3	0.3	0.3
	Min.	0.3	0.2	0.3	0.2

8.3 Lateral Earth Pressure at Abutments

Per the 60% Bridge Plans[28], the proposed Bridge 5823 Abutment 1 is approximately 83.8 feet wide including wingwalls, 5 feet thick, and founded at EL 98.0 feet, Abutment 2 is approximately 89.5 feet wide including wingwalls, 5 feet thick, and founded at EL 94.0 feet.

The proposed Bridge 6787 Abutment 1 is approximately 117.2 feet wide including wingwalls, 5 feet thick, and founded at EL 101.5 feet, Abutment 2 is approximately 117.8 feet wide including wingwalls, 5 feet thick, and founded at EL 99.0 feet.

Per the MaineDOT BDG Section 5.4.3, semi-integral abutments should be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the soil surcharge from the pavement surface to the bottom of the backwall. The superstructure backwall should be designed for full passive pressure. Per AASHTO LRFD Article 3.11.6.4, a live load Surcharge shall be applied where vehicular load is expected to act on the surface of the backfill. A live load surcharge is included in the lateral earth pressure acting over the superstructure back wall and the rigid abutment.

Per MaineDOT BDG Section 3.6.6, the passive earth pressure coefficient was determined using AASHTO LRFD Figure 3.11.5.4-1 and Figure 3.11.5.4-2. The Active earth pressure coefficient was determined using Rankine earth pressure coefficient (assuming level backfill) per MaineDOT BDG Section 5.4.3.

Engineering parameters and calculated earth pressure coefficients for the backfill material are presented in Table 8-5. We estimate that the resultant force generated from passive earth pressure on the superstructure backwall using the triangular pressure distribution. At Abutment 1 is an unfactored load of 13,071 pounds per foot of abutment length and acts at EL 116.8, at Abutment 2 the resultant force generated from passive earth pressure

on the superstructure backwall is an unfactored load of 20,734 pounds per foot of abutment length and acts at EL 113.7 feet.

We estimate the resultant force generated from active earth pressure on the rigid abutment using the Rankine method at Abutment 1 is an unfactored load of 9,613 pounds per linear foot of abutment length and acts at EL 104.9 feet. At Abutment 2 the resultant force generated from active earth pressure on the rigid abutment is an unfactored load of 10,893 pounds per linear foot of abutment length and acts at EL 101.2 feet.

These values assume the fill is free draining (i.e., no water pressure is allowed to build up behind the abutments). Per the 60% Bridge Plans[28], wingwalls are proposed to be in line, and cast monolithically, with the abutments. Therefore, we recommend their design is based on the K_p lateral earth pressure in Table 8-5. Refer to the full methodology of the analysis, material properties, and calculations in Appendix H.

Table 8-5: Recommended Lateral Earth Pressure Coefficients for Proposed Bridge Abutments

Structure	[1]Fill Material	[2]γ (pcf)	[3]ϕ (°)	[4]K_a	[5]K_p	[6]K_0
Bridge 5823 Abutments 1 & 2	Aggregate Subbase (Section 703.20)	135	36	0.26	8.1	0.41
	Granular Borrow (Section 703.19)	125	32	0.31	5.7	0.47
Bridge 6787 Abutments 1 & 2	Aggregate Subbase (Section 703.20)	135	36	0.26	8.1	0.41
	Granular Borrow (Section 703.19)	125	32	0.31	5.7	0.47
	Ultra-Lightweight Foamed Glass Aggregate (ULFGA)	20	35.5	0.27	7.5	0.42

Notes: [1]Materials identified in MaineDOT Standard Specifications. [2]Unit weight. [3]Friction Angle. [4]Active earth pressure, K_a, was determined using Rankine Method. [5]Passive earth pressure, K_p, was determined using AASHTO LRFD Figures 3.11.5.4-1 and 3.11.5.4-2. [6]At-rest lateral earth pressure coefficient, K_0, was determined as K_0 = 1-sin ϕ.

8.4 Elastic Inclusion Design for Abutments

The design for the bridge abutments put forward by VHB was to include elastic inclusions behind the portion of the abutments subject to passive earth pressure to allow movement of the abutment backwall associated with temperature-induced contraction and expansion of the bridges. VHB's request was for an elastic inclusion that is able to accommodate 60% of the estimated 2 inches of movement normal to the backwall of the abutments which was determined based on the movement rating determined using the MaineDOT BDG for each abutment which resulted in a maximum thermal movement of 2.8 inches in the direction of stationing that was corrected for bridge skew for both Bridge 5823 and Bridge 5787.

WSP recommends the use of an elastic inclusion made of Elasticized Expanded Polystyrene (EEPS). Conventional Expanded Polystyrene (EPS) exhibits linear elastic stress-strain behavior up to 1% strain, however EEPS exhibits linear elastic stress-strain behavior up to 10% strain and linear proportional stress-strain behavior

of 30%.[36] WSP determined the thickness of the EEPS using the method based on Hoppe and Eichenthal (2012)[37] for highly skewed integral abutment bridges and the method based on linear interpolation of deformation and strain. We also applied a factor of safety of 1.5 to the thickness. Calculations are provided in Appendix H.

WSP recommends a thickness of 30 inches of EEPS behind the backwall of the abutments over the height of the abutments where passive earth pressure acts. Based on our lateral earth pressure calculations (Section 8.3), The EEPS height is 4.0 feet and 5.0 feet Bridge 5823 Abutment 1 and Abutment 2, respectively. The proposed EEPS height is 5.3 feet and 5.2 feet for Bridge 6787 Abutment 1 and Abutment 2, respectively.

9.0 BRIDGE FOUNDATION DESIGN METHODOLOGY

The analyses described in the subsections below were conducted using the 9th edition of AASHTO LRFD[20], as it was the current edition when the design contract was awarded. These analyses include pile foundation design for Bridge 6787 abutments and piers, and pile design for the replacement abutments for Bridge 5823.

9.1 Pile Design Criteria

WSP evaluated all foundations using HP 14x89 piles with a yield strength of 50 ksi and assumed to have a section loss as described in Section 6.4. VHB provided WSP with a design life criterion of 100 years, and a maximum pile deflection criterion of 0.3 inches at the pile cap at the service limit state both parallel and perpendicular to centerline of bearings.

WSP analyzed the nominal and geotechnical pile resistance following the procedures outlined in AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG). We analyzed the structural and geotechnical resistance of the piles for driven and rock socketed piles using the resistance factors in Table 9-1.

Table 9-1: Pile Design Resistance Factors

AASHTO LRFD (2020) Reference	Loading and Resistance	Resistance Factor
Article 6.5.4.2	Structural: Pile resistance in axial compression, for piles subject to severe driving conditions, where a pile tip is necessary	$\phi_c = 0.50$
Article 6.5.4.2	Structural: Pile resistance in axial compression in the lower zone of the pile (since piles will be set into rock sockets rather than driven).	$\phi_c = 0.60$
Article 6.5.4.2	Structural: Combined axial flexural resistance of undamaged piles	$\phi_c = 0.70$
Table 10.5.5.2.4-1	Geotechnical: Axial tip resistance at the base of the rock socket	$\phi_{stat} = 0.50$
Article 5.5.4.2	Geotechnical: Bearing on concrete (grout)	$\phi_c = 0.70$
Table 10.5.5.2.3-1	Geotechnical: Nominal side and end bearing resistance, cohesionless soils, Static analysis	$\phi_{stat} = 0.45$
Table 10.5.5.2.3-1	Geotechnical: Nominal side and end bearing resistance, cohesive soils, Static analysis	$\phi_{stat} = 0.35$

[36] Horvath, John. (1995). Geofoam Geosynthetic.

[37] Hoppe, E.J, and Eichenthal, S.L. (2012). Thermal Response of a Highly Skewed Integral Abutment Bridge. FHWA / VCTIR 12-R10. https://vtrc.virginia.gov/media/vtrc/vtrc-pdf/vtrc-pdf/12-r10.pdf

AASHTO LRFD (2020) Reference	Loading and Resistance	Resistance Factor
Table 10.5.5.2.3-1	Geotechnical: Nominal side and end bearing resistance (rock), Static analysis	ϕ_{stat} = 0.45
Table 10.5.5.2.3-1	Geotechnical: Nominal bearing resistance, Dynamic analysis	ϕ_{dyn} = 0.65

9.2 Pile Loading

VHB provided WSP with unfactored loads at each bridge abutment and pier girder bearing location. WSP then used the FB-MultiPier[38] software to factor these loads to determine the Strength I, and Service I for Abutment loading, and Strength I, Strength III, Strength V, Service I, and Service IV for pier load combinations using maximum and minimum load factors per AASHTO LRFD[20]. One model was developed incorporating the geometry and stiffness for each substructure element (abutment and pier), and the range of pile lengths from WSP's interpreted bedrock surface. The bedrock surface was highly variable at this site with rock surfaces sloping from north to south and west to east, WSP interpreted the bedrock surface using the encountered bedrock depths at both sides of the proposed structures and a linear interpolation between the measured bedrock depths.

FB-MultiPier uses finite element analysis methods to evaluate foundation systems consisting of an abutment stem wall with girder bearings and wingwalls on the pile cap supported on piles either driven through soil and bearing on rock or within rock sockets. Linear or nonlinear structural finite element analysis is coupled with nonlinear static soil models to determine axial, lateral, and torsional response and loading on each pile.

9.3 Downdrag Loading

Per AASHTO LRFD Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. The settlement analyses performed and discussed in Section 8.2 was used to determine the settlement at the abutments along the length of the piles. While settlements at the abutment pile locations are expected to vary along the length of the abutment, WSP calculated a maximum downdrag loading value for the maximum loaded axial pile determined from the FB-MultiPier analyses of each substructure (Section 9.2) and at the pile location where the maximum settlement occurs beneath the abutment. The downdrag loading at the pile with the maximum axial load controlled the design.

WSP calculated downdrag loads in accordance with the methods described in AASHTO LRFD Article 10.7.3.7. We used the Beta Method (AASHTO LRFD Article 10.7.3.8.6c) to estimate skin-friction axial shaft resistance for both cohesionless and cohesive soils. Beta values from Dixon and Sandford (1998)[39] for cohesionless soils, which are similar to those reported in FHWA GEC 012.[24] Beta values from FHWA GEC 012 were used for cohesive soils, as these were slightly larger than those reported by Dixon and Sandford (1998). A downdrag load factor of 1.05 (Strength I) was used for the cohesionless soils, and a downdrag load factor of 1.40 (Strength I) was used for the cohesive soils, in accordance with AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2. Downdrag load along the pile

[38] Florida Bridge Software Institute. FB-MultiPier, Version 6.0.0

[39] Dixon, L.D. and Sandford, T.C. (1998). Construction Behavior Report: Effects of Bitumen Coating on the Axial and Lateral Loading of Abutment Piles Subject to Downdrag, Technical Report No. 95-04, Maine Department of Transportation Bureau of Planning, Research, and Community Services, Transportation Research Division.

increases with depth, and we assumed the maximum load occurs at the depth where the remaining settlement is below 0.4 inches.

9.4 Axial Loading

WSP analyzed the nominal structural and geotechnical pile resistance following the design procedures outlined in AASHTO LRFD and the MaineDOT BDG. In accordance with the MaineDOT BDG, the maximum factored axial design load applied to the piles is controlled by the lesser of the factored structural pile resistance, the factored geotechnical pile resistance, and the drivability resistance.

One model was developed for each substructure element (abutment or pier). For abutments, this consisted of an abutment stem wall with girder bearings and wingwalls on the pile cap supported on either piles socketed in soil and bedrock or driven to bedrock. For piers, this consisted of a pier cap with girder bearings, pier columns, and the pile cap supported on either piles socketed in soil and bedrock or driven to bedrock

9.4.1 Structural Resistance

Refer to Table 9-1 for structural pile resistance factors used in our evaluations.

9.4.2 Geotechnical Axial Resistance for Piles in Rock-Sockets

WSP analyzed tip resistance for rock-socketed piles assuming a 30-inch diameter rock socket, the diameter of which is approximately twice the pile flange width for HP 14x89 piles. For the rock-socketed piles, geotechnical axial resistance is based solely on tip resistance of the pile in the rock socket with no contribution from side resistance. Due to the weathered nature and poor RQD values in the upper portions of the bedrock, the upper 1-foot was ignored in the rock socket design depths. The rock sockets were assumed to begin 1-foot below the top of bedrock surface; the total rock socket depth reported includes the upper 1-foot for quantities. Refer to Table 9-1 for resistance factors used in our evaluations of resistance for a pile tip on bedrock.

We assumed that the rock sockets will be backfilled with grout of compressive strength 6 ksi in the annular space between the pile and the socket wall and between the bottom of the pile and the bottom of the socket; and will be backfilled with Backfill Type C above the rock sockets in the annular space between the pile and the soil shaft wall. We assumed that the pile installation will include a steel plate welded to the tip of the pile to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions of 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. Following the AISC design method for column base plates[40] a minimum plate thickness of 1 inch was calculated for the pile shoe plates. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between the base of the shoe plate and the base of the rock socket to permit full contact with the bedrock surface.

9.4.3 Geotechnical Axial Resistance for Driven Piles – Static Analysis

For driven piles, WSP analyzed the factored geotechnical resistance of a single pile in axial compression using a static analysis prediction. The static analysis consisted of calculating the shaft resistance along the length of the piles and tip resistance at the base of the piles using the Nordlund method for cohesionless soils, the Alpha method for cohesive soils, and the Canadian Geotechnical Society method for toe and shaft resistance in rock as described in AASHTO LRFD Article 10.5.5.2.3. Due to the weathered nature and poor RQD values in the upper

[40] American Institute of Steel Construction, Steel Design Guide Series, Column Base Plates, 3rd Edition, October 2003

portions of the bedrock, the driven piles were evaluated assuming the piles would be embedded 1-foot into bedrock. Refer to Table 9-1 for resistance factors used in our evaluations of geotechnical static resistance.

The Kulhawy and Goodman method for hard rock outlined in FHWA GEC 012[24] Section 7.2.1.4.2 was used to estimate the nominal pile toe resistance (R_p) for hard rock as $0.33q_u$ when RQD ≤ 70% and $0.80q_u$ for RQD = 100%. The following parameters were used for each of the substructure analyses:

- Rock Quality Designation (RQD) observed in the recovered rock core from the 100- and 200-series borings was very low, ranging from 0% to 76%, with an average of 25%, and a median of 25%. Refer to Appendix A, Appendix B, and Table 3 (attached) for more details.
- Low laboratory-measured unconfined compressive strength (q_u) from the most intact rock core, which ranged from 908 ksf to 3,107 ksf (refer to Table 6, attached).

9.4.4 Geotechnical Axial Resistance for Driven Piles – Wave Equation Analysis

For driven piles, WSP performed a wave equation analysis (WEAP) to estimate the nominal resistance of the proposed HP 14x89 piles driven to bedrock. We performed the WEAP using an American Pile Driving Equipment APE D25-42 single acting diesel pile driving hammer, a locally available diesel hammer used on similar projects with similar pile sizes. We assumed the limiting pile driving criterion used by MaineDOT of 10 blows per 0.5 inches of penetration at final driving.

A geotechnical resistance in rock was evaluated using the wave equation analysis program GRLWEAP[41] by Pile Dynamics, Inc. by comparing axial loads with the estimated driving stresses required to drive the piles to meet the driving criterion of 10 blows not exceeding 0.5 inches of penetration and limiting driving stress to 90% of the 50 ksi pile yield strength in accordance with the MaineDOT Standard Specifications[21] Section 501.042. The analysis evaluated resistance assuming the pile was driven to rock and a resistance distribution of 10% in the shaft and 90% at the toe for the shortest pile length each substructure, determined based on the driven pile lengths and bedrock elevation. Refer to Table 9-1 for resistance factors used in our evaluations of geotechnical dynamic resistance where driving criteria is established by dynamic testing with quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles. For the foundation analyses, WSP evaluated the axial geotechnical pile resistance using a wave equation analysis for the driven piles, as our opinion is the static analyses yield unrealistically low resistances.

9.4.5 Drivability Analysis

We performed drivability analyses using GRLWEAP[42] software to determine that the required pile resistance can be achieved for driven piles at Bridge 5823 Abutment 2 and Bridge 6787 Abutment 1, Abutment 2, and Pier 4 considering locally available diesel hammers based on the design axial loads. WSP understands it is possible to obtain additional axial resistance with the hammer evaluated and with other hammers; however, the reported nominal pile driving resistance is based on the analyzed maximum axial loads per structure. Per the MaineDOT Standard Specifications[8] Section 501.042, the driving stresses were limited to 45 ksi (90% of the pile steel yield stress), and the number of hammer blows at the required resistance was limited to between 3 and 15 blows per inch. The nominal pile driving resistance was calculated using a resistance factor ϕ_{dyn} = 0.65 for driving criteria

[41] Pile Dynamics, Inc. GRLWEAP Software Package Version 14

[42] GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.

that will be established by dynamic testing of at least two piles per site condition or 2% of the production piles, whichever is greater.

9.5 Lateral and Combined Loading Pile Response

WSP evaluated lateral response of the proposed abutment piles, whether driven or in rock sockets, using FB-MultiPier using the load combinations described in Section 9.1. We assumed the pile head to abutment connection is fixed. The input parameters were developed based on layer response models defined by the software, laboratory test results, correlations to soil properties determined from the field investigations, correlations with soil properties identified in the FB-MultiPier User Manual[43], and standard properties provided in the MaineDOT BDG[22]. The input parameters are summarized in the substructure design appendices. Refer to Table 9-1 for combined axial and flexural resistance factors used in our evaluations.

10.0 BRIDGE 5823 ABUTMENT FOUNDATION ANALYSES

Per the design, the existing Bridge 5823 abutments will be removed in their entirety and the existing HP 10x42 piles will be cut off a minimum of 1-foot below the proposed replacement Abutment 1 and Abutment 2 bottom of pile cap elevations of EL 98.0 feet and EL 94.0 feet, respectively.

Abutment 1 will be supported by 20 HP 14x89 rock-socketed piles. Piles will have a minimum center to center spacing of 6 feet parallel to centerline of bearings and 7 feet perpendicular to the centerline of bearings, with piles oriented with the strong axis parallel with the centerline of bearing. Rock sockets are required at Abutment 1 due to shallow bedrock sloping down from northeast to southwest to maintain a minimum pile length of 10 feet and to achieve fixity. Rock-sockets will be 30-inch in diameter and various socket lengths with the annular space between the socket wall and the pile backfilled with grout.

Abutment 2 will be supported by twenty 20 HP 14x89 piles driven to bedrock. Piles will have a minimum center to center spacing of 5.5 feet parallel to centerline of bearings and 7 feet perpendicular to centerline of bearings, with piles oriented with the strong axis parallel with the centerline of bearing. The Abutment 2 back row of piles will be vertical and the front row of piles will be battered at 3H:12V.

The sections below present further details on pile design, the full analysis methodology and calculations are provided in Appendix I.

10.1 Bridge 5823 Pile Loading

VHB provided WSP with unfactored loads[44] at each bridge girder bearing location and WSP used the FB-MultiPier software to factor these loads per AASHTO LRFD. WSP generated an HP 14x89 pile layout to reduce potential conflicts with the existing HP 10x42 piles that will be cut off and remain in-place. Pile lengths used in the analysis were taken from WSP's interpreted bedrock surface shown on attached Figure 7 and Figure 8. The pile layout was coordinated between WSP and VHB to provide sufficient spacing between the existing abutment piles that will remain in-place and the new proposed driven or rock socketed piles.

[43] Bridge Software Institute. 2018. FB-MultiPier Soil Parameter Table (US Customary Units).

[44] VHB Computations, Bangor DDI Hogan Rd Eastbound (1983) bridge Bearing Loads, dated 4/23/2025.

The settlement analyses performed and discussed in Section 8.2 at Abutment 1 and Abutment 2 indicated settlement below the abutments along the length of the pile was estimated to be less than 0.4 inches. Thus, downdrag loading was not considered in our pile evaluation.

10.2 Bridge 5823 Abutment Axial Pile Resistance

WSP analyzed the nominal structural pile resistance of the piles at Abutment 1 and Abutment 2 following the design procedures outlined in Section 9.4.1.WSP evaluated the axial geotechnical pile resistance for rock-socketed piles at Abutment 1 following the design procedures outlined in Section 9.4.2. WSP evaluated the axial geotechnical pile resistance using a wave equation analysis for Abutment 2 following the design procedures outlined in Section 9.4.4 and evaluated the required nominal pile driving resistance using procedures outlined in Section 9.4.5. Table 10-1 provides a summary of the strength limit state factors structural and geotechnical resistances for the Bridge 5823 abutment piles.

Table 10-1: Bridge 5823 Abutment Strength Limit State Factored Structural and Geotechnical Axial Pile Resistance

Abutment	Max. Applied Factored Axial Load (kips)	Factored Structural Resistance[1] (kips)	Factored Geotechnical Resistance[2] (kips)	Required Nominal Pile Driving Resistance[3] (kips)
Abutment 1 (Rock-Sockets)	259	534	432	-
Abutment 2 (Driven Piles)	230	439	434	353

Note: [1]Factored structural resistance in HP14x89 piles with ϕ_c = 0.60 for non-driven piles, and ϕ_c = 0.50 for driven piles. [2]Factored Geotechnical Resistance based in rock with ϕ_{stat} = 0.50 for Abutment 1 and based on wave equation analysis with ϕ_{dyn} = 0.65 for Abutment 2. [3]The nominal pile driving resistance is based on the design axial load divided by the dynamic resistance factor ϕ_{dyn} = 0.65.

10.3 Bridge 5823 Lateral and Combined Loading Pile Response

FB-MultiPier input parameters are summarized in Appendix I for the abutment piles.

For Abutment 1, WSP analyzed a pile length of 17 feet corresponding to the pile with the maximum axial load. The analysis indicates a corresponding moment of 188.6 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.23 inches occurs in the pile cap at the Service Limit State. The piles in Abutment 1 piles achieve fixity, with negligible lateral deflection at the pile tips.

For Abutment 2, WSP analyzed a pile length of 38.3 feet corresponding to the pile with the maximum axial load. The analysis indicates a corresponding moment of 122.6 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.19 inches occurs in the pile cap at the Service Limit State. The piles in Abutment 2 achieve fixity, with negligible lateral deflection at the pile tips.

Results of the FB-MultiPier analyses are summarized in Table 10-2. The lateral deflection in the strong axis (the direction of loading) is less than the design deflection criterion of 0.3 inches at the Service Limit State. The

combined axial and flexural interaction indicate that at both abutments the piles are stable against buckling, where the combined axial and bending ratio is less than 1.0.

Table 10-2: Bridge 5823 Summary of Abutment Pile Lateral and Combined Loading Response

Location	Max. Applied Factored Axial Load (kips)	Max. Lateral Deflection Weak Axis (in)	Max. Lateral Deflection Strong Axis (in)	Max. Moment at Pile Head (in-kips)	Combined Axial and Flexural Interaction[1] (-)
Abutment 1 (Rock Sockets)	259	0.02	0.23	2263	0.86
Abutment 2 (Driven Piles)	230	0.03	0.19	1472	0.69

Notes: [1]The combined axial and bending ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1.

10.4 Bridge 5823 Summary of Pile Analyses

We recommend the Abutment 1 piles be socketed in 30-inch diameter rock sockets with piles oriented with the strong axis parallel to I-95 with socket lengths ranging from 5 feet to 8 feet. We recommend the Abutment 2 piles be driven to, and seated on, bedrock and oriented with the strong axis parallel to I-95.

The recommended maximum factored loads for design of HP 14x89 piles at Abutment 1 and Abutment 2, pile lengths, and pile orientation are summarized in Table 10-3. Refer to Table 7 (attached) for estimated pile lengths, rock socket lengths, and bedrock elevation for each pile location for the Bridge 5823 abutments.

Table 10-3: Bridge 5823 Summary of Abutment HP 14x89 Pile Evaluations

Location	Recommended Nominal Pile Driving Resistance[1] (kips)	Recommended Maximum Factored Axial Load Per Pile (kips)	Estimated Pile Length (feet)	Pile Axis Orientation
Abutment 1 (Rock Sockets)	-	259	11. to 17	Strong
Abutment 2 (Driven Piles)	353	230	35 to 38	Strong

Notes: [1]Using a resistance factor of 0.65 for driving criteria that will be established by dynamic testing of no less than 2% of the production piles.

11.0 BRIDGE 6787 ABUTMENT FOUNDATION ANALYSES

Abutment 1 and Abutment 2 will each be supported by 24 HP 14x89 Grade 50 piles driven vertically to bedrock. Piles will have a minimum center to center spacing of 8.5 feet parallel to the centerline of bearings and 6 feet perpendicular to the centerline of bearings, with piles oriented with the strong axis parallel with the centerline of bearing. The sections below present further details on pile design, the full analysis methodology and calculations are provided in Appendix J.

11.1 Bridge 6787 Abutment Pile Loading

VHB provided WSP with unfactored loads[45] at each bridge girder bearing location and WSP used the FB-MultiPier software to factor these loads per AASHTO LRFD. The pile layout was coordinated by WSP and VHB based on the pile cap design deflection constraint of 0.3. The pile lengths used in the analysis were taken from WSP's interpreted bedrock surface shown on attached Figure 3 and Figure 4.

11.2 Bridge 6787 Abutment Downdrag Loading

The settlement analyses performed and discussed in Section 8.2 at the abutments indicate settlement below the abutments along the length of the piles was estimated to be 2.1 inches and 1.5 inches at Abutment 1 and Abutment 2, respectively. The resulting factored maximum downdrag load for an HP 14x89 pile in Abutment 1 is 21.9 kips for the pile with the maximum axial load and 34.5 kips for the pile where the maximum settlement is estimated. The resulting maximum downdrag load for an HP 14x89 pile in Abutment 2 is 10.3 kips for the pile with the maximum axial load and 15.3 kips for the pile where the maximum settlement is estimated. Under Strength I loading on the piles, the downdrag load for the pile with the maximum axial load controls the design.

11.3 Bridge 6787 Abutment Axial Pile Resistance

WSP analyzed the nominal structural pile resistance the piles at Abutment 1 and Abutment 2 following the design procedures outlined Section 9.4.1. WSP evaluated the axial geotechnical pile resistance using a wave equation analysis for Abutment 2 following the design procedures outlined in Section 9.4.4 and evaluated the required nominal pile driving resistance using procedures outlined in Section 9.4.5. Table 11-1 provides a summary of the strength limit state factors structural and geotechnical resistances for the Bridge 6787 abutment piles.

Table 11-1: Bridge 6787 Abutment Strength Limit State Factored Structural and Geotechnical Axial Pile Resistance

Abutment	Max. Applied Factored Axial Load (kips)	Factored Structural Resistance[1] (kips)	Factored Geotechnical Resistance[2] (kips)	Required Nominal Pile Driving Resistance[3] (kips)
Abutment 1 (Driven Piles)	237	414	460	364
Abutment 2 (Driven Piles)	229	405	453	353

Note: [1]Factored structural resistance in HP14x89 piles with ϕ_c = 0.50. [2]Factored Geotechnical Resistance based on wave equation analysis with ϕ_{dyn} = 0.65 for Abutment 2. [3]The nominal pile driving resistance is based on the design axial load divided by the dynamic resistance factor ϕ_{dyn} = 0.65.

11.4 Bridge 6787 Abutment Lateral and Combined Loading Response

FB-Multipier input parameters are summarized in Appendix J for the abutment piles

[45] VHB Computations, Bangor DDI Hogan Rd Westbound Bearing Loads, dated 11/12/2024.

For Abutment 1, WSP analyzed a pile length of 30.8 feet corresponding to the pile with the maximum axial load. The analysis indicates a corresponding moment of 133.9 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.18 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip.

For Abutment 2, WSP analyzed a pile length of 33.6 feet corresponding to the pile with the maximum axial load. The analysis indicates a corresponding moment of 144.1 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.37 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip. This analyzed deflection exceeds the design limit of 0.3 inches provided by VHB. To maintain a consistent pile count and layout between Abutment 1 and Abutment 2, VHB took no exception to the maximum deflection value of 0.37 inches estimated in the analysis at Abutment 2.

Results of the FB-MultiPier analyses at both abutments are summarized in Table 11-2. The combined axial and flexural interaction indicate that at both abutments the piles are stable against buckling, where the combined axial and bending ratio is less than 1.0.

Table 11-2: Bridge 6787 Summary of Abutment Pile Lateral and Combined Loading Response

Location	Max. Applied Factored Axial Load (kips)	Max. Lateral Deflection Weak Axis (in)	Max. Lateral Deflection Strong Axis (in)	Max. Moment at Pile Head (in-kips)	Combined Axial and Flexural Interaction[1] (-)
Abutment 1 (Driven Piles)	237	0.04	0.18	1607	0.74
Abutment 2 (Driven Piles)	229	0.06	0.37	1729	0.75

Notes: [1]The combined axial and bending ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1.

11.5 Bridge 6787 Summary of Abutment Pile Analyses

We recommend the Abutment 1 and Abutment 2 piles be driven to, and seated on, bedrock and oriented with the strong axis parallel to I95. The recommended maximum factored loads for design of HP 14x89 piles at Abutment 1 and Abutment 2, pile lengths, and pile orientation are summarized in Table 11-3. Refer to Table 8 (attached) for Bridge 6787 Abutment 1 and Abutment 2 estimated pile lengths for each pile location.

Table 11-3: Bridge 6787 Summary of Abutment HP 14x89 Pile Evaluations

Location	Recommended Nominal Pile Driving Resistance[1] (kips)	Recommended Max. Factored Axial Load Per Pile (kips)	Estimated Pile Length (feet)	Pile Axis Orientation
Abutment 1 (Driven Piles)	364	237	27 to 31	Strong
Abutment 2 (Driven Piles)	353	229	34 to 38	Strong

Location	Recommended Nominal Pile Driving Resistance[1] (kips)	Recommended Max. Factored Axial Load Per Pile (kips)	Estimated Pile Length (feet)	Pile Axis Orientation

Notes: [1]Using a resistance factor of 0.65 for driving criteria that will be established by dynamic testing of no less than 2% of the production piles.

12.0 BRIDGE 6787 PIER PILE FOUNDATION ANALYSES

WSP evaluated the proposed foundations for Bridge 6787 piers 1A/1B, 2A/2B, 3A/3B, and 4A/4B. All “A” piers will be supported on eight (8) vertical HP 14x89 Grade 50 piles with a center-to-center spacing of 9 feet parallel and perpendicular to the centerline of bearings. All “B” piers will be supported by ten (10) vertical HP 14x89 Grade 50 piles with a center-to-center spacing of 6.75 feet parallel to the centerline of bearings and 9 feet perpendicular to the centerline of bearings. All pier piles are oriented with the strong axis parallel with the centerline of bearing.

Rock sockets are required at Piers 1A/1B, 2A/2B, and 3A/3B due to shallow bedrock sloping down from northeast to southwest, to maintain a minimum pile length of 10 feet, and to achieve fixity. Rock sockets will be 30-inch diameter at various socket lengths with the annular space between the socket wall and the pile backfilled with grout. Pile foundations at Pier 4A/4B will be driven vertically to bedrock.

The sections below present further details on pile design with the full analysis methodology and calculations provided in Appendix K.

12.1 Bridge 6787 Pier Pile Loading

VHB provided WSP with unfactored loads[46] at each bridge girder bearing location on the piers and WSP used the FB-MultiPier software to factor these loads per AASHTO LRFD. The pile lengths used in the analysis were taken from WSP’s interpreted bedrock surface shown on attached Figure 5 and Figure 6. The pile layout was coordinated between WSP and VHB based on the pile cap design deflection constraint of 0.3 inches parallel and perpendicular to the centerline of bearings as provided by VHB.

12.2 Bridge 6787 Pier Downdrag Loading

Downdrag loading was not considered in our pile evaluation for the piers, as there are no planned increases in grade that would result in loading-induced settlement.

12.3 Bridge 6787 Pier Axial Pile Resistance

WSP analyzed the nominal structural pile resistance of the piles at Piers 1A/1B, 2A/2B, 3A/3B, and 4A/4B following the design procedures outlined in Section 9.4.1. WSP analyzed axial pile tip resistance for rock-socketed piles at Piers 1A/1B, 2A/2B, and 3A/3B following the design procedures outlined in Section 9.4.2. WSP evaluated the axial geotechnical pile resistance using a wave equation analysis for Abutment 2 following the design procedures outlined in Section 9.4.4 and evaluated the required nominal pile driving resistance using procedures outlined in Section 9.4.5.

Table 12-1 provides a summary of the strength limit state factors structural and geotechnical resistances for the Bridge 6787 pier piles.

[46] VHB Computations, Bangor DDI Hogan Rd Westbound Bearing Loads, dated 10/23/2024.

Table 12-1: Bridge 5823 Pier Strength Limit State Factored Structural and Geotechnical Axial Pile Resistance

Abutment	Max Applied Factored Axial Load (kips)	Factored Structural Resistance[1] (kips)	Factored Geotechnical Resistance[2] (kips)	Required Nominal Pile Driving Resistance[3] (kips)
Pier 1A (Rock-Sockets)	407	467	424	-
Pier 1B (Rock-Sockets)	385	484	432	-
Pier 2A (Rock-Sockets)	399	462	416	-
Pier 2B (Rock-Sockets)	386	480	432	-
Pier 3A (Rock-Sockets)	376	502	432	-
Pier 3B (Rock-Sockets)	390	438	424	-
Pier 4A (Driven Piles)	380	367	520	604
Pier 4B (Driven Piles)	393	393	520	604

Note: [1]Factored structural resistance in HP14x89 piles with ϕ_c = 0.60 for non-driven piles, and ϕ_c = 0.50 for driven piles. [2]Factored Geotechnical Resistance based op resistance in rock with ϕ_{stat} = 0.50 for Pier 1A/1B, 2A/2B, and 3A/3B and based on wave equation analysis with ϕ_{dyn} = 0.65 for Pier 4A/4B. [3]The nominal pile driving resistance is based on the design axial load divided by the dynamic resistance factor ϕ_{dyn} = 0.65.

12.4 Bridge 6787 Pier Lateral and Combined Loading Response

FB-Multipier input parameters are summarized in Appendix K for the abutment piles.

For Pier 1A and 1B, WSP analyzed total pile lengths of 14.1 feet and 19.5 feet respectively, including the rock socket, corresponding to the pile with the maximum axial load. For Pier 1A, the analysis indicates a corresponding moment of 34.9 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.09 inches occurs in the pile cap at the Service Limit State. For Pier 1B, the analysis indicates a corresponding moment of 31.2 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.09 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip.

For Pier 2A and 2B, WSP analyzed total pile lengths of 14.7 feet and 20.4 feet respectively, including the rock socket, corresponding to the pile with the maximum axial load. For Pier 2A, the analysis indicates a corresponding moment of 33.7 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.10 inches occurs in the pile cap at the Service Limit State. For Pier 2B, the analysis indicates a corresponding moment of 31.7 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of

0.09 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip.

For Pier 3A and 3B, WSP analyzed total pile lengths of 10.7 feet and 17.8 feet respectively, including the rock socket, corresponding to the pile with the maximum axial load. For Pier 3A, the analysis indicates a corresponding moment of 31.0 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.07 inches occurs in the pile cap at the Service Limit State. For Pier 3B, the analysis indicates a corresponding moment of 31.1 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.08 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip.

For Pier 4A and 4B, WSP analyzed total pile lengths of 21.2 feet and 17.9 feet respectively, including the rock socket, corresponding to the pile with the maximum axial load. For Pier 4A, the analysis indicates a corresponding moment of 28.6 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.12 inches occurs in the pile cap at the Service Limit State. For Pier 4B, the analysis indicates a corresponding moment of 27.4 kip-feet occurs in the pile under the Strength I load case, and a maximum lateral deflection of 0.11 inches occurs in the pile cap at the Service Limit State. The piles achieve fixity, with negligible lateral deflection at the pile tip.

Results of the FB-MultiPier analyses at both abutments are summarized in Table 12-2.

Table 12-2: Bridge 6787 Summary of Pier Pile Lateral and Combined Loading Response

Location	Max. Applied Factored Axial Load (kips)	Max. Lateral Deflection Weak Axis (in)	Max. Lateral Deflection Strong Axis (in)	Max. Moment at Pile Head (in-kips)	Combined Axial and Flexural Interaction[1] (-)
Pier 1A (Rock-Sockets)	407	0.09	0.08	419.3	0.96
Pier 1B (Rock-Sockets)	385	0.09	0.06	374.0	0.91
Pier 2A (Rock-Sockets)	399	0.10	0.08	404.1	0.95
Pier 2B (Rock-Sockets)	386	0.09	0.06	380.7	0.92
Pier 3A (Rock-Sockets)	376	0.07	0.04	372.1	0.88
Pier 3B (Rock-Sockets)	390	0.08	0.06	373.5	0.99
Pier 4A (Driven Piles)	380	0.12	0.10	337.4	0.88
Pier 4B (Driven Piles)	393	0.11	0.08	330.1	0.82

Notes: [1]The combined axial and bending ratio should be less than 1.0, as per AASHTO LRFD Article 6.9.2.2.1.

The lateral deflection in the strong axis (the direction of loading) is less than the maximum deflection value of 0.3 inches at the service limit state, provided by VHB. The combined axial and flexural interactions indicate the piles at Piers 1A/1B, 2A/2B, 3A/3B, and 4A/4B are stable against buckling, where the combined axial and bending ratio is less than 1.0.

12.5 Bridge 6787 Summary of Pier Pile Analyses

We recommend Pier 1A/1B, 2A/2B, and 3A/3B piles be socketed in a 30-inch diameter rock sockets with piles oriented with the strong axis parallel to the centerline of bearing. We recommend the Pier 4A/4B piles be driven to, and seated on, bedrock and oriented with the strong axis parallel to the centerline of bearing. The recommended maximum factored loads for design of HP 14x89 piles at Pier 1A/1B, 2A/2B, 3A/3B, and 4A/4B, pile lengths, and pile orientation are summarized in Table 12-3. Refer to Table 9 (attached) for pile lengths, rock socket lengths, and estimated bedrock elevation for each pile location for the Bridge 6787 piers.

Table 12-3: Bridge 6787 Summary of Pier HP 14x89 Pile Evaluations

Location	Recommended Nominal Pile Driving Resistance[1] (kips)	Recommended Maximum Factored Axial Load Per Pile (kips)	Estimated Pile Length[2] (feet)	Pile Axis Orientation
Pier 1A (Rock-Sockets)	-	407	14 to 18	Strong
Pier 1B (Rock-Sockets)	-	385	20	Strong
Pier 2A (Rock-Sockets)	-	399	15 to 17	Strong
Pier 2B (Rock-Sockets)	-	386	20 to 22	Strong
Pier 3A (Rock-Sockets)	-	376	11 to 15	Strong
Pier 3B (Rock-Sockets)	-	390	18 to 23	Strong
Pier 4A (Driven Piles)	607	380	19 to 21	Strong
Pier 4B (Driven Piles)	604	393	18 to 20	Strong

Notes: [1]Using a resistance factor of 0.65 for driving criteria that will be established by dynamic testing of no less than 2% of the production piles. [2]Estimated pile length includes pile cap embedment and rock socket embedment or assumed driven embedment into bedrock.

13.0 GEOTECHNICAL & CONSTRUCTION RECOMMENDATIONS

13.1 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 Section 203.09. which requires a firm, unyielding surface. Embankment fill should meet the requirements of MaineDOT Standard Specification Section 703.19 for Granular Borrow. Embankment excavation and fill placement should follow MaineDOT Specification 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, 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) 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.

13.2 Ultra-Lightweight Foamed Glass Aggregate (ULFGA)

WSP has identified two manufacturers/distributors of ULFGA within the United States, AeroAggregates of North America, LLC[47] headquartered in Eddystone, PA; and Glavel, Inc.[48] headquartered in Burlington, VT. Both of 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[49] foaming agent and Glavel uses a wet manufacturing process and glycerin, sodium silicate, and water as foaming agents.

Nicks et al (2024)[50] provide comprehensive laboratory-measured parameters and differences between wet-process and dry-process manufactured ULFGA material engineering properties, including gradation, particle

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

[48] Glavel, Inc. Company website. https://www.glavELcom. Accessed June 2023.

[49] 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

[50] 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

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[51,52,53,54] specifications and recommendations:

- ULFGA placed in embankments should be benched in accordance with MaineDOT Standard Specifications 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 and ULFGA does not become clogged due to fines migration from surrounding fill materials.
- 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

[51] 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

[52] Glavel, Inc. Lightweight Fill Guide. https://www.glavELcom/lightweight-fill-guide.

[53] Glavel, Inc. Commercial Construction. https://www.glavELcom/commercial-construction.

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

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).

13.3 Bridge Abutments and Piles

Adequate drainage should be installed behind the proposed semi-integral abutments at Bridge 5823 and Bridge 6787. For the Bridge 5823 semi-integral abutments with Granular Borrow backfill, we recommend the use of French drains and weap holes per MaineDOT Standard Specification Section 512, to assure efficacy of abutment drainage throughout the design life of the bridge. For the Bridge 6787 semi-integral abutments, although the ULFGA backfill is highly permeable we recommend the use of French drains and weap holes per MaineDOT Standard Specification Section 512, to assure efficacy of abutment drainage throughout the design life of the bridge. To prevent the migration of fines and potential clogging of the ULFGA backfill we recommend encapsulation in a geotextile as outlined in Section 13.2.

13.3.1 Elastic Inclusion

WSP makes the following construction recommendations with respect to the use of elasticized expanded polystyrene (EEPS) behind a portion of the bridge abutments.

- A 30-inch thick EEPS layer should be placed behind the abutment backwall per the Contract Plans and Special Provision 203 Excavation and Embankment (Elastic Inclusion).
- EEPS should have a minimum compressive strength of 720 psf at 10% strain and linear-elastic stress-strain behavior to 10% strain and linear proportional stress-strain behavior up to 30%.
- EEPS should be protected from organic solvents and hydrocarbons, fire, and prolonged exposure to sunlight.
- A minimum of 3 inches of Aggregate Sand for Leveling (MaineDOT Standard Specifications Item 703.05) be placed between ULFGA and the EEPS as shown on the Contract Plans.

13.3.2 Driven Piles

The estimated piles lengths provided in this report were based on our interpreted depth to bedrock which is inherently variable. To establish actual pile lengths needed to develop the required axial resistance of the driven piles, we recommend implementing a field-verification program consisting of a wave equation analysis and dynamic testing with signal matching including the following:

- Prior to the beginning of pile driving, a wave equation analysis should be conducted on the contractor's proposed driving system for each substructure element to ensure the hammer is capable of driving the piles to the required capacities without overstressing the piles to the required penetration depths and within a reasonable number of hammer blow counts, typically 3 blows per inch to 15 blows per inch at end of driving (EOD).
- Dynamic testing in accordance with MaineDOT specified procedures should be used to establish the driving criteria at the beginning of production pile driving. At a minimum, two percent of the production piles or a minimum of one pile per substructure should be subject to dynamic testing. However, we recommend the following dynamic load tests per substructure:
  - Two (2) plumb piles and two (2) battered piles at Bridge 5823 Abutment 2 from opposite ends of the foundation,
  - Two (2) dynamic load tests at Bridge 6787 Abutment 1 from opposite ends of the foundation,
  - Two (2) dynamic load tests at Bridge 6787 Abutment 2 at opposite ends of the foundation,
  - One (1) dynamic load test at Pier 4A, and one (1) dynamic load test at Pier 4B for Bridge 6787.

  We recommend the first production pile for each substructure be tested during the initial drive and at the 24-hour (minimum) restrike and the additional piles subject to dynamic load testing shall be selected by the Resident and the Engineer of Record. Dynamic testing and field inspection should include verification of hammer stroke or bounce chamber pressures and hammer blows throughout the pile driving operations. To facilitate field inspection, the MaineDOT Bridge Program should provide the Resident Engineer with a working Saximeter for the duration of pile testing and production pile driving. As outlined in Special Provision 501 Saximeter for Pile Driving from WIN 021726.10 (Freeport Approach Road and Merrill Road over I-295 and signalized intersections (Exit 22 and Exit 20 Interchanges, respectively) we recommend the following:

  > Saximeter. The Resident should use a Saximeter or equivalent device to collect data to monitor the blow count (for all hammer types), the stroke (for open-end diesel hammers only), or the kinetic energy (if the hammer is equipped with proximity switches for measuring impact velocity). The Saximeter should be completely charged and in sound working order prior to use and should be available for the duration of the pile driving operation. Pile driving operations should not be conducted without the use of a Saximeter.
  >
  > The Saximeter should be capable of performing the following functions:
  >
  > 1. Detect hammer blows automatically using sound recognition circuits, or manually with a keypad.
  > 2. Automatically count blows and determine the blows per minute (BPM) for all impact hammers.

3. Calculate the stroke for open-end diesel hammers.
4. Store blow count, penetration, average stroke or BPM data in memory.
5. Permit viewing of results on built-in screen.
6. Permit data transfer to PCs or printers.

For hammers equipped with proximity switches, the Saximeter should be deployed to acquire hammer impact velocity data by communicating with a transmitter mounted on the hammer and use this information to calculate the hammer's kinetic energy.

The Saximeter should operate on rechargeable batteries, with batteries and charger available.

- Piles should be driven to achieve bearing at the top of bedrock. We recommend that MaineDOT's typical refusal criteria of 10 blows per 0.5 inches be implemented to reduce the potential for structural damage to the piles. The piles should be fitted with standard prefabricated driving shoes meeting MaineDOT Standard Specification 501.048 to reduce potential for damaging the piles during driving and to engage the piles on sloping bedrock.
- The Engineer should review and approve all dynamic pile testing results before the piles are cut off.
- Signal matching analysis of the dynamic test data using methods described by Rausche et al. (1972)[41] should be conducted to determine pile bearing resistance.

13.3.3 Rock Socketed Piles

Socketed piles at Bridge 5823 Abutment 1 and Bridge 6787 Piers 1A/1B, 2A/2B, and 3A/3B should be installed in temporarily cased holes with a 30-inch-diameter socket drilled at the lengths specified in the attached Table 7, Table 8, and Table 9 (attached) per structure. Piles should be placed centered and plumb in the cased holes. The annular space around the piles should be filled with tremie-placed grout to the top of the bedrock surface after the piles are placed in the socket. We recommend that excavation and backfilling for the socketed piles be specified in accordance with Special Provision 501 for Rock Socketed Piles to supplement Section 501 of the Standard Specifications. Following pile installation and grout placement, the grout should be allowed to set for a minimum of 24 hours. After this 24-hour period, the remainder of the temporary casing above the top of the grout should be filled with Underdrain Backfill Material, Type C (MaineDOT Standard Specification 703.22). The piles should be supported vertically and laterally in their final positions during removal of the temporary casing as indicated in Special Provision 501.

13.4 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 to enable the 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.
- Assessment of seepage control requirements for permanent cut slopes.
- Subgrade preparation operations in site areas overlain with foamed glass aggregate and embankment fills.

- Placement, compaction, and testing of ULFGA and embankment fills for Special Provision 203 for Ultra-Lightweight Foamed Glass Aggregate.
- Preparation of subgrades for bridge abutments.
- Installation of the Elastic Inclusion at the bridge abutments per Special Provision 203 Excavation and Embankment (Elastic Inclusion)
- Pile driving per MaineDOT Standard Specifications Section 501 Foundation Piles
- Rock-socket drilling per Special Provision 501 Foundation Piles (Rock-Socketed H-Pile Foundations)

14.0 REPORT AND EXPLORATION LIMITATIONS

WSP prepared this report for the exclusive use of MaineDOT for specific application to the stability, settlement, and foundation design for the rehabilitation of the Hogan Road I-95 Bridge 5823, and the proposed Hogan Road No. 2 Bridge 6787 at I-95 Exit 187 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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https://wsponlinenam.sharepoint.com/sites/gld-157336/project files/6 deliverables/gdr bridge/win 018595.11 hogan road bridge gdr wsp 20251205.docx

Tables

Table 1: Boring Locations
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation[1]	As-Drilled Locations[2,3,4]		Existing Ground Surface Elevation[3,4] (ft)	Boring Depth[5] (ft)	Top of Bedrock[5] (ft)	Nearby Feature
	Stationing	Offset				
BB-BHR-101	210+11	17.8 LT	99.7	34.9	24.5	Proposed WB Abutment 1
BB-BHR-102	212+61	49.0 LT	92.4	29.7	17.7	Proposed WB Pier 3
BB-BHR-103	214+67	11.9 LT	88.5	38.0	22.5	Proposed WB Abutment 2
BB-BHR-201	210+99	53.8 RT	97.4	35	25	Proposed WB Abutment 1
BB-BHR-202	211+13	9.7 LT	91.5	32.1	22.1	Proposed WB Pier 1
BB-BHR-203	212+66	37.2 RT	91.4	36.2	26.2	Proposed WB Pier 2
BB-BHR-204	213+52	45.9 RT	88.9	39.6	29.6	Proposed WB Pier 3
BB-BHR-205	214+43	4.1 LT	87.9	30	20	Proposed WB Pier 4
BB-BHR-206	215+34	14.5 RT	87.4	30.8	18.8	Proposed WB Abutment 2
BP-BHR-207	211+61	37.2 RT	90.0	26.7	21.7	Proposed WB Pier 1
BP-BHR-208	211+13	6.3 LT	93.1	28	23	Proposed WB Pier 2
BP-BHR-209	213+09	22.2 LT	90.6	19.2	14.2	Proposed WB Pier 3
BP-BHR-210	214+95	26.8 RT	84.9	28.8	23.8	Proposed WB Pier 4

Notes

1. Borings BB-BHR-101 through BB-BHR-103 were drilled by Seaboard Drilling, LLC between April 26, 2022 and May 2, 2022; Borings BB-BHR-201 through BB-BHR-206 were drilled by Seaboard Drilling, LLC between September 25, 2023 and October 2, 2023.

2. Test borings locations are illustrated in the "Hogan Rd Bridge WIN 18595.10 WSP All Borings".

3. As-drilled locations, offsets, and elevations for borings BB-BHR-101 through BB-BHR-103 are derived from electronic file "Borings.dgn" received by WSP on May 24, 2022 from MaineDOT.

4. As-drilled locations and elevations for borings BB-BHR-201 through BB-BHR-206 and BP-BHR-207 through BP-BHR-210 are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.

5. Boring logs with descriptions of the soil and rock encountered at the site are provided in Appendix A.

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

Table 2: Summary of In-Situ Vane Shear Testing Results
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, 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)		
BB-BHR-101	99.7	MV	12.0 - 13.0	87.7 - 86.7	55 x 110	-	-	-	-	-	Could not push into soil
BB-BHR-101	99.7	V1	17.0 - 18.0	82.7 - 81.7	55 x 110	30	1340	10	447	3.0	
BB-BHR-101	99.7	MV	18.0 - 19.0	81.7 - 80.7	55 x 110	-	-	-	-	-	Could not push into soil
BB-BHR-102	92.4	MV	10.0 - 11.0	82.4 - 81.4	55 x 110	-	-	-	-	-	Could not push into soil
BB-BHR-103	88.5	MV	5.0 - 6.0	83.5 - 82.5	55 x 110	-	-	-	-	-	Could not push into soil
BB-BHR-103	88.5	MV	9.0 - 10.0	79.5 - 78.5	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
BB-BHR-201	97.4	MV	15.0 - 16.0	82.4 - 81.4	65 x 130	>44	>1217	-	-	-	Reached max. torque without failure
BB-BHR-204	89.0	V1	17.0 - 18.0	72.0 - 71.0	65 x 130	24	659	8	220	3.0	
BB-BHR-204	89.0	V2	18.0 - 19.0	71.0 - 70.0	65 x 130	37	1016	11	302	3.4	
BB-BHR-206	87.4	V1	15.0 - 16.0	72.4 - 71.4	65 x 130	36	989	4	110	9.0	
BB-BHR-206	87.4	MV	16.0 - 17.0	71.4 - 70.4	65 x 130	>44	>1217	-	-	-	Reached max. torque without failure

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: TJ
Checked By: KAR
Reviewed By: MEL

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										
BB-BHR-101	NQ (1.88)	99.7	R1	1.8	24.9	27.7	26.3	2.8	2.5	89%	1.10	39%	Poor	Fresh (W1)	Very Strong (R5)	60	24.9-27.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to steeply dipping, very closely to closely spaced, open; average 2.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-101	NQ (1.88)	99.7	R2	3.7	27.7	28.7	28.2	1.0	0.9	90%	0.00	0%	Very Poor	Moderately Weathered (W3)	Strong (R4)	44	27.7-28.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, moderately weathered; discontinuities moderately to steeply dipping, open; average 5.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-101	NQ (1.88)	99.7	R3	6.7	28.7	33.7	31.2	5.0	4.7	94%	2.45	49%	Poor	Fresh (W1)	Strong (R4)	55	28.7-33.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steeply dipping, very closely to moderately closely spaced, open; average 2.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-101	NQ (1.88)	99.7	R4	9.8	33.7	34.9	34.3	1.2	1.2	100%	0.55	46%	Poor	Fresh (W1)	Very Strong (R5)	60	33.7- 34.9 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities steeply dipping, closely spaced, open, thin (0.25 in) milky quartz and calcite vein present at 34.9 bgs; average 1.7 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-102	NQ (1.88)	92.4	R1	1.7	14.7	17.7	16.2	3.0	1.6	53%	-	-	-	-	-	-	14.7-17.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderately dipping, closely spaced, open, milky quartz and calcite veins (0.1 to 0.2 in thick) present throughout; average 1.3 fractures per foot [BOULDER]
BB-BHR-102	NQ (1.88)	92.4	R2	4.6	17.7	20.5	19.1	2.8	2.8	100%	1.10	39%	Poor	Slightly Weathered (W2)	Medium Weak (R3)	47	17.7-20.5 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, slightly weathered; discontinuities low angle to steeply dipping, very closely to closely spaced, open with grey sand and silt infilling, milky quartz and calcite veins (0.1 to 0.5 in thick) present at 18.5 ft bgs to 19.2 ft bgs and 20.2 ft bsg to 20.5 ft bgs; average 2.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-102	NQ (1.88)	92.4	R3	8.3	20.5	25.0	22.8	4.5	4.3	96%	3.40	76%	Good	Fresh (W1)	Medium Weak (R3)	63	20.5-25 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, moderately weathered (24.5-25 ft bgs) to fresh; discontinuities low angle to moderately dipping, very closely to moderately closely spaced, open, milky quartz and calcite vein (0.25 in thick) present at 34.9 bgs; average 0.9 fractures per foot; quartz veins 22.5-22.7 ft bgs and 23.6-23.8 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-102	NQ (1.88)	92.4	R4	11.6	25.0	27.2	26.1	2.2	2.2	100%	0.00	0%	Very Poor	Moderately Weathered (W3)	Medium Weak (R3)	38	25-27.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, moderately weathered; discontinuities low angle to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (1 in to 1.5 in thick) present at 26 ft bgs and 26.3 bgs; broken core [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-102	NQ (1.88)	92.4	R5	14.0	27.2	29.7	28.5	2.5	2.5	100%	0.70	28%	Poor	Slightly Weathered (W2)	Strong (R4)	54	27.2-29.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, slightly weathered; discontinuities low angle to vertical, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 29 ft bgs and 29.7 bgs; average 3.6 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										
BB-BHR-103	NQ (1.88)	88.5	R1	2.3	23.0	26.6	24.8	3.6	3.5	97%	0.45	13%	Very Poor	Fresh (W1)	Strong (R4)	50	23.0-26.6 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 23.3 ft bgs to 24.2 bgs and at 26.5ft bgs; average 3.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-103	NQ (1.88)	88.5	R2	5.1	26.6	28.6	27.6	2.0	2.0	100%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	50	26.6-28.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 27 ft bgs to 27.7 bgs; average 5.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-103	NQ (1.88)	88.5	R3	7.8	28.6	32.0	30.3	3.4	3.4	100%	0.70	21%	Very Poor	Fresh (W1)	Strong (R4)	50	28.6-32 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in to 1 in thick) present from 28.6 ft bgs to 29.3 bgs and 31.7 ft bgs to 32 ft bgs; average 3.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-103	NQ (1.88)	88.5	R4	10.2	32.0	33.4	32.7	1.4	1.4	100%	0.65	46%	Poor	Fresh (W1)	Strong (R4)	55	32.0-33.4 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 throughout; highly weathered fracture at 32 ft bgs; average 4.3 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION].
BB-BHR-103	NQ (1.88)	88.5	R5	11.6	33.4	34.8	34.1	1.4	1.0	71%	0.90	64%	Fair	Fresh (W1)	Strong (R4)	62	33.4-34.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle, closely spaced, open, milky quartz and calcite veins (0.1 in to 0.2 in thick) present throughout; average 1.4 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-103	NQ (1.88)	88.5	R6	13.9	34.8	38.0	36.4	3.2	3.2	100%	0.60	19%	Very Poor	Fresh (W1)	Strong (R4)	50	34.8-38 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 to 0.5 in thick) present throughout; average 2.5 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	Slightly Weathered (W2)	Very Strong (R5)	61	24-27 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.13 in to 1 in thick) present from 24 ft bgs 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)	Very Strong (R5)	54	27-29 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities moderately to steeply dipping, very closely to moderately spaced, open, milky quartz and calcite veins (0.25 in to 0.5 in thick) present from 28 ft bgs to 29 ft bgs; average 3.5 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

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MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										
BB-BHR-201	NQ (1.88)	97.4	R1	2.5	25.0	27.0	26.0	2.0	1.7	85%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	44	25-27 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, wide to close/very closely spaced, open to tight, extremely fractured throughout upper 6" of run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-201	NQ (1.88)	97.4	R2	5.3	27.0	30.5	28.8	3.5	2.5	71%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	42.5	27-30.5 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steep/vertical dipping, very close to closely spaced, tight to open, extremely fractured throughout bottom 6" of run, milky quartz intrusion present [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-201	NQ (1.88)	97.4	R3	7.5	30.5	31.4	31.0	0.9	0.6	67%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	43	30.5-31.4 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-201	NQ (1.88)	97.4	R4	9.7	31.4	35.0	33.2	3.6	3.6	100%	0.75	21%	Very Poor	Fresh (W1)	Medium Strong (R3)	48	31.4 -35 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, fresh; discontinuities low angle to steep dipping, very close to closely spaced, tight, milky quartz intrusion present [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-202	NQ (1.88)	91.5	R1	2.5	22.1	27.1	24.6	5.0	4.8	96%	1.70	34%	Poor	Fresh (W1)	Strong (R4)	55	22.1 - 27.1 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep angle and vertical dipping, close to very close spacing, open to tight, milky quartz intrusion at 1.2 to 2.2 ft bgs, frequent calcite intrusions, chemical weathering at 0.6 to 2.5 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-202	NQ (1.88)	91.5	R2	7.5	27.1	32.1	29.6	5.0	4.9	98%	2.80	56%	Fair	Fresh (W1)	Strong (R4)	62	27.1 - 32.1 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep angle dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-203	NQ (1.88)	91.4	R1	2.5	26.2	31.2	28.7	5.0	4.7	94%	3.20	64%	Fair	Fresh (W1)	Strong (R4)	60	26.2 - 31.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to steep dipping, very close spacing, tight, frequent calcite intrusions, chemical weathering at 29.7 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-203	NQ (1.88)	91.4	R2	6.5	31.2	34.2	32.7	3.0	3.0	100%	0.75	25%	Very Poor	Fresh (W1)	Strong (R4)	54	31.2 - 34.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, chemical weathering from 32.03 to 34.2 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-203	NQ (1.88)	91.4	R3	8.5	34.2	35.2	34.7	1.0	0.7	71%	0.30	30%	Poor	Slightly Weathered (W2)	Medium Strong (R3)	48	34.2 - 35.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, slightly weathered, severe chemical weathering on discontinuity surfaces; discontinuities moderate to steep dipping, close spacing, open [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-203	NQ (1.88)	91.4	R4	9.5	35.2	36.2	35.7	1.0	0.7	67%	0.00	0%	Very Poor	Slightly Weathered (W2)	Strong (R4)	46	35.2 - 36.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, slight chemical weathering; discontinuities moderate to steep dipping, very close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										
BB-BHR-204	NQ (1.88)	89.0	R1	1.5	29.6	32.6	31.1	3.0	2.7	90%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	49	29.6 - 32.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to vertical dipping, very close to close spacing, tight to open, frequent milky quartz and calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-204	NQ (1.88)	89.0	R2	4.3	32.6	35.1	33.9	2.5	2.3	92%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	47	32.6 - 35.1 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; extremely fractured, discontinuities steep to vertical dipping, close spacing, open, milky quartz intrusion 34.6 to 35.1 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-204	NQ (1.88)	89.0	R3	6.8	35.1	37.6	36.4	2.5	2.5	100%	0.33	13%	Very Poor	Fresh (W1)	Strong (R4)	47	35.1 - 37.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; highly fractured, discontinuities low angle to moderate dipping, very close spacing, tight, frequent milky quartz intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-204	NQ (1.88)	89.0	R4	9.0	37.6	39.6	38.6	2.0	1.6	80%	0.33	17%	Very Poor	Fresh (W1)	Strong (R4)	50	37.6 - 39.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; highly fractured, discontinuities horizontal to steep dipping, very close spacing, tight, milky quartz intrusion 37.6 to 38.06 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-205	NQ (1.88)	87.9	R1	2.5	20.0	23.9	22.0	3.9	3.6	92%	1.42	36%	Poor	Fresh (W1)	Strong (R4)	57	20 - 23.9 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep to vertical dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-205	NQ (1.88)	87.9	R2	6.8	23.9	28.6	26.3	4.7	4.5	96%	3.54	75%	Fair	Fresh (W1)	Strong (R4)	66	23.9 - 28.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-205	NQ (1.88)	87.9	R3	9.8	28.6	30.0	29.3	1.4	1.1	79%	0.58	41%	Poor	Fresh (W1)	Strong (R4)	57	28.6 - 30 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderate dipping, very 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 Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										
BB-BHR-206	NQ (1.88)	87.4	R1	3.1	18.8	22.3	20.6	3.5	1.5	43%	0.33	9%	Very Poor	Fresh (W1)	Strong (R4)	47	18.8 - 22.3 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle dipping, very close spacing, tight, extremely fractured 18.8 to 19.2 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-206	NQ (1.88)	87.4	R2	5.6	22.3	23.8	23.1	1.5	1.5	100%	0.41	27%	Poor	Fresh (W1)	Strong (R4)	48	22.3 - 23.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, extremely fractured 22.3 to 22.8 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-206	NQ (1.88)	87.4	R3	6.8	23.8	24.8	24.3	1.0	0.9	92%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	50	23.8 - 24.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep angle dipping, very close to close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-206	NQ (1.88)	87.4	R4	8.4	24.8	26.9	25.9	2.1	1.2	57%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	44	24.8 - 26.9 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate angle dipping, close to very close spacing, open to tight, extremely fractured 24.8 to 25.2 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-206	NQ (1.88)	87.4	R5	10.3	26.9	28.6	27.8	1.7	1.7	100%	0.00	0%	Very Poor	Fresh (W1)	Medium Strong (R3)	42	26.9 - 28.6 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, fresh; discontinuities moderate to steep angle dipping, very close spacing, tight, extremely fractured 27.8 to 28.6 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BB-BHR-206	NQ (1.88)	87.4	R6	12.2	28.6	30.8	29.7	2.2	2.1	95%	0.00	0%	Very Poor	Fresh (W1)	Strong (R4)	50	28.6 - 30.8 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, very close spacing, tight to open, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BP-BHR-207	NQ (1.88)	90.0	R1	2.5	21.7	26.7	24.2	5.0	4.2	84%	2.40	48%	Poor	Fresh (W1)	Strong (R4)	52	21.7 - 26.7 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to low angle dipping, very close to close spacing, tight, frequent milk quartz intrusions throughout entire run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BP-BHR-208	NQ (1.88)	93.1	R1	2.5	23.0	28.0	25.5	5.0	4.6	92%	1.83	37%	Poor	Fresh (W1)	Very Strong (R5)	57	23.0 - 28.0 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to steep dipping, very close to close spacing, tight to open, frequent calcite intrusions, milk quartz intrusion at 3.8 to 3.9 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BP-BHR-209	NQ (1.88)	90.6	R1	2.5	14.2	19.2	16.7	5.0	3.9	78%	1.50	30%	Poor	Fresh (W1)	Very Strong (R5)	55	14.2 - 19.2 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to vertical dipping, close to very close spacing, open to tight, frequent calcite intrusions, extremely fractured 14.9 to 15.7 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]
BP-BHR-210	NQ (1.88)	84.9	R1	2.5	23.8	28.8	26.3	5.0	4.8	96%	2.10	42%	Poor	Moderately Weathered (W3) to Fresh (W1)	Strong (R4)	48	23.5 - 28.5 ft: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, moderately weathered (25.2 to 26.9 ft bgs) to Fresh; discontinuities low angle dipping, close to very close spacing, open to tight, extremely fractured 25.2 to 26.9 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]

Table 3: Summary of Rock Core Quality
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation (-)	Core Size (in)	Existing Ground Surface Elevation[1] (ft)	Run						TCR [2]		RQD [3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description[6,7]
			No. (-)	Midpoint Depth Below Bedrock Surface (ft)	Depth Below Ground Surface (ft)			Length (ft)	Length (ft)	%	Length (ft)	%	Designation (-)	Weathering[4] (-)	Estimated Field Strength[4] (-)	Rock Mass Rating [RMR][5] (-)	
					Start	End	Midpoint										

Notes:

1. As-Drilled elevations for BB-BHR-101 through BB-BHR-103 and HB-BHR-105 are derived from the survey file received from MaineDOT on May 24, 2022 entitled: "Borings.dgn".
As-Drilled elevations for borings BB-BHR-201 through BP-BHR-204 are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.
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 Wyllie, 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. Mapped bedrock formation taken from Pollock, S.G., 2011, Bedrock Geology of the Veazie Quadrangle, Maine, Maine Geological Survey Open-File No. 11-58, scale 1:24,000, 1 sheet.

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

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)			Approximate Sample Elevation (feet)			Laboratory Testing[4,5,6,7,8,9,10]													Soil Classification	
									Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Organic Content (%)	pH of Soil in Distilled Water	pH of soil in Calcium Chloride	Chloride (ppm)	Sulfates (ppm)	Electrical Resistivity (ohm-cm)	Electrical Conductivity (ohm-cm)$^{-1}$	AASHTO[11]	USCS[12]
BB-BHR-101	99.7	2D	5.0	-	7.0	94.7	-	92.7	-	-	-	-	-	-	-	6.8	6.5	58	10	2169	4.6E-04	-	-
BB-BHR-101	99.7	3D	10.0	-	12.0	89.7	-	87.7	99.3	33.0	39	23	16	0.6	-	-	-	-	-	-	-	A-6 (18)	CL
BB-BHR-101	99.7	5D	20.0	-	22.0	79.7	-	77.7	23.0	10.2	-	-	-	-	-	-	-	-	-	-	-	A-1-b (0)	*GM*
BB-BHR-101	99.7	1U	15.0	-	17.0	84.7	-	82.7	99.7	33.0	31	18	13	0.9	0.7	-	-	-	-	-	-	A-6 (13)	CL
BB-BHR-102	92.4	1D-B	0.6	-	2.0	91.8	-	90.4	-	-	-	-	-	-	-	7.3	6.4	31	<10	3409	2.9E-04	-	-
BB-BHR-102	92.4	2D	5.0	-	7.0	87.4	-	85.4	-	32.5	36	22	14	0.7	-	-	-	-	-	-	-	-	CL
BB-BHR-102	92.4	3D-B	10.3	-	12.0	82.1	-	80.4	29.9	10.9	-	-	-	-	-	-	-	-	-	-	-	A-2-4 (0)	*GM*
BB-BHR-103	88.5	1D	0.0	-	2.0	88.5	-	86.5	-	-	-	-	-	-	-	7.8	7.2	120	<10	1446	6.9E-04	-	-
BB-BHR-103	88.5	2D	5.0	-	7.0	83.5	-	81.5	99.5	27.4	37	21	16	0.4	-	-	-	-	-	-	-	A-6 (17)	CL
BB-BHR-103	88.5	3D	10.0	-	12.0	78.5	-	76.5	99.6	29.8	36	21	15	0.6	-	-	-	-	-	-	-	A-6 (16)	CL
BB-BHR-103	88.5	4D	20.0	-	22.0	68.5	-	66.5	15.5	9.9	-	-	-	-	-	-	-	-	-	-	-	A-1-b (0)	*GM*
BB-BHR-103	88.5	1U	15.00	-	16.20	73.5	-	72.3	92.6	22.0	27	15	12	1.3	0.5	-	-	-	-	-	-	A-6 (9)	CL
BB-BHR-201	97.4	2D	5.0	-	7.0	92.4	-	90.4	99.7	24.1	36	20	16	0.3	-	-	-	-	-	-	-	A-6 (17)	CL
BB-BHR-201	97.4	1U	17.0	-	19.0	80.4	-	78.4	98.3	21.0	28	18	10	0.3	-	-	-	-	-	-	-	A-4 (9)	CL
BB-BHR-202	91.5	1D	0.0	-	2.0	91.5	-	89.5	-	-	-	-	-	-	-	7.80	-	-	-	1516	6.6E-04	-	-
BB-BHR-202	91.5	2D	5.0	-	7.0	86.5	-	84.5	-	-	-	-	-	-	-	8.20	-	37	<10	2473	4.0E-04	-	-
BB-BHR-202	91.5	3D	10.0	-	12.0	81.5	-	79.5	-	18.8	25	17	8	0.2	-	-	-	-	-	-	-	-	CL
BB-BHR-202	91.5	5D	20.0	-	22.0	71.5	-	69.5	91.6	21.7	-	-	-	-	-	-	-	-	-	-	-	A-4 (0)	*CL-ML*
BB-BHR-203	91.4	1D	0.0	-	2.0	91.4	-	89.4	-	-	-	-	-	-	-	8.17	-	78	<10	1212	8.3E-04	-	-
BB-BHR-203	91.4	2D	5.0	-	7.0	86.4	-	84.4	99.6	26.3	38	20	18	0.3	-	-	-	-	-	-	-	A-6 (19)	CL
BB-BHR-203	91.4	3D	10.0	-	12.0	81.4	-	79.4	-	-	-	-	-	-	-	7.51	-	-	-	2212	4.5E-04	-	-
BB-BHR-203	91.4	4D	15.0	-	17.0	76.4	-	74.4	-	33.9	33	19	14	1.1	-	-	-	-	-	-	-	-	CL
BB-BHR-204	89.0	1D	0.0	-	2.0	89.0	-	87.0	-	-	-	-	-	-	-	-	-	18	<10	-	-	-	-
BB-BHR-204	89.0	2D	5.0	-	7.0	84.0	-	82.0	99.0	25.1	36	22	14	0.2	-	-	-	-	-	-	-	A-6 (15)	CL
BB-BHR-204	89.0	3D	10.0	-	12.0	79.0	-	77.0	-	-	-	-	-	-	-	7.68	-	-	-	2603	3.8E-04	-	-
BB-BHR-204	89.0	5D	17.0	-	19.0	72.0	-	70.0	-	18.8	24	15	9	0.4	-	-	-	-	-	-	-	-	CL
BB-BHR-205	87.9	2D	5.0	-	7.0	82.9	-	80.9	99.4	27.1	40	20	20	0.4	-	-	-	-	-	-	-	A-6 (21)	CL
BB-BHR-205	87.9	3D	10.0	-	12.0	77.9	-	75.9	-	-	-	-	-	-	-	7.54	-	93	10	1038	9.6E-04	-	-
BB-BHR-205	87.9	4D-A	15.0	-	16.0	72.9	-	71.9	-	29.0	23	15	8	1.8	-	-	-	-	-	-	-	-	CL
BB-BHR-206	87.4	2D	5.0	-	7.0	82.4	-	80.4	99.5	26.4	35	19	16	0.5	-	-	-	-	-	-	-	A-6 (16)	CL
BB-BHR-206	87.4	4D	15.0	-	17.0	72.4	-	70.4	99.2	25.0	24	16	8	1.1	-	-	-	-	-	-	-	A-4 (6)	CL

Table 4: Summary of Laboratory Soil Index and Classification Testing Results
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Sample Number[3]	Sample Depth Below Ground Surface (feet)			Approximate Sample Elevation (feet)			Laboratory Testing[4,5,6,7,8,9,10]												Soil Classification	
									Sieve Minus No. 200 (%)	Moisture Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Organic Content (%)	pH of Soil in Distilled Water	pH of soil in Calcium Chloride	Chloride (ppm)	Sulfates (ppm)	Electrical Resistivity (ohm-cm)	Electrical Conductivity (ohm-cm)$^{-1}$	AASHTO[11]	USCS[12]
BB-BHR-301	106.0	1D	1.0	-	1.6	105.0	-	104.4	-	-	-	-	-	-	-	-	-	-	-	7666	1.30E-04	-	-
BB-BHR-301	106.0	2D	2.0	-	2.6	104.0	-	103.4	-	-	-	-	-	-	-	-	-	<10	30			-	-
BB-BHR-302	94.0	4D	4.0	-	4.6	90.0	-	89.4	-	-	-	-	-	-	-	-	-	112	10	-	-	-	-
BB-BHR-303	110.0	1D	1.0	-	1.6	109.0	-	108.4	-	-	-	-	-	-	-	-	-	-	-	7523	1.33E-04	-	-
BB-BHR-303	110.0	2D	2.0	-	2.6	108.0	-	107.4	-	-	-	-	-	-	-	-	-	-	-			-	-
BB-BHR-303	110.0	3D	3.0	-	3.6	107.0	-	106.4	-	-	-	-	-	-	-	-	-	<10	20			-	-

Notes:

1. Boring locations are illustrated in Boring Location Plans of the Geotechnical Design Report

2. As-drilled elevations for borings BB-BHR-101 through BB-BHR-103, HB-BHR-105, and HB-BHR-107 are derived from the electronic file "borings.dgn" provided by MaineDOT to WSP Golder on May 24, 2022. As-drilled elevations 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. As-drilled elevations for borings BB-BHR-301 through 303 were obtained by WSP staff.

3. Laboratory testing was performed by Geotesting Express of Acton, MA

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 performed in accordance with ASTM D2216, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.

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

7. Chloride testing was performed in accordance with ASTM D512-12, Standard Test Methods for Chloride Ion in Water Method B for 100-series borings, and in accordance with AASHTO T 291-Chloride Method B for 200-series and 300-series borings.

8. Sulfates testing was performed in accordance with ASTM D516-16, Standard Test Method for Sulfate Ion in Water for 100-series borings, and in accordance with AASHTO T290-Sulfates (Soluble) for 200-series borings.

9. pH of soil testing was performed in accordance with ASTM D4972, Standard Test Method for pH of Soils for 100-series borings, and in accordance with AASHTO T 289 for pH of Soils for 200-series borings.

10. Soil Resistivity testing was performed in accordance with ASTM G57, Standard Test Method for Measurement of Soil Resistivity Using the Wenner Four-Electrode Method.

11. 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.

12. 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.

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

Prepared By: ATM/LDN
Checked By: MA/DEB
Reviewed By: MEL

Table 5: Summary of Laboratory Soil Engineering Testing Results
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation[1] (-)	Ground Surface Elevation[2] (feet) (-)	Sample Number[3] (-)	Sample Depth Below Ground Surface (feet)	Approximate Sample Elevation (feet)	Tube X-ray (-)	Test Type[4] (-)	Test No. (-)	Bulk Unit Weight[6] (pcf)	Void Ratio, e (-)	Estimated In Situ Vertical Effective Stress, σ'_{v0}[7] (psf)	Consolidation Vertical Effective Stress, σ'_{vc}[8] (psf)	Consolidation Horizonal Effective Stress, σ'_{vc} (psf)	Undrained Shear Strength, s_u[9] (psf)	Pre-consolidation Stress, σ'_p[10] (psf)	Over-consolidation Ratio, OCR[11] (-)	Recompression Index, C_r[12] (-)	Compression Index, C_c[12] (-)	Coefficient of Consolidation, c_v[13] (ft^2/day)	Secondary Compression Index, C_a[14] (-)	Secondary Recompression Index, C_{ar}[14] (-)
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	0.0040
			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	-	-
BB-BHR-201	97.4	1U	17.3 - 17.9	80.1 - 79.5	YES	CAUC	CAU-1	124	0.73	1214	1139	916	1132	-	-	-	-	-	-	-
			18.5 - 18.8	78.9 - 78.7		ILC	IP-1	132	0.57	1272	-	-	-	2250	1.8	0.003	0.07	4.8	0.007	0.0004
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	-	-

Notes:

1. Boring locations are illustrated on the Boring Location Plans in the Geotechnical Design Report
2. As-drilled elevations for borings BB-BHR-101 through BB-BHR-103 are derived from the electronic file "borings.dgn" provided by MaineDOT to WSP on May 24, 2022. As-drilled elevations for boring BB-BHR-201 is derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.
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 = Anisotropically Consolidated Triaxial Shear in 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 and CAUC tests.
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. Highway 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: KAR/DEB
Checked By: MEL/RJN
Reviewed By: JEL/MEL

Table 6: Summary of Rock Laboratory Testing Results
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Test Boring Designation[1]	Ground Surface Elevation[2] (feet)	Top of Bedrock Depth[3] (feet)	Top of Bedrock Elevation (feet)	Sample Number[4] (-)	Sample Depth Below Top of Bedrock (feet)	Sample Depth Below Ground Surface (feet)	Sample Elevation (feet)	Unconfined Compressive Strength[5] (ksf)	Elastic Modulus[5] (ksf)	Poissons Ratio[5] (-)	Bulk Density[6] (pcf)
BB-BHR-201	97.4	25.0	72.4	R4 (31.4-35.0')	7.9 - 8.3	32.9 - 33.3	64.5 - 64.1	908	820,800	0.38	172
BB-BHR-206	87.4	18.8	68.6	R1 (18.8-22.3')	0.3 - 0.7	19.1 - 19.47	68.3 - 67.9	1811	904,320	0.31	169
BP-BHR-208	93.1	23.0	70.1	R1 (23.0-28.0')	2.6 - 3.0	25.6 - 26.0	67.5 - 67.2	3107	976,320	0.44	171
BP-BHR-209	90.6	14.2	76.4	R1 (14.2-19.2')	2.7 - 3.1	16.9 - 17.3	73.7 - 73.3	848	792,000	0.16	171

Notes:

1. Boring locations are illustrated on the Boring Location Plan of the Geotechnical Data Report.

2. As-drilled locations and elevations for borings BB-BHR-201 through BP-BHR-203 are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.

3. Top of bedrock surface refers to the interpreted top of competent rock based on roller cone/auger resistance or material observed in rock core barrel. Notes regarding weathered rock, if present, are provided in Appendix A of the Geotechnical Design Report.

4. Laboratory testing was performed by Geotesting Express of Acton, MA.

5. Testing was performed based ASTM D7012 Method D Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures. Young's modulus and Poisson's ratio values are presented for compressive stress ranges 2300 to 4000 psi (BB-BHR-201), 4600 to 8000 psi (BB-BHR-206), 13700 to 194000 psi (BP-BHR-202), and 11600 to 16500 psi (BP-BHR-203). The stress range reported correspond to the lowest Young modulus measured.

6. Density determined by GTX on core samples by measuring dimensions and weight before testing and then calculating.

7. Complete laboratory test results for rock core testing are provided in Appendix C.

8. ft = feet; ksf = kilopounds per square foot; pcf = pounds per cubic foot

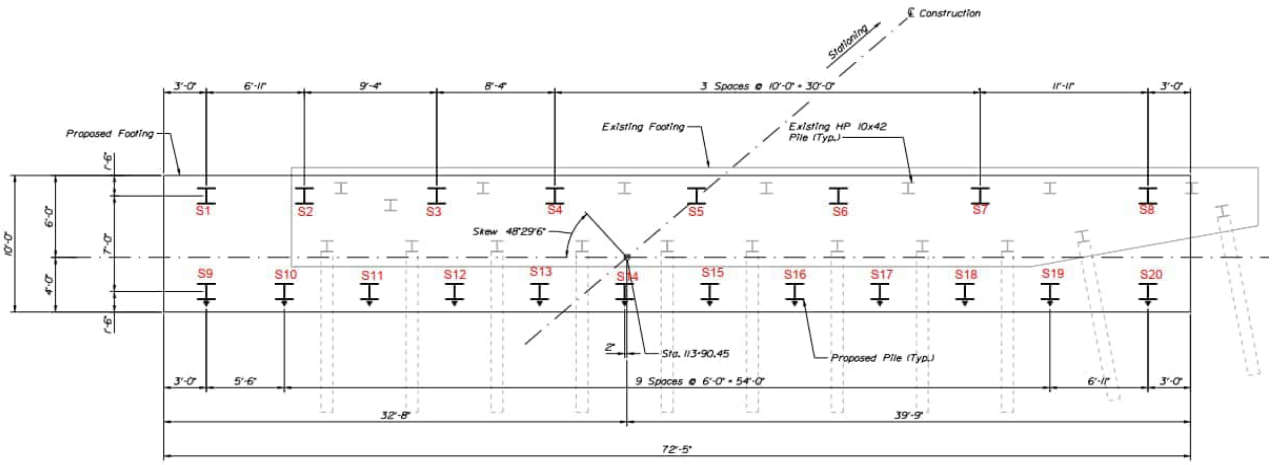
Prepared By: LMP
Checked By: KAR
Reviewed By: MEL

Table 7: Summary Abutment Pile Lengths for Bridge 5823 (Hogan Road Eastbound)
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Pile	Bottom of Abutment Pile Cap Elevation (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Top of Grout Elevation (feet)	Approximate Bottom of Pile Elevation (feet)	Estimated Pile Length[1] (feet)	Minimum Rock Socket Length (feet)
Eastbound Abutment 2						
S1	94	61.6	-	60.6	35	-
S2	94	61.6	-	60.6	35	-
S3	94	61.5	-	60.5	35	-
S4	94	60.7	-	59.7	36	-
S5	94	59.8	-	58.8	37	-
S6	94	59.8	-	58.8	37	-
S7	94	59	-	58	38	-
S8	94	58.2	-	57.2	38	-
S9	94	61.6	-	60.6	36	-
S10	94	61.6	-	60.6	36	-
S11	94	61.5	-	60.5	36	-
S12	94	61.5	-	60.5	36	-
S13	94	60.7	-	59.7	37	-
S14	94	60.7	-	59.7	37	-
S15	94	59.8	-	58.8	38	-
S16	94	59.8	-	58.8	38	-
S17	94	59	-	58	39	-
S18	94	59	-	58	39	-
S19	94	58.2	-	57.2	40	-
S20	94	58.2	-	57.2	40	-

Note: [1]Due to the weathered nature and poor RQD, the piles are assumed to penetrate 1-foot in to bedrock. The estimated pile length includes the assumed 1-foot penetration into bedrock.

Prepared By: ATM
Checked By: LDN
Reviewed By: CCB



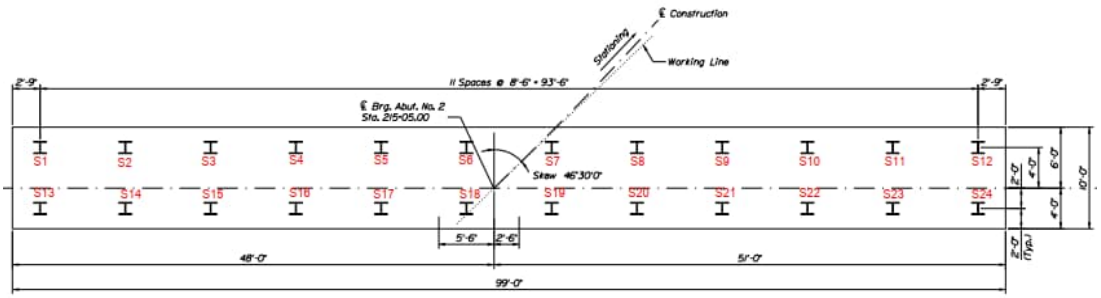
ABUTMENT NO. 2
FOOTING PLAN PILE LAYOUT

Table 8: Summary Abutment Pile Lengths for Bridge 6787 (Hogan Road Westbound)
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Pile	Bottom of Abutment Pile Cap Elevation (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Top of Grout Elevation (feet)	Approximate Bottom of Pile Elevation (feet)	Estimated Pile Length[1] (feet)	Minimum Rock Socket Length (feet)
Abutment 2						
S1	99	64.4	-	63.4	37	-
S2	99	64.4	-	63.4	37	-
S3	99	65.9	-	64.9	36	-
S4	99	65.9	-	64.9	36	-
S5	99	65.9	-	64.9	36	-
S6	99	64.7	-	63.7	37	-
S7	99	64.7	-	63.7	37	-
S8	99	64.7	-	63.7	37	-
S9	99	64	-	63	38	-
S10	99	64	-	63	38	-
S11	99	64	-	63	38	-
S12	99	67.9	-	66.9	34	-
S13	99	64.4	-	63.4	37	-
S14	99	64.4	-	63.4	37	-
S15	99	65.9	-	64.9	36	-
S16	99	65.9	-	64.9	36	-
S17	99	65.9	-	64.9	36	-
S18	99	64.7	-	63.7	37	-
S19	99	64.7	-	63.7	37	-
S20	99	64.7	-	63.7	37	-
S21	99	64	-	63	38	-
S22	99	64	-	63	38	-
S23	99	64	-	63	38	-
S24	99	67.9	-	66.9	34	-

Note: [1]Due to the weathered nature and poor RQD, the piles are assumed to penetrate 1-foot in to bedrock. The estimated pile length includes the assumed 1-foot penetration into bedrock.

Prepared By: ATM
Checked By: LDN
Reviewed By: CCB



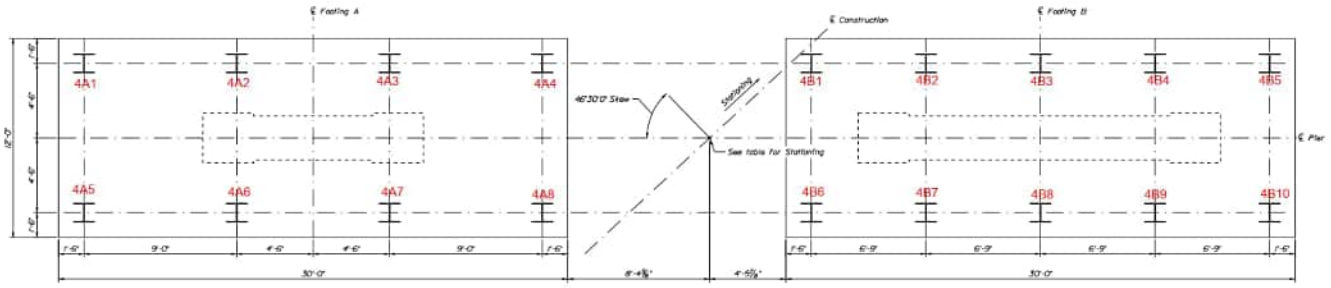
FOOTING PLAN & PILE LAYOUT

Table 9: Summary of Pier Pile Lengths for Bridge 6787 (Hogan Road Westbound)
Geotechnical Design Report
Hogan Road Bridge 5823 and Bridge 6787, Bangor, Maine
MaineDOT WIN 018595.11

Pile	Bottom of Abutment Pile Cap Elevation (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Top of Grout Elevation (feet)	Approximate Bottom of Pile Elevation (feet)	Estimated Pile Length[1] (feet)	Minimum Rock Socket Length (feet)
Pier 4A						
4A1	82	64.3	-	63.3	20	-
4A2	82	65.1	-	64.1	19	-
4A3	82	65.7	-	64.7	19	-
4A4	82	66.4	-	65.4	18	-
4A5	82	64.3	-	63.3	20	-
4A6	82	65.1	-	64.1	19	-
4A7	82	65.7	-	64.7	19	-
4A8	82	66.4	-	65.4	18	-
Pier 4B						
4B1	82	67.5	-	66.5	17	-
4B2	82	67.6	-	66.6	17	-
4B3	82	66.8	-	65.8	18	-
4B4	82	66	-	65	19	-
4B5	82	65.2	-	64.2	19	-
4B6	82	67.5	-	66.5	17	-
4B7	82	67.6	-	66.6	17	-
4B8	82	66.8	-	65.8	18	-
4B9	82	66	-	65	19	-
4B10	82	65.2	-	64.2	19	-

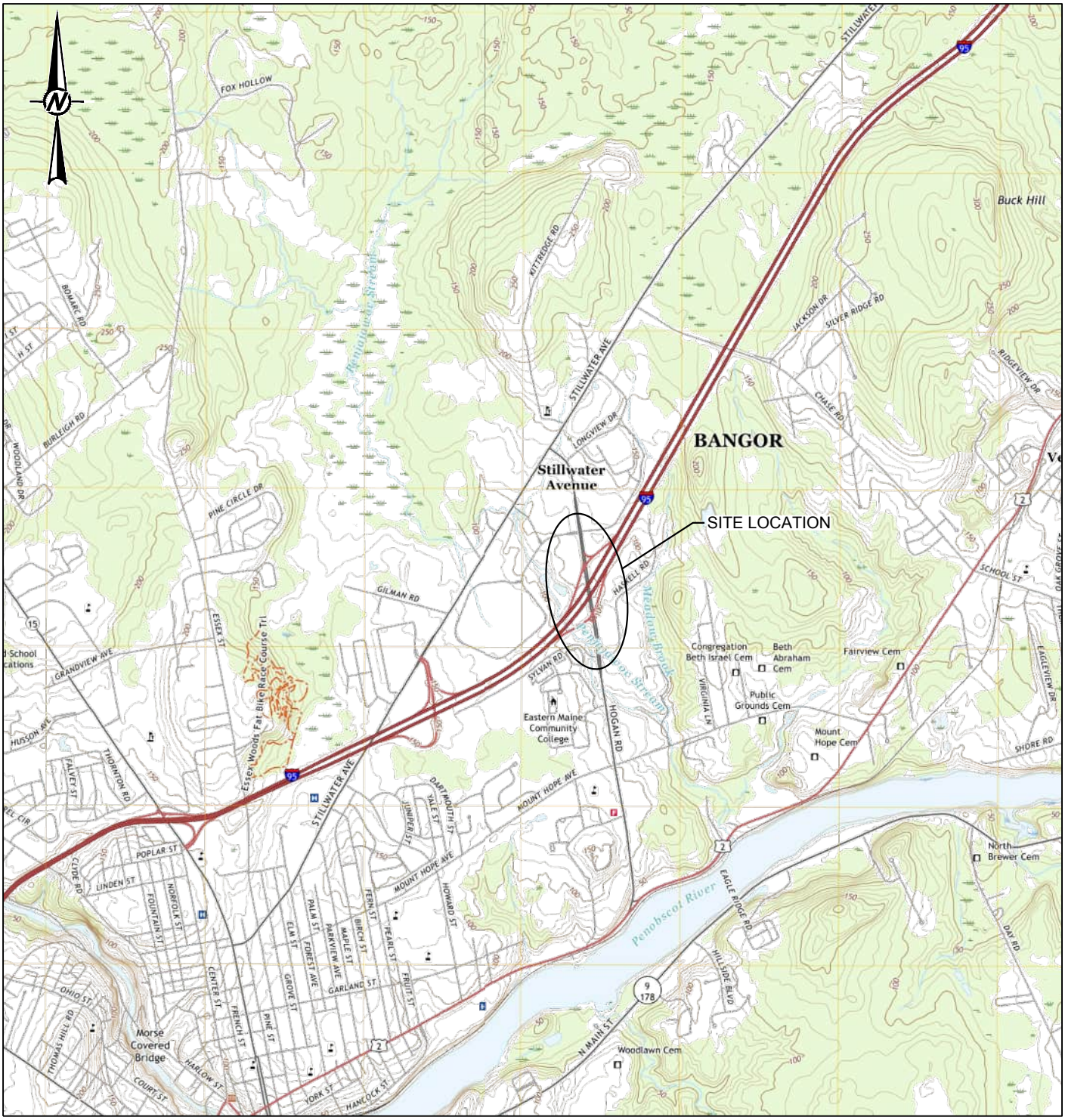
Note: [1]Due to the weathered nature and poor RQD, the piles are assumed to penetrate 1-foot in to bedrock. The estimated pile length includes the assumed 1-foot penetration into bedrock.

Prepared By: ATM
Checked By: LDN
Reviewed By: CCB



FOOTING PLAN & PILE LAYOUT

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
31404817.004	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_PRO\J31404817.003_hogan\rd0001_geotech\design\report\40_PRO\D\ | File Name: 31404817003-0001-001.dwg

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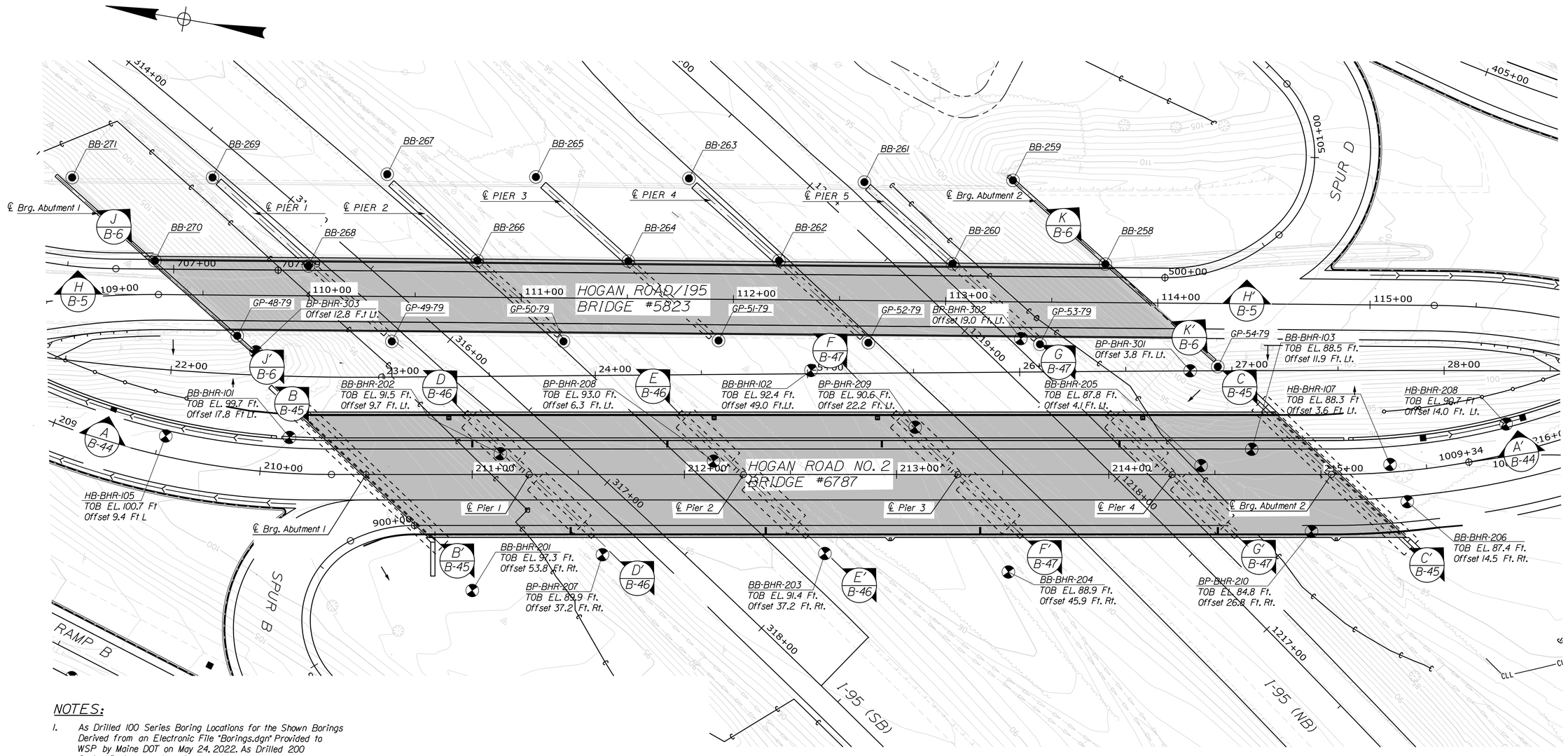


NOTES:

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 "18595.10BORLOC.200SERIES.10.11.23" Provided to WSP by Maine DOT 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 elements shown derived from Electronic file "Bridge-Proposed.dgn" Provided by VHB on August 27, 2025.
3. Proposed Stationing Provided to WSP by VHB on August 27, 2022 in an Electronic File "Alignments.dgn".
4. Approximate Historic Boring Locations or the Shown Borings Derived from Historic Plan Set "Department of Transportation Bureau of Highways, Bangor Penobscot County, Maine Federal an Urban Project No. IR-95-8(129) I75 Hogan Road/ I-95 Marked, As-Built 1983".
5. For Detailed Lithologic Descriptions see Boring Logs included in the Geotechnical Design Report (GDR).

BORING LOCATION PLAN

25 0 25 50
Scale of Feet

LEGEND

BB-BHR-101	Completed Bridge Series Borings
BP-BHR-207	Completed Bridge Series Rock Probe
HB-BHR-105	Completed Highway Series Borings
B-268, GP-48-79	Historic Borings
TOB EL. 99.7	Elevation of Top of Boring
X / X-XX	Interpretive Subsurface Section

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE NO. 6787

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	WEG	9/2022
CHECKED-REVIEWED	CCB	10/2025
DESIGN2-DETAILED2	-	-
DESIGN3-DETAILED3	-	-
REVISIONS 1	-	-
REVISIONS 2	-	-
REVISIONS 3	-	-
REVISIONS 4	-	-
FIELD CHANGES	-	-

SIGNATURE

P.E. NUMBER

DATE

HOGAN ROAD/NO. 2 BRIDGE
195
BANGOR PENOBSCOT

BORING LOCATION PLAN

SHEET NUMBER
2
2 OF 8

Filename: ...\MST\A\BorLocPlan_5Span_wb.dgn Username: Division: BRIDGE Date:11/7/2025

Filename: ...\MSTA\GeoProfile_5Span_wb.dgn Username: Date:11/7/2025 Division: BRIDGE

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+52.00
℄ PIER 1 Sta. 211+27.00
℄ PIER 2 Sta. 212+28.00
℄ PIER 3 Sta. 213+29.00
℄ PIER 4 Sta. 214+30.00
℄ Brg. Abutment 2 Sta. 215+05.00

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.
BP-BHR-207 EL. 89.9 Ft. Offset 37.2 Ft. Rt.
BP-BHR-208 EL. 93.0 Ft. Offset 6.3 Ft. Lt.
BB-BHR-102 EL. 92.4 Ft. Offsett 49.0 Ft. Lt.
BP-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.
HB-BHR-107 EL. 88.3 Ft. Offsett 3.6 Ft. Lt.
BB-BHR-202 EL. 91.5 Ft. Offset 9.7 Ft. Lt.
BB-BHR-203 EL. 91.4 Ft. Offsett 37.2 Ft. Rt.
BB-BHR-103 EL. 88.5 Ft. Offsett 11.9 Ft. Lt.
BB-BHR-206 EL. 87.4 Ft. Offsett 14.5 Ft. Rt.
BP-BHR-210 EL. 84.8 Ft. Offsett 26.8 Ft. Rt.

Aggregate Subbase
Existing Grade
Granular Borrow
Silt
Fill
I-95 SB
I-95 NB
Stiff to Very Stiff Clay
Medium Stiff to Soft Clay
Glacial Till
Weathered Bedrock
Medium Stiff to Soft Clay
Stiff to Very Stiff Clay
Medium Stiff to Soft Clay
Glacial Till
Weathered Bedrock
Metawacke

VI-1161/357 V2-1116/223
VI-1339/446
VI > 1217
VI - 659/220 V2 - 1016/302
VI - 989/110 V2 > 1217

Run 1 = 52% Run 2 = 20% EOB 29' BGS
Run 1 = 39% Run 2 = 0% Run 3 = 49% Run 4 = 46% EOB 34.9' BGS
Run 1 = 0% Run 2 = 0% Run 3 = 0% Run 4 = 21% EOB 35.0' BGS
Run 1 = 34% Run 2 = 56% EOB 31.2' BGS
Run 1 = 48% EOB 26.7' BGS
Run 1 = 29% Run 2 = 39% Run 3 = 76% EOB 19.2' BGS
Run 1 = 37% Run 2 = 10% Run 3 = 28% Run 4 = 64% Run 5 = 28% Run 6 = 64% EOB 26.7' BGS
Run 1 = 10% EOB 29.7' BGS
Run 1 = 64% Run 2 = 25% Run 3 = 30% Run 4 = 0% EOB 36.2' BGS
R Run 1 = 30% RUN 2 = 75% RUN 3 = 41% EOB 30.0' BGS
Run 1 = 0% Run 2 = 0% Run 3 = 13% Run 4 = 17% EOB 39.6' BGS
Run 1 = 36% Run 2 = 0% Run 3 = 20% Run 4 = 46% Run 5 = 64% Run 6 = 19% EOB 38.0' BGS
Run 1 = 13% EOB 23.5' BGS
R Run 1 = 42% EOB 28.8' BGS
Run 1 = 9% Run 2 = 27% Run 3 = 0% Run 4 = 0% Run 5 = 0% Run 6 = 0% EOB 30.8' BGS

SECTION A-A'

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

PROFILE 28

Scale of Feet

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)

Topsoil
Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)
Olive-Brown, Medimum Stiff to Very Stiff Silt, Trace Fine Sand and Gravel, Slightly Plastic (Silt)
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

GENERAL NOTES:

1. As-drilled boring locations for the shown borings derived from electronic file "Hogan Road borings.dgn" provided to WSP by MaineDOT on October 13, 2023.
2. For detailed lithologic decriptions see Boring Logs included in the Geotechnical Desiogn Report (DGR).
3. Groundwater surface is interpreted from localized surface water levels and measurements taken during the subsurface exploration programs.
4. This generalized subsurface section is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed based on interpretations of widely spaced explorations actual soil and rock transitions may vary amd are probably more erratic.
5. Proposed abutment geometry interpreted from electronic file "xsect_bridge_abutments_piers.dgn" provided to WSP by VHB on August 22, 2025.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE PLANS

BRIDGE No. 6787

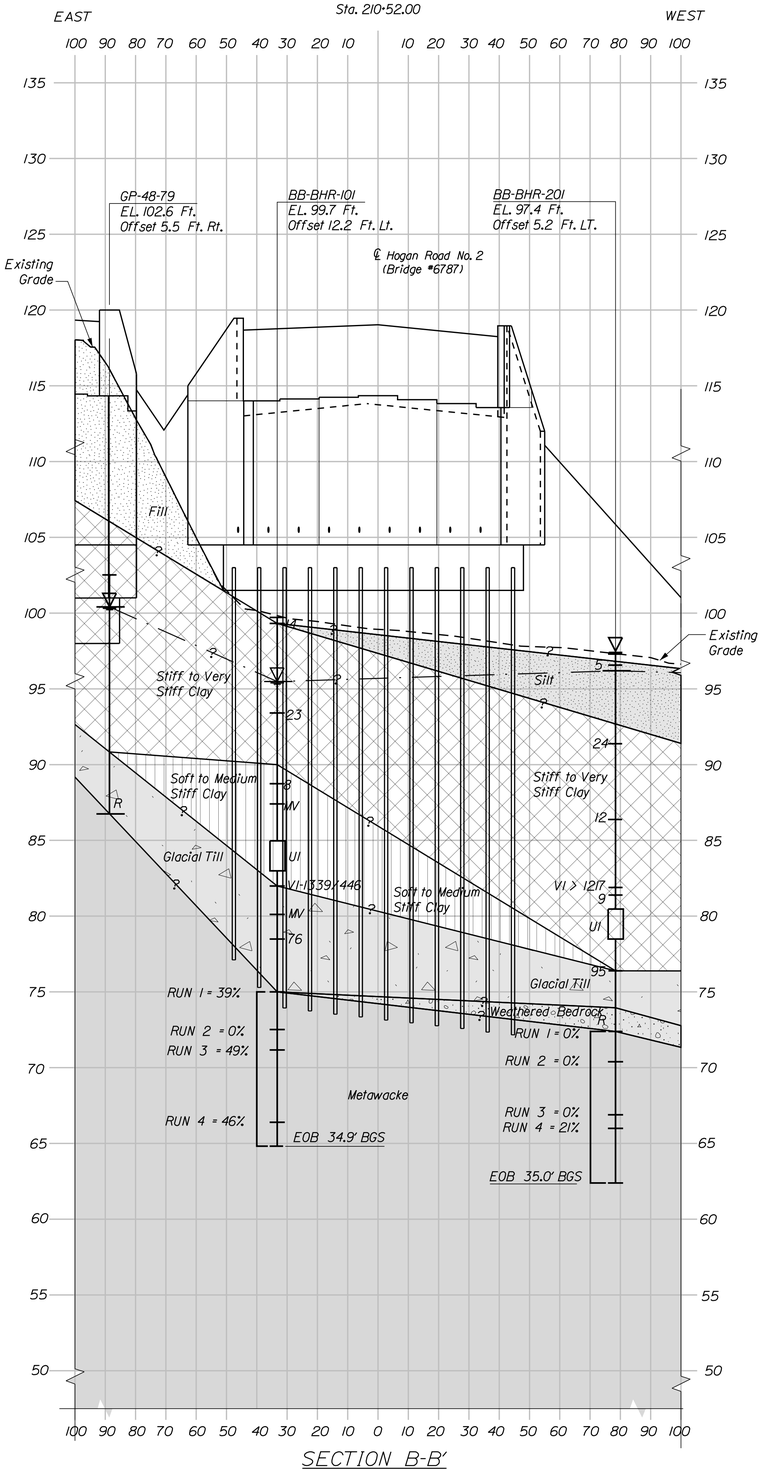
PROJ. MANAGER	M. LANDON		
DESIGN-DETAILED	NVW	5/2024	
CHECKED-REVIEWED	WEG	10/2025	
DESIGN2-DETAILED2	DEB	CCB	
DESIGN3-DETAILED3			
REVISIONS 1	XXX	XXX	
REVISIONS 2	XXX	XXX	
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

BY | P.E. NUMBER | SIGNATURE | DATE

HOGAN ROAD/NO. 2 BRIDGE
195
BANGOR PENOBSCOT

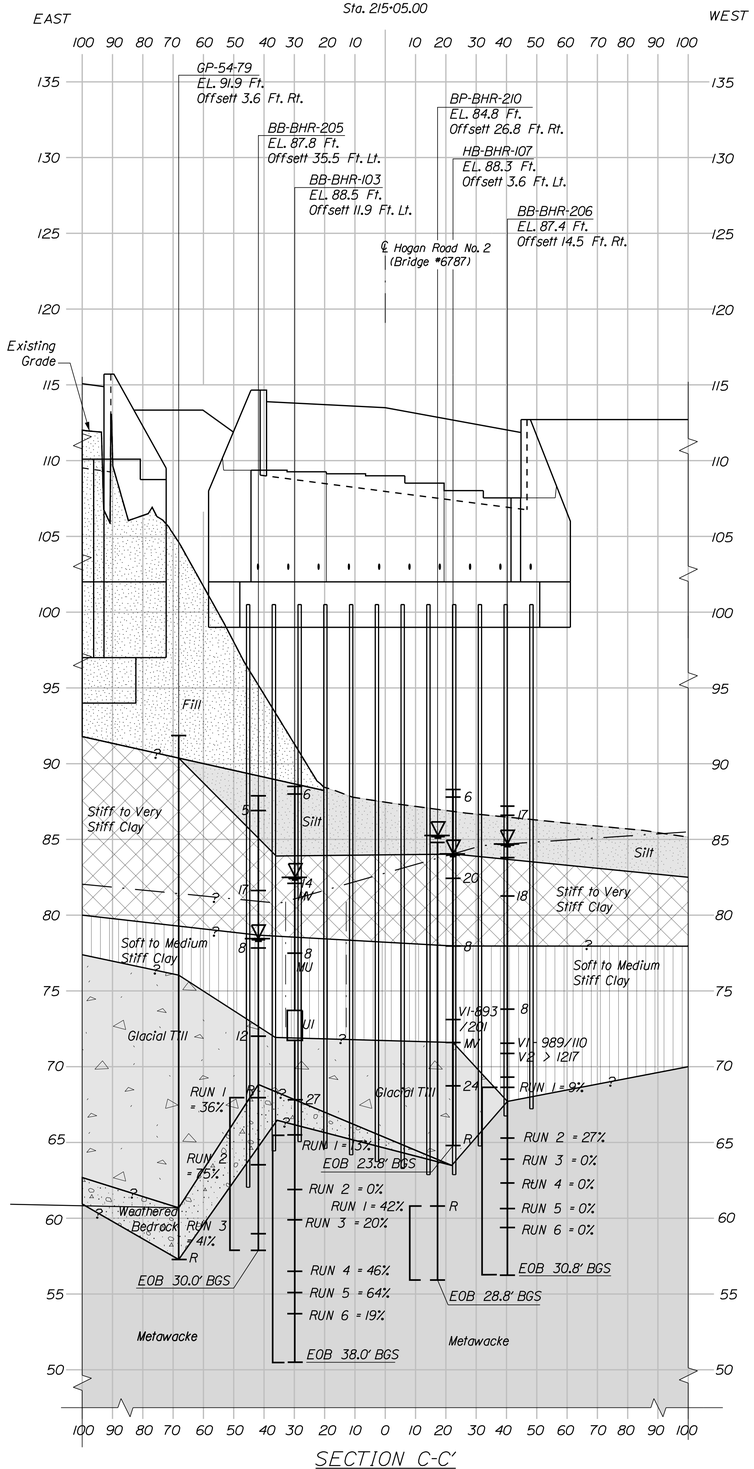
INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 1 OF 4)

SHEET NUMBER
3
3 OF 8



SECTION B-B'

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE



SECTION C-C'

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

- Topsoil
- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)
- Olive-Brown, Medium Stiff to Very Stiff Silt, Trace Fine Sand and Gravel, Slightly Plastic (Silt)
- Grey-Brown, Medium Stiff to Soft, Low Plasticity, Silt and Clay, Trace Sand (Clay)
- Silt to Very Stiff (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
- Boring Location I.D.
  Elevation
  Offset (Ft.) From Section Line
- Top of Boring Probe
- 30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
- VI — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
- MV — Missed Vane
- 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)

NOTE:

1. See Sheets B-43 and B-44 for General Notes.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE PLANS

BRIDGE No. 6787

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

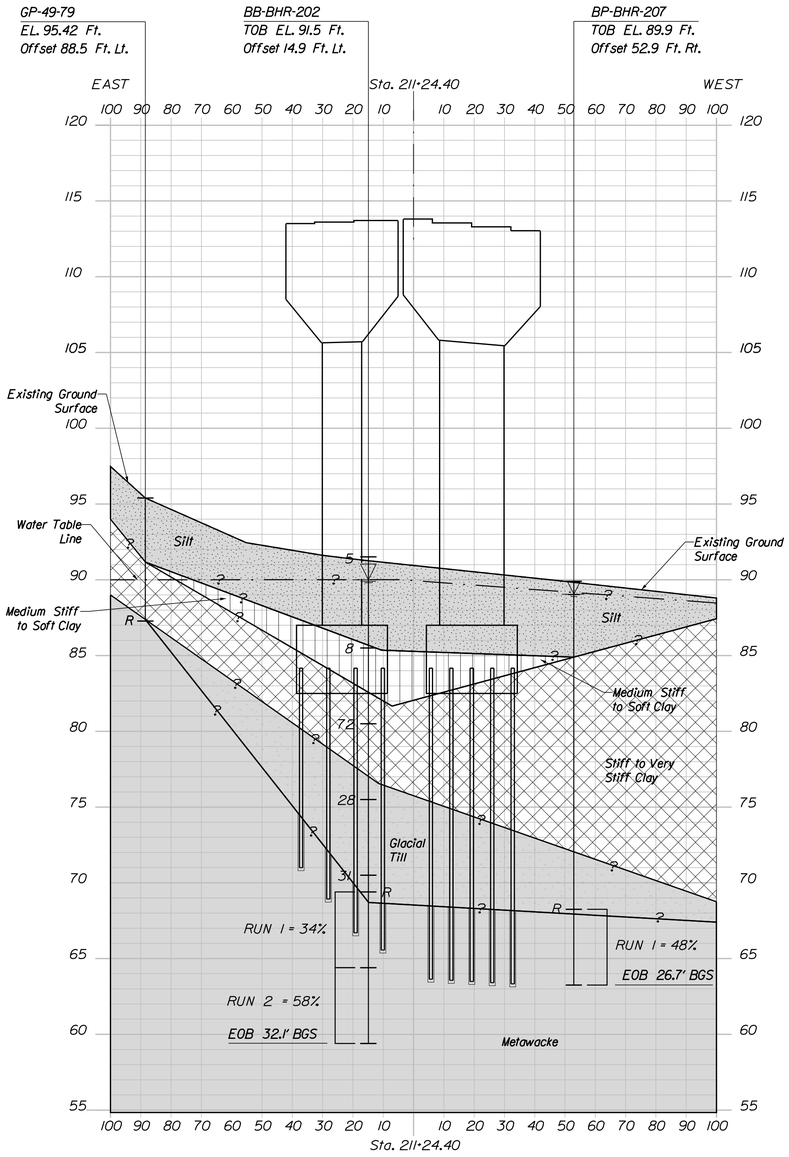
BY | DATE | SIGNATURE | P.E. NUMBER | DATE

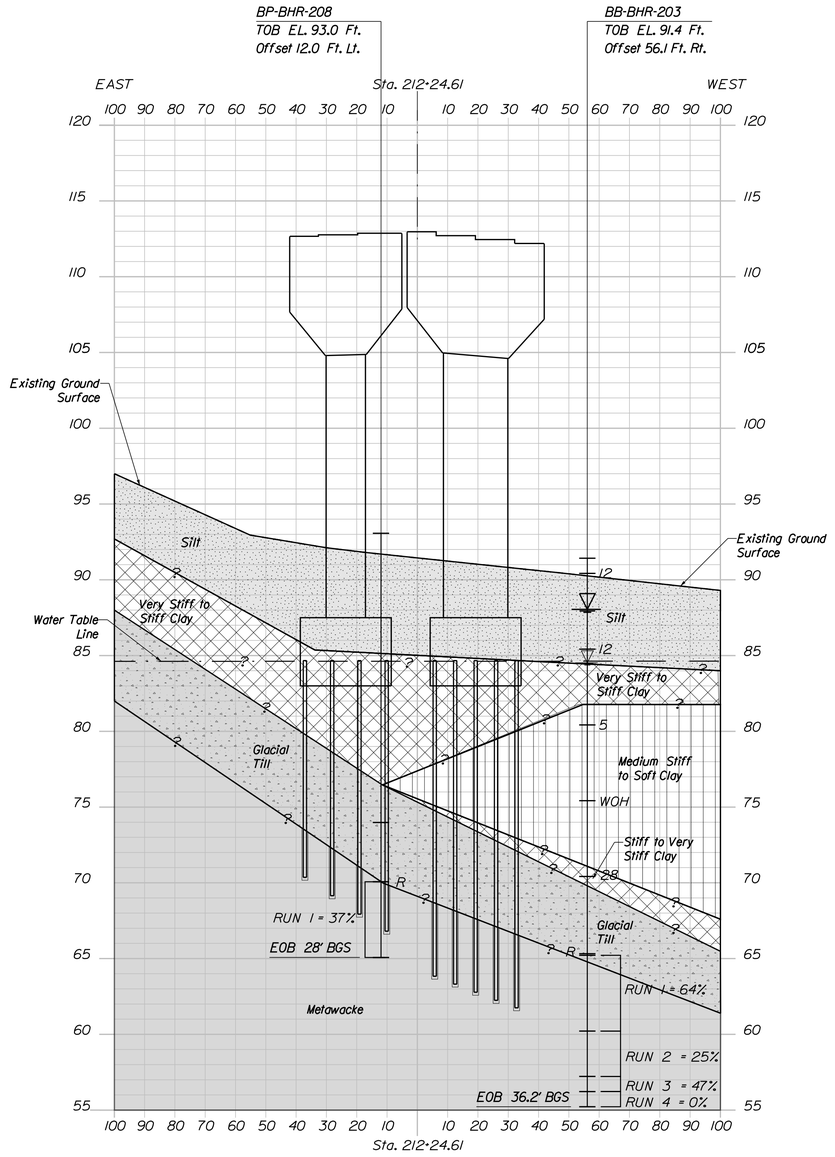
HOGAN ROAD/NO. 2 BRIDGE
195
PENOBSCOT
BANGOR

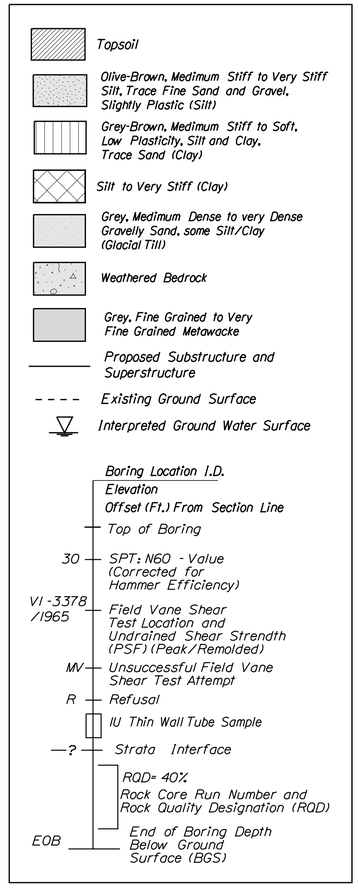
INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 2 OF 4)

SHEET NUMBER
4
4 OF 8

Filename: ...\MSTA\Geoxsect_01_5Span_wb.dgn Division: BRIDGE Username: Date:11/7/2025





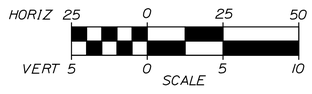


NOTE:

1. See Sheets B-43 and B-44 for General Notes.

SECTION D-D'
(PIER 1)

SECTION E-E'
(PIER 2)



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE No. 6787

BRIDGE PLANS

P.E. NUMBER

DATE

SIGNATURE

	M. LANDON	BY	DATE
DESIGN-DETAILED	ATM	DJH	9/2025
CHECKED-REVIEWED	DEB	CCB	10/2025
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

PROJ. MANAGER

HOGAN ROAD/NO. 2 BRIDGE
195
PENOBSCOT
BANGOR

INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 3 OF 4)

SHEET NUMBER

5

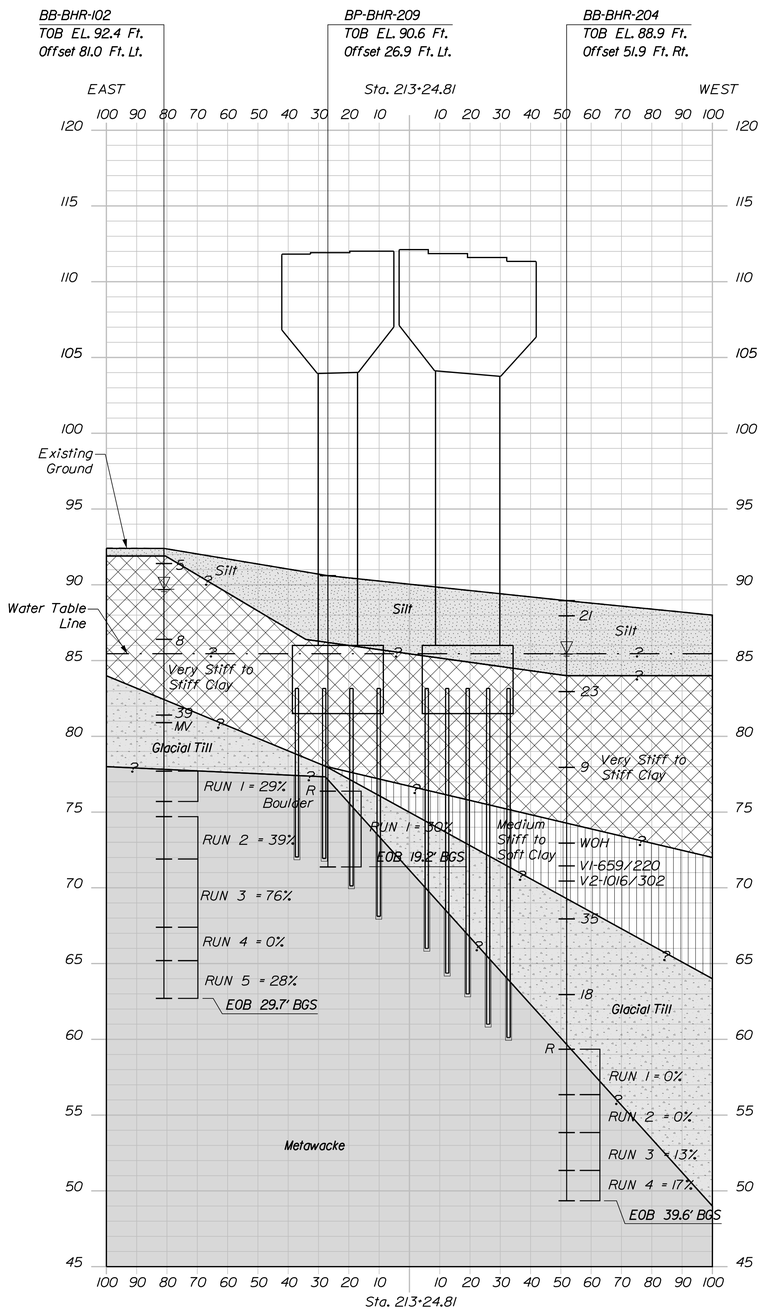
5 OF 8

Date:11/7/2025

Username:

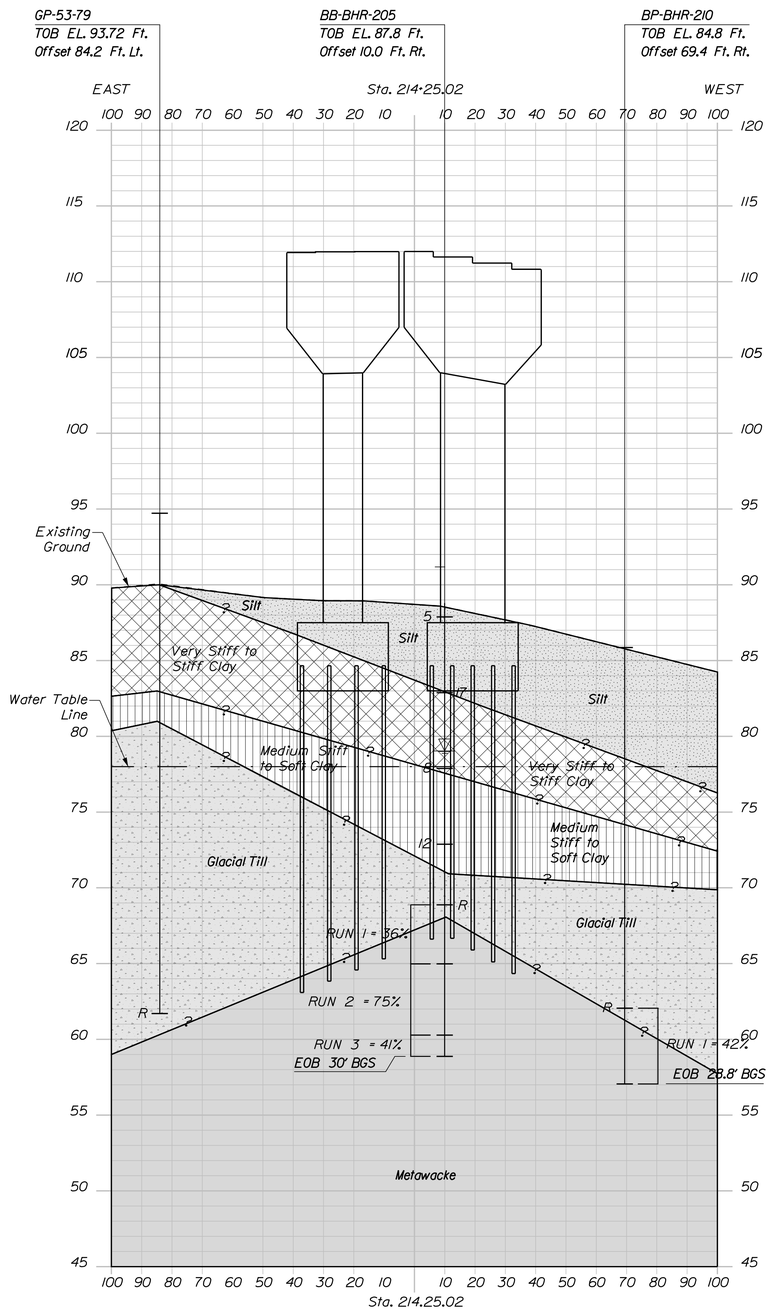
Division: BRIDGE

Filename: ...\MSTA\Geoxsect_WB_Pier1-2.dgn



SECTION F-F'
(PIER 3)

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE



SECTION G-G'
(PIER 4)

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

- Topsoil
- Olive-Brown, Medimum Stiff to Very Stiff Silt, Trace Fine Sand and Gravel, Slightly Plastic (Silt)
- Grey-Brown, Medimum Stiff to Soft, Low Plasticity, Silt and Clay, Trace Sand (Clay)
- Medium Stiff to Soft 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

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

- Top of Boring Probe
- 30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
- VI — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
- MV — Missed Vane
- 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)

NOTE:

1. See Sheets B-43 and B-44 for General Notes.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE PLANS

BRIDGE No. 6787

	BY	DATE
PROJ. MANAGER	M. LANDON	
DESIGN-DETAILED	DJH	9/2025
CHECKED-REVIEWED	CCB	10/2025
DESIGN2-DETAILED2	ATM	
DESIGN3-DETAILED3	DEB	
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

HOGAN ROAD/NO. 2 BRIDGE
195
PENOBSCOT
BANGOR

INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 4 OF 4)

SHEET NUMBER
6
5 OF 8

Filename: ...\MSTA\Geoxsect_WB_Pier3-4.dgn Username: Division: BRIDGE Date:11/7/2025

D NORTH

D' SOUTH

108+00 109+00 110+00 111+00 112+00 113+00 114+00 115+00

℄ Brg. Abutment 1 Sta. 109+43.83

℄ PIER 1 Sta. 110+16.90

℄ PIER 2 Sta. 110+99.00

℄ PIER 3 Sta. 111+70.00

℄ PIER 4 Sta. 112+41.20

℄ PIER 5 Sta. 113+22.90

℄ Brg. Abutment 2 Sta. 113+91.73

BB-270 El 104.2 Ft. Offset 16.0 Ft. Lt.

GP 48-79 El. 102.6 Ft. Offset 19.1 Ft. Rt.

BB-268 El. 103.6 Ft. Offset 14.0 Ft. Lt.

GP 49-79 El 94.4 Ft. Offset 21.1 Ft. Rt.

BB-266 El. 102.4 Ft. Offset 17.1 Ft. Lt.

GP 50-79 El. 96.76 Ft. Offset 20.4 Ft. Rt.

BB-264 El. 100.9 Ft. Offset 17.5 Ft. Lt.

GP 51-79 El. 95.5Ft. Offset 19.4 Ft. Rt.

BB-262 El. 98.8 Ft. Offset 18.5 Ft. Lt.

GP 52-79 El. 94.8 Ft. Offset 19.8 Ft. Rt.

BB-260 El. 97.5Ft. Offset 18.5 Ft. Lt.

GP 53-79 El. 93.7 Ft. Offset 19.8 Ft. Rt.

BB-258 El. 95.8 Ft. Offset 18.5 Ft. Lt.

GP 54-79 El 91.9 Ft. Offset 30.0 Ft. Rt.

Fill

Stiff to Very Stiff Clay

Glacial Till

I-95 S

I-95 Southbound

Fill

Stiff to Very Stiff Clay

Glacial Till

Stiff to Very Stiff Clay

Stiff to Very Stiff Clay

Glacial Till

Metawacke

I-95 NB

I-95 Northbound

Fill

Stiff to Very Stiff Clay

Fill

Stiff to Very Stiff Clay

Stiff to Very Stiff Clay

Glacial Till

Glacial Till

Weathered Rock

Metawacke

Topsoil

Asphalt

Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)

Medimum to Stiff, Gray to Brown Mottled and Weathered Silty Clay with Sandy Silt Lines and Lenses (Clay)

Loose to Dense, Grey to Brown, Silty Fine Sand, Angular Rocks and Gravel (Glacial Till)

Fractured Rock, Weathered Rock

Grey, Fine Grained to very Fine Grained Metawacke

Proposed Bridge Substructure and Superstructure

Existing Ground Surface

Boring Location I.D.
Elevation
Offset (Ft.) From Section Line
Top of Boring Probe
? Strata Interface
R End of Boring

GENERAL NOTES:

1. As-drilled boring locations for the shown borings derived from electronic file "Hogan Road borings.dgn" provided to WSP by MaineDOT on October 13, 2023.
2. For detailed lithologic decriptions see Boring Logs included in the Geotechnical Desiogn Report (DGR).
3. Groundwater surface is interpreted from localized surface water levels and measurements taken during the subsurface exploration programs.
4. This generalized subsurface section is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed based on interpretations of widely spaced explorations actual soil and rock transitions may vary amd are probably more erratic.
5. Proposed abutment geometry interpreted from electronic file "xsect_bridge_abutments_piers.dgn" provided to WSP by VHB on August 22, 2025.

SECTION H-H'

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

140 135 130 125 120 115 110 105 100 95 90 85 80 75 70 65 60 55 50 45

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

018595.11

WIN 018595.11

BRIDGE No. 5823

BRIDGE PLANS

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

PROJ. MANAGER

SIGNATURE

P.E. NUMBER

DATE

HOGAN ROAD/I95 BRIDGE
I95
PENOBSCOT
BANGOR

INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 1 OF 2)

SHEET NUMBER

7

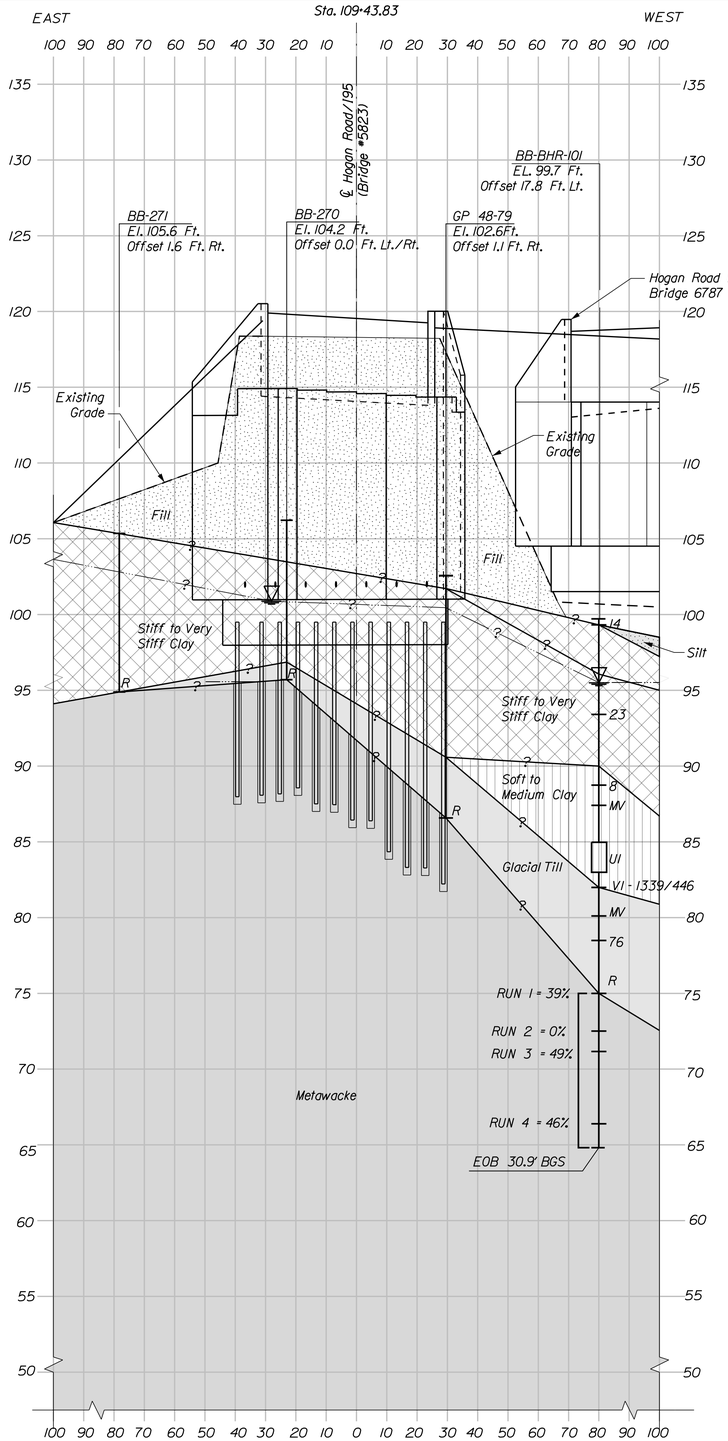
7 OF 8

Username:

Date:11/7/2025

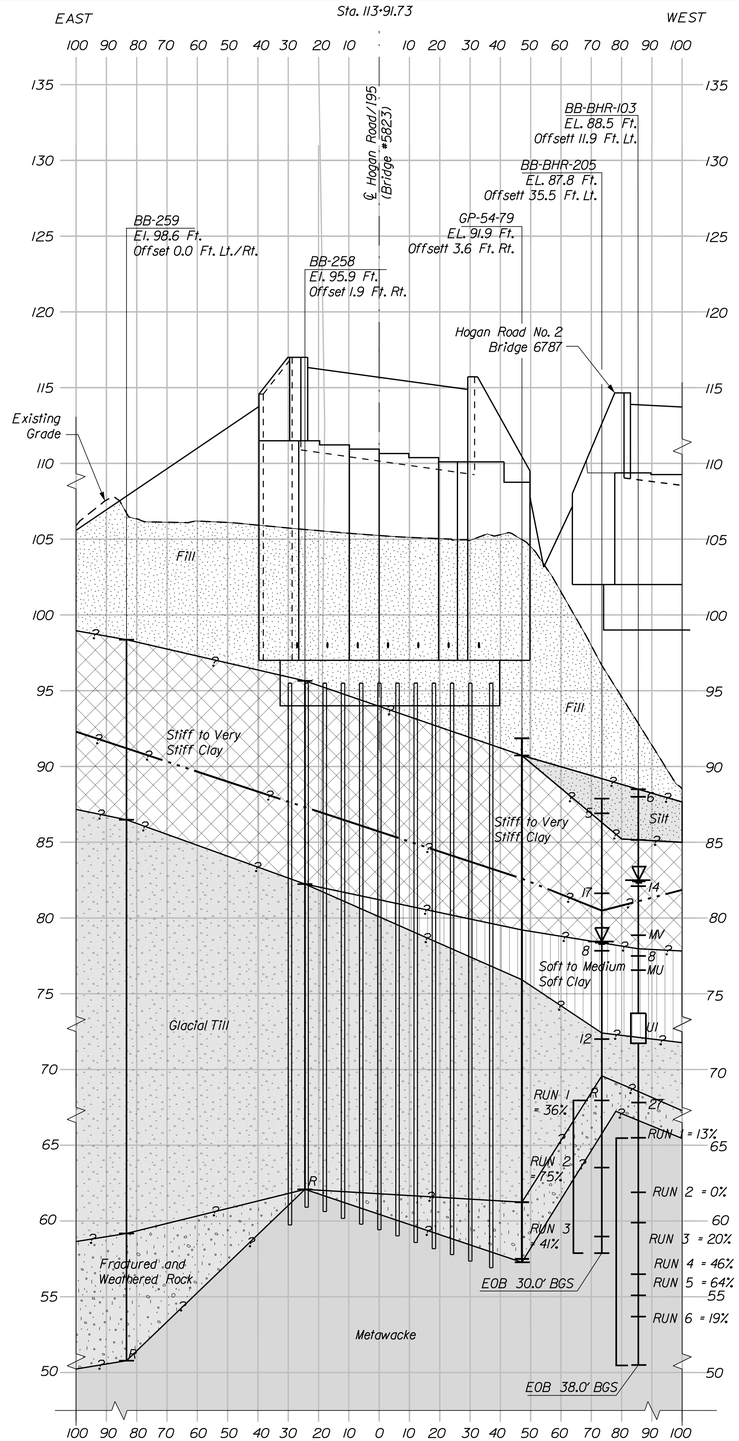
Division: BRIDGE

Filename: ...\MSTA\GeoProfile_5Span_eb.dgn



SECTION J-J'

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE



SECTION K-K'

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

- Grey, Medium Dense, Gravelly Fine to Coarse Sand, some Silt (Fill)
- Olive-Brown, Medimum Stiff to Very Stiff Silt, Trace Fine Sand and Gravel, Slightly Plastic (Silt)
- Medimum to Soft, Grey to Brown Mottled and Weathered Silty Clay with Sandy Silt Lines and Lenses (Clay)
- Stiff, Grey to Brown Mottled and Weathered Silty Clay with Sandy Silt Lines and Lenses (Clay)
- Fractured and Weathered Rock
- Loose to Dense, Grey to Brown Silty Fine Sand, Angular Rocks and Gravel (Glacial Till)
- Grey, Fine Grained to Very Fine Grained Metawacke
- Proposed Bridge Substructure and Superstructure
- Existing Ground Surface
- Interpreted Ground Water Surface
- Boring Location I.D.
  Elevation
  Offset (Ft.) From Section Line
- Top of Boring Probe
- —?— Strata Interface
- 30 — SPT: N60 - Value (Corrected for Hammer Efficiency)
- VI — Field Vane Shear Test Location and Undrained Shear Strength (PSF) (Peak/Remolded)
- UI Thin Wall Tube Sample
- MV — Missed Vane
- R — End of Boring Depth Below Ground Surface (BGS)

NOTE:

1. See Sheets B-4 and B-5 for General 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	NVW	5/2024
CHECKED-REVIEWED	WEG	
DESIGN2-DETAILED2	CCB	10/2025
DESIGN3-DETAILED3	DEB	
REVISIONS 1	XXX	
REVISIONS 2	XXX	
REVISIONS 3	XXX	
REVISIONS 4	XXX	
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

HOGAN ROAD/195 BRIDGE
195
PENOBSCOT
BANGOR

INTERPRETIVE SUBSURFACE SECTIONS
(SHEET 2 OF 2)

SHEET NUMBER
8
8 OF 8

Filename: ...\MSTA\Geoxsect_01_5Span_eb.dgn Division: BRIDGE Username: Date:11/7/2025

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

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	99.7	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (07:10); 4/29/22 (10:30)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 486077.832, E: 1739764.759	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	4.1 ft bgs on 4/29/22 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	24/13	0.00 - 2.00	1-2-7-9	9	14	OPEN	99.3		Brown, moist, stiff, SILT, some fine to medium sand, trace gravel, trace grass roots in top 5 inches (TOPSOIL).	
										0.4	
5	2D	24/24	5.00 - 7.00	4-6-9-8	15	23	31			Grey-brown, moist, very stiff, Silty CLAY, trace fine sand (CLAY). q_p = 5.7 ksf (Pocket Penetrometer), T_v = 409 psf	
							29				
							43				
							45				
							48				
10	3D	24/24	10.00 - 12.00	2-2-3-3	5	8	35	89.7		10.0	WC = 33.0% Fines = 99.3% LL = 39 PL = 23 PI = 16 LI = 0.6 A-6 (18), CL
							39			Grey-brown, wet, medium stiff, Silty CLAY, trace fine sand, moderately plastic (CLAY). q_p = 4.0 ksf (Pocket Penetrometer), T_v = 655 psf	
							44				
	MV			Would Not Push			48			Failed 55x110 mm vane, would not push to 13 ft bgs. Driller notes wash water color change to grey at 13.5 ft bgs.	
							50				
15	1U	24/24	15.00 - 17.00	Piston Sampler			38			Grey-brown, wet, Silty CLAY, moderately plastic (CLAY).	WC = 33.0% Fines = 99.7% LL = 31 PL = 18 PI = 13 LI = 0.9 A-6 (13), CL
							41				
	4D	24/12	17.00 - 19.00	WOH(12")-10-12	-		43	82.7		17.0	
	V1			S_u=1339/446 psf			64	81.7		Grey, wet, stiff, Silty CLAY, trace gravel in tip of spoon (CLAY). 18.0	
							72			55x110 mm vane raw torque readings: V1: 30/10 ft-lbs MV: Failed vane, would not push to 19 ft bgs.	
20	5D	24/10	20.00 - 22.00	7-13-37-33	50	76	OPEN			Grey, wet, very dense, GRAVEL, some fine to coarse sand, some silt (GLACIAL TILL).	WC = 10.2% Fines = 23.0% A-1-b (0)
25	R1	33.6/30	24.90 - 27.70	RQD = 39%			NQ2	75.2		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 Geosciences 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 test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made 5 minutes after completion of drilling with casing in the ground. 5. Station 210+11, offset 17.8 Ft. Lt.

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	99.7	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (07:10); 4/29/22 (10:30)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 486077.832, E: 1739764.759	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	4.1 ft bgs on 4/29/22 at 10:35
Hammer Efficiency Factor: 0.908		**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
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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										Top of Bedrock at Elev. 75.2 ft. Advanced with roller bit to 24.9 ft bgs.	
										R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to steeply dipping, very closely to closely spaced, open; average 2.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 89% Recovery Rock Core Rate (min:sec) 24.9-25.9 ft (3:42) 25.9-26.9 ft (3:58) 26.9-27.7 ft (3:12)	
	R2	12/10.8	27.70 - 28.70	RQD = 0%							
	R3	60/56.4	28.70 - 33.70	RQD = 49%							
30											
	R4	14.4/14.4	33.70 - 34.90	RQD = 46%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, moderately weathered; discontinuities moderately to steeply dipping, open; average 5.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 90% Recovery Rock Core Rate (min:sec) 27.7-28.7 ft (3:04)	
35								64.8			
										R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steeply dipping, very closely to moderately closely spaced, open; average 2.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 94% Recovery Rock Core Rate (min:sec) 28.7-29.7 ft (2:54) 29.7-30.7 ft (3:12) 30.7-31.7 ft (3:48) 31.7-32.7 ft (3:42) 32.7-33.7 ft (2:59)	
40											
										R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities steeply dipping, closely spaced, open, thin (0.25 in) milky quartz and calcite vein presenet at 34.9 bgs; average 1.7 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 33.7-34.9 ft (2:41)	
45										34.9 **Bottom of Exploration at 34.9 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
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 Geosciences 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 test before sampling; blow counts not representative of undisturbed material. 4. Water level reading was made 5 minutes after completion of drilling with casing in the ground. 5. Station 210+11, offset 17.8 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	92.41	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (11:50); 4/27/22 (13:50)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485839.891, E: 1739836.204	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.7 ft bgs on 4/27/22 at 13:55
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
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	0.00 - 2.00	1-1-2-4	3	5	OPEN	91.8		1DA, Top 7 in: Brown, moist, medium stiff, SILT, some fine to coarse sand, trace gravel (TOPSOIL).	
										0.6	
										1DB, Bottom 12 in: Grey-brown, moist, medium stiff, Silty CLAY, trace fine to medium sand (CLAY). q_p = 8.2 ksf (Pocket Penetrometer), T_v = 716 psf	
5	2D	24/24	5.00 - 7.00	3-2-3-3	5	8	12			Grey-brown, moist, medium stiff, Silty CLAY, trace fine sand, moderately plastic (CLAY). q_p = 1.8 ksf, T_v = 593 psf	WC = 32.5% LL = 36 PL = 22 PI = 14 LI = 0.7 A-6 (18), CL
							21				
							26				
							31				
							44				
10	3D	24/17	10.00 - 12.00	11-14-12-11	26	39	22	82.1		3DA, Top 4 in: Grey, wet, hard, Silty CLAY (CLAY).	
										10.3	
	MV			Would Not Push			25			3DB, Bottom 13 in: Grey, wet, dense, GRAVEL, some silt, some fine to coarse sand, trace clay (GLACIAL TILL). Failed 55x110 mm vane, would not push to 11 ft bgs.	WC = 10.9% Fines = 29.9% A-2-4 (0)
							41				
							54				
	R1	36/19.2	14.70 - 17.70	RQD = 29%			104 NQ2	77.9		14.5	
15										Casing refusal at 14.5 ft bgs. Advanced with roller bit to 14.7 ft bgs. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderately dipping, closely spaced, open, milky quartz and calcite veins (0.1 to 0.2 in thick) present throughout; average 1.3 fractures per foot [BOULDER].	
								75.8			
	R2	33.6/33.6	17.70 - 20.50	RQD = 39%				74.7		Rock Mass Quality = Poor 53% Recovery Rock Core Rate (min:sec) 14.7-15.7 ft (1:56) 15.7-16.7 ft (2:10) 16.7-17.7 ft (1:43)	
										16.6	
20	R3	54/51.6	20.50 - 25.00	RQD = 76%						Driller notes fast advancement of core barrel from 16.6 ft bgs to 17.7 ft bgs through possible zone of fractured bedrock or gravel seam. No recovery of this material present upon retrieval of core sample	
										17.7	
										Top of Bedrock at Elev. 74.7 ft. R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, slightly weathered; discontinuities low angle to steeply dipping, very closely to closely spaced, open with grey sand and silt infilling, milky quartz and calcite veins (0.1 to 0.5 in	
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 Geosciences 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 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 14.5 ft bgs. 5. Station 212+61, offset 49.0 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	92.41	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (11:50); 4/27/22 (13:50)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485839.891, E: 1739836.204	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.7 ft bgs on 4/27/22 at 13:55
Hammer Efficiency Factor: 0.908		**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 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	R4	26.4/26.4	25.00 - 27.20	RQD = 0%						thick) present at 18.5 ft bgs to 19.2 ft bgs and 20.2 ft bsg to 20.5 ft bgs; average 2.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 17.7-18.7 ft (2:13) 18.7-19.7 ft (2:38) 19.7-20.5 ft (3:10)	
	R5	30/30	27.20 - 29.70	RQD = 28%							
30								62.7		R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, moderately weathered (24.5-25 ft bgs) to fresh; discontinuities low angle to moderately dipping, very closely to moderately closely spaced, open, milky quartz and calcite vein (0.25 in thick) present at 34.9 bgs; average 0.9 fractures per foot; quartz veins 22.5-22.7 ft bgs and 23.6-23.8 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Good 96% Recovery Rock Core Rate (min:sec) 20.5-21.5 ft (2:42) 21.5-22.5 (2:36) 22.5-23.5 (2:20) 23.5-24.5 (3:14) 24.5-25.0 (2:48)	
35											
40										R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, moderately weathered; discontinuities low angle to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (1 in to 1.5 in thick) present at 26 ft bgs and 26.3 bgs; broken core [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 25-26 ft (3:16) 26-27 ft (3:34) 27-27.2 ft (2:10)	
45										R5: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, slightly weathered; discontinuities low angle to vertical, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 29 ft bgs and 29.7 bgs; average 3.6 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 27.2-28.2 ft (3:12) 28.2-29.2 ft (2:18) 29.2-29.7 ft (1:42)	
50										29.7	

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 Geosciences 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 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 14.5 ft bgs. 5. Station 212+61, offset 49.0 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	92.41	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (11:50); 4/27/22 (13:50)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485839.891, E: 1739836.204	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.7 ft bgs on 4/27/22 at 13:55
Hammer Efficiency Factor:	0.908	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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										**Bottom of Exploration at 29.7 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 Geosciences 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 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 14.5 ft bgs. 5. Station 212+61, offset 49.0 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	88.48	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (09:15); 4/26/22 (12:20)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485628.925, E: 1739834.412	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.8 ft bgs on 4/26/22 at 10:25
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	24/19	0.00 - 2.00	1-1-3-5	4	6	OPEN	88.0		Grey-brown, moist, medium stiff, SILT, some fine to medium sand, trace roots in top 6 inches (TOPSOIL).	
										0.5	
5	2D	24/24	5.00 - 7.00	4-4-5-6	9	14	5			Grey, moist, stiff, Silty CLAY, moderately plastic (CLAY). q_p = 4.0 ksf (Pocket Penetrometer), T_v = 716 psf	WC = 27.4% Fines = 99.5% LL = 37 PL = 21 PI = 16 LI = 0.4 A-6 (17), CL
	MV			Would Not Push			10			Failed 55x110 mm vane, would not push to 6 ft bgs.	
							19				
							31				
	MV			Would Not Push			33				
10	3D	24/24	10.00 - 12.00	2-2-3-3	5	8	36	78.5		Failed 55x110 mm vane, would not push to 10 ft bgs.	
							39			10.0 Grey, wet, medium stiff, Silty CLAY, moderately plastic (CLAY). q_p = 3.0 ksf (Pocket Penetrometer), T_v = 716 psf	WC = 29.8% Fines = 99.6% LL = 36 PL = 21 PI = 15 LI = 0.6 A-6 (16), CL
	MU	0/0	12.00 - 12.00	Piston Sampler			44			Failed Shelby tube.	
							40				
							45				
15	1U	14.4/9.6	15.00 - 16.20	Piston Sampler			43			Grey, wet, Silty CLAY, moderately plastic (CLAY).	WC = 22.0% Fines = 92.6% LL = 27 PL = 15 PI = 12 LI = 1.3 A-6 (9), CL
							71	72.3		16.2	
							78				
							91				
							99				
20	4D	24/8	20.00 - 22.00	7-6-12-20	18	27	41			Grey, wet, medium dense, Sandy GRAVEL, little silt, trace clay (GLACIAL TILL).	WC = 9.9% Fines = 15.5% A-1-b (0), GM
							45				
							64	66.0		22.5	
	R1	43.2/42	23.00 - 26.60	RQD = 13%			NQ2			Top of Bedrock at Elev. 66.0 ft. Advanced with roller bit to 23 ft bgs. R1: 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	
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 Geosciences 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 test before sampling; blow counts not representative of undisturbed material. 4. Station 214+67, offset 11.9 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	88.48	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (09:15); 4/26/22 (12:20)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485628.925, E: 1739834.412	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.8 ft bgs on 4/26/22 at 10:25
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 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										veins (0.1 in thick) present from 23.3 ft bgs to 24.2 bgs and at 26.5ft bgs; average 3.1 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 97% Recovery Rock Core Rate (min:sec) 23-24 ft (2:18) 24-25 ft (1:59) 25-26 ft (2:02) 26-26.6 ft (2:32)	
	R2	24/24	26.60 - 28.60	RQD = 0%							
	R3	40.8/40.8	28.60 - 32.00	RQD = 20%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in thick) present from 27 ft bgs to 27.7 bgs; average 5.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 26.6-27.6 ft (2:19) 27.6-28.6 ft (3:04)	
30											
	R4	16.8/16.8	32.00 - 33.40	RQD = 46%							
	R5	16.8/12	33.40 - 34.80	RQD = 64%							
	R6	38.4/38.4	34.80 - 38.00	RQD = 19%						R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, very closely to closely spaced, open, milky quartz and calcite veins (0.1 in to 1 in thick) present from 28.6 ft bgs to 29.3 bgs and 31.7 ft bgs to 32 ft bgs; average 3.0 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 28.6-29.6 ft (2:41) 29.6-30.6 ft (2:36) 30.6-31.6 ft (3:11) 31.6-32 ft (1:52)	
35								50.5			
40										R4: 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 throughout; highly weathered fracture at 32 ft bgs; average 4.3 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 32-33 ft (2:51) 33-33.4 ft (2:12)	
45										R5: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle, closely spaced, open, milky quartz and calcite veins (0.1 in to 0.2 in thick)	
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 Geosciences 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 test before sampling; blow counts not representative of undisturbed material. 4. Station 214+67, offset 11.9 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole	**Elevation (ft.)**	88.48	**Auger ID/OD:**	N/A
Operator:	Kevin Hanscom	**Datum:**	Maine East Zone	**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 (09:15); 4/26/22 (12:20)	**Drilling Method:**	Cased Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485628.925, E: 1739834.412	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	5.8 ft bgs on 4/26/22 at 10:25
Hammer Efficiency Factor: 0.908		**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
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.
50										present throughout; average 1.4 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Fair 71% Recovery Rock Core Rate (min:sec) 33.4-34.4 ft (2:34) 34.4-34.8 ft (2:48) R6: 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 to 0.5 in thick) present throughout; average 2.5 fractures per foot [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]. Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 34.8-35.8 ft (2:41) 35.8-36.8 ft (2:36) 36.8-38 ft (2:14)	
55											
60										38.0 **Bottom of Exploration at 38.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
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 Geosciences 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 test before sampling; blow counts not representative of undisturbed material. 4. Station 214+67, offset 11.9 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W.Cole Explorations LLC	**Elevation (ft.)**	97.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / U
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	11:56 AM 9/25/23-3:23 PM 9/25/23	**Drilling Method:**	Solid Stem Auger/Cased wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485981.059, E: 1739707.931	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.4 ft bgs on 9/25/23 at 15:02
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	24/17	0.00 - 2.00	1-1-2-5	3	5	SSA	96.6		(TOPSOIL) 0.8	
										Olive with brown, dry, medium stiff, Clayey SILT (SILT).	
5	2D	24/22	5.00 - 7.00	5-7-9-10	16	24		92.4		5.0 Olive, moist, very stiff, CLAY, moderately plastic (CLAY).	WC = 24.1% Fines = 99.7% LL = 36 PL = 20 PI = 16 LI = 0.3 A-6 (17), CL
10	3D	24/24	10.00 - 12.00	3-4-4-4	8	12	45			Olive, wet, stiff, Silty CLAY (CLAY). q_p = 3.5 ksf (pocket penetrometer)	
							48				
							54				
							59				
							39				
15	4D	24/23	15.00 - 17.00	3-3-3-4	6	9	OPEN			Grey, wet, stiff, CLAY, low plasticity (CLAY). q_p = 2.8 ksf (pocket penetrometer) 65x130 mm vane raw torque readings: V1: Reached maximum torque 44.3 ft-lbs without failure.	WC = 21.0% Fines = 98.3% LL = 28 PL = 18 PI = 10 LI = 0.3 A-4 (9), CL
	V1			S_u>1217 psf							
	1U	24/23	17.00 - 19.00								
							4				
							29				
20	5D	24/12	20.00 - 22.00	20-32-30-22	62	95	55	77.4		20.0 Grey, wet, very dense, fine to coarse SAND and GRAVEL, little silt, fractured rock fragment (GLACIAL TILL).	
							78				
							110				
							78/6"	73.9		23.5 Increase resistance to roller cone. Top of Bedrock at Elev. 73.9 feet.	
25							OPEN				

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 4D Sample interval disturbed by vane test before sampling; blow counts not represenative of undisturbed material. 4. Station 210+99, Offset 53.8 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W.Cole Explorations LLC	**Elevation (ft.)**	97.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / U
Logged By:	Daniel Burgess	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	11:56 AM 9/25/23-3:23 PM 9/25/23	**Drilling Method:**	Solid Stem Auger/Cased wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485981.059, E: 1739707.931	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.4 ft bgs on 9/25/23 at 15:02
Hammer Efficiency Factor: 0.918		**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	R1	24/20	25.00 - 27.00	RQD = 0%			NQ2	72.4		(WEATHERED ROCK), wash to 25 ft.	
										25.0	
	R2	42/30	27.00 - 30.50	RQD = 0%						R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steeply dipping, wide to close/very closely spaced, open to tight, extremely fractured upper 6" of run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 85% Recovery Rock Core Rate (min:sec) 25-26 ft (2:42) 26-27 ft (2:52)	
30	R3	10.8/7.2	30.50 - 31.40	RQD = 0%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderately to steep/ vertical dipping, very close to closely spaced, tight to open, extremely fractured bottom 6" of run, milky quartz intrusion present [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 71% Recovery Rock Core Rate (min:sec) 27-28 ft (4:29) 28-29 ft (3:02) 29-30 ft (2:41) 30-30.5 ft (1:06)	
	R4	43.2/43.2	31.40 - 35.00	RQD = 21%							
35								62.4		R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities, extremely fractured [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 67% Recovery Rock Core Rate (min:sec) 30.5-31 ft (1:20) 31-31.4 ft (1:37)	
40										R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, fresh; discontinuities low angle to steep angle dipping, very close to closely spaced, tight, milky quartz intrusion present [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 31.4-32 ft (1:54) 32-33 ft (2:54) 33-34 ft (3:14) 34-35 ft (4:28)	
45										35.0	
50										**Bottom of Exploration at 35.0 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 4D Sample interval disturbed by vane test before sampling; blow counts not represenative of undisturbed material. 4. Station 210+99, Offset 53.8 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.5	**Auger ID/OD:**	4.25 in
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:	2:37 PM 9/26/23-8:37 AM 9/27/23	**Drilling Method:**	Solid Stem Auger/Cased wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485977.643, E: 1739772.982	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.5 ft bgs on 9/27/23 at 07:04
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	1D	24/18	0.00 - 2.00	1-1-2-2	3	5	SSA	91.3		(TOPSOIL) 0.2	
										Olive with grey and brown,wet, medium stiff, Clayey SILT (SILT). q_p = 2.8 ksf (pocket penetrometer)	
5	2D	24/24	5.00 - 7.00	2-2-3-3	5	8		86.5		5.0 Olive, moist to wet, medium stiff, CLAY (CLAY). q_p= 2.5 ksf (pocket penetrometer)	
10	3D	24/15	10.00 - 12.00	WOH-17-30-16	47	72	18	81.5		10.0 Grey, wet, hard, CLAY, low plasticity, rock fragments at bottom of sample (CLAY).	WC = 18.8% LL = 25 PL = 17 PI = 8 LI = 0.2 CL
							51				
							149				
							87				
							70				
15	4D	24/10	15.00 - 17.00	7-10-8-10	18	28	15	76.5		15.0 Grey, wet, medium dense, fine to coarse SAND, some fine gravel, some silt (GLACIAL TILL).	
							25				
							44				
							23				
							63				
20	5D	18.1/-	20.00 - 21.51	8-11-9-50/1"	20	31	OPEN			Grey, wet, hard, Clayey SILT, trace fine to medium sand (GLACIAL TILL).	WC = 21.7% Fines = 91.6% A-4 (0)
								69.4		22.1	
	R1	60/57.6	22.10 - 27.10	RQD = 34%			NQ/NX			Top of Bedrock at 69.4 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep angle and vertical dipping, close to very close spacing, open to tight, milky quartz intrusion 1.2 to 2.2 ft, frequent calcite intrusions, chemical	
25											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+13, Offset 9.7 Ft. Lt.

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.5	**Auger ID/OD:**	4.25 in
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:	2:37 PM 9/26/23-8:37 AM 9/27/23	**Drilling Method:**	Solid Stem Auger/Cased wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485977.643, E: 1739772.982	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.5 ft bgs on 9/27/23 at 07:04
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										weathering 0.6-2.5ft [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 96% Recovery Rock Core Rate (min:sec) 22.1-23.1 ft (4:33) 23.1-24.1 ft (3:55) 24.1-25.1 ft (4:43) 25.1-26.1 ft (5:35) 26.1-27.1 ft (5:15)	
	R2	60/58.8	27.10 - 32.10	RQD = 56%							
30										R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep angle dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
								59.4		Rock Mass Quality = Fair 98% Recovery Rock Core Rate (min:sec) 27.1-28.1 ft (3:43) 28.1-29.1 ft (3:24) 29.1-30.1 ft (3:27) 30.1-31.1 ft (4:02) 31.1-32.1 ft (4:28)	
35										32.1 **Bottom of Exploration at 32.1 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+13, Offset 9.7 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:10 AM 9/28/23-12:00 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485818.985, E: 1739752.320	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	6.9 ft bgs on 9/28/23 at 11:37
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	24/15	0.00 - 2.00	1-2-6-4	8	12	SSA	91.2		(TOPSOIL) 0.3 Olive with grey and brown, mottled, dry to moist, stiff, Clayey SILT, trace fine sand, trace fine gravel (SILT). q_p = 9 ksf (pocket penetrometer)	
5	2D	24/24	5.00 - 7.00	3-4-4-5	8	12		86.4		5.0 Olive with grey, moist, stiff, CLAY, moderately plastic (CLAY). q_p = 4.8 ksf (pocket penetrometer)	WC = 26.3% Fines = 99.6% LL = 38 PL = 20 PI = 18 LI = 0.3 A-6 (19), CL
10	3D	24/24	10.00 - 12.00	1-1-2-3	3	5	5	81.4		10.0 Olive, wet, medium stiff, Silty CLAY (CLAY). q_p = 2.5 ksf (pocket penetrometer)	
							9				
							10				
							12				
							8				
15	4D	24/24	15.00 - 17.00	W.O.H/18"-2	-		12			Grey, wet, very soft, CLAY, moderately plastic (CLAY). q_p < 2 ksf (pocket penetrometer)	WC = 33.9% LL = 33 PL = 19 PI = 14 LI = 1.1 CL
							11				
							13				
							16				
							10				
20	5D	24/13	20.00 - 22.00	4-8-10-23	18	28	23	71.4		20.0 Grey, wet, very stiff, CLAY (CLAY). q_p = 3 ksf (pocket penetrometer)	
							55	69.9		21.5 Grey, wet, medium dense, fine to medium Silty SAND, some fine GRAVEL (GLACIAL TILL).	
							76				
							38				
25							17				

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.3. Station 212+66, Offset 37.2 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	91.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:10 AM 9/28/23-12:00 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485818.985, E: 1739752.320	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	6.9 ft bgs on 9/28/23 at 11:37
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	13/3	25.00 - 26.08	30-16-50/1"	R		45			Grey, wet, very dense, fine to medium Silty SAND, some fine GRAVEL, fractured rock at bottom of sample (GLACIAL TILL).	
	R1	60/56.4	26.20 - 31.20	RQD = 64%			50/2" NQ2	65.2		26.2 Top of Bedrock at 65.2 feet. Roller cone refusal encountered at 26.2 feet bgs. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to steep dipping, very close spacing, tight, frequent calcite intrusions, chemical weathering at 29.7 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Fair 94% Recovery Rock Core Rate (min:sec) 26.2-27.2 ft (2:11) 27.2-28.2 ft (2:20) 28.2-29.2 ft (2:13) 29.2-30.2 ft (2:37) 30.2-31.2 ft (2:33)	
30	R2	36/36	31.20 - 34.20	RQD = 25%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, chemical weathering from 32.03 to 34.2 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very poor 100% Recovery Rock Core Rate (min:sec) 31.2-32.2 ft (2:08) 32.2-33.2 ft (2:02) 33.2-34.2 ft (2:31)	
	R3	12/8.4	34.20 - 35.20	RQD = 30%						R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, severe chemical weathering on discontinuty surfaces; discontinuities moderate to steep dipping, close spacing, open [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 71% Recovery Rock Core Rate (min:sec) 34.2-35.2 ft (2:08)	
35	R4	12/8.4	35.20 - 36.20	RQD = 0%				55.2		R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, slight chemical weathering; discontinuities moderate to steep dipping, very close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 67% Recovery Rock Core Rate (min:sec) 35.2-36.2 ft (2:15) 36.2 **Bottom of Exploration at 36.2 feet below ground surface.** Boring backfilled with bentonite chips to ground surface.	
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.3. Station 212+66, Offset 37.2 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	89.0	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	1:17 PM 9/28/23-4:36 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485732.278, E: 1739758.223	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft bgs on 9/29/23 at 07:45
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	24/10	0.00 - 2.00	1-7-7-5	14	21	SSA	88.7		(TOPSOIL) 0.3 Olive, moist, very stiff, Clayey SILT, trace fine sand, trace fine gravel (SILT).	
5	2D	24/23	5.00 - 7.00	6-7-8-9	15	23		84.0		5.0 Olive, moist, very stiff, CLAY, moderately plastic (CLAY). q_p = 4.5 ksf (pocket penetrometer).	WC = 25.1% Fines = 99.0% LL = 36 PL = 22 PI = 14 LI = 0.2 A-6 (15), CL
10	3D	24/24	10.00 - 12.00	2-3-3-4	6	9	7	79.0		10.0 Olive, moist, stiff, Clayey SILT (SILT). q_p = 3.5 ksf (pocket penetrometer).	
							12				
							16				
							20				
							14				
15	4D	24/24	15.00 - 17.00	1-1-WOH/12"	-		23	74.0		15.0 Grey, wet, very soft, CLAY, low plasticity (CLAY). q_p < 2 ksf (pocket penetrometer)	
							18				
	5D	24/24	17.00 - 19.00	-	-		17			65x130 mm vane raw torque readings: V1: 24 ft-lbs/8 ft-lbs	WC = 18.8% LL = 24 PL = 15 PI = 9 LI = 0.4 CL
	V1/V2			~~S_u= 659/220 psf~~ S_u= 1016/302psf			17			65x130 mm vane raw torque readings: V2: 37 ft-lbs/11 ft-lbs	
							65				
20	6D	24/9	20.00 - 22.00	12-13-10-7	23	35	32	69.0		20.0 Grey, wet, dense, fine to coarse Silty SAND, some fine GRAVEL (GLACIAL TILL).	
							33				
							30				
							49				
25							44				

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 5D Sample sampling; blow counts not represenative of undisturbed material. 4. Station 213+52, Offset 45.9 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	89.0	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	1:17 PM 9/28/23-4:36 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485732.278, E: 1739758.223	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft bgs on 9/29/23 at 07:45
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
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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	7D	24/0	25.00 - 27.00	7-6-6-8	12	18	35			No recovery	
							40				
							86				
							130				
	R1	36/32.4	29.60 - 32.60	RQD = 0%			65/3" NQ2	59.4		29.6 Roller cone refusal. Top of Bedrock Elev. 59.4 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to vertical dipping, very close to close spacing, tight to open, frequent milky quartz and calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 90% Recovery Rock Core Rate (min:sec) 29.6 - 30.6 ft (2:19) 30.6 - 31.6 ft (2:34) 31.6 - 32.6 ft (2:57)	
30											
	R2	30/27.6	32.60 - 35.10	RQD = 0%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; extremely fractured, discontinuities steep to vertical dipping, close spacing, open, milky quartz intrusion 34.6 to 35.1 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 92% Recovery Rock Core Rate (min:sec) 32.6 - 33.6 ft (4:32) 33.6 - 34.6 ft (3:30) 34.6 - 35.1 ft (3:00	
35	R3	30/30	35.10 - 37.60	RQD = 13%						R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; highly fractured, discontinuities low angle to moderate dipping, very close spacing, tight, frequent milky quartz intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 35.1 - 35.6 ft (1:47) 35.6 - 36.6 ft (2:50) 36.6 - 37.6 ft (1:11)	
	R4	24/19.2	37.60 - 39.60	RQD = 17%				49.4		R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; highly fractured, discontinuities horizontal to steep dipping, very close spacing, tight, milky quartz intrusion 37.6 to 38.06 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 80% Recovery Rock Core Rate (min:sec)	
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 5D Sample sampling; blow counts not represenative of undisturbed material. 4. Station 213+52, Offset 45.9 Ft. Rt.

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	89.0	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	1:17 PM 9/28/23-4:36 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485732.278, E: 1739758.223	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.5 ft bgs on 9/29/23 at 07:45
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 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										37.6 - 38.6 ft (2:40) 38.6 - 39.6 ft (2:55) 39.6 **Bottom of Exploration at 39.6 feet below ground surface.** Boring backfilled with bentonite chips to match existing surface.	
55											
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 5D Sample sampling; blow counts not represenative of undisturbed material. 4. Station 213+52, Offset 45.9 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	87.9	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:19 AM 9/29/23-11:03 AM 9/29/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485651.132, E: 1739822.894	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.9 ft bgs on 9/29/23 at 10:56
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 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	1D	24/1	0.00 - 2.00	1-1-2-3	3	5	SSA	87.7		(TOPSOIL)	
										0.3	
										Wood in sample.	
5	2D	24/-	5.00 - 7.00	3-5-6-6	11	17		82.9		5.0	
										Olive with brown and grey, mottled, wet, very stiff, CLAY, trace fine sand, moderately plastic (CLAY). q_p = 6.5 ksf (pocket penetrometer)	WC = 27.1% Fines = 99.4% LL = 40 PL = 20 PI = 20 LI = 0.4 A-6 (21), CL
10	3D	24/24	10.00 - 12.00	2-2-3-3	5	8	15	77.9		10.0	
							6			Olive, wet, medium stiff, CLAY, frequent silt seams (CLAY). q_p = 3.5 ksf (pocket penetrometer)	
							7				
							9				
							7				
15	4D(A)	24/19	15.00 - 17.00	1/12"-8-16	8	12	4	72.9		15.0	
										Grey, wet, stiff, CLAY, low plasticity, frequent sand seams (CLAY). q_p < 2.0 ksf (pocket penetrometer)	WC = 29.0% LL = 23 PL = 15 PI = 8 LI = 1.8 CL
	4D(B)						25	71.9		16.0	
							43			Grey, wet, medium dense, Silty SAND, little fine gravel (GLACIAL TILL).	
							93				
							79/4"	68.4		19.5	
20	R1	46.8/43.2	20.00 - 23.90	RQD = 36%			NQ2	67.9		Increased roller cone resistance. Top of Bedrock Elev. 68.4 feet. (WEATHERED BEDROCK)	
										20.0	
										Roller cone refusal. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities steep to vertical dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor	
	R2	56.4/54	23.90 - 28.60	RQD = 75%							
25											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.3. Station 214+43, Offset 4.1 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	87.9	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:19 AM 9/29/23-11:03 AM 9/29/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485651.132, E: 1739822.894	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	9.9 ft bgs on 9/29/23 at 10:56
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										92% Recovery Rock Core Rate (min:sec) 20-21 ft (3:18) 21-22 ft (2:58) 22-23 ft (3:39) 23-23.9 ft (4:45)	
	R3	16.8/13.2	28.60 - 30.00	RQD = 41%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
30								57.9		Rock Mass Quality = Fair 96% Recovery Rock Core Rate (min:sec) 23.9-24 ft (0:30) 24-25 ft (2:49) 25-26 ft (3:16) 26-27 ft (2:49) 27-28 ft (3:28) 28-28.6 ft (1:24)	
35										R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to moderate dipping, very close spacing, tight, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 79% Recovery Rock Core Rate (min:sec) 28.6-29 ft (1:12) 29-30 ft (3:34)	
40										30.0 **Bottom of Exploration at 30.0 feet below ground surface.** Boring backfilled with bentonite chips to match existing surface.	
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT.3. Station 214+43, Offset 4.1 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	87.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:20 AM 10/2/23-12:23 AM 10/2/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485552.504, E: 1739822.376	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.3 ft bgs on 10/2/23 at 12:00
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	24/7	0.00 - 2.00	2-3-8-13	11	17	SSA	87.1		(TOPSOIL) 0.3 Dark brown, dry, very stiff, Clayey SILT, fine to medium sand, trace organics (roots) (SILT).	
5	2D	24/24	5.00 - 7.00	3-6-6-7	12	18		82.4		5.0 Olive with brown, moist, very stiff, CLAY, trace fine sand, moderately plastic (CLAY). q_p = 6.0 ksf (pocket penetrometer)	WC = 26.4% Fines = 99.5% LL = 35 PL = 19 PI = 16 LI = 0.5 A-6 (16), CL
10	3D	24/24	10.00 - 12.00	2-2-3-3	5	8	8	77.4		10.0 Olive, wet, medium stiff, CLAY (CLAY). q_p = 3.3 ksf (pocket penetrometer)	
							11				
							15				
							24				
							14				
15	4D	24/24	15.00 - 17.00	- S_u = 989/110psf	-		OPEN			Grey, wet, medium stiff, CLAY, low plasticity, interbedded brown sand beams (CLAY). 65x130 mm vane raw torque readings: V1: 36 ft-lbs/4 ft-lbs 65x130 mm vane raw torque readings: V2: Reached maximum torque 44.3 ft-lbs without failure.	WC = 25.0% Fines = 99.2% LL = 24 PL = 16 PI = 8 LI = 1.1 A-4 (6), CL
	V1/V2			S_u > 1217 psf							
	MU		17.00 - 17.50				30/4" 55/1"	69.9		Missed tube 17-19 ft bgs. Tube penetrated about 6-in before refusing.	
	R1	42/18	18.80 - 22.30	RQD = 9%			NQ2	68.6		17.5 Increased roller cone resistance. Top of Bedrock Elev. 69.9 feet. (WEATHERED ROCK).	
20										18.8 Roller cone refusal. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle dipping, very close spacing, tight, extremely fractured 18.8 to 19.2 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 43% Recovery Rock Core Rate (min:sec)	
	R2	18/18	22.30 - 23.80	RQD = 27%							
	R3	12/10.8	23.80 - 24.80	RQD = 0%							
25	R4	25.2/14.4	24.80 - 26.90	RQD = 0%							

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 4D Sample interval disturbed by vane test before sampling; blow counts not represenative of undisturbed material. 4. Station 215+34, Offset 14.5 Ft . Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	87.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:20 AM 10/2/23-12:23 AM 10/2/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485552.504, E: 1739822.376	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.3 ft bgs on 10/2/23 at 12:00
Hammer Efficiency Factor: 0.918		**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										18.8-19.8 ft (2:18) 19.8-20.8 ft (2:04) 20.8-21.8 ft (1:30) 21.8-22.3 ft (1:32)	
	R5	20.4/20.4	26.90 - 28.60	RQD = 0%						R2: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep dipping, very close spacing, tight, extremely fractured 22.3 to 22.8 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 100% Recovery Rock Core Rate (min:sec) 22.3-22.8 ft (1:25) 22.8-23.8 ft (1:55)	
	R6	26.4/25.2	28.60 - 30.80	RQD = 0%						R3: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate to steep angle dipping, very close to close spacing, tight [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 92% Recovery Rock Core Rate (min:sec) 23.8-24.8 ft (3:26)	
30								56.6		R4: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities moderate angle dipping, close to very close spacing, open to tight, extremely fractured 24.8 to 25.2 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 57% Recovery Rock Core Rate (min:sec) 24.8-25.8 ft (2:44) 25.8-26.8 ft (2:52) 26.8-26.9 ft (0:45)	
35										R5:Grey, fine grained to very fine grained, METAWACKE [metasandstone], medium strong, fresh; discontinuities moderate to steep angle dipping, very close spacing, tight, extremely fractured 27.8 to 28.6 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 100% Recovery Rock Core Rate (min:sec) 26.9-27.8 ft (1:59) 27.8-28.6 ft (2:09)	
40										R6: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities low angle to steep dipping, very close spacing, tight to open, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Very Poor 95% Recovery Rock Core Rate (min:sec) 28.6-28.8 ft (0:32) 28.8-29.8 ft (1:58)	
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 4D Sample interval disturbed by vane test before sampling; blow counts not represenative of undisturbed material. 4. Station 215+34, Offset 14.5 Ft . Rt.

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	87.4	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:20 AM 10/2/23-12:23 AM 10/2/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485552.504, E: 1739822.376	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	2.3 ft bgs on 10/2/23 at 12:00
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.
50										29.8-20.8 ft (2:14)	
										30.8	
										Bottom of Exploration at 30.8 feet below ground surface. Backfilled with bentonite chips to match existing ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. 4D Sample interval disturbed by vane test before sampling; blow counts not represenative of undisturbed material. 4. Station 215+34, Offset 14.5 Ft . Rt.

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 Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	90.0	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	10:43 AM 9/26/23-12:35 PM 9/26/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485922.242, E: 1739734.177	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.0 ft bgs on 9/26/23 at 12:18

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			Olive, moist to wet, Clayey SILT, classification based on auger cuttings (SILT).	
5											
10							OPEN	80.0		10.0 Grey, wet, Silty CLAY, classification based on auger cuttings and interpreted depth (CLAY).	
15											
20								72.3		17.7 Increased auger resistance. Possible glacial till or weathered rock.	
	R1	60/50.4	21.70 - 26.70	RQD = 48%			NQ2	68.3		21.7 Top of Bedrock Elev. 68.3 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, fresh; discontinuities horizontal to low angle dipping, very close to close spacing, tight, frequent milky quartz intrusions throughout entire run [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
25											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+61, Offset 37.2 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	90.0	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	10:43 AM 9/26/23-12:35 PM 9/26/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485922.242, E: 1739734.177	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	1.0 ft bgs on 9/26/23 at 12:18
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
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								63.3		Rock Mass Quality = Poor 84% Recovery Rock Core Rate (min:sec) 21.7-22.7 ft (4:02) 22.7-23.7 ft (5:04) 23.7-24.7 ft (3:28) 24.7-25.7 ft (5:06) 25.7-26.7 ft (5:16)	
										26.7	
										Bottom of Exploration at 26.7 feet below ground surface. Backfilled with bentonite chips to match existing ground surface.	
30											
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+61, Offset 37.2 Ft. Rt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	93.1	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:59 AM 9/27/23-11:32 AM 9/27/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485878.004, E: 1739786.412	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.6 ft bgs on 9/27/23 at 11:19
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	92.8		(TOPSOIL) 0.3	
										Brown, dry to moist, fine to medium Sandy SILT, classification based on auger cuttings (SILT).	
								90.1		3.0	
										Olive, wet, CLAY, classificaiton based on cuttings and interpreted depth (CLAY).	
5											
10										Grey, wet, Silty CLAY, classification based on auger cuttings and interpreted depth (CLAY).	
15								76.6		16.5	
							17			Silty SAND and GRAVEL, inferred from casing penetration, resistance and roller cone resistance (GLACIAL TILL).	
							70				
							53				
20							40			(GLACIAL TILL), inferred from casing resistance penetration and roller cone resistance.	
							46				
							83				
							87/7"				
	R1	60/55.2	23.00 - 28.00	RQD = 37%			NQ2	70.1		23.0	
25										Roller cone refusal. Top of Bedrock Elev. 70.1 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to steep	

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+13, Offset 6.3 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	93.1	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	9:59 AM 9/27/23-11:32 AM 9/27/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485878.004, E: 1739786.412	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	3.6 ft bgs on 9/27/23 at 11:19
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										dipping, very close to close spacing, tight to open, frequent calcite intrusions, milky quartz intrusion at 3.8 to 3.9 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Poor	
										92% Recovery	
								65.1		Rock Core Rate (min:sec)	
										23-24 ft (2:38)	
										24-25 ft (3:06)	
30										25-26 ft (3:23)	
										26-27 ft (3:39)	
										27-28 ft (3:01)	
										28.0	
										Bottom of Exploration at 28.0 feet below ground surface. Backfilled with bentonite chips to match existing ground surface.	
35											
40											
45											
50											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 211+13, Offset 6.3 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	90.6	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	12:12 PM 9/28/23-1:08 PM 9/28/23	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485787.036, E: 1739818.087	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	8.1 ft bgs on 9/28/23 at 12:58
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	90.3		(TOPSOIL)	
										0.3	
										Olive, moist to wet, Clayey SILT, classification based on auger cuttings and interpreted depth inferred from auger resistance (SILT).	
5								85.6		5.0	
										Olive, moist to wet, CLAY, classification based on auger cuttings and interpreted depth inferred from auger resistance (CLAY).	
10										Grey, wet, Silty CLAY, classification based on auger cuttings and interpreted depth inferred from auguer resistance (CLAY).	
							50/1"	77.1		13.5	
	R1	60/46.8	14.20 - 19.20	RQD = 30%			NQ2	76.4		Increased roller cone resistance, possible glacial till or weathered bedrock.	
15										14.2	
										Roller cone refusal. Top of Bedrock Elev. 76.4 feet. R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], very strong, fresh; discontinuities low angle to vertical dipping, close to very close spacing, open to tight, frequent calcite intrusions,extremely fractured 14.9 to 15.7 ft bgs [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION] Rock Mass Quality = Poor 78% Recovery Rock Core Rate (min:sec) 14.2-15.2 ft (1:21) 15.2-16.2 ft (1:01) 16.2-17.2 ft (2:00) 17.2-18.2 ft (2:02) 18.2-19.2 ft (2:20)	
20								71.4		19.2	
										Bottom of Exploration at 19.2 feet below ground surface.	
25											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 213+09, Offset 22.2 Ft. Lt.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: I-95 Hogan Road Bridge Replacement #5823 (Exit 187)
Location: Bangor, Maine

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

Driller:	S.W. Cole Explorations LLC	**Elevation (ft.)**	84.9	**Auger ID/OD:**	4.25 in
Operator:	Kevin Hanscom	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	Daniel Burgess	**Rig Type:**	Dierich D-50	**Hammer Wt./Fall:**	140 lbs/30 in
Date Start/Finish:	11:18 AM 9/29/23-12:55	**Drilling Method:**	Solid Stem Auger/Drive and Wash	**Core Barrel:**	1.98 inch I.D. - NQ2
Boring Location:	N: 485594.479, E: 1739801.153	**Casing ID/OD:**	4.0/4.5 in	**Water Level*:**	0.5 ft ags on 9/29/23 at 12:43
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	84.7		(TOPSOIL)	
										0.2	
										Olive, moist to wet, Clayey SILT, inferred from auger cuttings (SILT).	
5											
								77.9		7.0	
										Olive, wet, Silty CLAY, inferred from auger cuttings (CLAY).	
10							5				
							9				
							11				
							11				
							31	70.4		14.5	
15							59			Silty SAND and GRAVEL, inferred from roller cone resistance and wash cuttings (GLACIAL TILL).	
							45				
							34				
							32				
							42				
20							35				
							54				
							61				
	R1	60/57.6	23.80 - 28.80	RQD = 42%			92/6" NQ2	61.1		23.8	
										Roller cone refusal. Top of Bedrock Elev. 61.1 feet.	
25											

Remarks:

1. Hammer Efficiency Factor provide 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 Geosciences Testing and Research, Inc., dated 9/23/2021. 2. As-drilled locations and elevations were provided by MaineDOT, and are derived from electronic file "18595.10.BORINGLOC.200SERIES.10.11.23.csv" received by WSP on October 13, 2023 from MaineDOT. 3. Station 214+95, Offset 26.8 Ft. Rt.

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

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WC = Water Content, percent
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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.
25										R1: Grey, fine grained to very fine grained, METAWACKE [metasandstone], strong, moderately weathered (25.2 to 26. 9 ft bgs) to fresh; discontinuities low angle dipping, close to very close spacing, open to tight, extremely fractured 25.2 to 26.9 ft bgs, frequent calcite intrusions [MEDIUM BEDDED FACIES, PENOBSCOT RIVER MEMBER, BANGOR FORMATION]	
										Rock Mass Quality = Poor	
										96% Recovery	
								56.1		Rock Core Rate (min:sec)	
										23.8-24.8 ft (3:01)	
30										24.8-25.8 ft (3:34)	
										25.8-26.8 ft (3:42)	
										26.8-27.8 ft (3:33)	
										27.8-28.8 ft (3:39)	
										28.8	
35										**Bottom of Exploration at 28.8 feet below ground surface.**	
40											
45											
50											

Remarks:

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APPENDIX B

Rock Core Photographs

APPENDIX B
ROCK CORE PHOTOGRAPHS
I-95 HOGAN ROAD BRIDGE 5823 AND BRIDGE 6787
BANGOR, MAINE
MAINEDOT WIN 018595.11

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

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Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BB-BHR-102	R1	14.7 - 17.7	1.6 / 3.0	53	0.88 / 3.0	29	METAWACKE	1	4/27/2022
	R2	17.7 - 20.5	2.8 / 2.8	100	1.10 / 2.8	39	METAWACKE	1	4/27/2022
	R3	20.5 - 25.0	4.3 / 4.5	96	3.40 / 4.5	76	METAWACKE	2	4/27/2022
	R4	25.0 - 27.2	2.2 / 2.2	100	0.00 / 2.2	0	METAWACKE	3	4/27/2022
	R5	27.2 - 29.7	2.5 / 2.5	100	0.70 / 2.5	28	METAWACKE	3	4/27/2022

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3. Rock core wetted with water for photographs
4. BB-BHR-102 Run 1 suspected boulder

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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	12/17/2019
	R2	31.3 - 34.0	2.4 / 2.7	89	1.30 / 2.7	48	METAWACKE	4	12/17/2019

Notes: 1. "Box row" indicates the section of the box where the core is contained: 1 = top, 4 = bottom.
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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

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Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BP-BHR-207	R1	21.7 - 26.7	4.2 / 5.0	84	2.40 / 5.0	48	METAWACKE	1	9/26/2023
BB-BHR-202	R1	22.1 - 27.1	4.8 / 5.0	96	1.70 / 5.0	34	METAWACKE	2	9/27/2023
	R2	27.1 - 32.1	4.9 / 5.0	98	2.80 / 5.0	56	METAWACKE	3	9/27/2023
BP-BHR-208	R1	23.0 - 28.0	4.6 / 5.0	92	1.83 / 5.0	37	METAWACKE	4	9/27/2023

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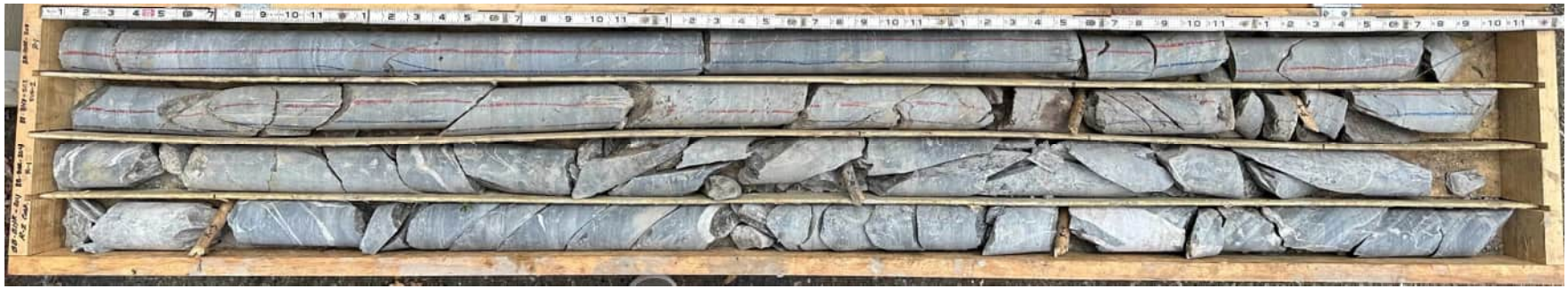
Boring	Run	Depth Below Surface Feet	Recovery Feet	Recovery %	RQD Feet	RQD %	Rock Type	Box Row	Date Cored
BB-BHR-203	R1	26.2 - 31.2	4.7 / 5.0	94	3.20 / 5.0	64	METAWACKE	1	9/28/2023
	R2	31.2 - 34.2	3.0 / 3.0	100	0.75 / 3.0	25	METAWACKE	2	9/28/2023
	R3	34.2 - 35.2	0.7 / 1.0	71	0.30 / 1.0	30	METAWACKE	2	9/28/2023
	R4	35.2 - 36.2	0.7 / 1.0	67	0.00 / 1.0	0	METAWACKE	2	9/28/2023
BB-BHR-204	R1	29.6 - 32.6	2.7 / 3.0	90	0.00 / 3.0	0	METAWACKE	3	9/28/2023
	R2	32.6 - 35.1	2.3 / 2.5	92	0.00 / 2.5	0	METAWACKE	3, 4	9/28/2023
	R3	35.1 - 37.6	2.5 / 2.5	100	0.33 / 2.5	13	METAWACKE	4	9/28/2023
	R4	37.6 - 39.6	1.6 / 2.0	80	0.33 / 2.0	17	METAWACKE	4	9/28/2023



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BB-BHR-205	R1	20.0 - 23.9	3.6 / 3.9	92	1.42 / 3.9	36	METAWACKE	1	9/29/2023
	R2	23.9 - 28.6	4.5 / 4.7	96	3.54 / 4.7	75	METAWACKE	1, 2	9/29/2023
	R3	28.6 - 30.0	1.1 / 1.4	79	0.58 / 1.4	41	METAWACKE	2	9/29/2023
BP-BHR-210	R1	23.8 - 28.8	4.8 / 5.0	96	2.10 / 5.0	42	METAWACKE	3	9/29/2023

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BP-BHR-209	R1	14.2 - 19.2	3.9 / 5.0	78	1.50 / 5.0	30	METAWACKE	1	9/28/2023
BB-BHR-206	R1	18.8 - 22.3	1.5 / 3.5	43	0.33 / 3.5	9	METAWACKE	2	10/2/2023
	R2	22.3 - 23.8	1.5 / 1.5	100	0.41 / 1.5	27	METAWACKE	2	10/2/2023
	R3	23.8 - 24.8	0.9 / 1.0	92	0.00 / 1.0	0	METAWACKE	2	10/2/2023
	R4	24.8 - 26.9	1.2 / 2.1	57	0.00 / 2.1	0	METAWACKE	3	10/2/2023
	R5	26.9 - 28.6	1.7 / 1.7	100	0.00 / 1.7	0	METAWACKE	3	10/2/2023
	R6	28.6 - 30.8	2.1 / 2.2	95	0.00 / 2.2	0	METAWACKE	4	10/2/2023

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APPENDIX C

Laboratory Testing Results

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	05/11/22	Checked By:	jdt
Depth :	---	Test Id:	667325		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BHR-101	S- 3	10-12	Moist, light olive brown clay	33.0
BB-BHR-101	S- 5	20-22	Moist, gray silty gravel with sand	10.2
BB-BHR-102	S- 2	5-7	Moist, light olive brown clay	32.5
BB-BHR-102	S- 3B	10.3-12	Moist, olive silty gravel with sand	10.9
BB-BHR-103	S- 2	5-7	Moist, light olive brown clay	27.4
BB-BHR-103	S- 3	10-12	Moist, light olive brown clay	29.8
BB-BHR-103	S- 4	20-22	Moist, olive silty gravel with sand	9.9

Notes: Temperature of Drying : 110º Celsius

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	---	Sample Type:	---	Tested By:	cam
Sample ID:	---	Test Date:	05/19/22	Checked By:	bfs
Depth :	---	Test Id:	668250		

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content,%	Ash Content,%	Organic Matter,%
BB-BHR-101	1U	15-17	Moist, gray clay	33	99.3	.7
BB-BHR-103	1U	15-16.2	Wet, gray clay	22	99.5	.5

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 Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	---	Sample Type:	---	Tested By:	amp
Sample ID:	---	Test Date:	05/21/22	Checked By:	bfs
Depth :	---	Test Id:	668924		

pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
BB-BHR-101	S-2	5-7	Moist, olive brown clay	6.8	6.5
BB-BHR-102	S-1B	0.6-2	Moist, olive brown clay	7.3	6.4
BB-BHR-103	S-1	0-2	Moist, olive brown clay	7.8	7.2

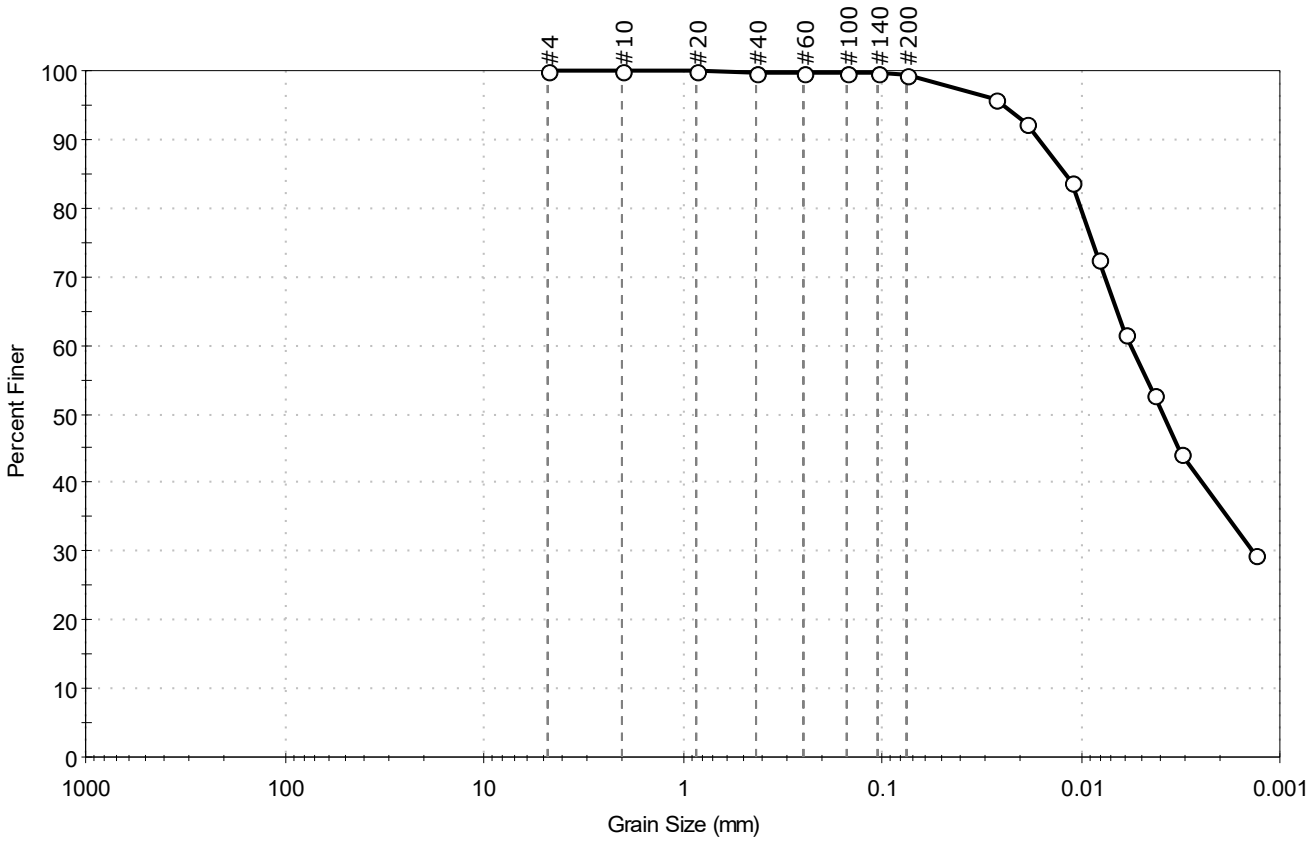
Notes: Sample Preparation: screened through #10 sieve

Method A, pH meter used

GeoTesting EXPRESS

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-101	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3	Test Date:	05/11/22	Checked By:	jdt
Depth :	10-12	Test Id:	667313		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.7	99.3

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	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
---	0.0271	96		
---	0.0190	92		
---	0.0111	84		
---	0.0082	73		
---	0.0060	62		
---	0.0043	53		
---	0.0031	44		
---	0.0013	30		

Coefficients

D_{85} = 0.0121 mm	D_{30} = 0.0014 mm
D_{60} = 0.0056 mm	D_{15} = N/A
D_{50} = 0.0039 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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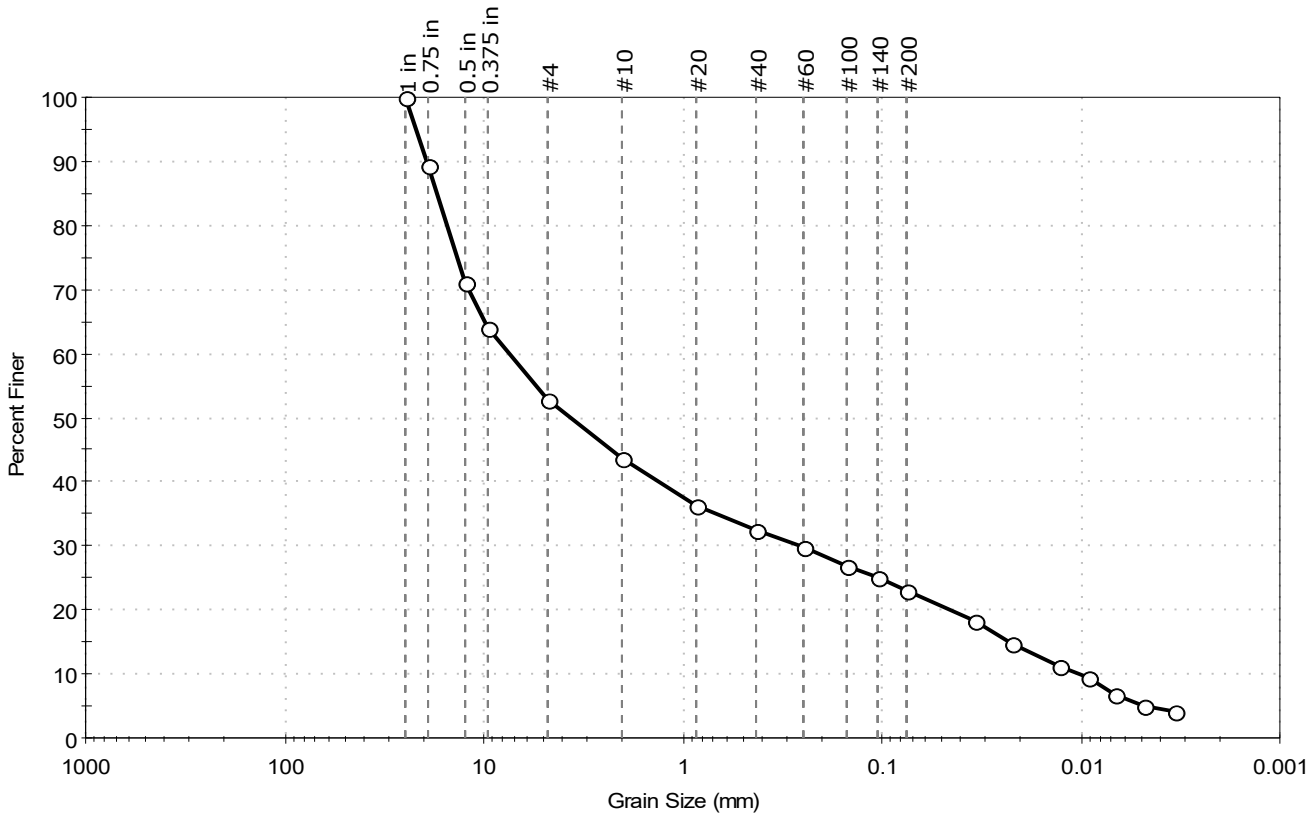
Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-101	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-5	Test Date:	05/11/22	Checked By:	jdt
Depth :	20-22	Test Id:	667314		
Test Comment:	---				
Visual Description:	Moist, gray silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
–	47.2	29.8	23.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	89		
0.5 in	12.50	71		
0.375 in	9.50	64		
#4	4.75	53		
#10	2.00	44		
#20	0.85	36		
#40	0.42	32		
#60	0.25	30		
#100	0.15	27		
#140	0.11	25		
#200	0.075	23		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0343	18		
---	0.0223	15		
---	0.0129	11		
---	0.0093	10		
---	0.0067	7		
---	0.0048	5		
---	0.0034	4		

Coefficients

D_{85} = 17.2292 mm
D_{30} = 0.2670 mm
D_{60} = 7.4306 mm
D_{15} = 0.0229 mm
D_{50} = 3.6730 mm
D_{10} = 0.0102 mm
C_u = 728.490
C_c = 0.941

Classification

ASTM N/A

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

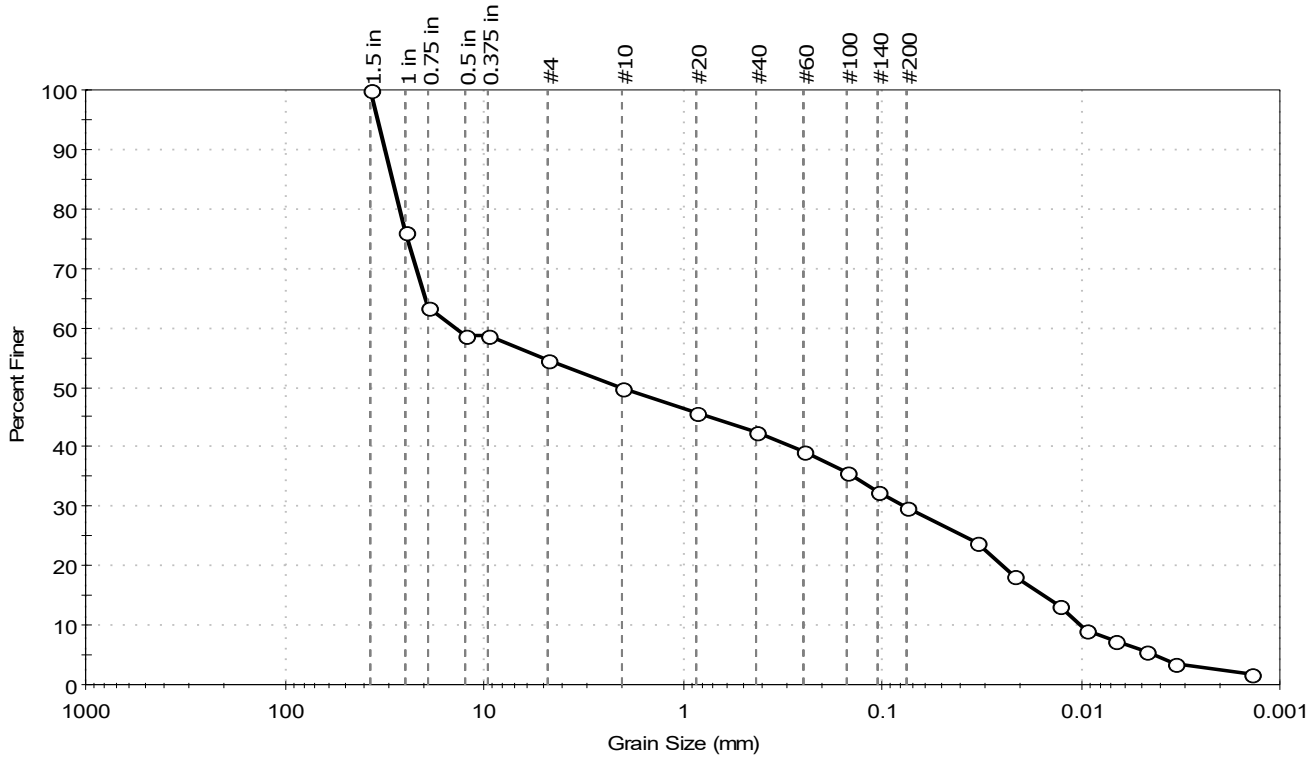
Sample/Test Description

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

GeoTesting EXPRESS

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3B	Test Date:	05/11/22	Checked By:	jdt
Depth :	10.3-12	Test Id:	667315		
Test Comment:	---				
Visual Description:	Moist, olive silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	45.5	24.6	29.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	76		
0.75 in	19.00	63		
0.5 in	12.50	59		
0.375 in	9.50	59		
#4	4.75	55		
#10	2.00	50		
#20	0.85	46		
#40	0.42	42		
#60	0.25	39		
#100	0.15	36		
#140	0.11	32		
#200	0.075	30		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0333	24		
---	0.0216	18		
---	0.0128	13		
---	0.0093	9		
---	0.0067	7		
---	0.0048	5		
---	0.0034	4		
---	0.0014	2		

Coefficients

D_{85} = 29.0509 mm D_{30} = 0.0759 mm
D_{60} = 13.9063 mm D_{15} = 0.0154 mm
D_{50} = 2.0190 mm D_{10} = 0.0100 mm
C_u = 1390.630 C_c = 0.041

Classification

ASTM N/A

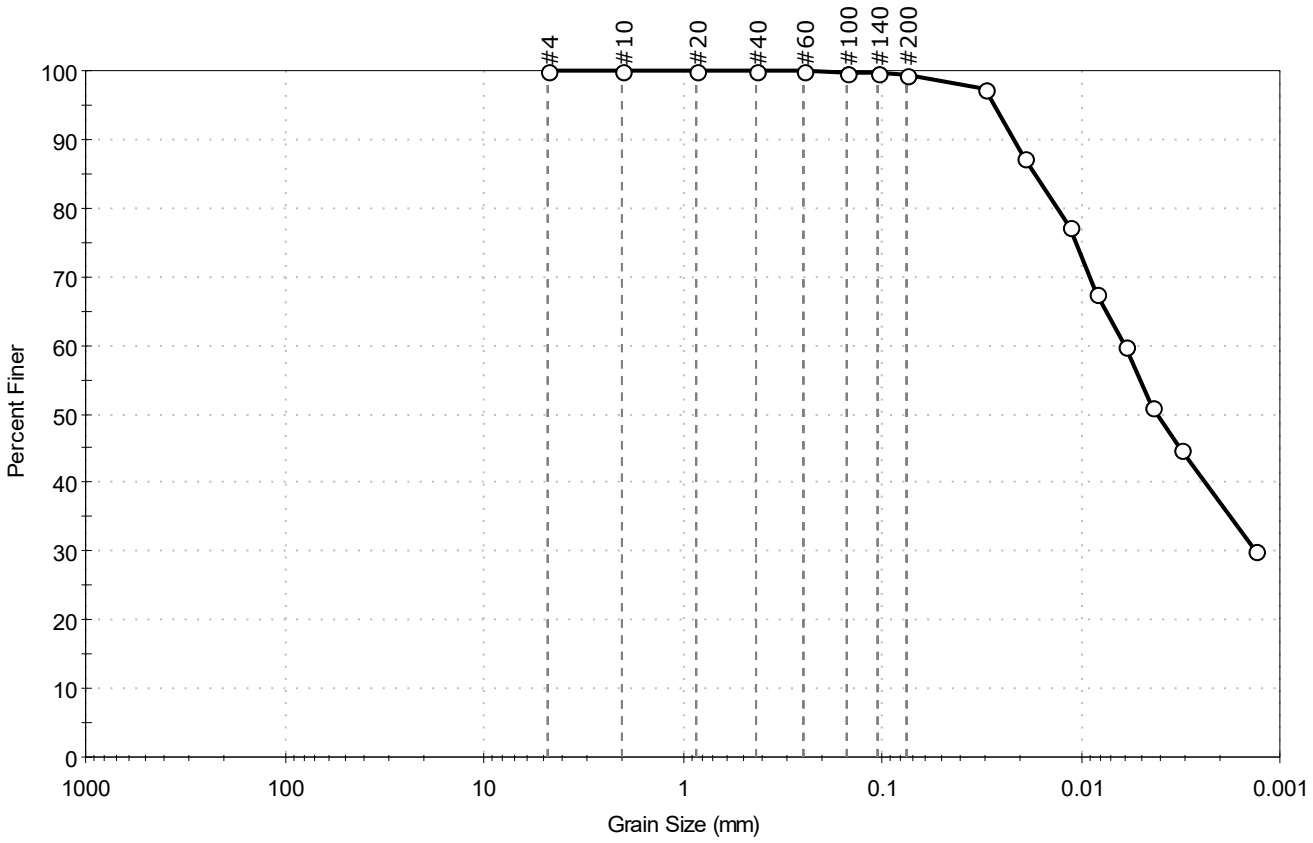
AASHTO Silty Gravel and Sand (A-2-4 (0))

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, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	05/11/22	Checked By:	jdt
Depth :	5-7	Test Id:	667316		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% 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	100		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0300	97		
---	0.0194	87		
---	0.0115	77		
---	0.0083	67		
---	0.0060	60		
---	0.0044	51		
---	0.0031	45		
---	0.0013	30		

Coefficients

D_{85} = 0.0171 mm	D_{30} = 0.0013 mm
D_{60} = 0.0060 mm	D_{15} = N/A
D_{50} = 0.0041 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Clayey Soils (A-6 (17))

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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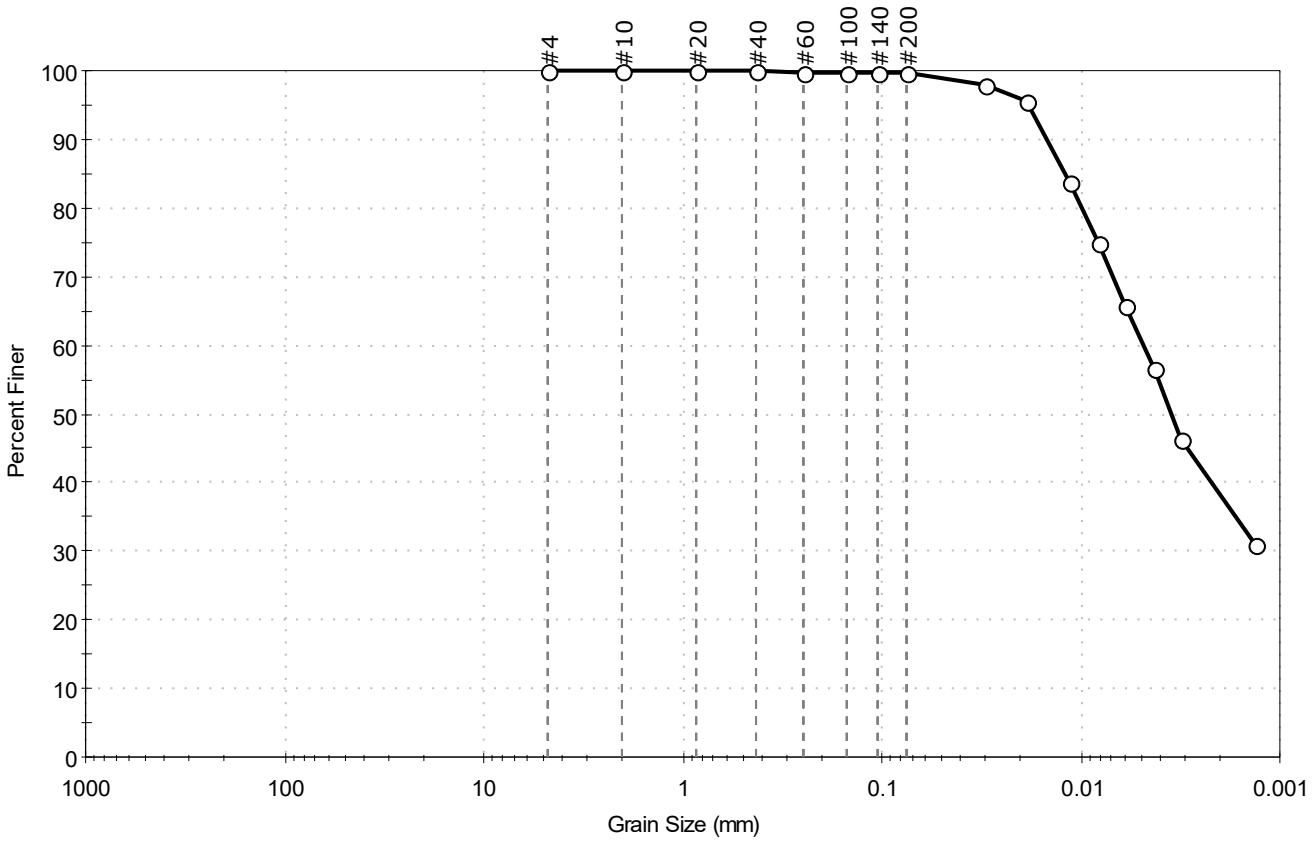
Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-3	Test Date:	05/11/22	Checked By:	jdt
Depth :	10-12	Test Id:	667317		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.4	99.6

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
---	0.0305	98		
---	0.0186	95		
---	0.0113	84		
---	0.0081	75		
---	0.0060	66		
---	0.0043	57		
---	0.0031	46		
---	0.0013	31		

Coefficients

D_{85} = 0.0118 mm	D_{30} = N/A
D_{60} = 0.0048 mm	D_{15} = N/A
D_{50} = 0.0035 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

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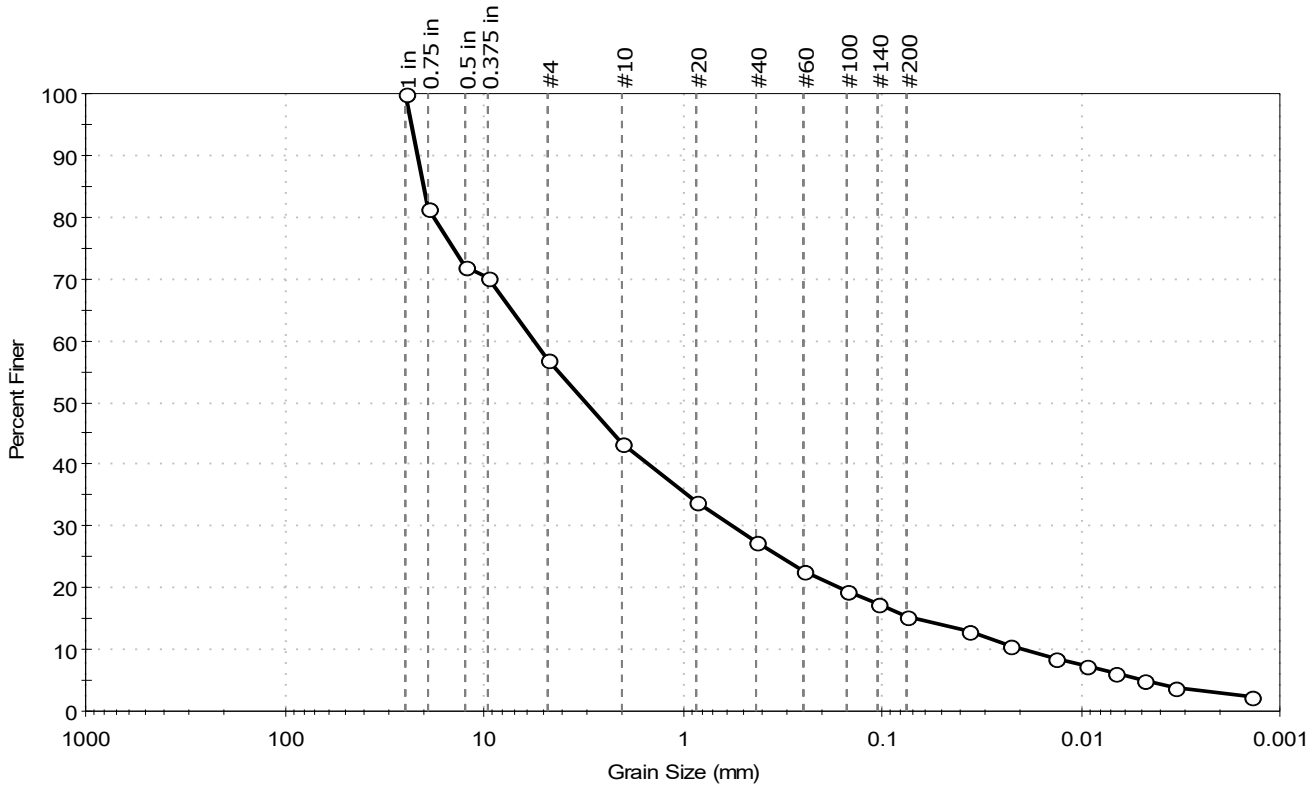
GeoTesting EXPRESS

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-4	Test Date:	05/11/22	Checked By:	jdt
Depth :	20-22	Test Id:	667318		
Test Comment:	---				
Visual Description:	Moist, olive silty gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
--	43.2	41.3	15.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	81		
0.5 in	12.50	72		
0.375 in	9.50	70		
#4	4.75	57		
#10	2.00	43		
#20	0.85	34		
#40	0.42	27		
#60	0.25	23		
#100	0.15	19		
#140	0.11	17		
#200	0.075	15		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0364	13		
---	0.0229	11		
---	0.0133	8		
---	0.0094	7		
---	0.0067	6		
---	0.0048	5		
---	0.0034	4		
---	0.0014	2		

Coefficients

D_{85} = 20.0266 mm	D_{30} = 0.5629 mm
D_{60} = 5.5995 mm	D_{15} = 0.0649 mm
D_{50} = 3.0813 mm	D_{10} = 0.0192 mm
C_u = 291.641	C_c = 2.947

Classification

ASTM — Silty GRAVEL with Sand (GM)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

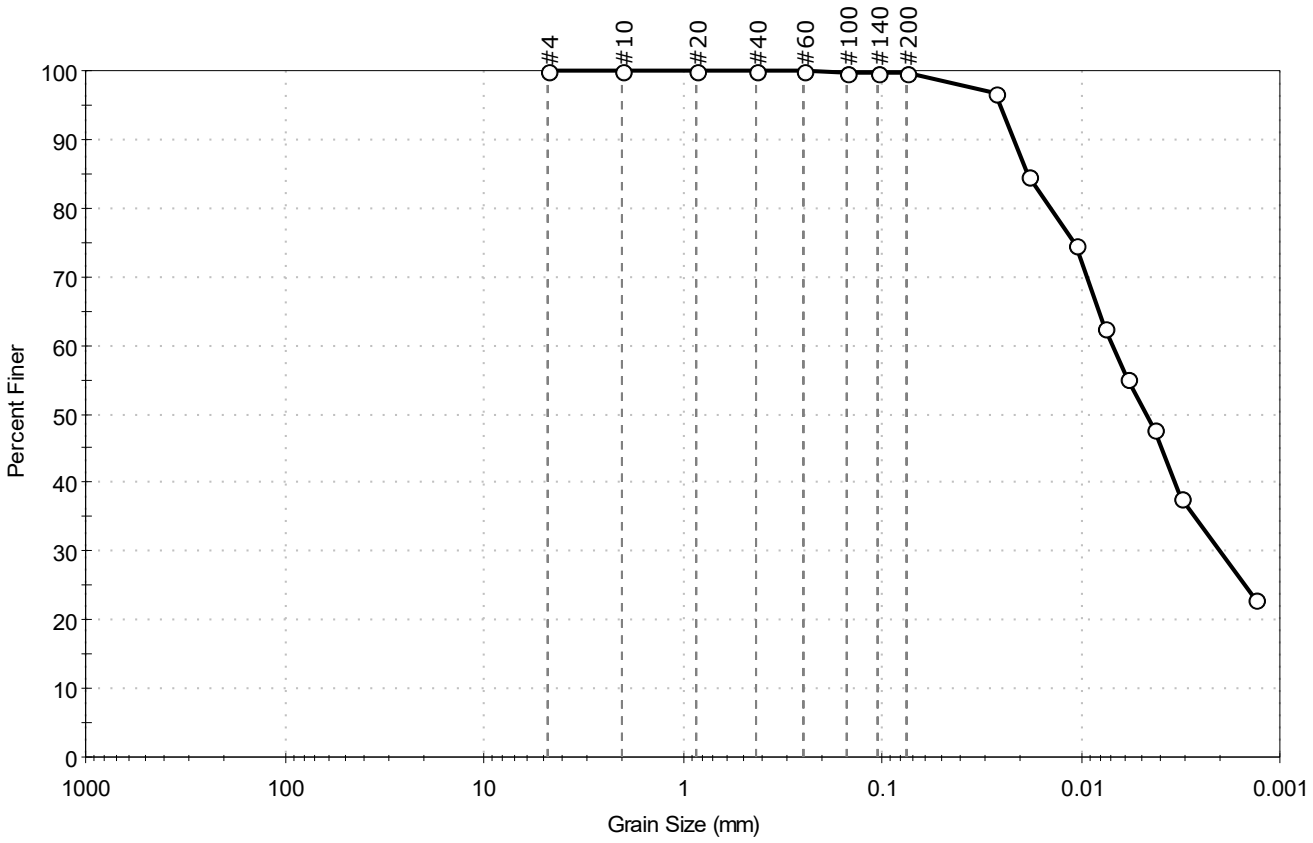
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-101	Sample Type:	tube	Tested By:	ckg
Sample ID:	1U	Test Date:	05/23/22	Checked By:	bfs
Depth :	15-17	Test Id:	668247		
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
---	0.0267	97		
---	0.0182	85		
---	0.0107	75		
---	0.0076	62		
---	0.0058	55		
---	0.0043	48		
---	0.0031	38		
---	0.0013	23		

Coefficients

D_{85} = 0.0185 mm	D_{30} = 0.0020 mm
D_{60} = 0.0070 mm	D_{15} = N/A
D_{50} = 0.0047 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

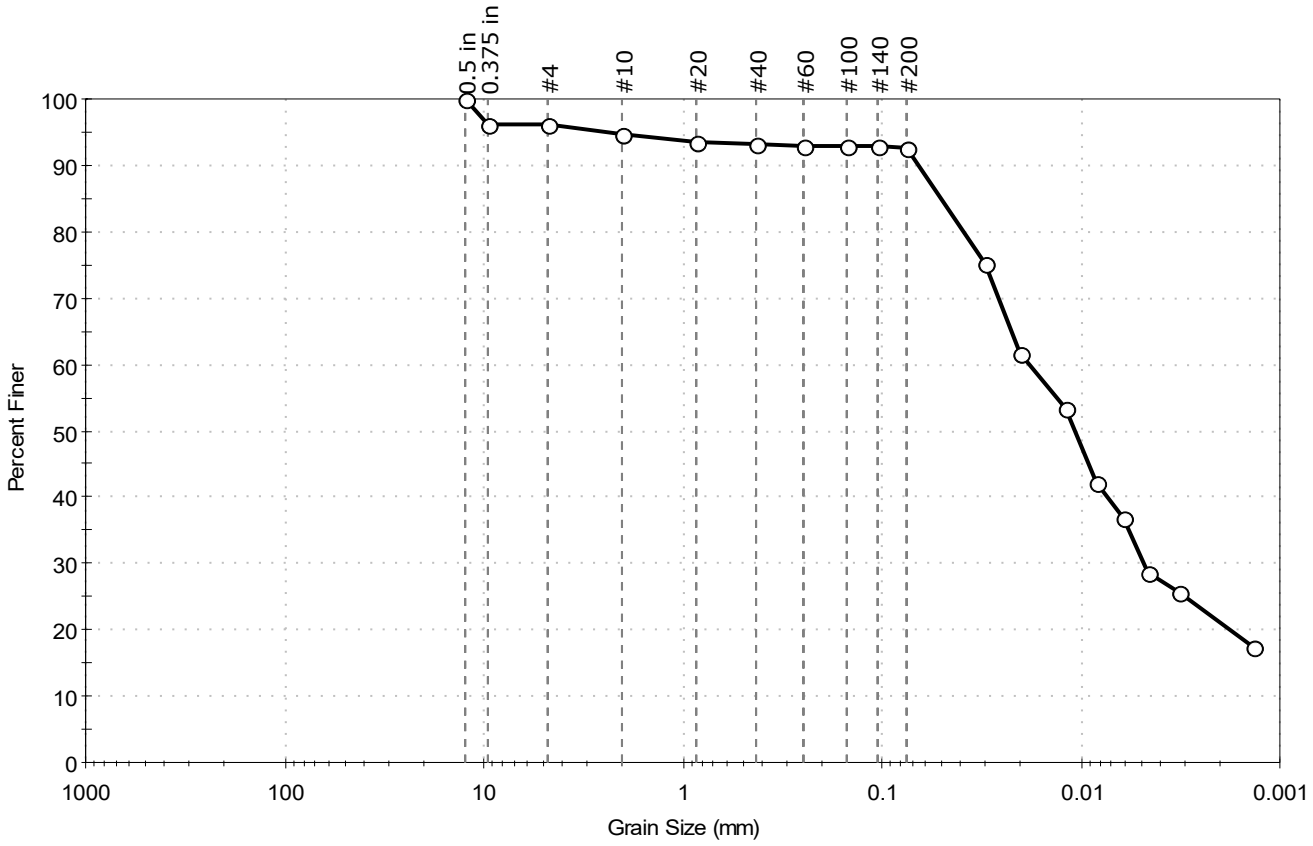
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	tube	Tested By:	ckg
Sample ID:	1U	Test Date:	05/23/22	Checked By:	bfs
Depth :	15-16.2	Test Id:	668248		
Test Comment:	---				
Visual Description:	Wet, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	3.7	3.7	92.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	96		
#4	4.75	96		
#10	2.00	95		
#20	0.85	94		
#40	0.42	93		
#60	0.25	93		
#100	0.15	93		
#140	0.11	93		
#200	0.075	93		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0305	75		
---	0.0202	62		
---	0.0120	53		
---	0.0083	42		
---	0.0061	37		
---	0.0046	29		
---	0.0032	26		
---	0.0014	18		

Coefficients

D_{85} = 0.0505 mm	D_{30} = 0.0048 mm
D_{60} = 0.0184 mm	D_{15} = N/A
D_{50} = 0.0107 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

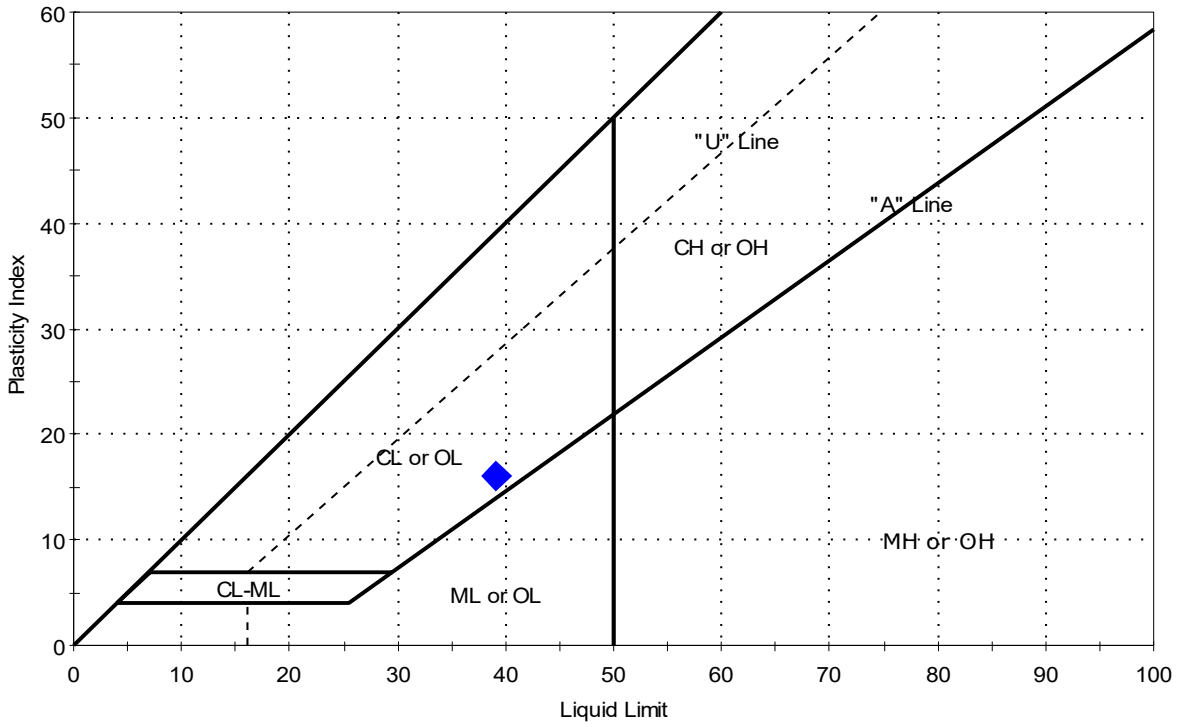
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-101	Sample Type:	jar	Tested By:	cam
Sample ID:	S-3	Test Date:	05/11/22	Checked By:	jdt
Depth :	10-12	Test Id:	667308		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

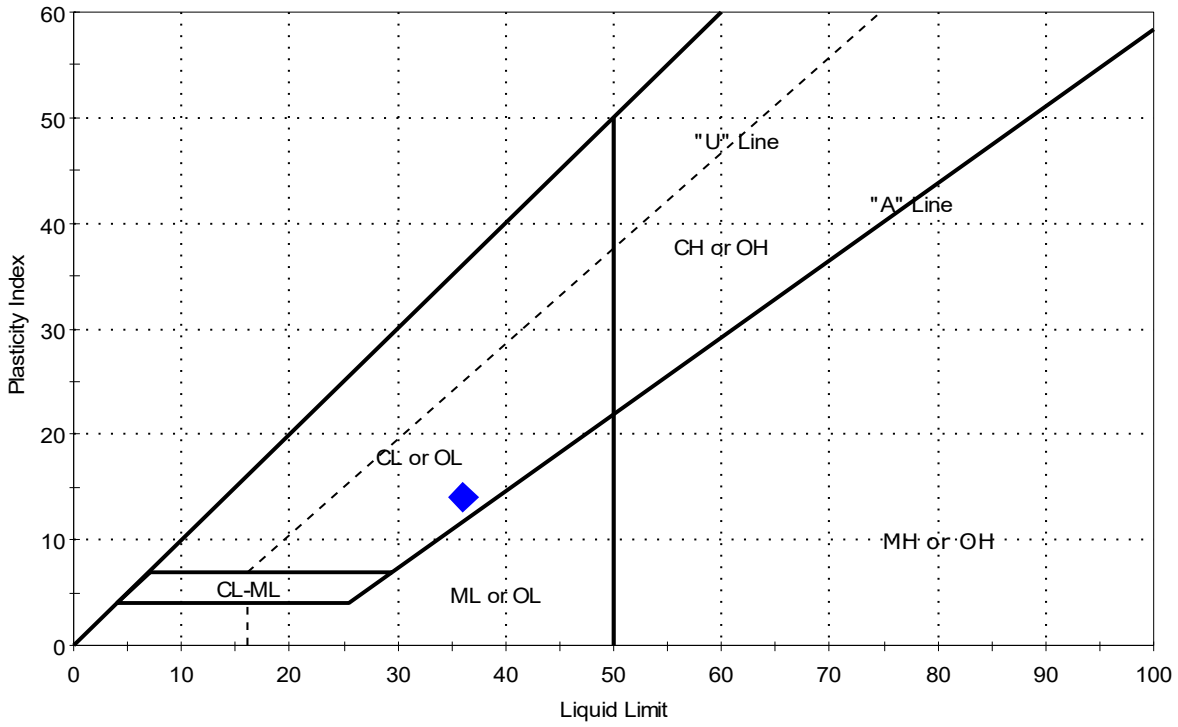


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-3	B-BHR-10	10-12	33	39	23	16	0.6	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	05/11/22	Checked By:	jdt
Depth :	5-7	Test Id:	667309		
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
◆	S-2	B-BHR-10	5-7	32	36	22	14	0.7	

Sample Prepared using the WET method

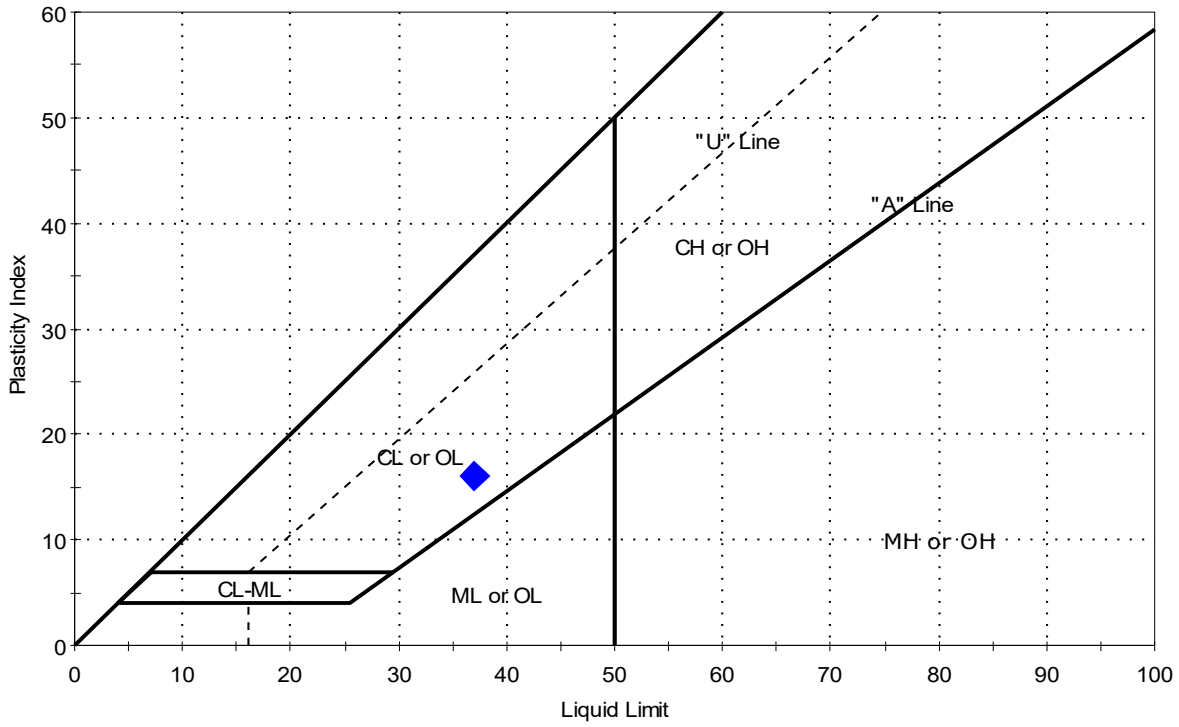
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	05/11/22	Checked By:	jdt
Depth :	5-7	Test Id:	667310		
Test Comment:	---				
Visual Description:	Moist, light olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



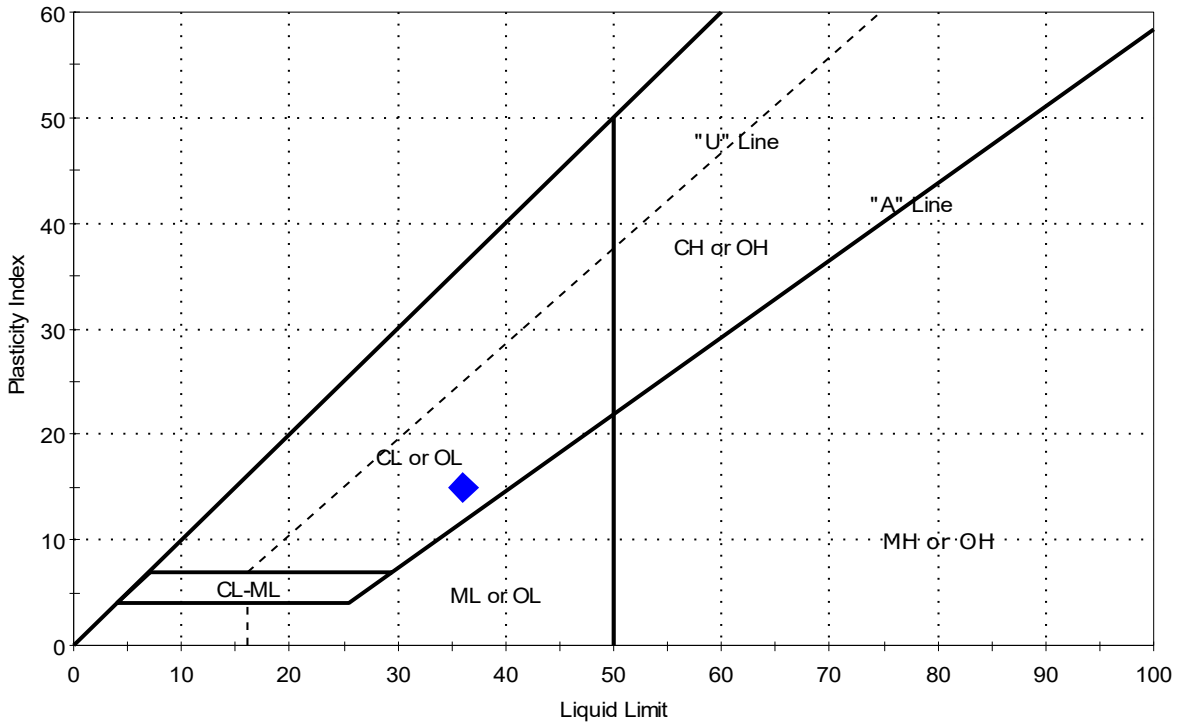
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-10	5-7	27	37	21	16	0.4	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

GeoTesting EXPRESS

Client:	Golder Associates, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	cam
Sample ID:	S-3	Test Date:	05/11/22	Checked By:	jdt
Depth :	10-12	Test Id:	667311		
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
◆	S-3	B-BHR-10	10-12	30	36	21	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, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	jar	Tested By:	cam
Sample ID:	S-4	Test Date:	05/10/22	Checked By:	jdt
Depth :	20-22	Test Id:	667312		
Test Comment:	---				
Visual Description:	Moist, olive silty gravel with sand				
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
◆	S-4	B-BHR-10	20-22	10	n/a	n/a	n/a	n/a	Silty GRAVEL with Sand (GM)

73% Retained on #40 Sieve

Dry Strength: LOW

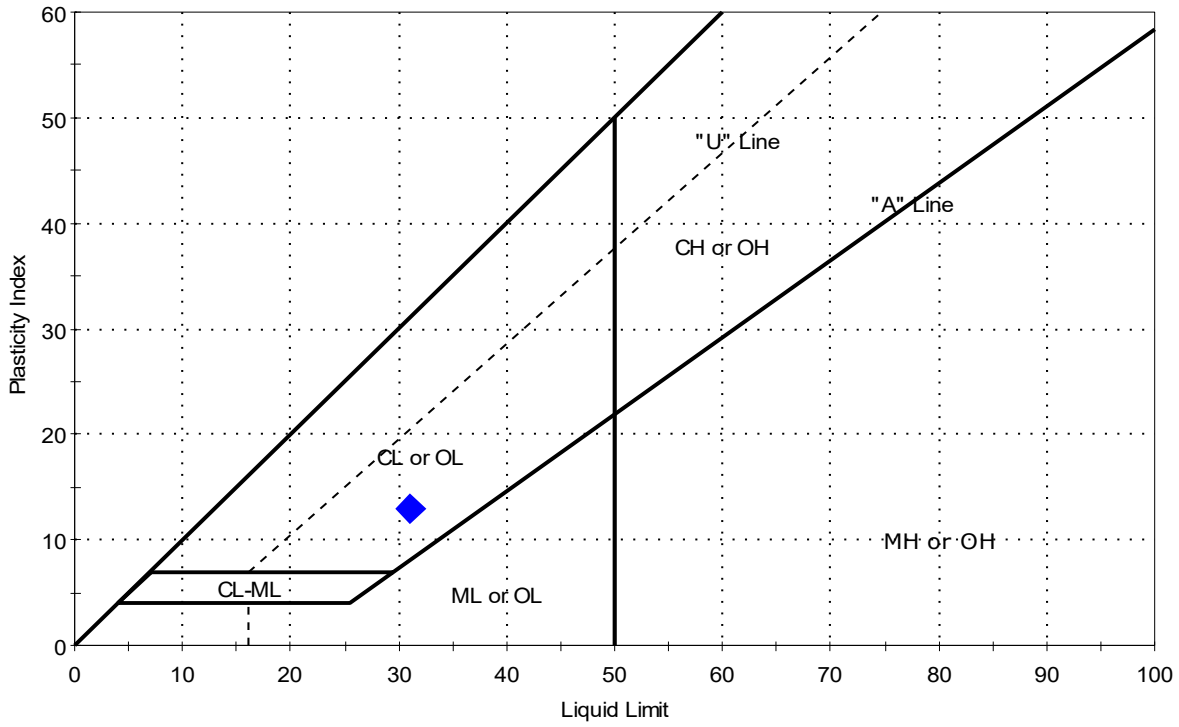
Dilatancy: RAPID

Toughness: n/a

The sample was determined to be Non-Plastic

Client:	Golder Associates USA, Inc.				
Project:	Hogan Road Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-101	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	05/18/22	Checked By:	bfs
Depth :	15-17	Test Id:	668245		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

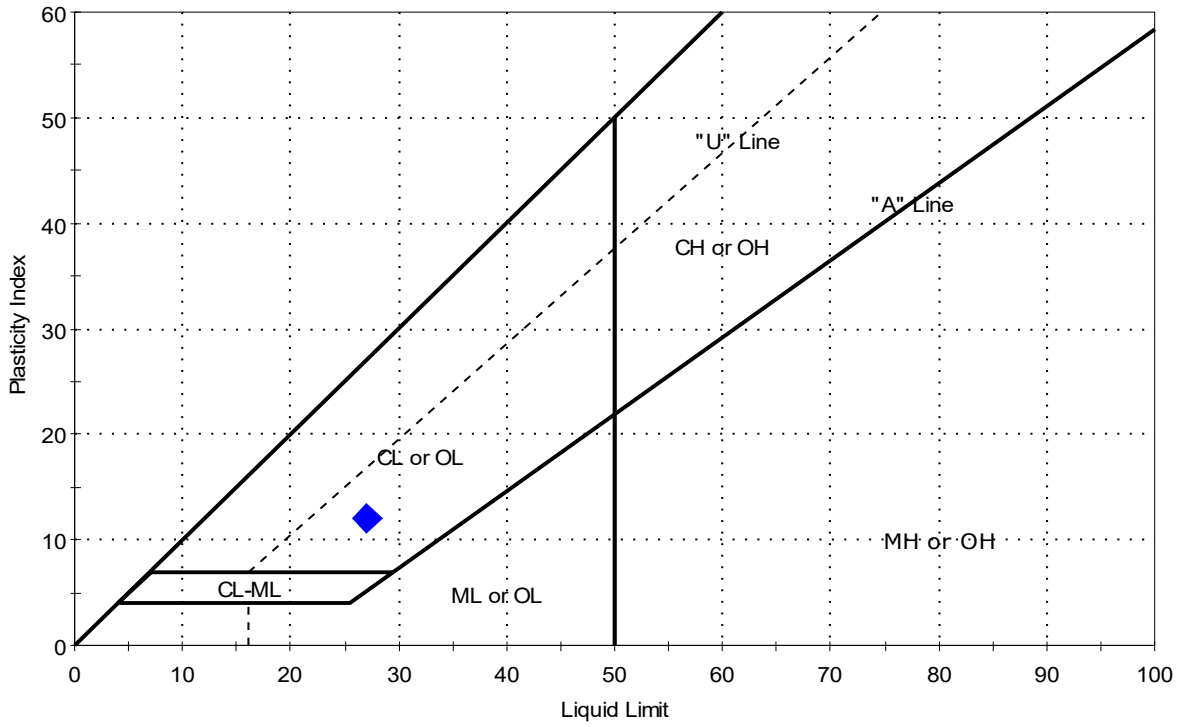


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	1U	B-BHR-10	15-17	30	31	18	13	0.9	Lean CLAY (CL)

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 Bridge				
Location:	Bangor, ME			Project No:	GTX-315431
Boring ID:	BB-BHR-103	Sample Type:	tube	Tested By:	cam
Sample ID:	1U	Test Date:	05/19/22	Checked By:	bfs
Depth :	15-16.2	Test Id:	668246		
Test Comment:	---				
Visual Description:	Wet, 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
◆	1U	B-BHR-10	15-16.2	31	27	15	12	1.3	Lean CLAY (CL)

Sample Prepared using the WET method
7% Retained on #40 Sieve
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#:	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.

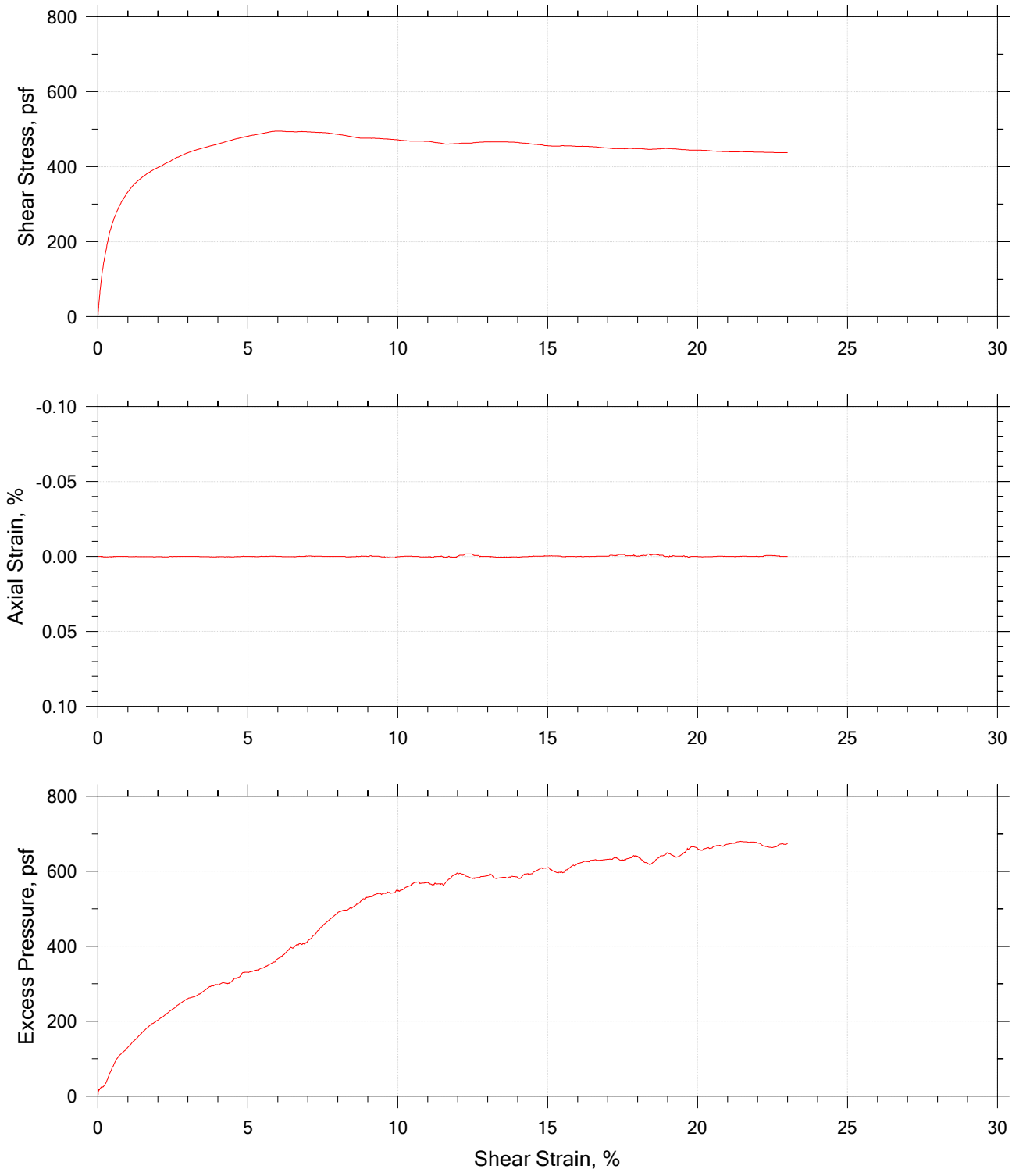
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	30.1				
Initial Dry Density, pcf	91.5				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	1,175				
Vertical Consolidation Stress at time of shear, psf	1,175				
Final 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

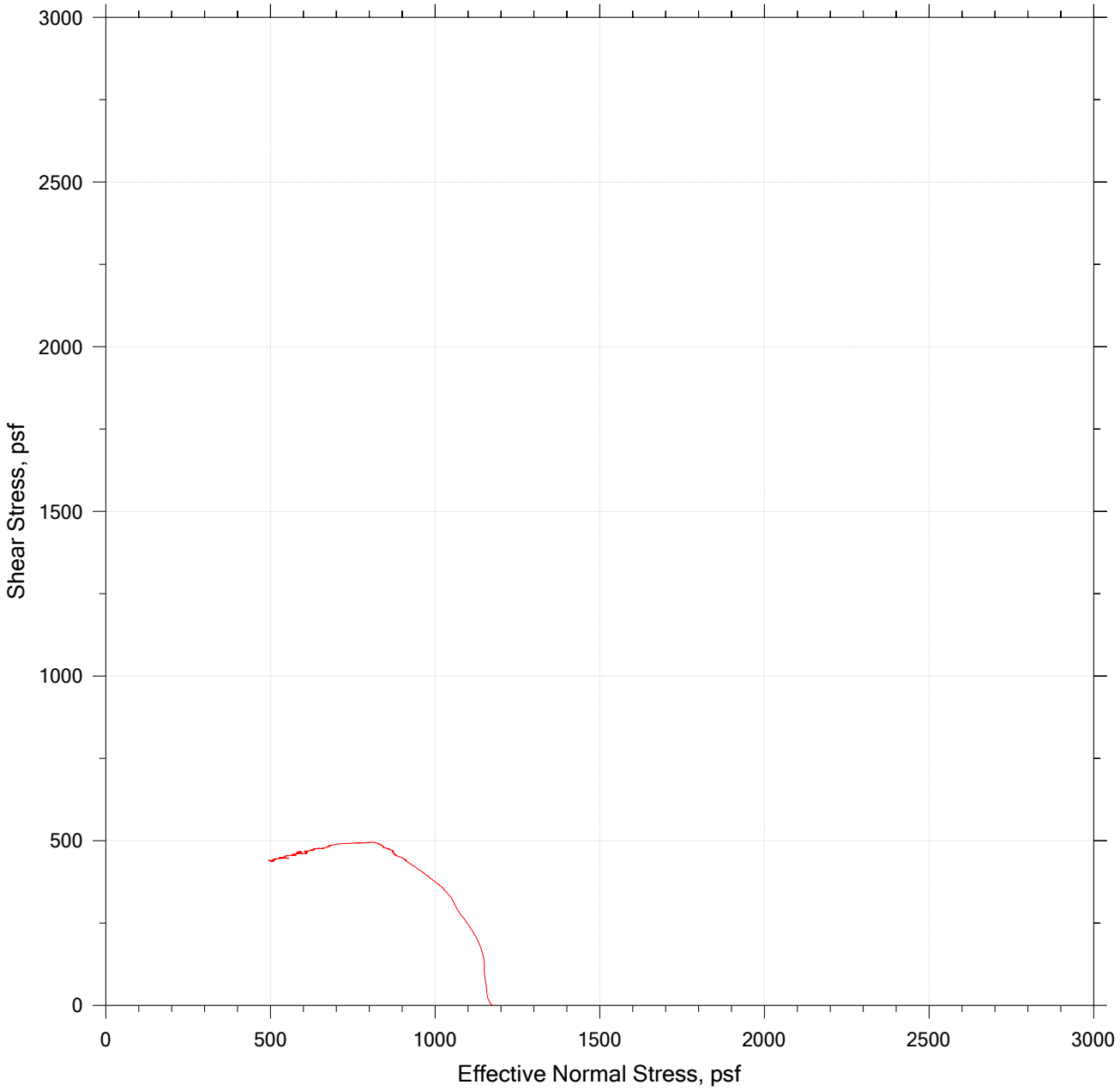
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

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:		

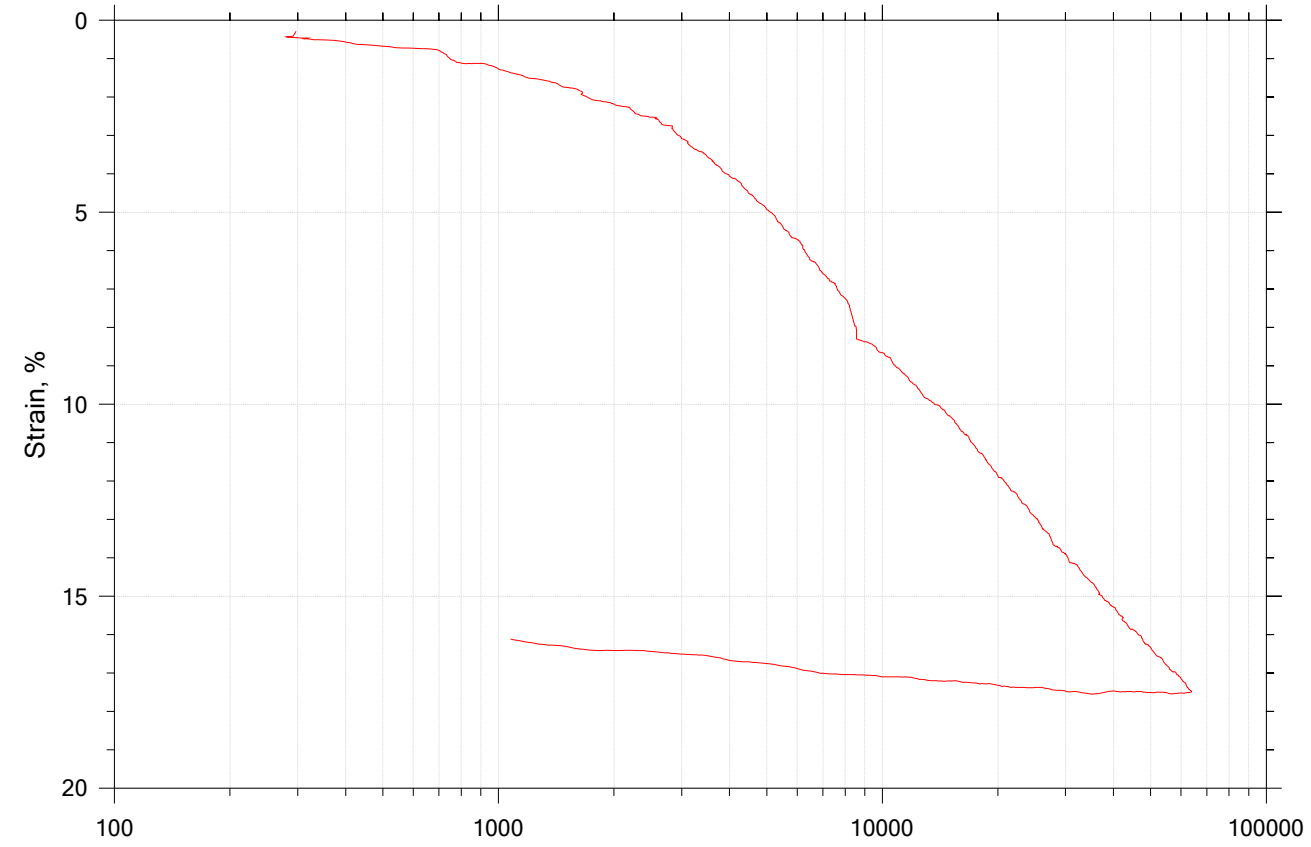
DIRECT SIMPLE SHEAR TEST by ASTM D6528

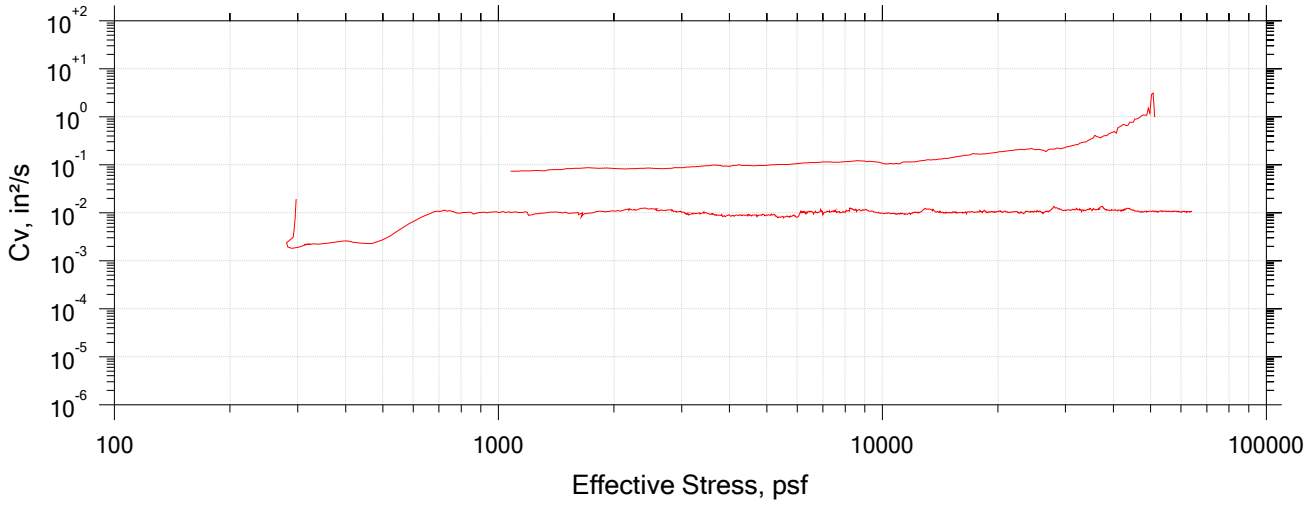


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: njh
	Sample No.: 1U	Test Date: 05/17/22	Depth: 15-17
	Test No.: DSS-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks:		

CRC Test

Summary

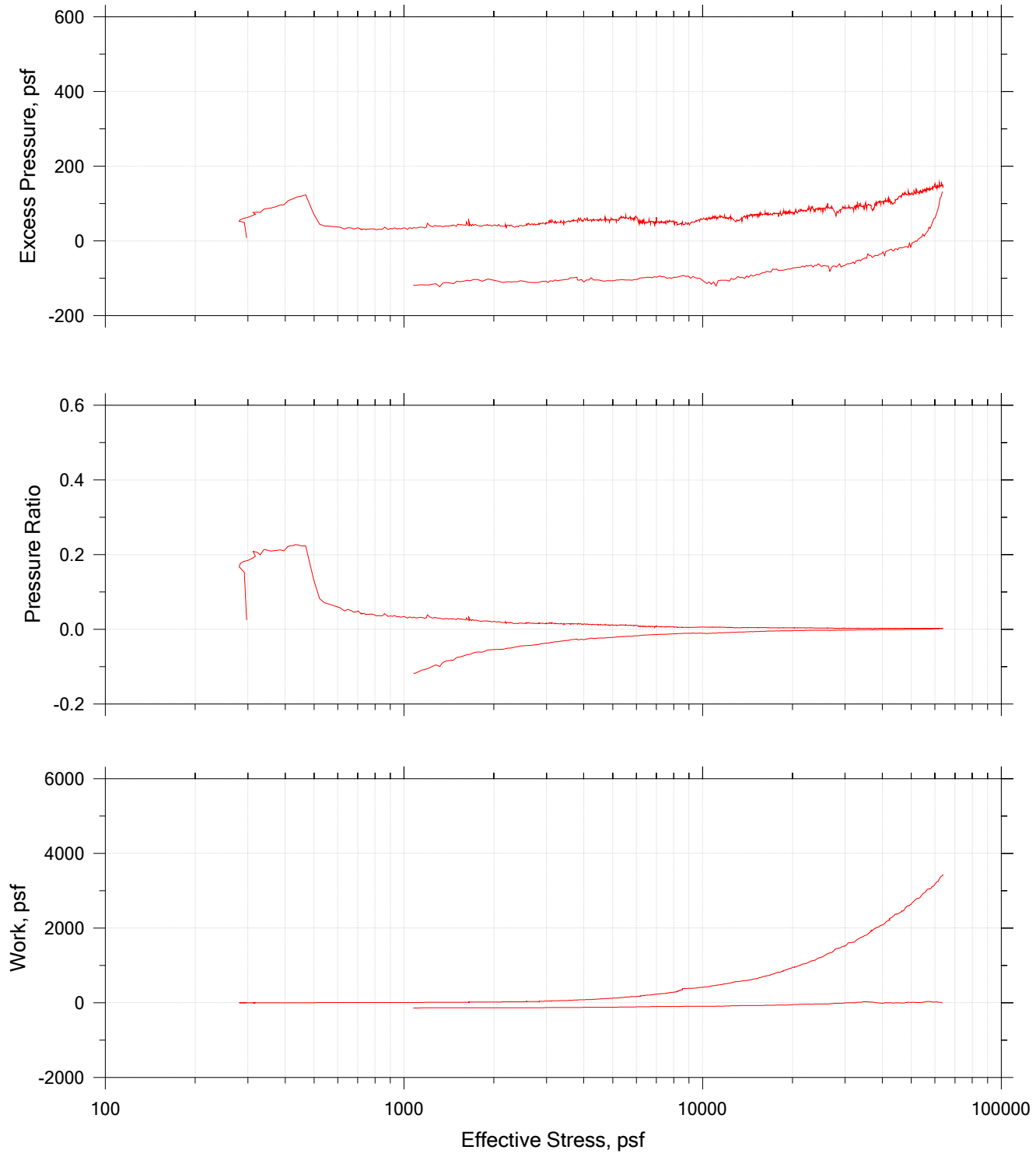




Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-31315431
Boring Number: BB-BHR-101	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 05/17/22	Depth: 15-17
Test Number: CRC-1	Preparation: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-003		

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

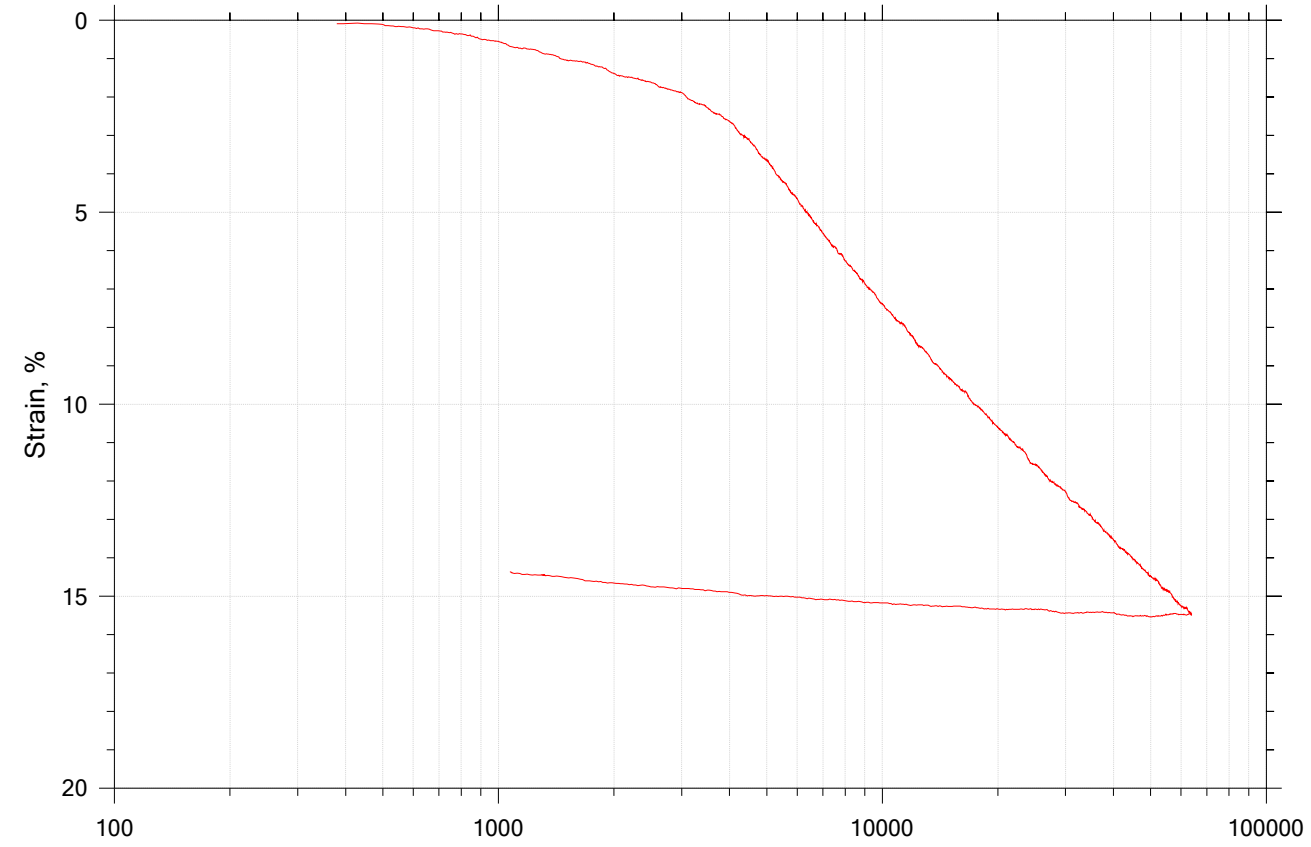
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-2020	---		E3267
Mass Container, gm	8.29	111.06	111.06	8.2
Mass Container + Wet Soil, gm	210	268.99	259.6	155.29
Mass Container + Dry Soil, gm	163.3	233.03	233.03	128.98
Mass Dry Soil, gm	155.01	121.97	121.97	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	---

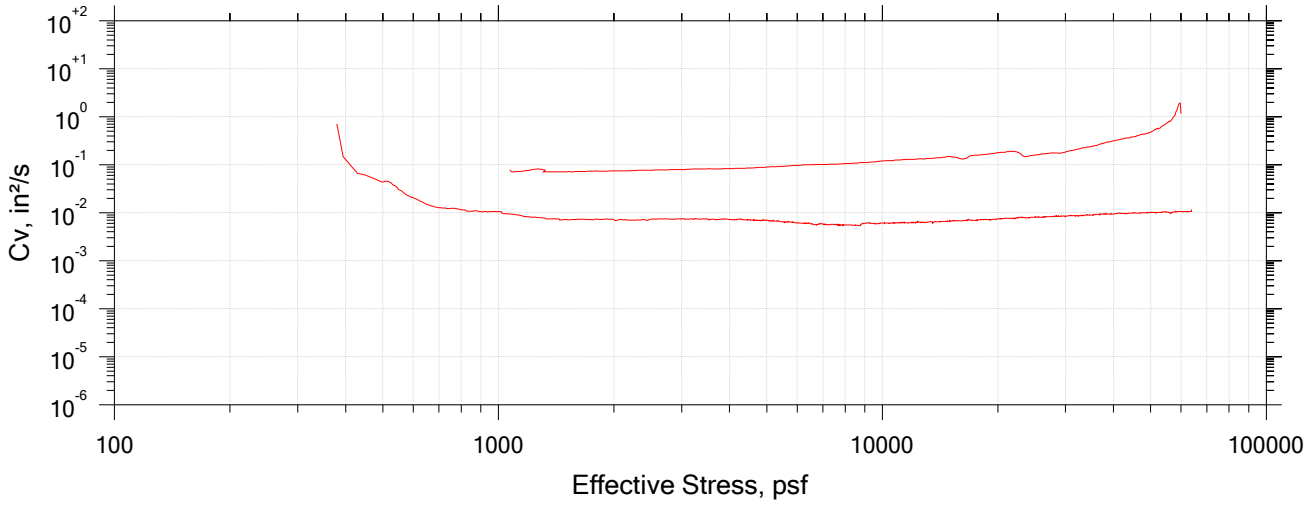
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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

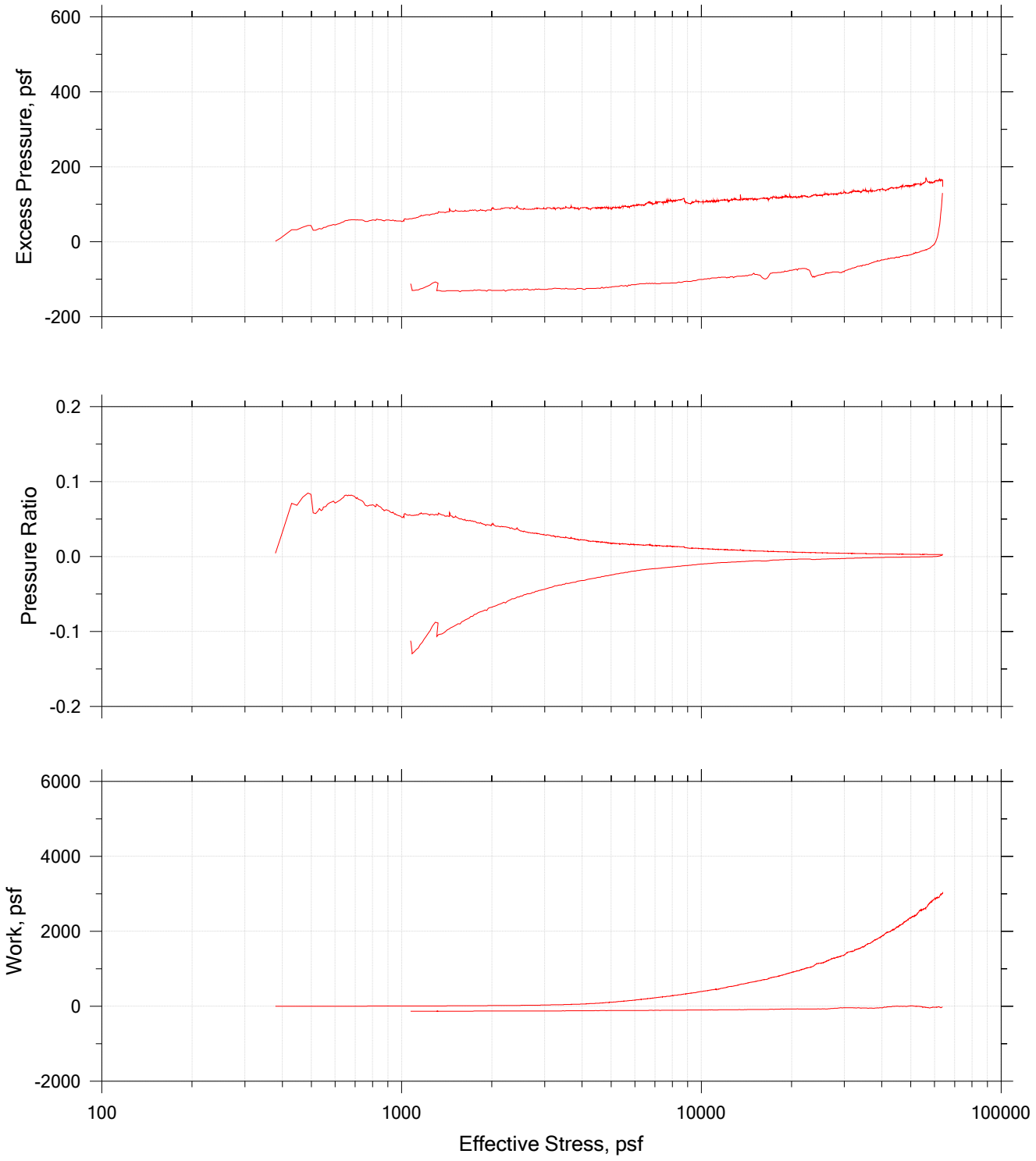




Project Name: Hogan Road Bridge	Location: Bangor, ME	Project Number: GTX-315431
Boring Number: BB-BHR-103	Tester: md	Checker: njh
Sample Number: 1U	Test Date: 05/17/22	Depth: 15-16.2
Test Number: CRC-2	Preparation: intact	Elevation:
Description: Moist, gray clay		
Remarks: System JJ		

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

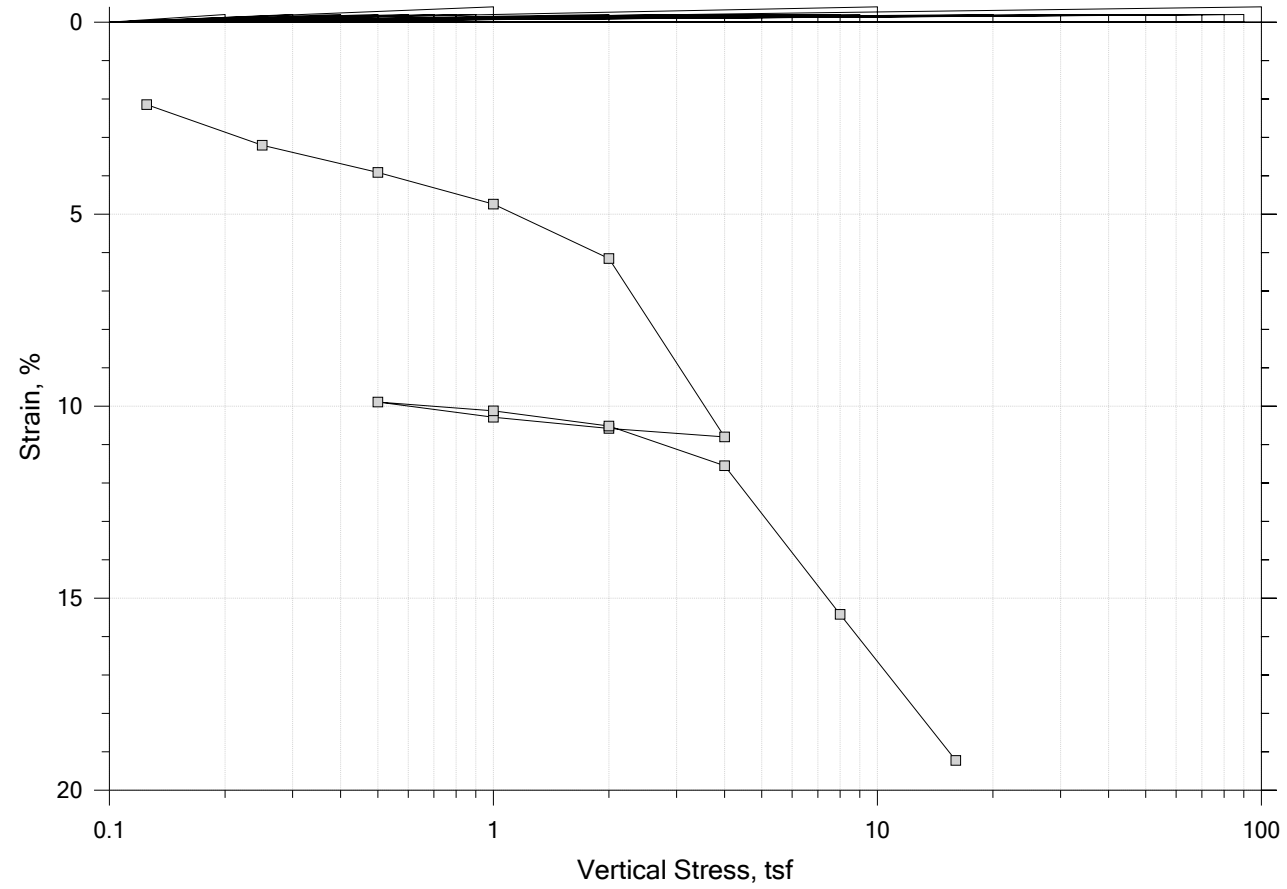
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-4975	---		E3466
Mass Container, gm	8.01	109.76	109.76	8.32
Mass Container + Wet Soil, gm	235.9	268.97	256.54	156.54
Mass Container + Dry Soil, gm	187.49	233.92	233.92	133.7
Mass Dry Soil, gm	179.48	124.16	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	---

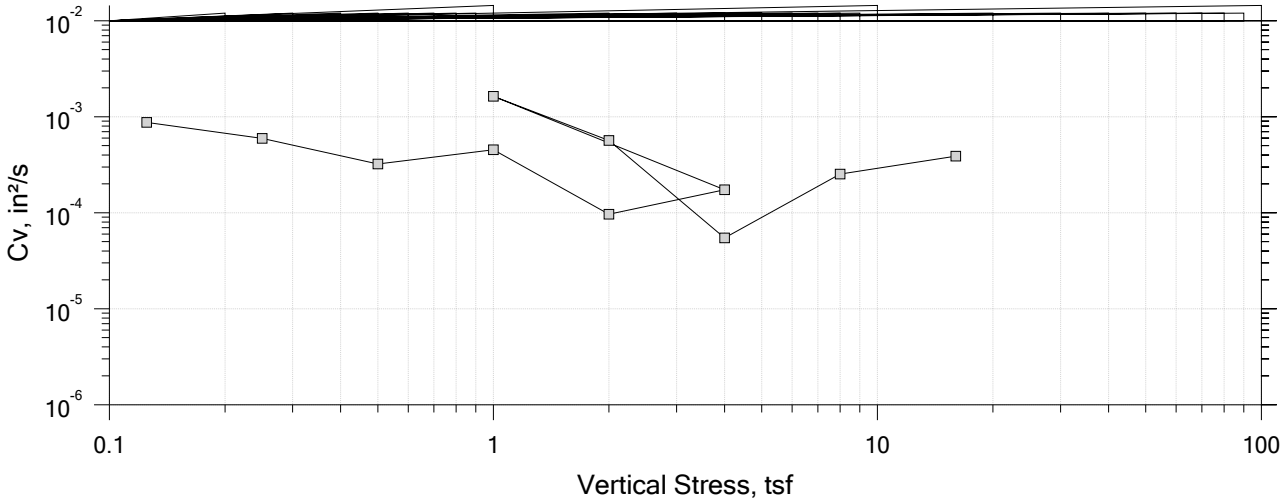
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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

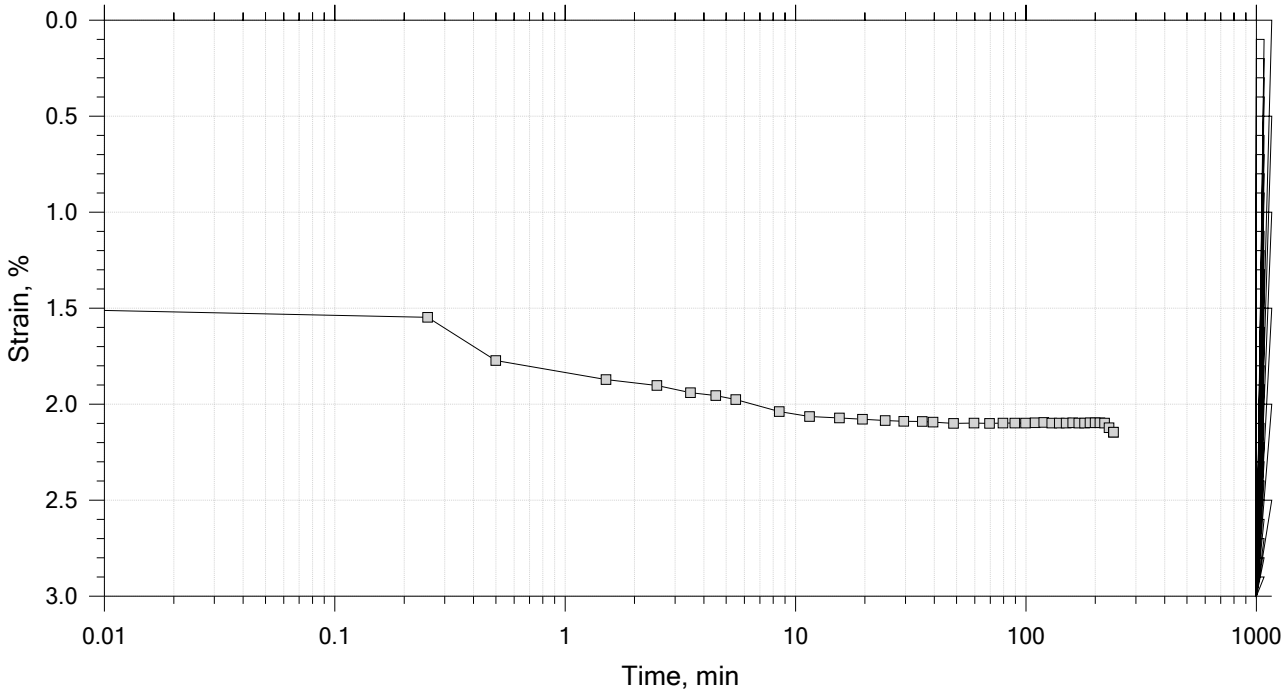


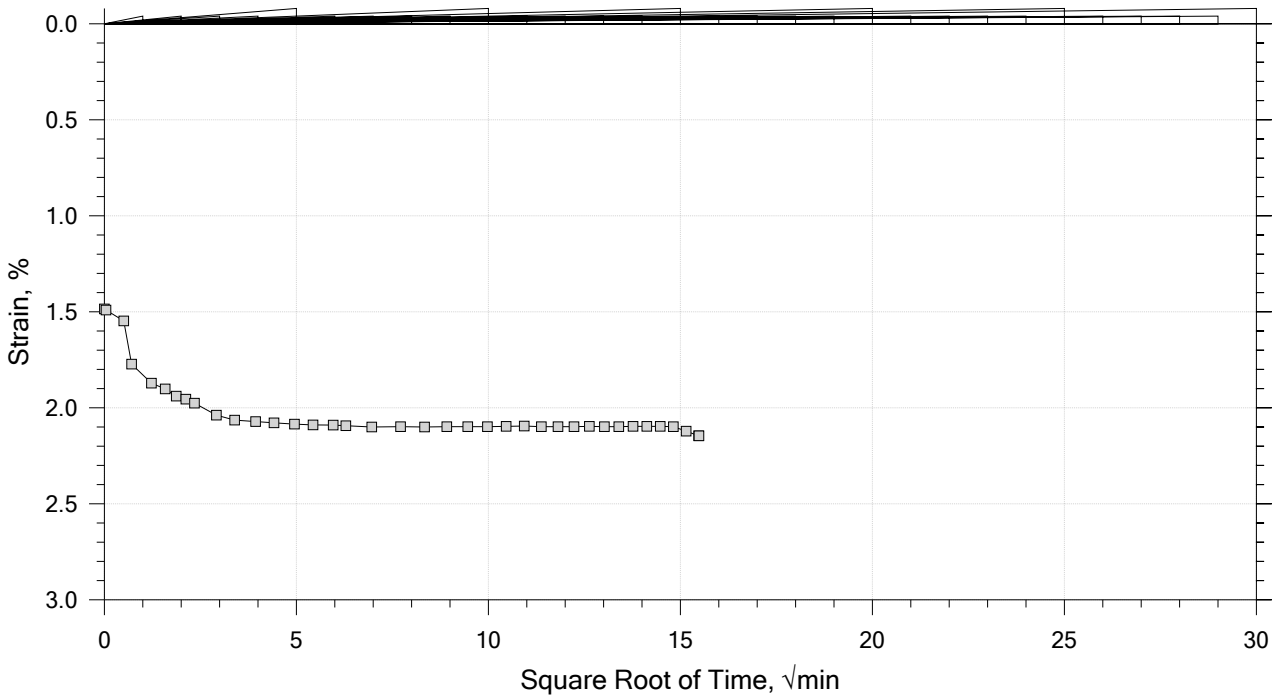


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		
	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





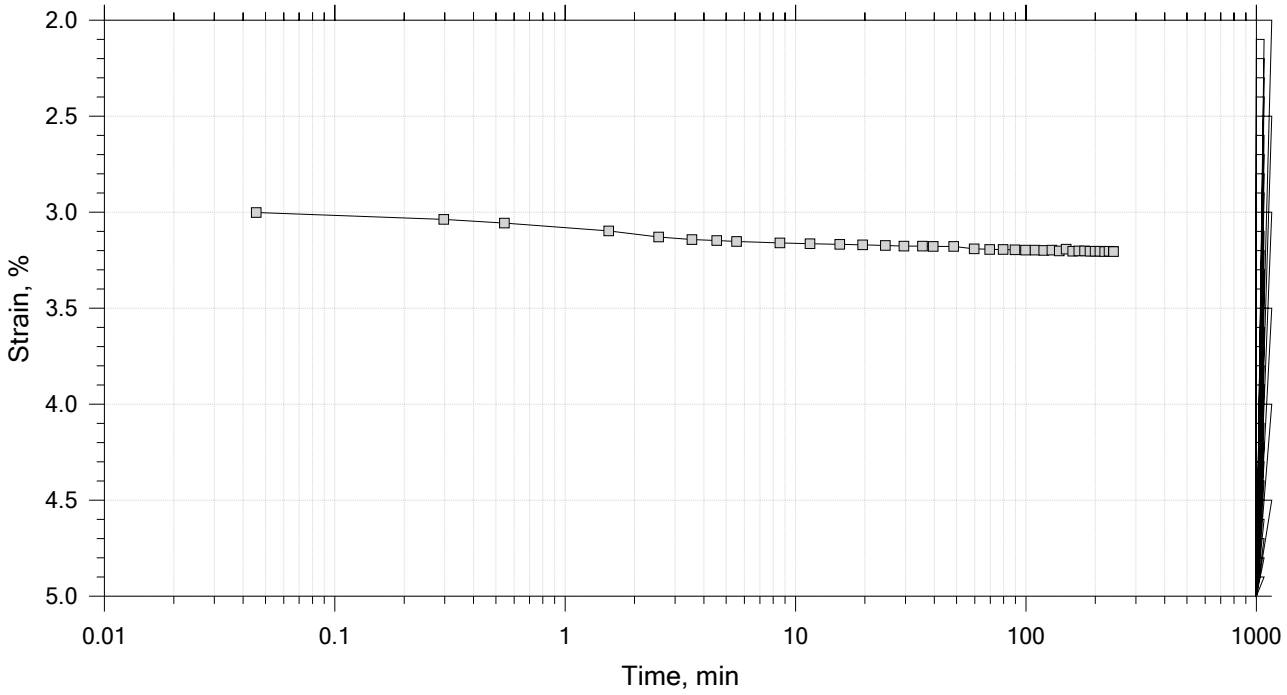
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

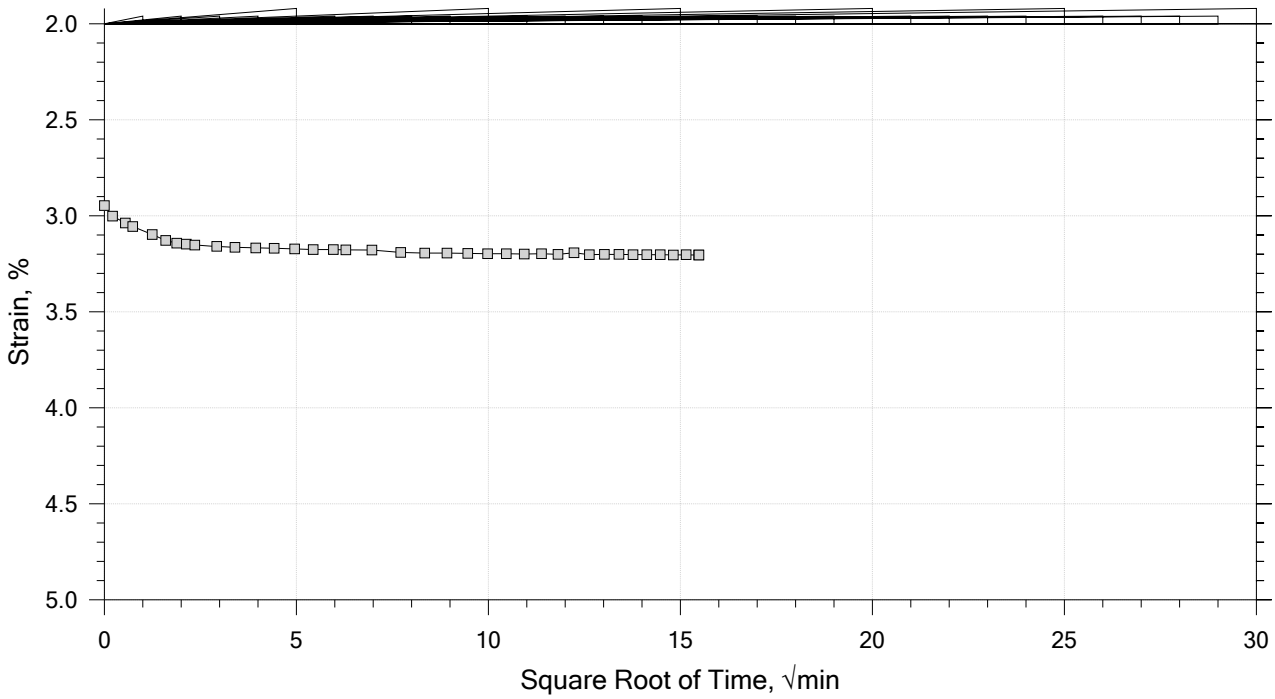
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14

Constant Load Step

Stress: 0.25 tsf

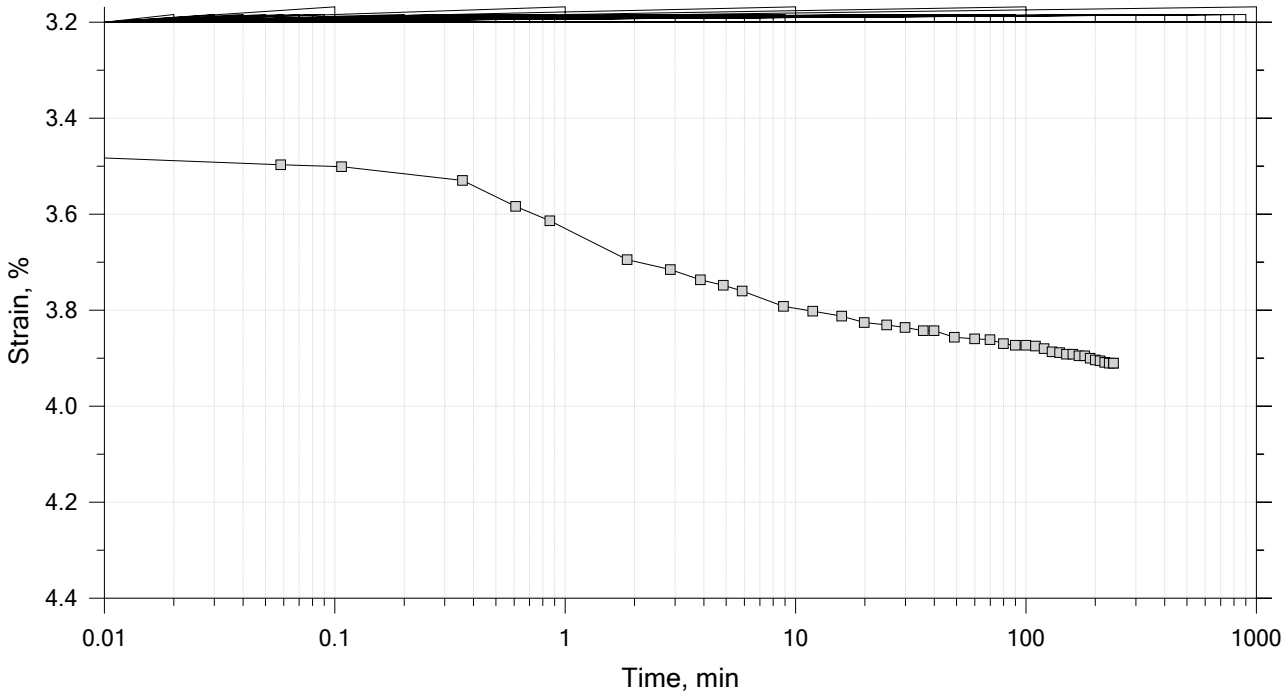


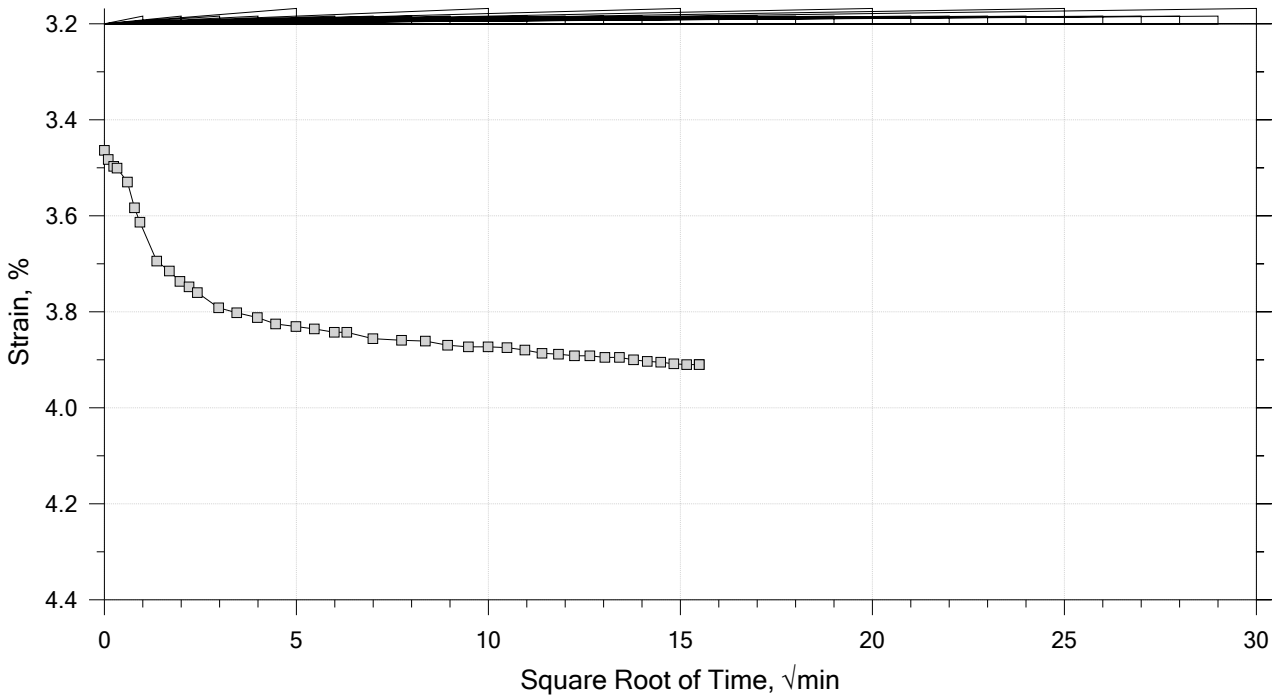


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf





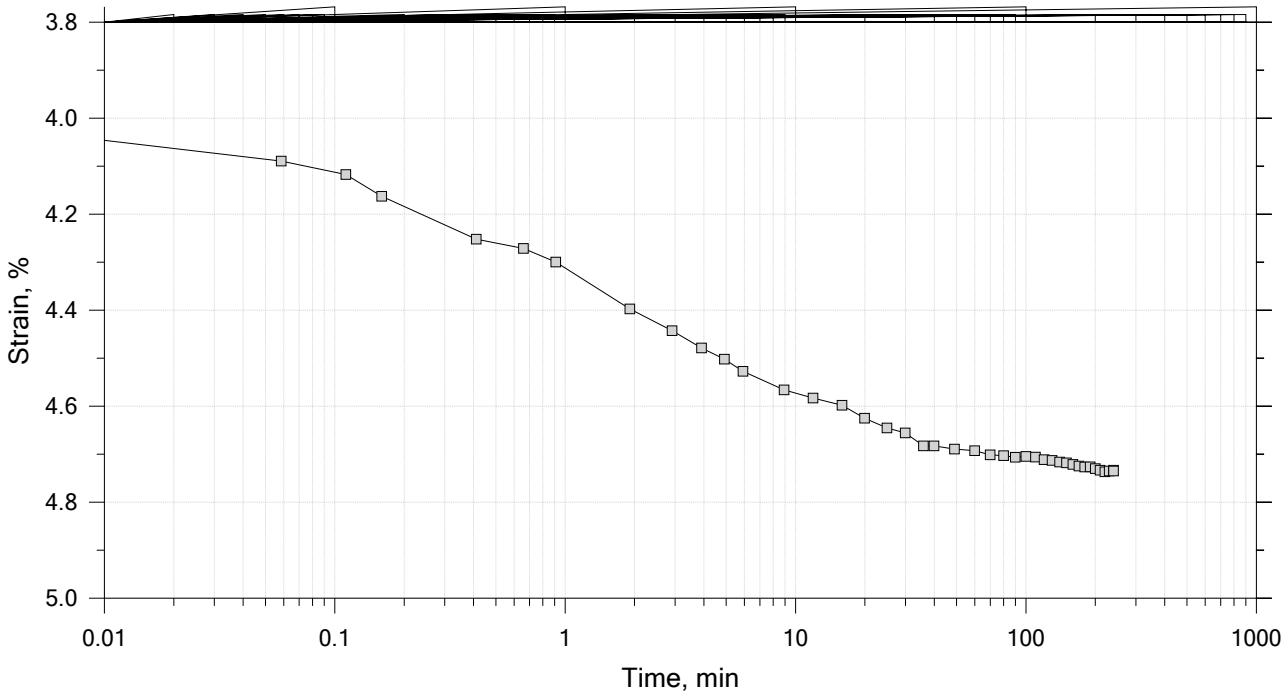
Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B		

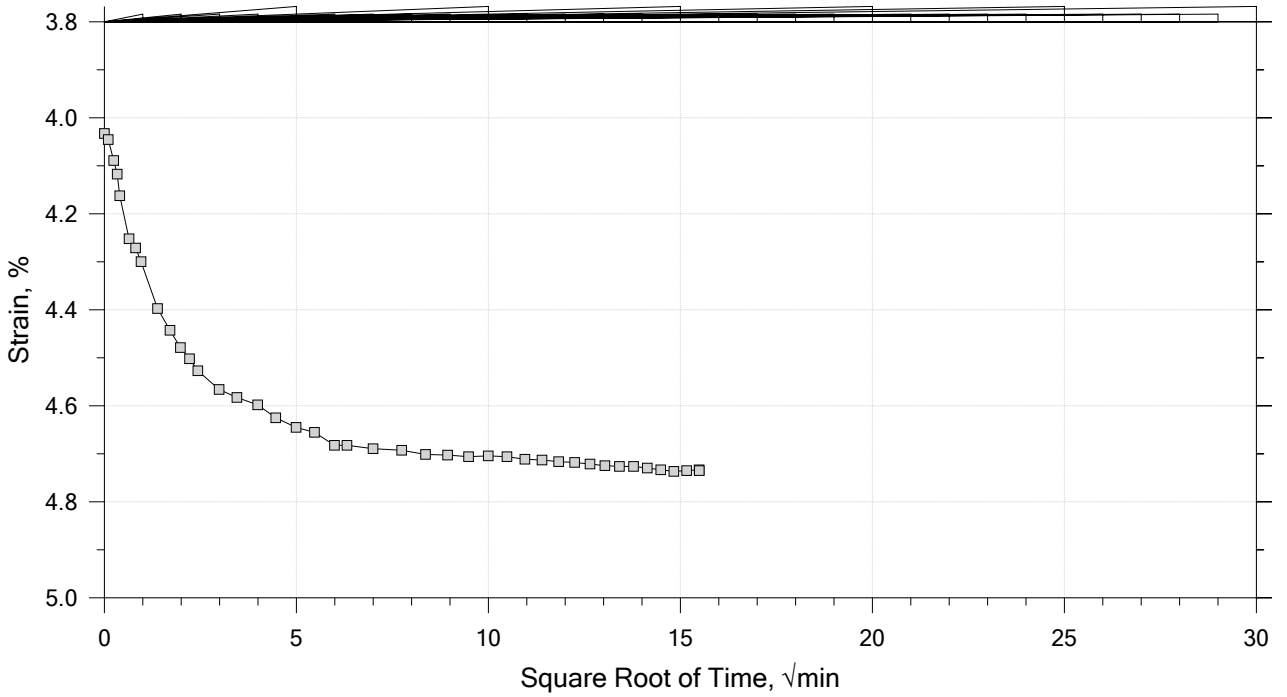
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 1 tsf

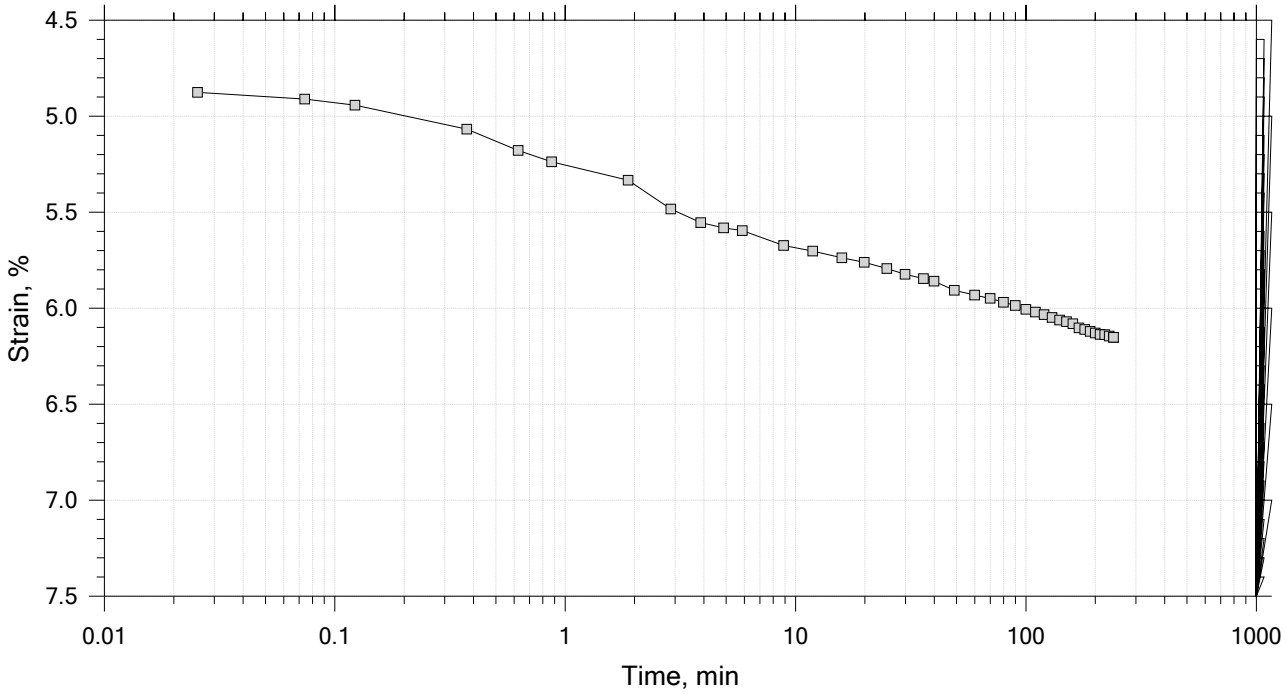


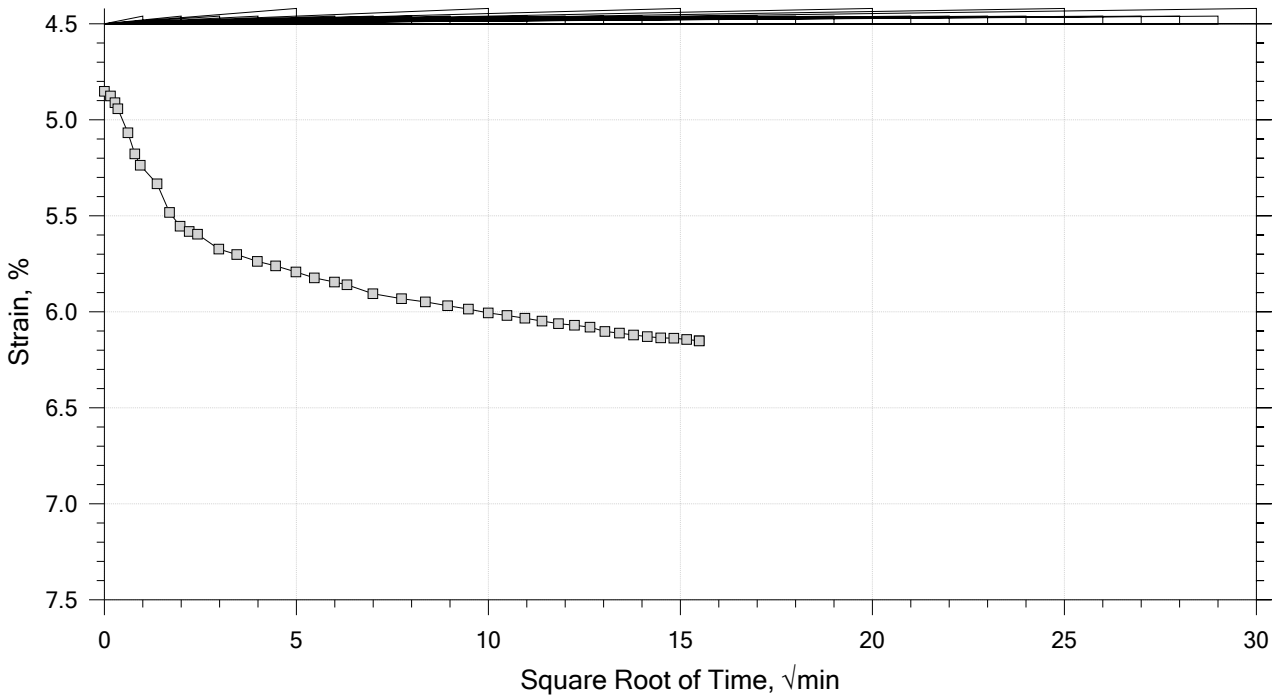


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14
Constant Load Step
Stress: 2 tsf





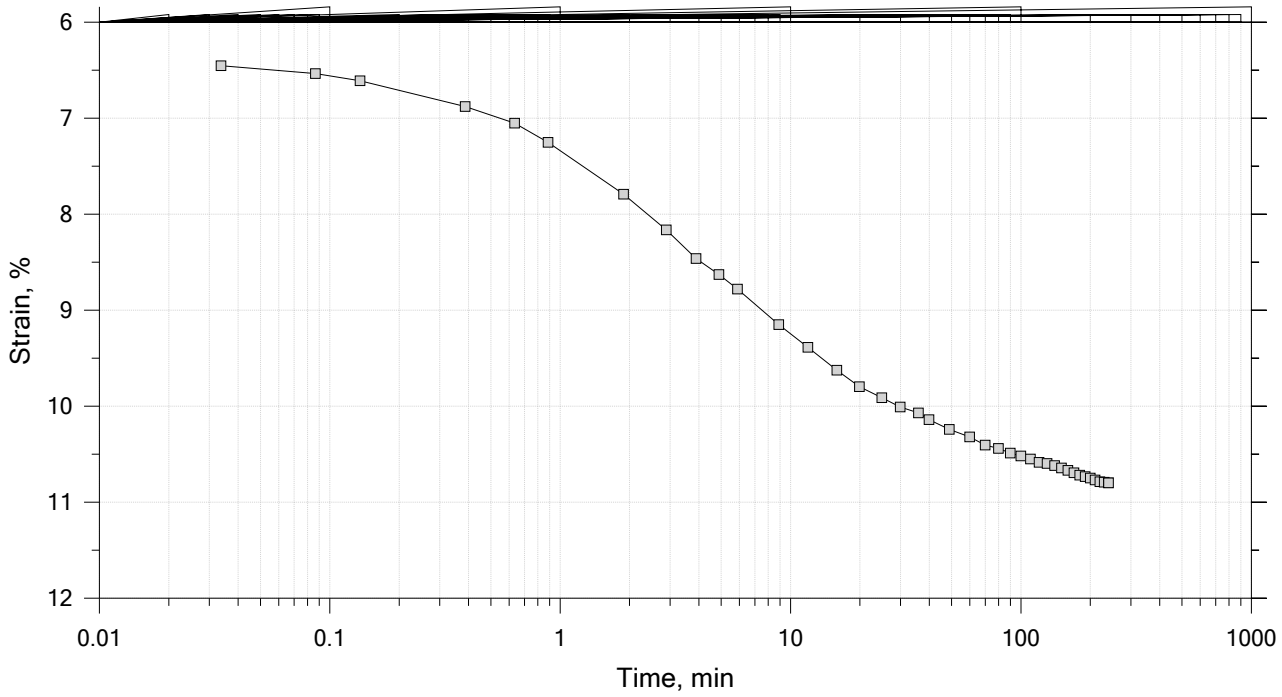
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

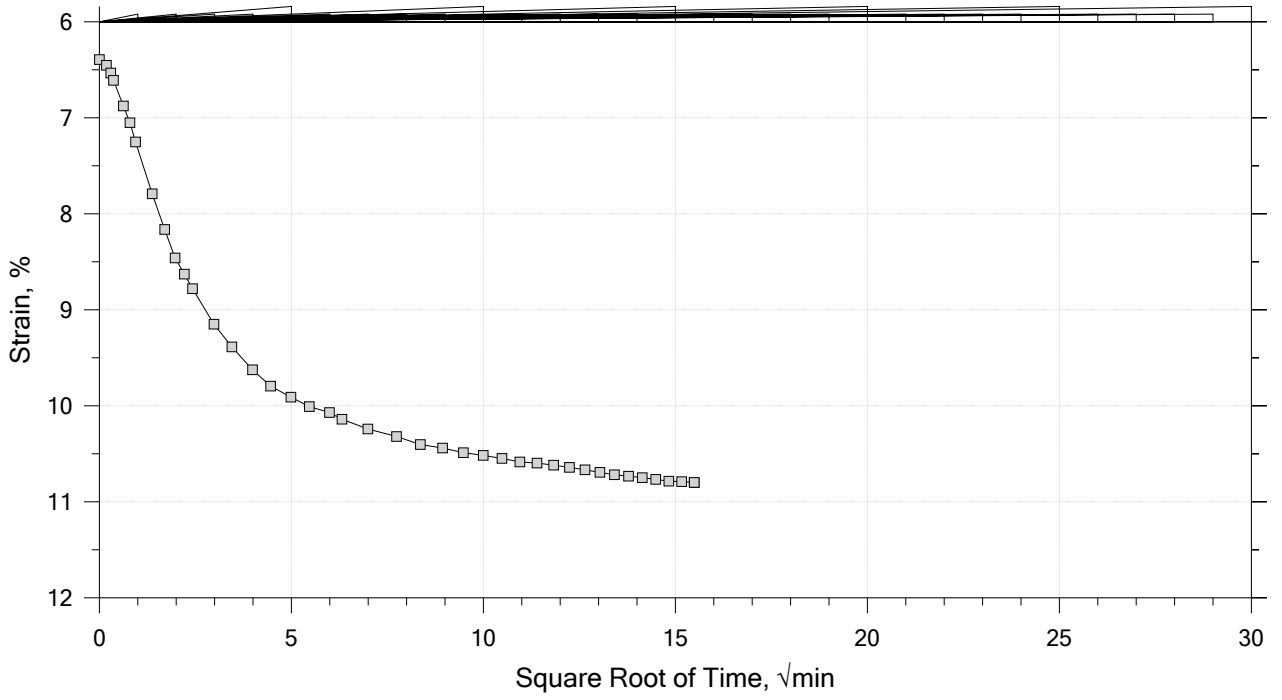
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 4 tsf





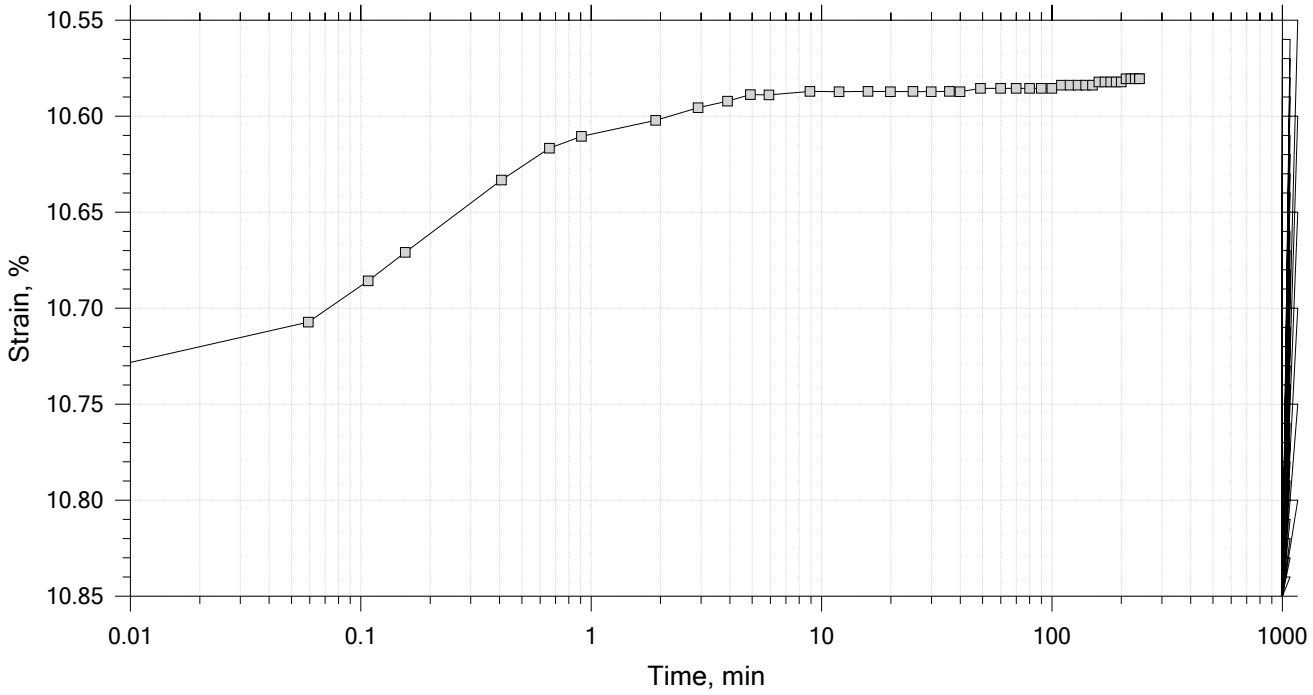
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

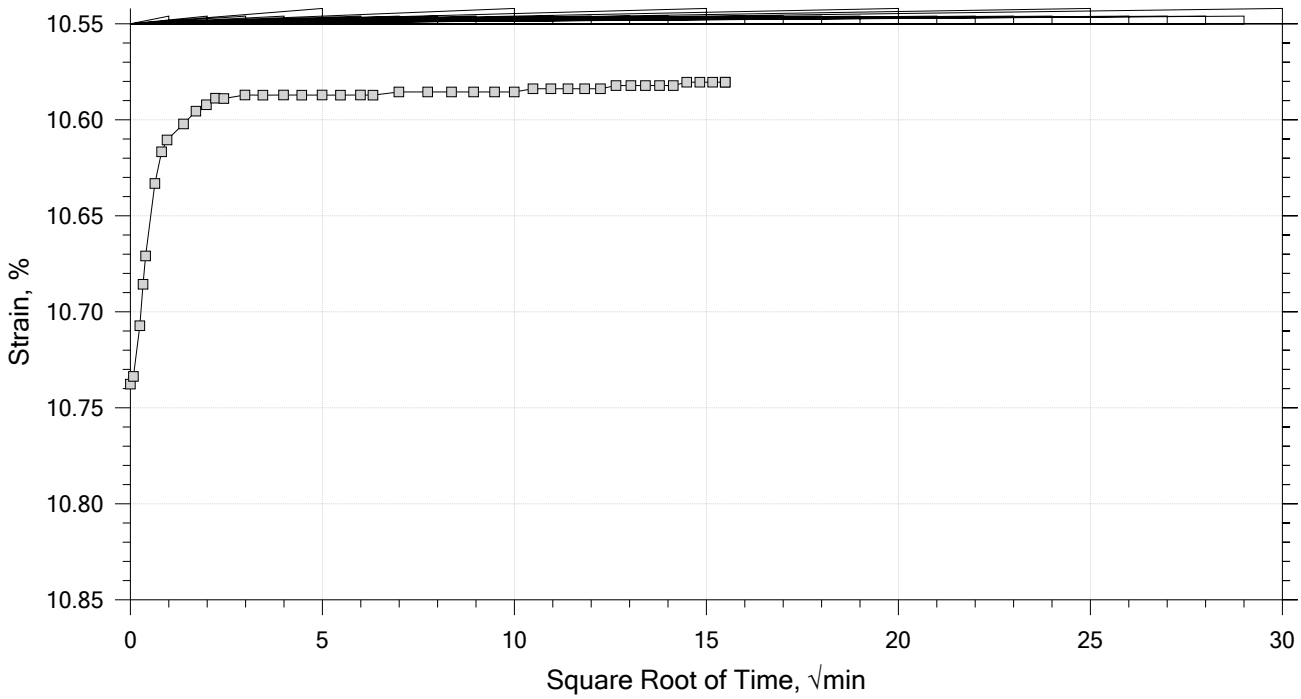
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14

Constant Load Step

Stress: 2 tsf





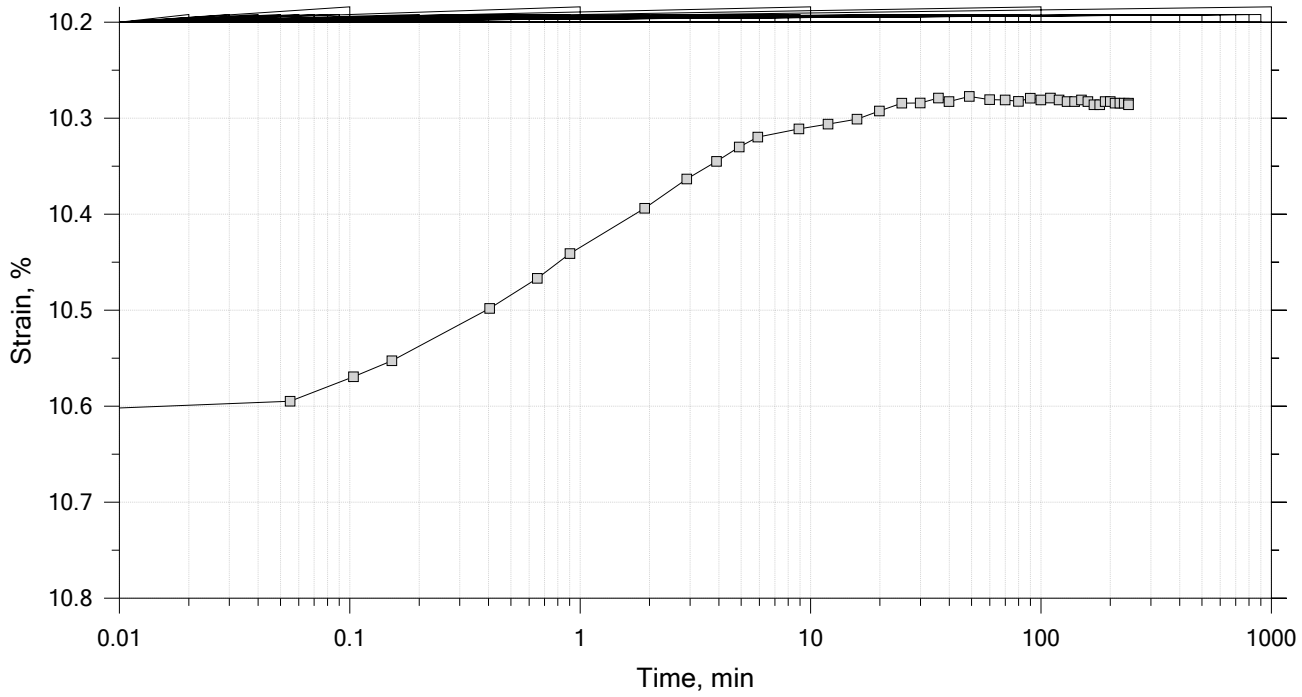
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

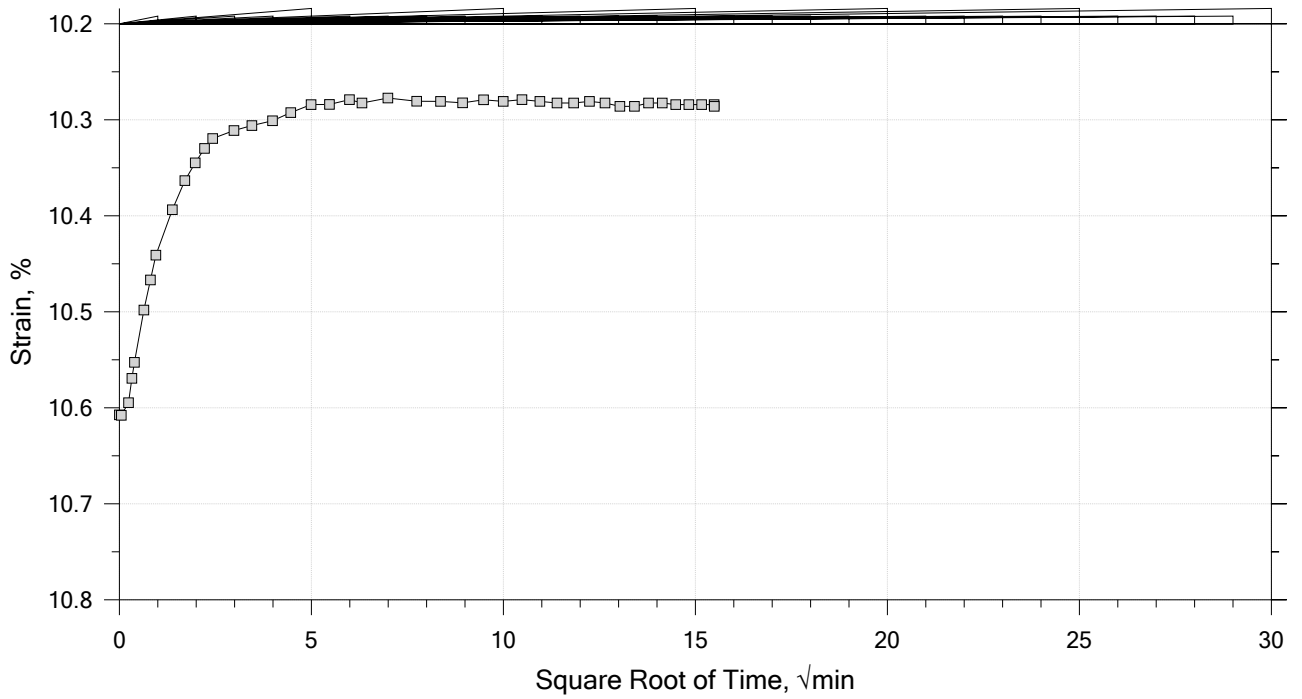
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf





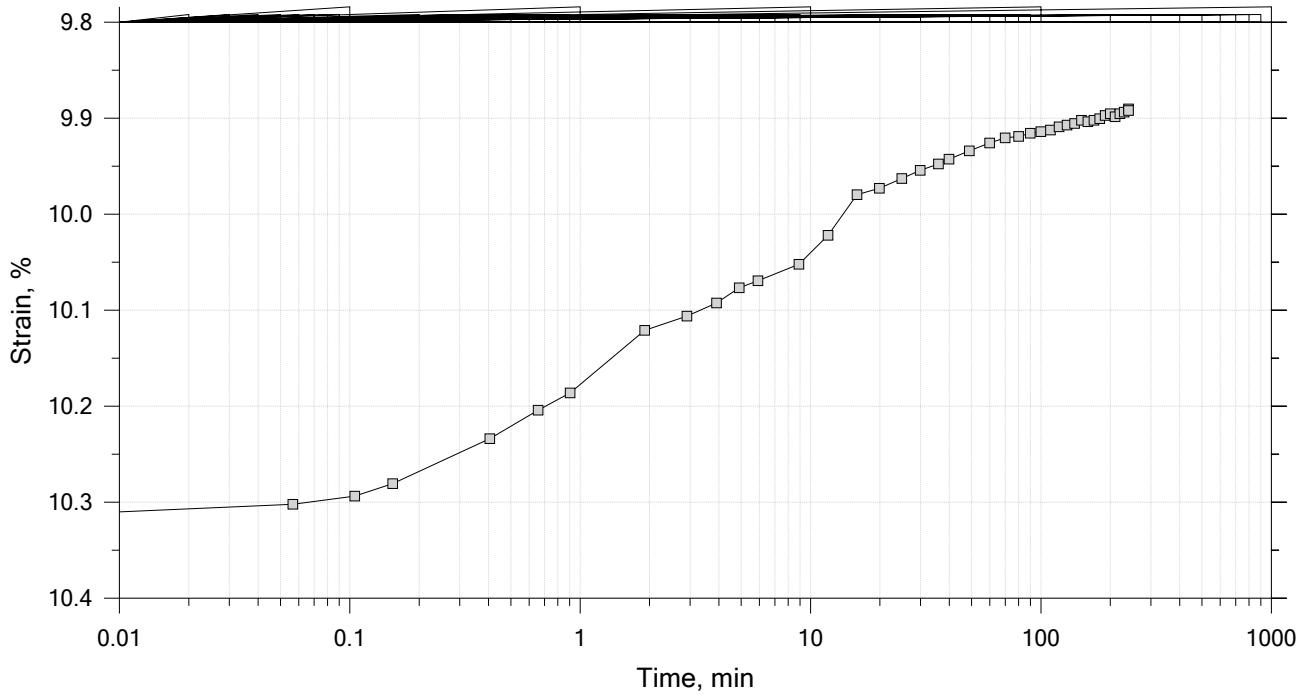
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

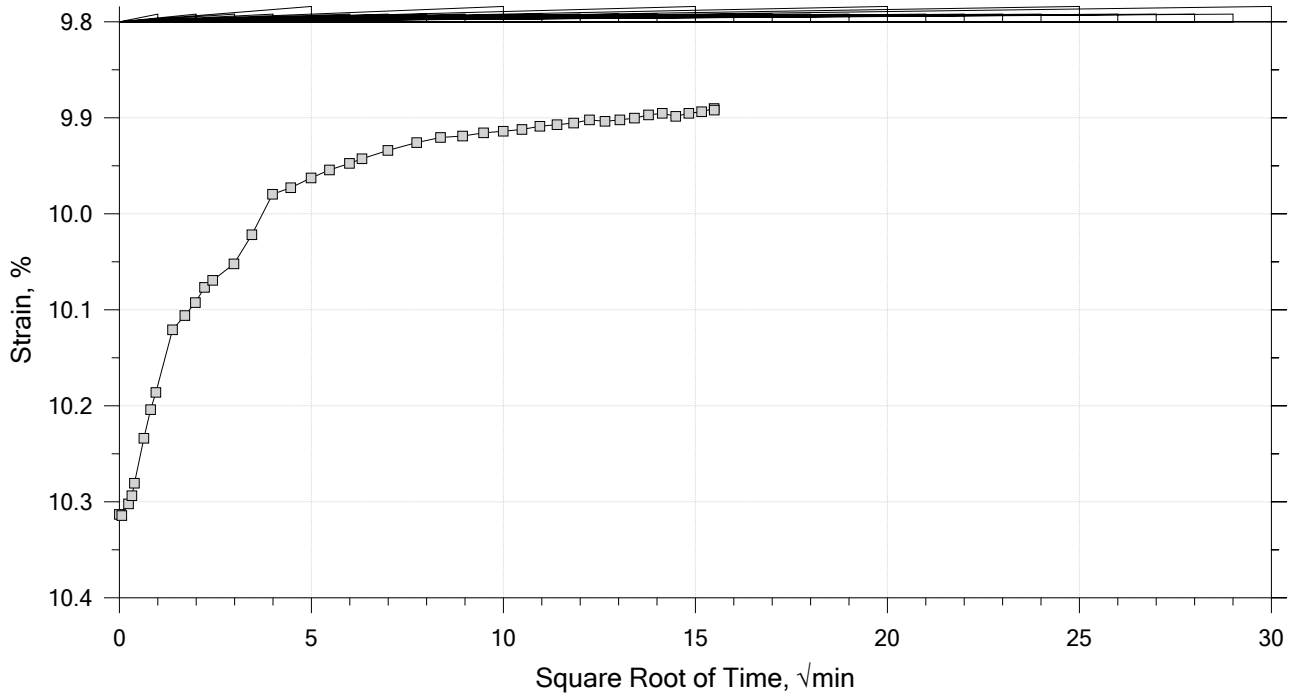
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14

Constant Load Step

Stress: 0.5 tsf





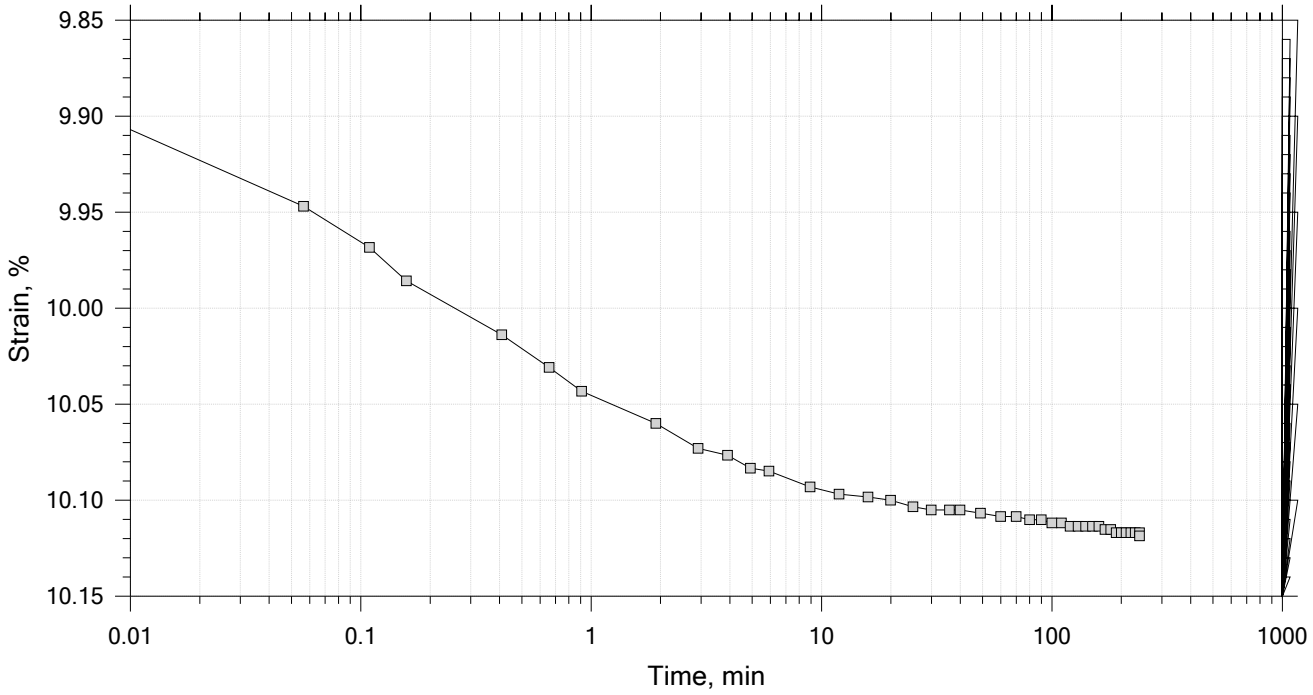
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

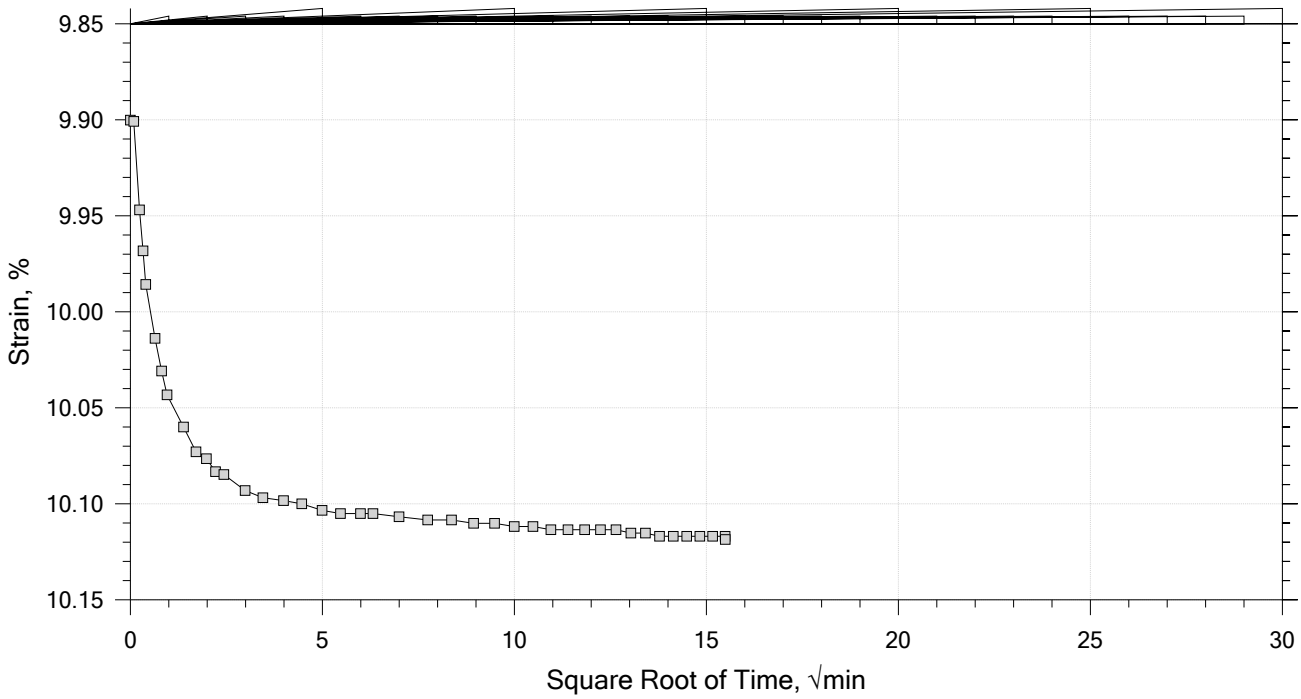
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14

Constant Load Step

Stress: 1 tsf

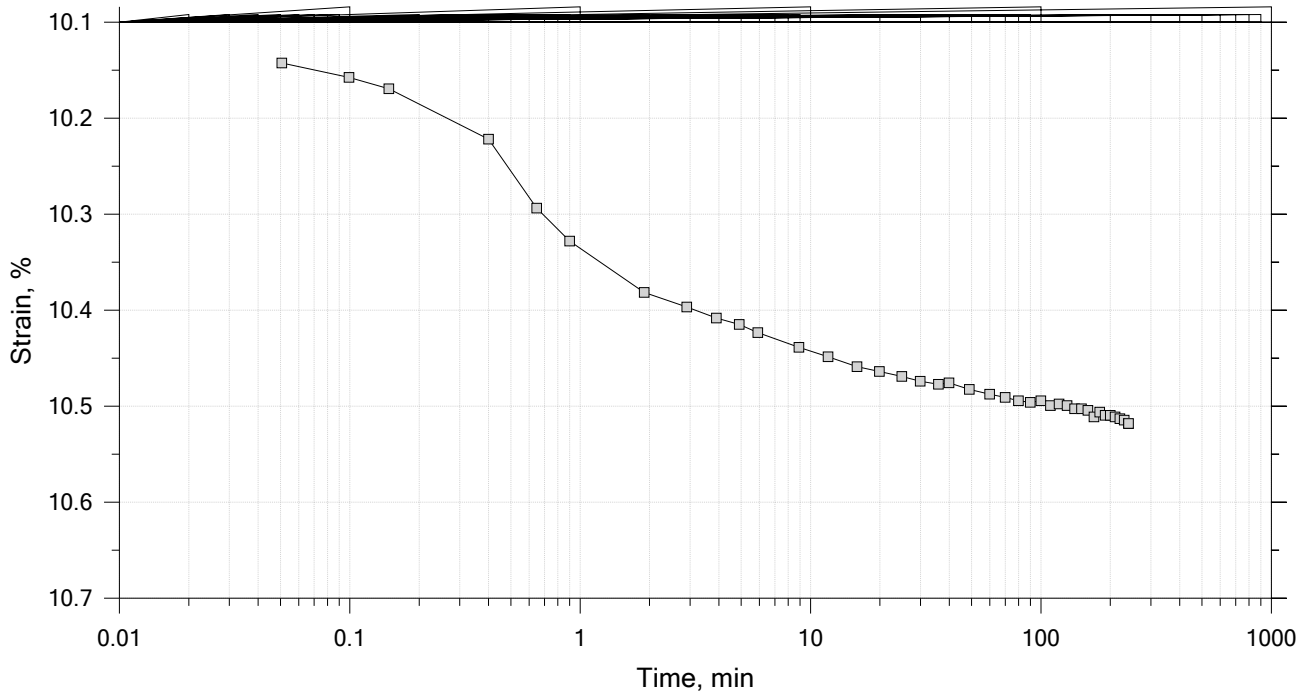


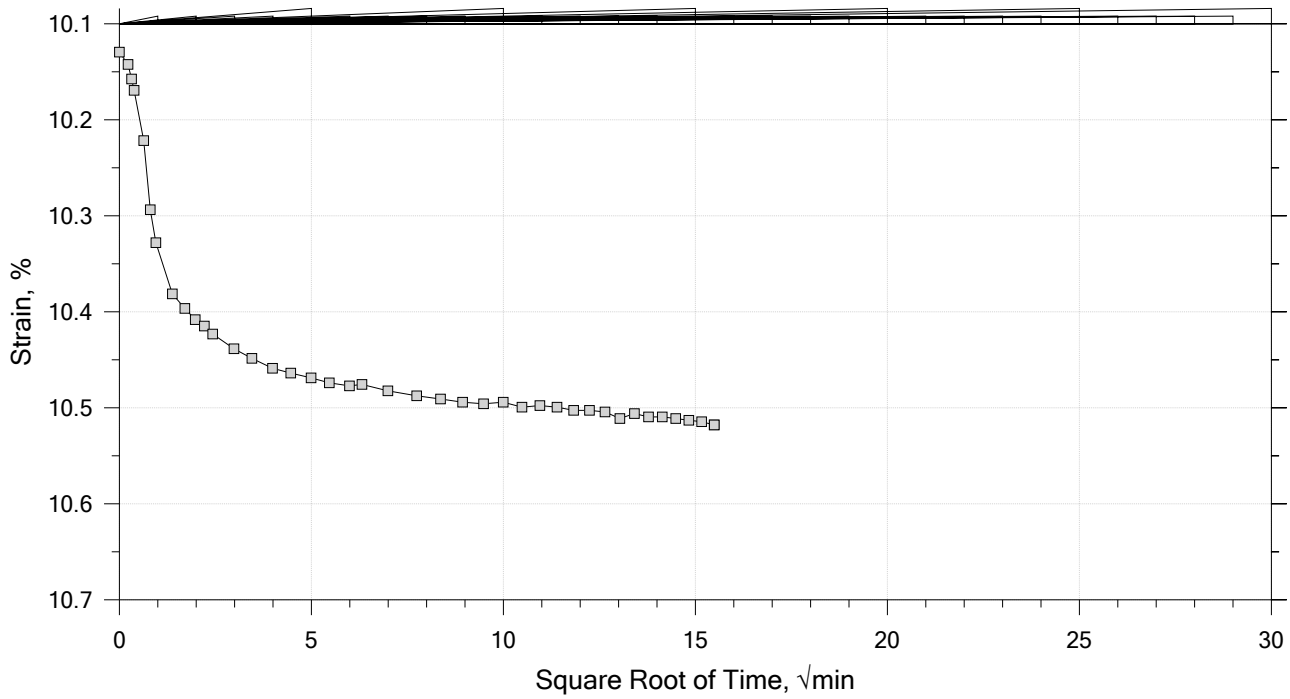


Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

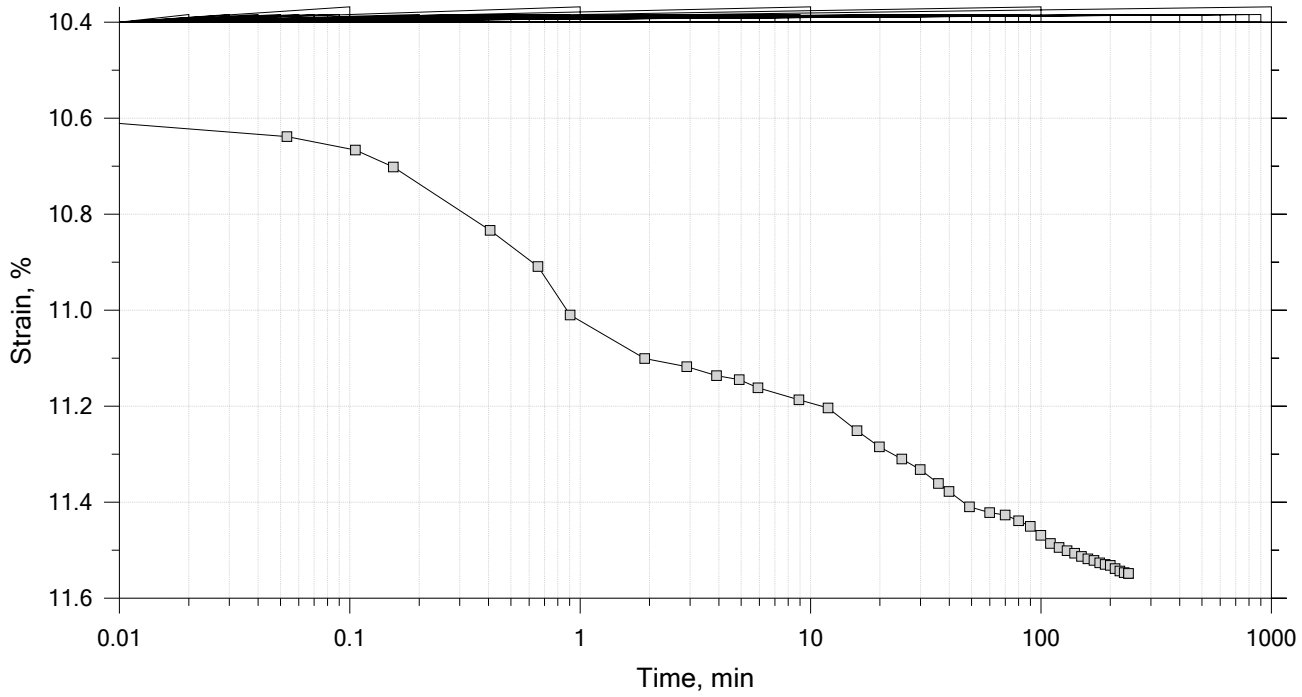


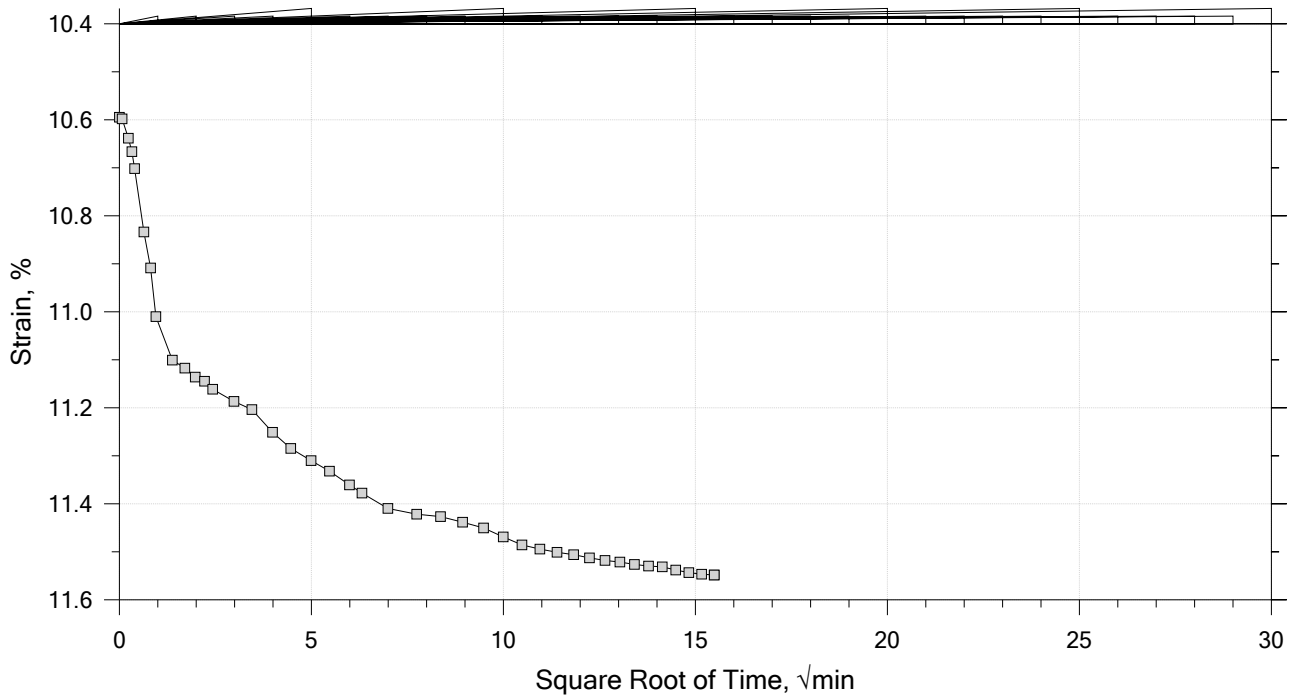


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf





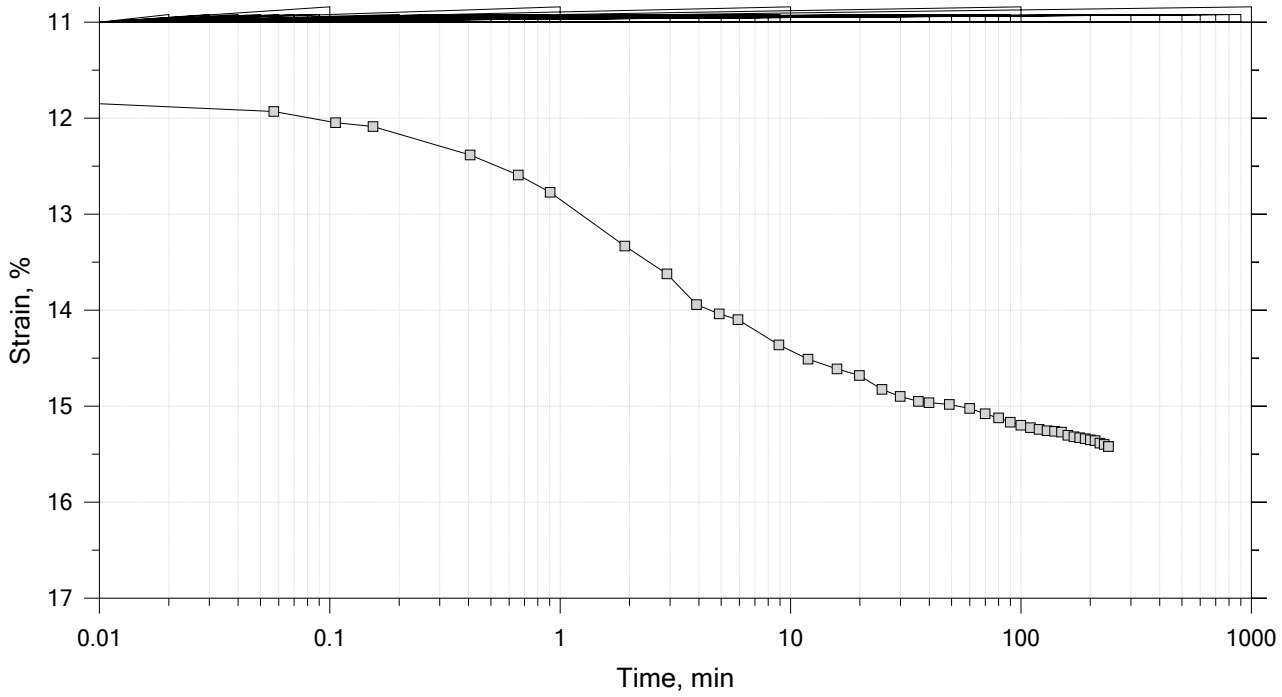
GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

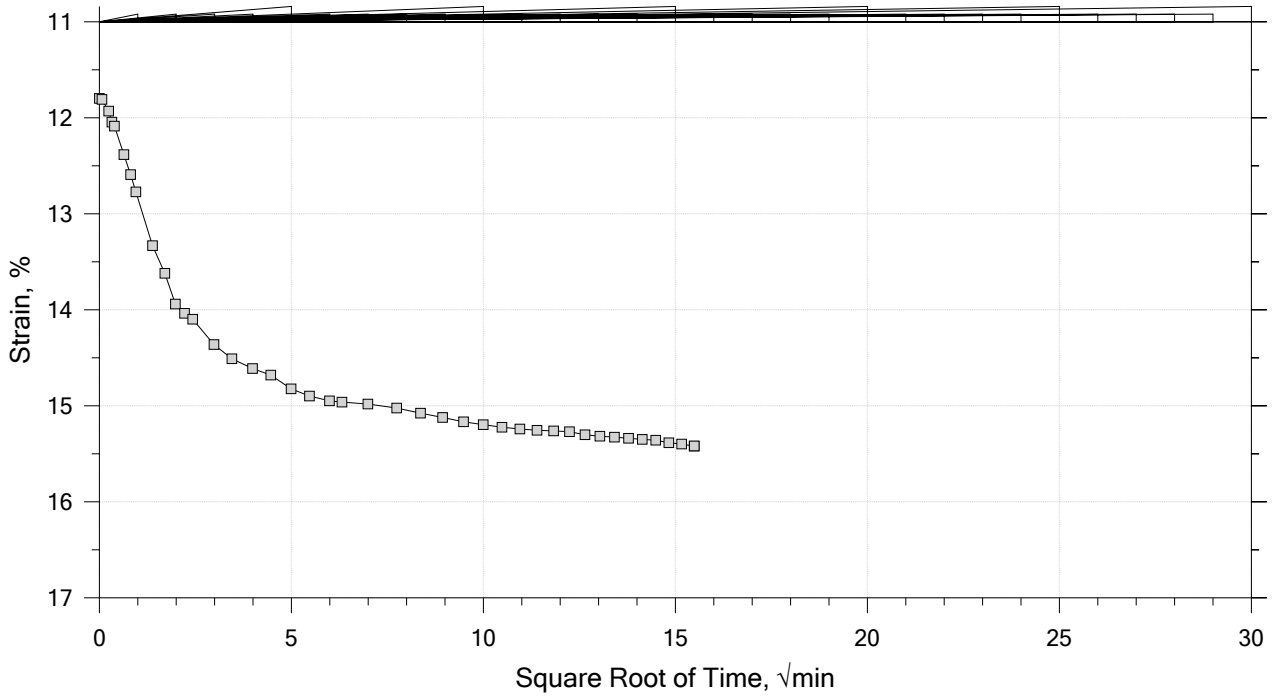
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 8 tsf

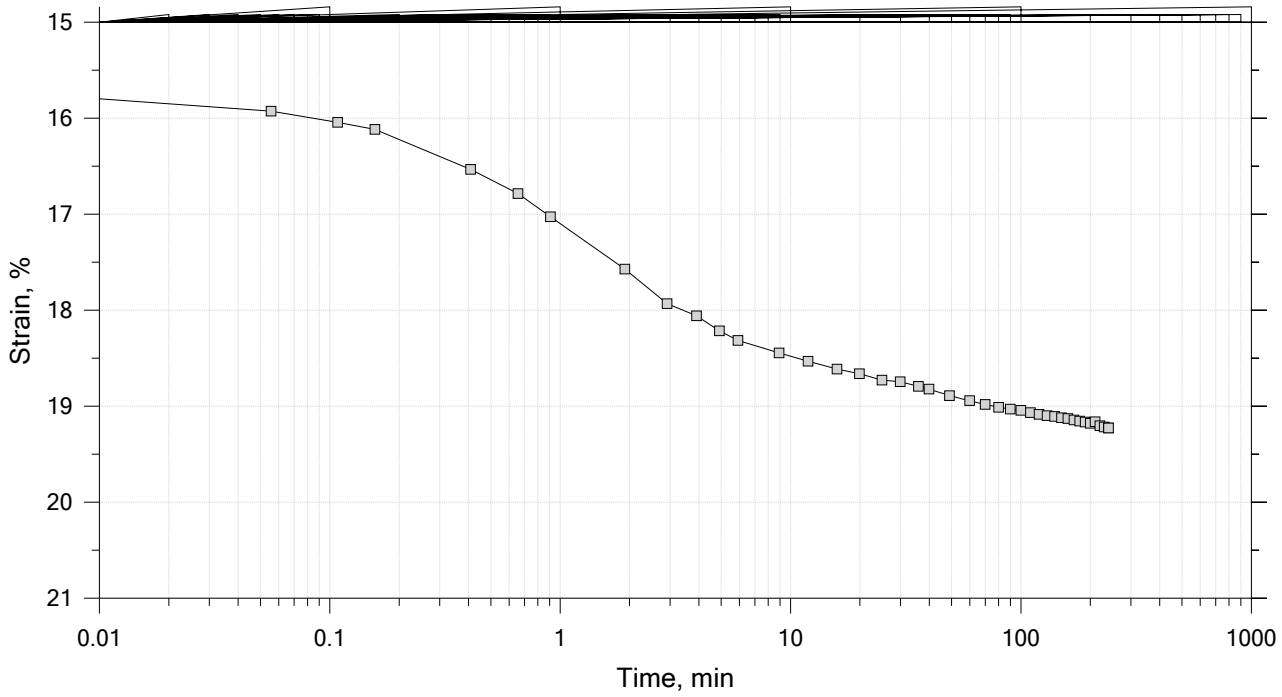


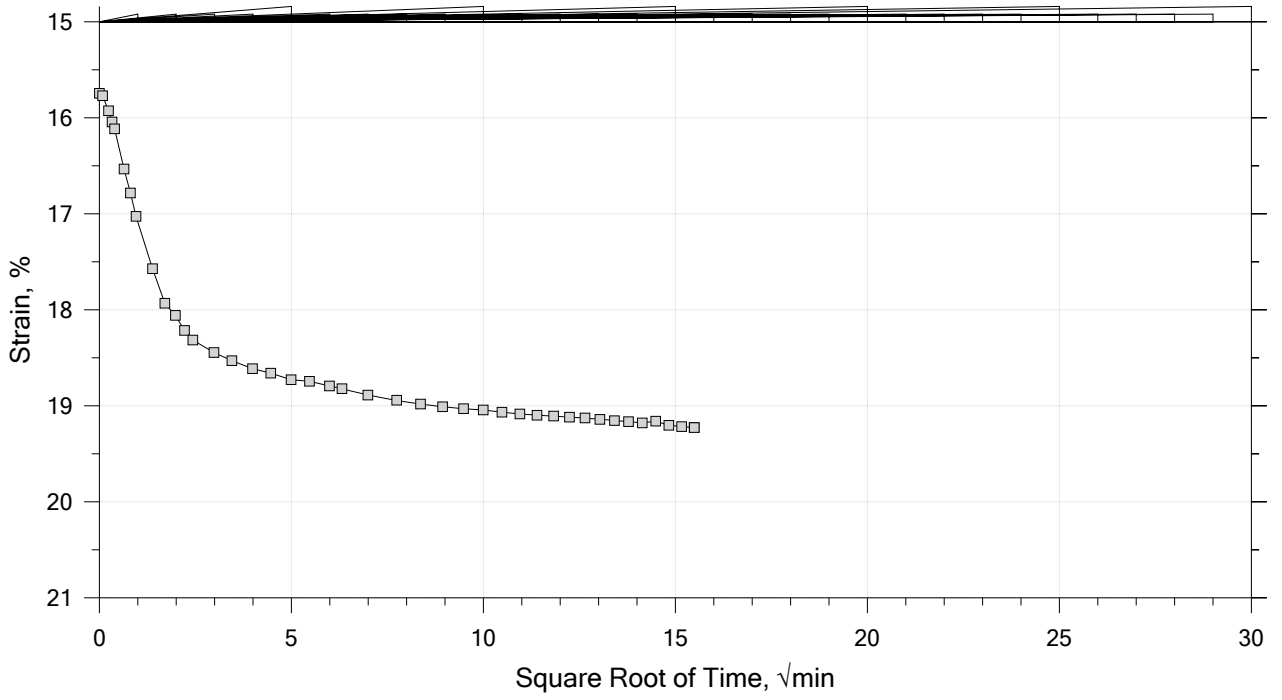


GeoTesting EXPRESS	Project: Hogan Road Bridge	Location: Bangor, ME	Project No.: GTX-315431
	Boring No.: BB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/16/22	Depth: 15-17
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: System LTIII-B		

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:	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	
3	
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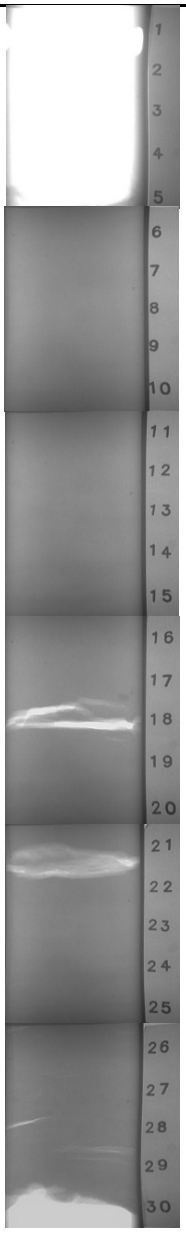
	Bottom of Tube



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:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

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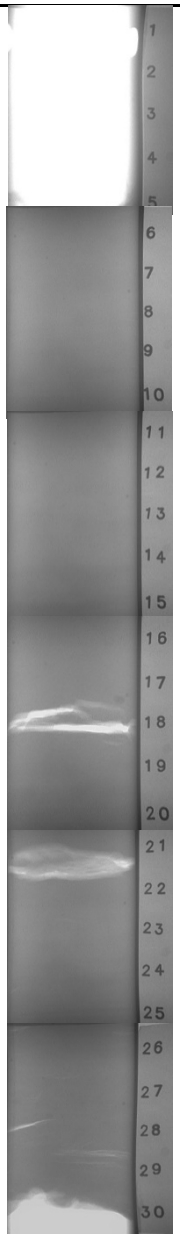
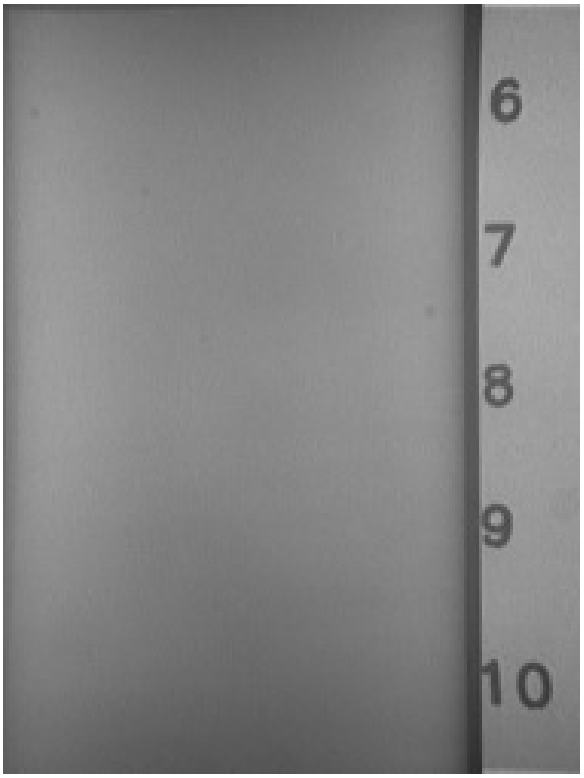


SECTION 2



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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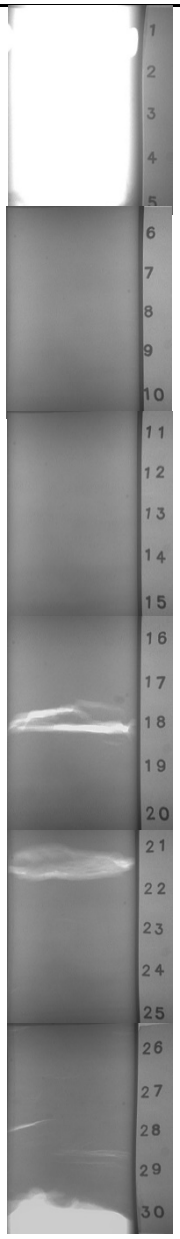

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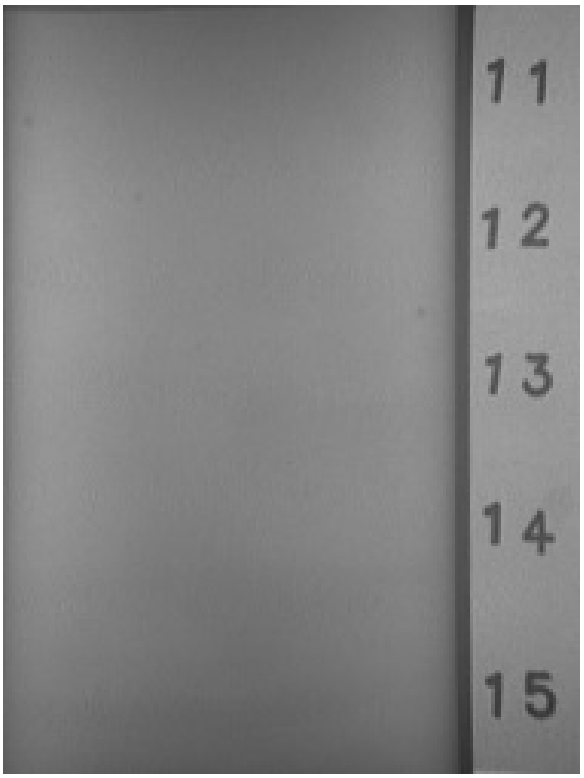

Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

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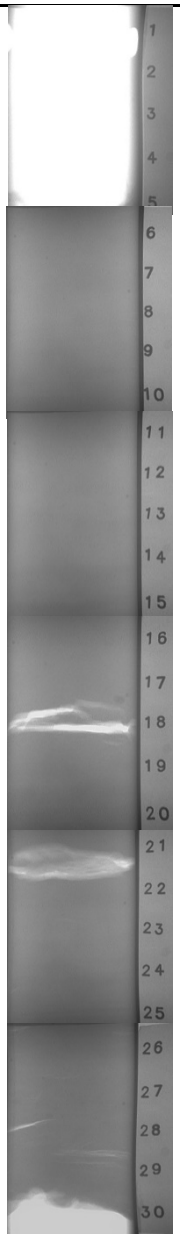

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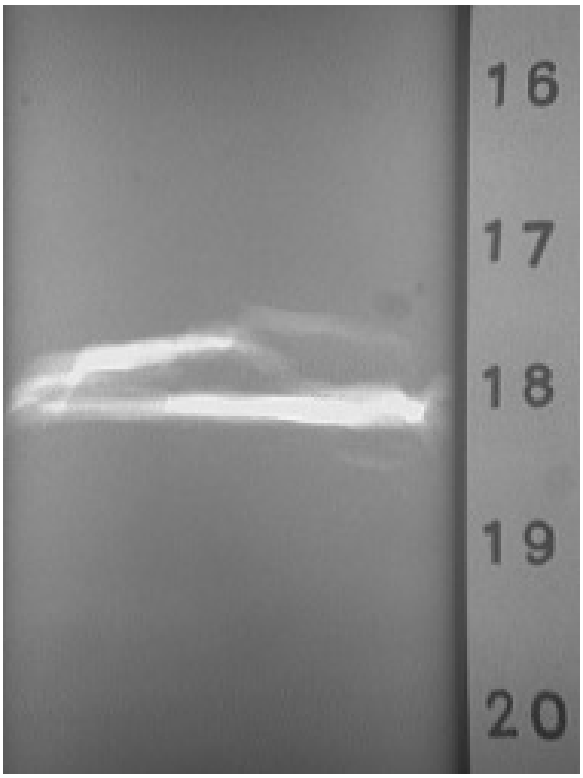

Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

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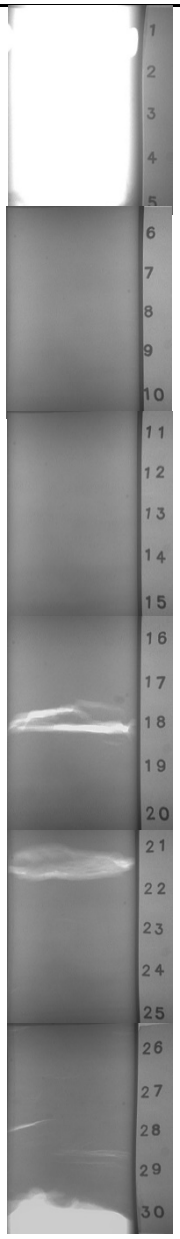

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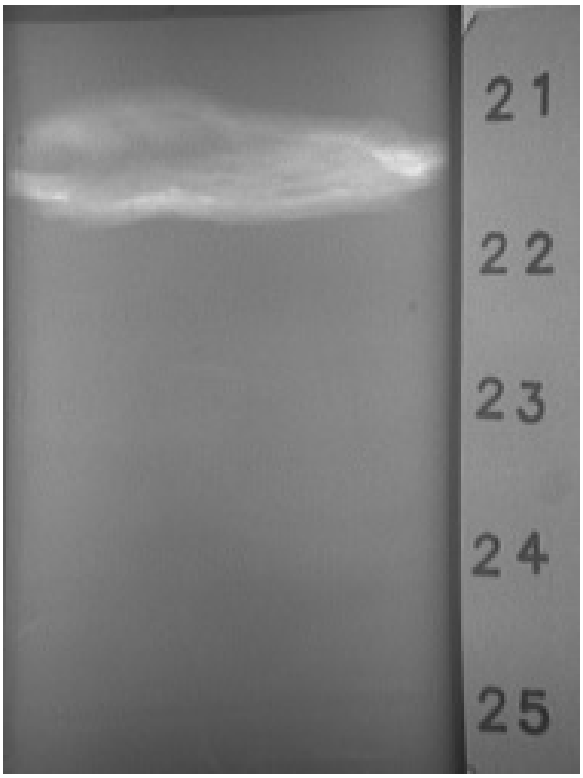

Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-101
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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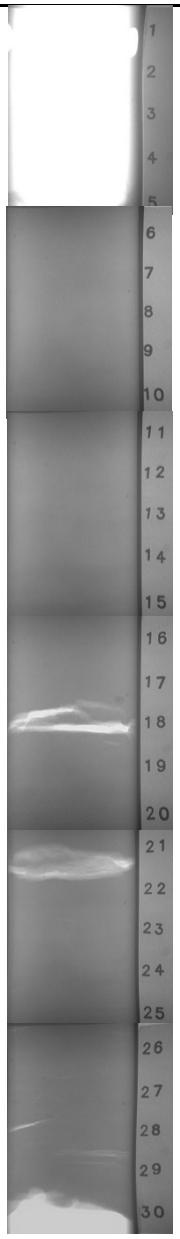
	Bottom of Tube



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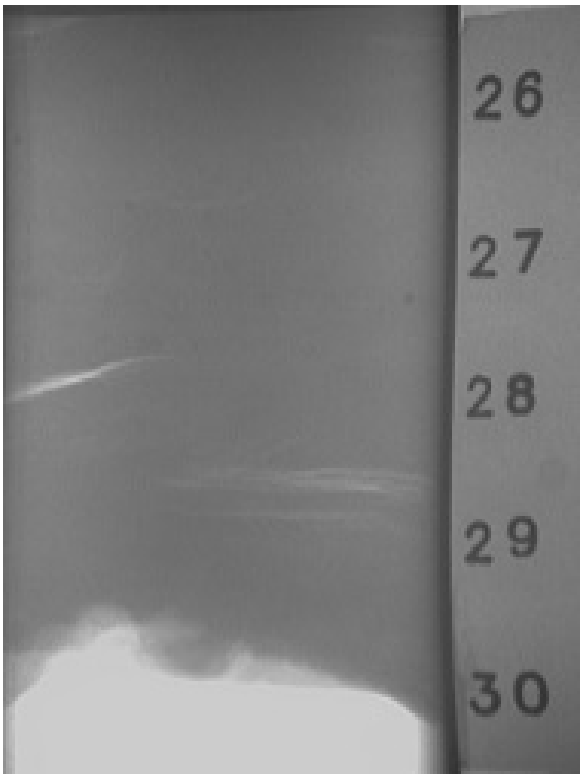

Client:	Golder Associates, Inc.
Project Name:	Hogan Road Bridge
Project Location:	Bangor, ME
GTX #:	315431
Test Date:	05/04/22
Tested By:	md
Checked By:	bfs
Boring ID:	BB-BHR-103
Sample ID:	1U
Depth:	15-17

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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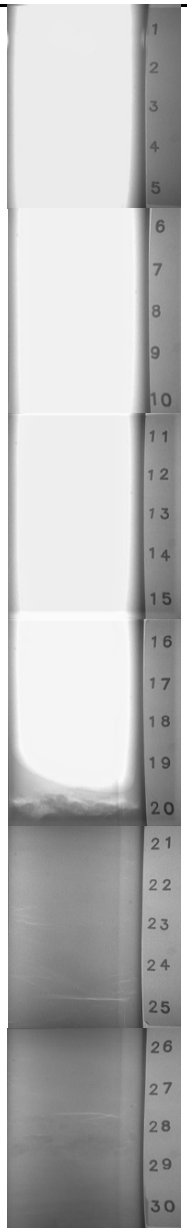
	Bottom of Tube



SECTION 1

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
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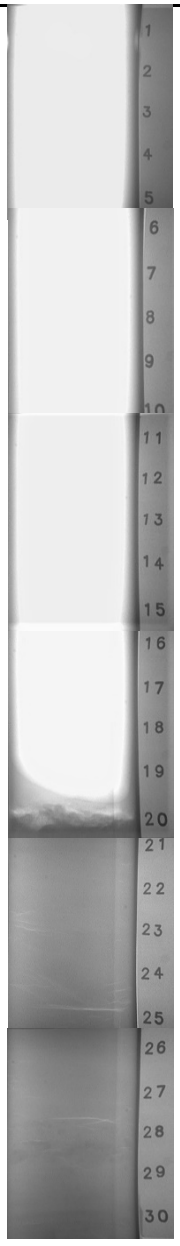
	Bottom of Tube



SECTION 2

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
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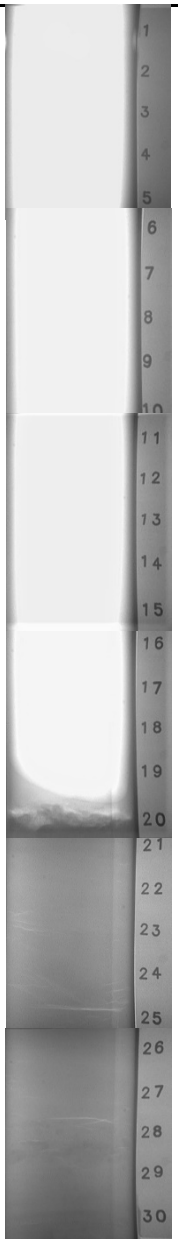
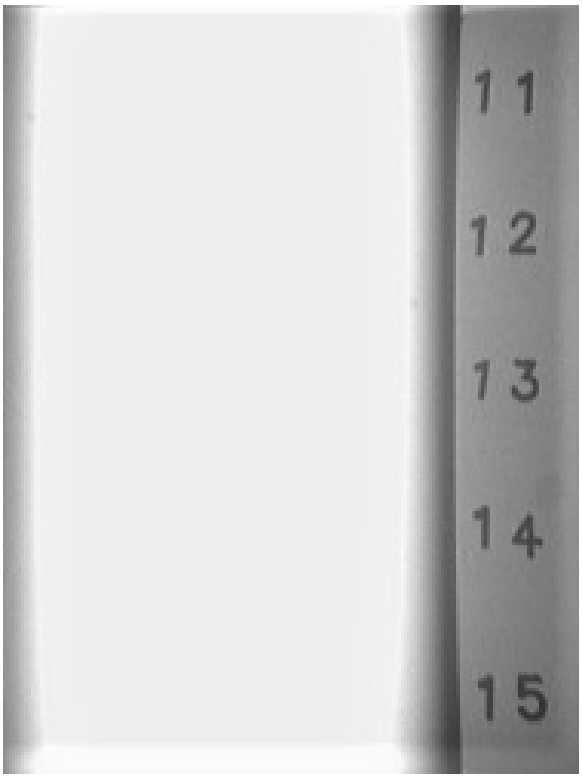
	Bottom of Tube



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
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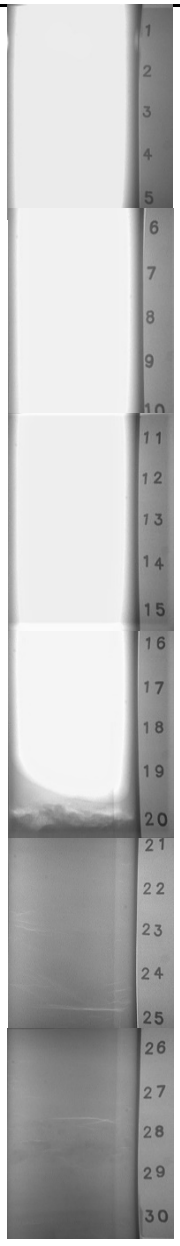
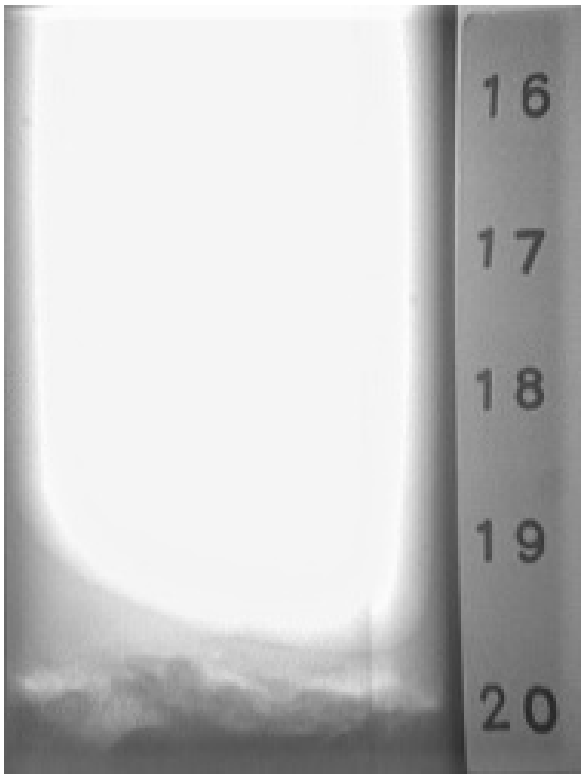


SECTION 4



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
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2	
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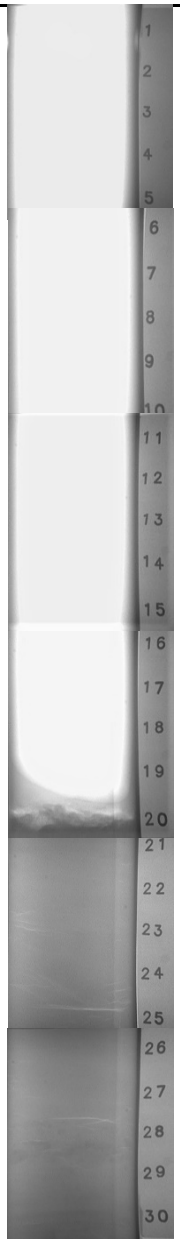
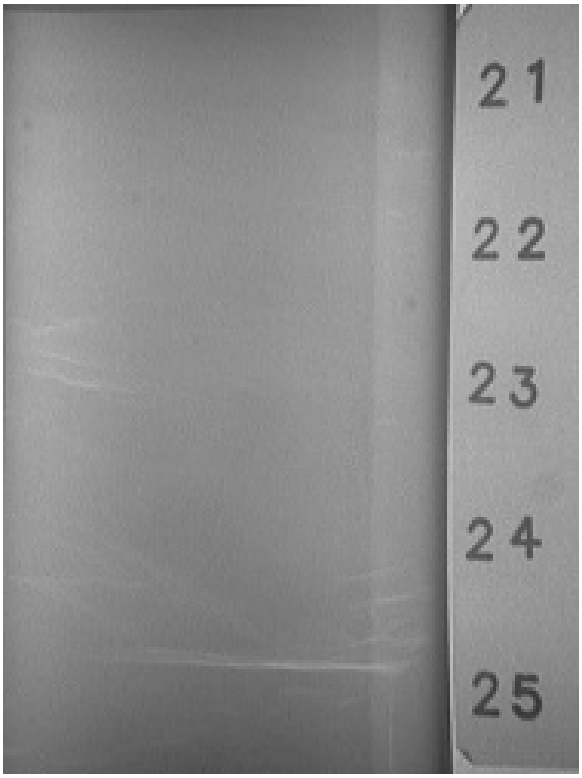
	Bottom of Tube



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	
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5	
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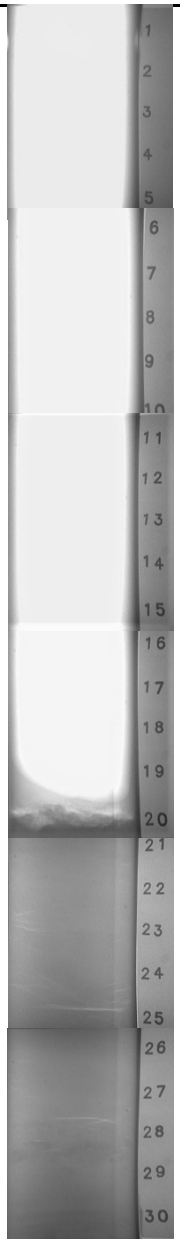
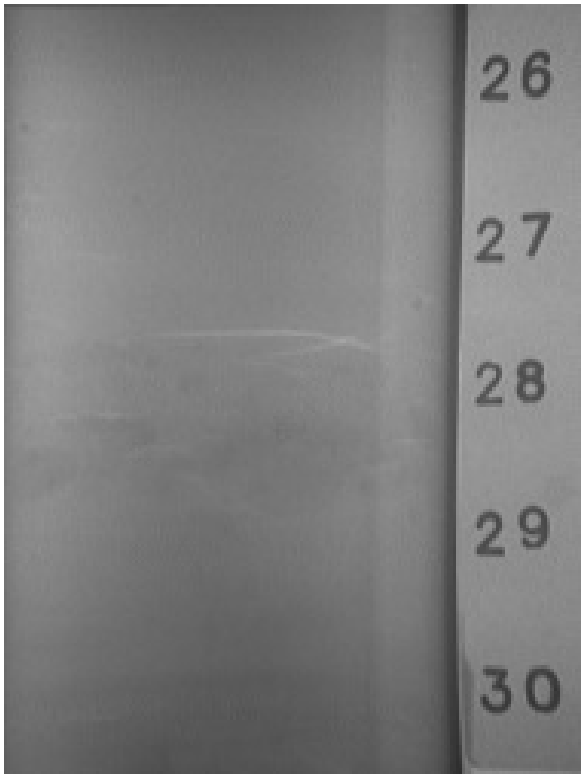
	Bottom of Tube



SECTION 6



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TEL +1 801 262 2448 · FAX +1 801 262 9870 · www.TEi-TS.com

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

Analysis No.	TS-A2210307
Report Date	26 May 2022
Date Sampled	19 May 2022
Date Received	25 May 2022
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Soil: Project: Hogan Road Bridge; Site Location: Bangor, ME; Job Number: GTX-315431

When examined to the applicable requirements of:

ASTM D 512-12*	“Standard Test Methods for Chloride Ion in Water” Method B
ASTM D 516-16	“Standard Test Method for Sulfate Ion in Water”

Results:

ASTM D512 - Chloride Method B

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
BB-BHR-101		58.	0.0058	10.
S-2	5 – 7’			
BB-BHR-102		31.	0.0031	
S-1B	0.6 – 2’			
BB-BHR-103		120.	0.0120	
S-1	0 – 2’			

NOTE: [1]Percent by weight after drying and prepared as per the Standard. *Withdrawn 2021 without Replacement

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

ASTM D 516 – Sulfates (Soluble)

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
BB-BHR-101		10.	0.0010	10.
S-2	5 – 7’			
BB-BHR-102		< 10.	< 0.0010	
S-1B	0.6 – 2’			
BB-BHR-103		< 10.	< 0.0010	
S-1	0 – 2’			

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

END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

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Client:	Golder Associates USA, Inc.
Project:	Hogan Road Bridge
Location:	Bangor, ME
GTX#:	315431
Test Date:	05/20/22
Tested By:	amp
Checked By:	bfs

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

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(ohm\text{-}cm)^{-1}$
BB-BHR-101	S-2	5.0-7.0	Moist, olive brown clay	2,169	4.61E-04
BB-BHR-102	S-1B	0.6-2.0	Moist, olive brown clay	3,409	2.93E-04
BB-BHR-103	S-1	0.0-2.0	Moist, olive brown clay	1,446	6.91E-04

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

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/03/23	Checked By:	ank
Depth :	---	Test Id:	741480		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BHR-201	S- 2	5-7'	Moist, grayish brown clay	24.1
BB-BHR-202	S- 3	10-12'	Moist, gray clay	18.8
BB-BHR-202	S- 5	20-22'	Moist, gray silt	21.7
BB-BHR-203	S- 2	5-7'	Moist, grayish brown clay	26.3
BB-BHR-203	S- 4	15-17'	Moist, gray clay	33.9
BB-BHR-204	S- 2	5-7'	Moist, olive gray clay	25.1
BB-BHR-204	S- 5	17-19'	Moist, gray clay	18.8

Notes: Temperature of Drying : 110º Celsius

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	11/03/23	Checked By:	ank
Depth :	---	Test Id:	741484		

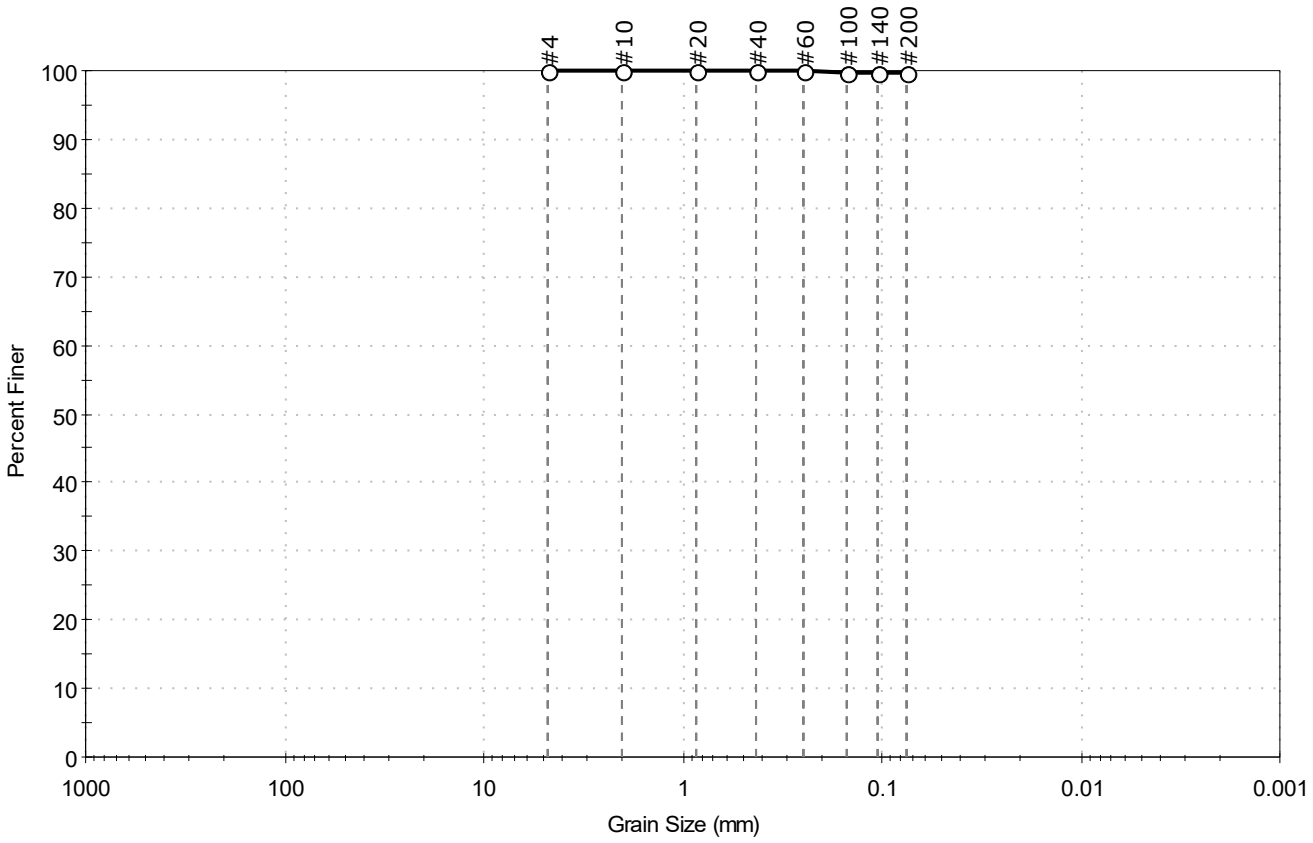
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-BHR-205	S- 2	5-7'	Moist, grayish brown clay	27.1
BB-BHR-205	S- 4A	15-16'	Wet, gray clay	29.0
BB-BHR-206	S- 2	5-7'	Moist, olive brown clay	26.4
BB-BHR-206	S- 4	15-17'	Moist, grayish brown clay	25.0

Notes: Temperature of Drying : 110º Celsius

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-201	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741439		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% 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		

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 (17))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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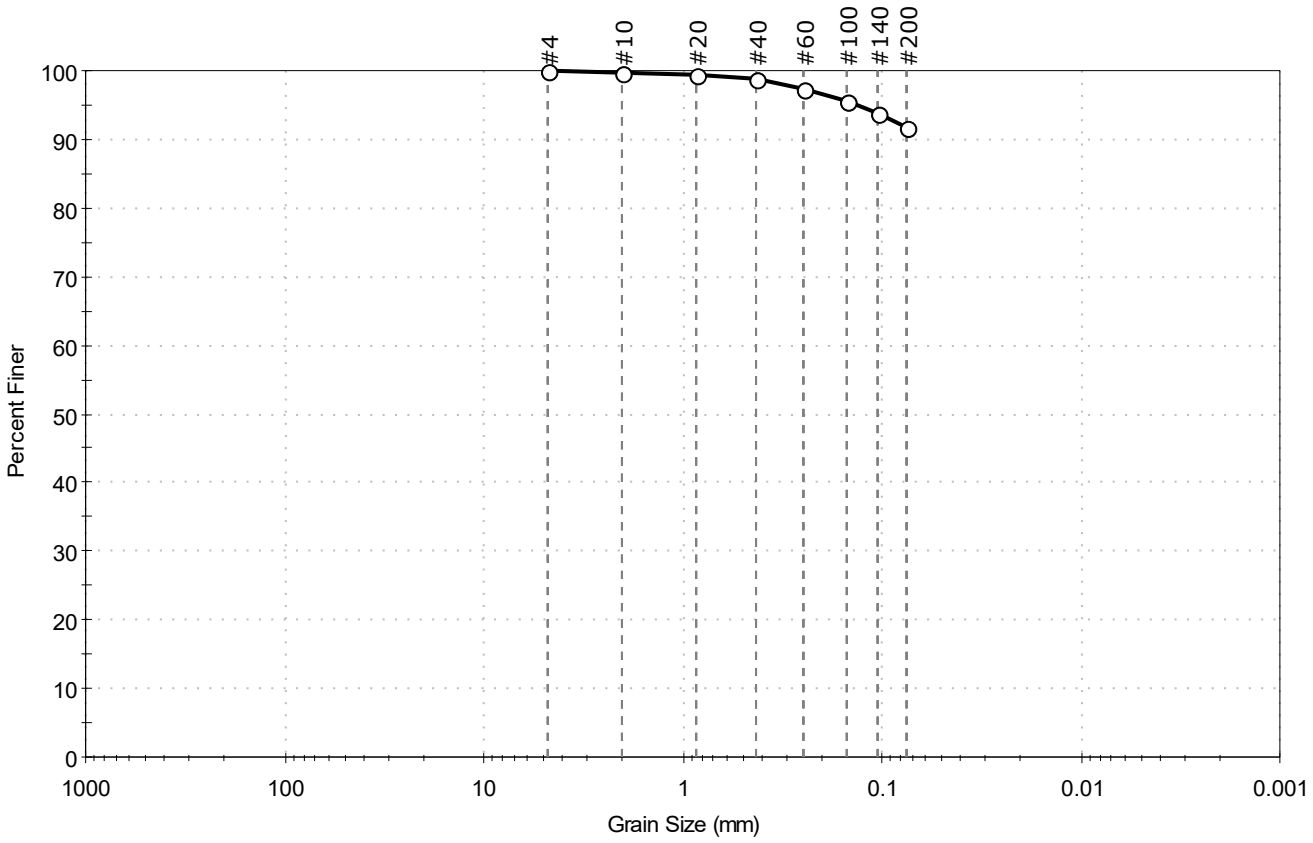
Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-202	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-5	Test Date:	11/01/23	Checked By:	ank
Depth :	20-22'	Test Id:	741440		
Test Comment:	---				
Visual Description:	Moist, gray silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	8.4	91.6

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	97		
#100	0.15	96		
#140	0.11	94		
#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))

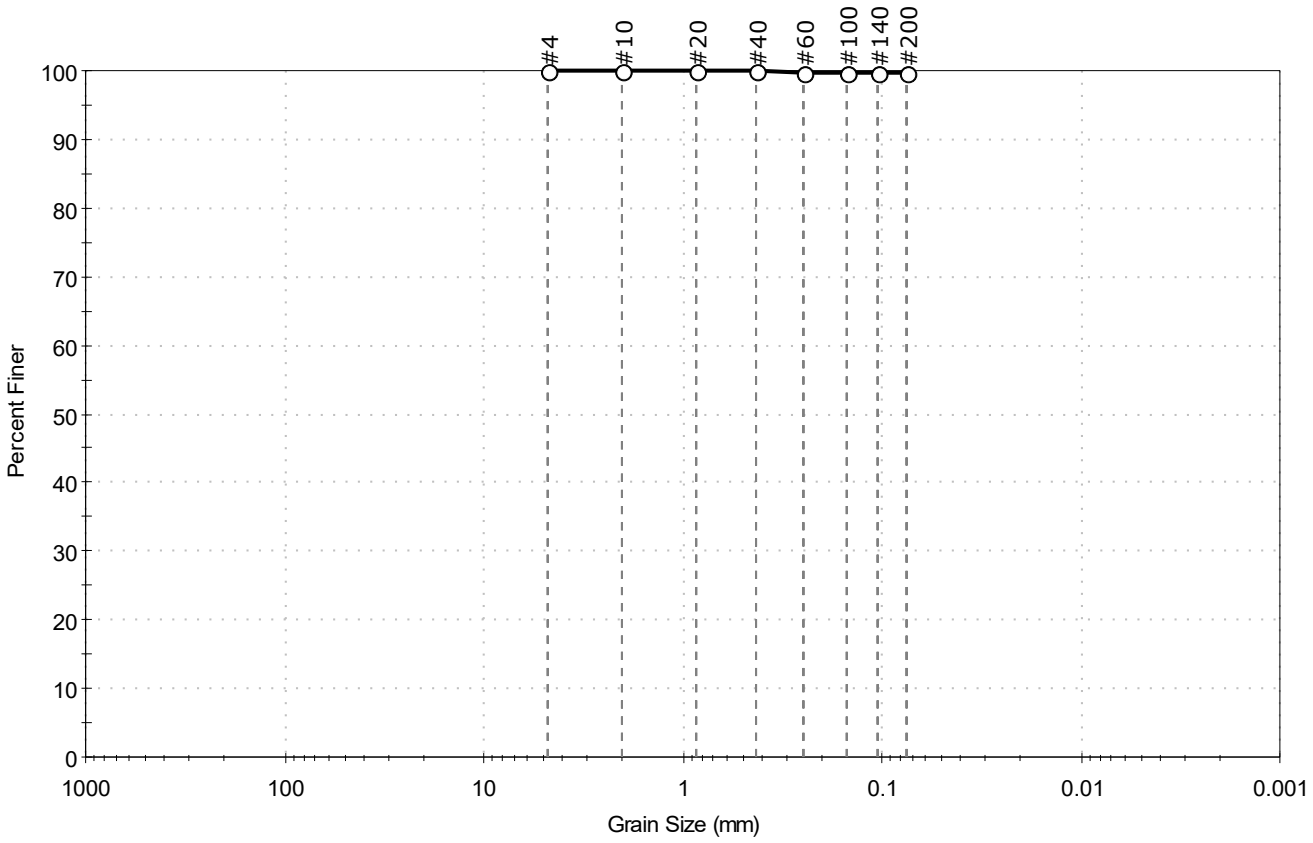
Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-203	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/03/23	Checked By:	ank
Depth :	5-7'	Test Id:	741441		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	0.4	99.6

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		

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 : ---

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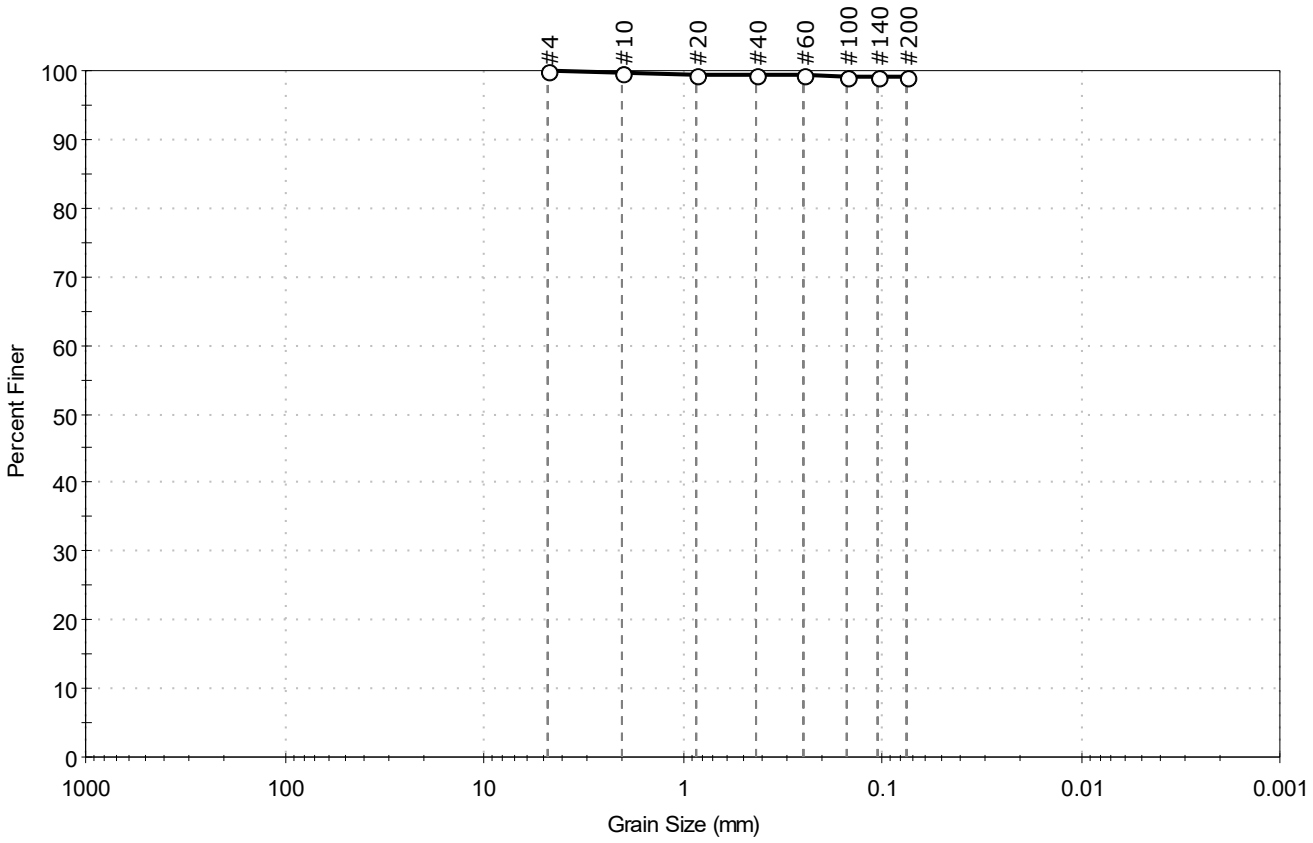
Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-204	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/03/23	Checked By:	ank
Depth :	5-7'	Test Id:	741442		
Test Comment:	---				
Visual Description:	Moist, olive gray 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 (15))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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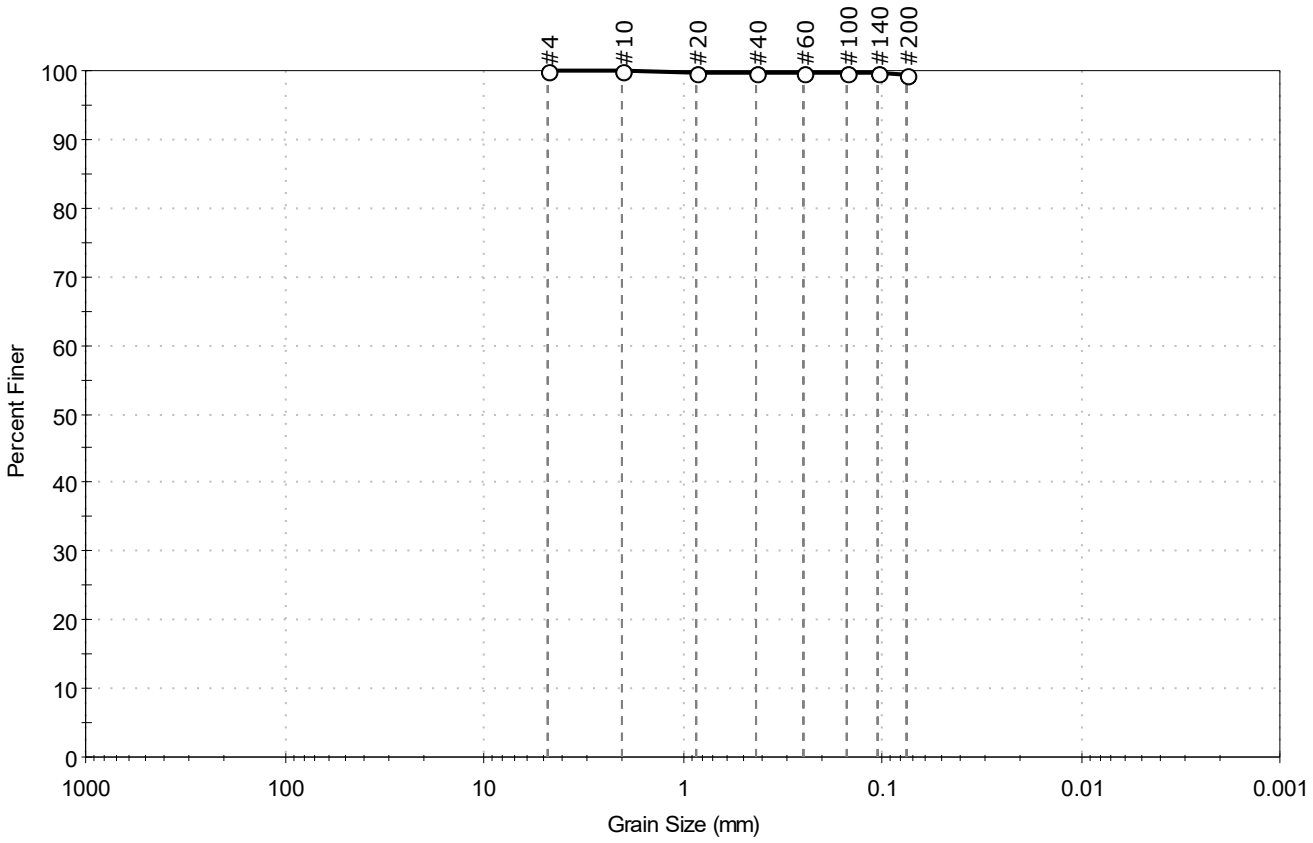
Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-205	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741443		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% 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		

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 (21))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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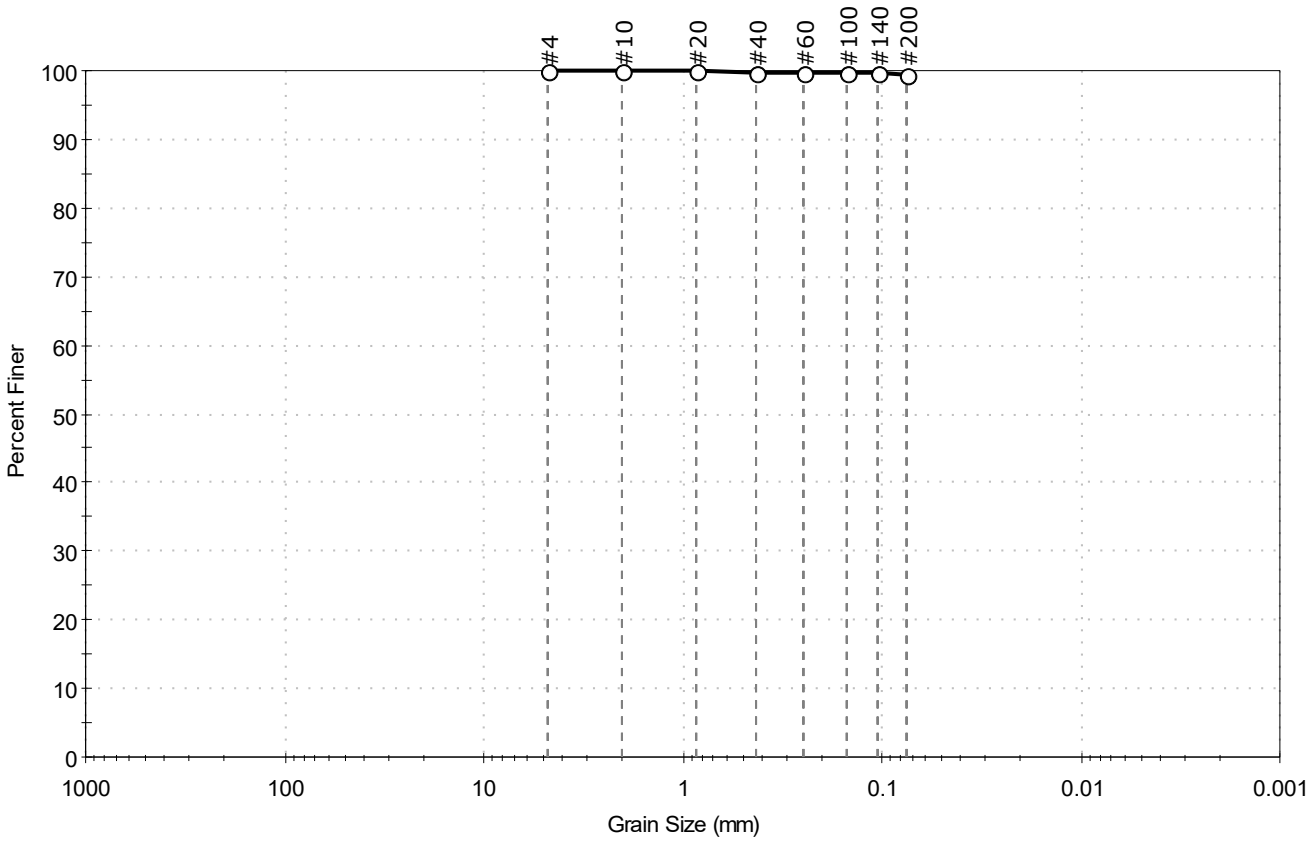
Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-206	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-2	Test Date:	11/06/23	Checked By:	ank
Depth :	5-7'	Test Id:	741444		
Test Comment:	---				
Visual Description:	Moist, olive 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	100		

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 : ---

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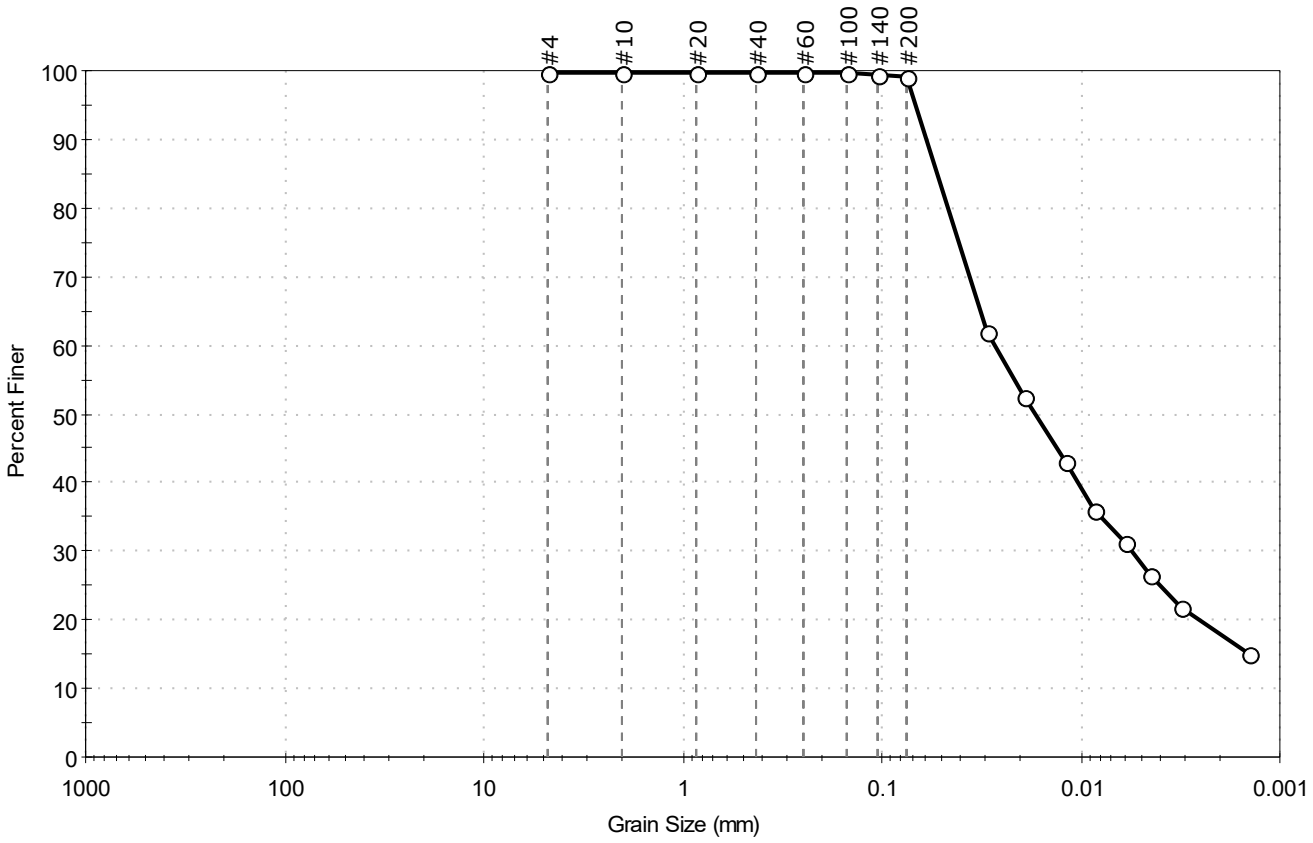
Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-206	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-4	Test Date:	11/03/23	Checked By:	ank
Depth :	15-17'	Test Id:	741488		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.2	0.6	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	100		
#200	0.075	99		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0296	62		
---	0.0192	52		
---	0.0119	43		
---	0.0085	36		
---	0.0060	31		
---	0.0044	27		
---	0.0032	22		
---	0.0014	15		

Coefficients

D_{85} =0.0526 mm	D_{30} =0.0055 mm
D_{60} =0.0272 mm	D_{15} =0.0014 mm
D_{50} =0.0169 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Silty Soils (A-4 (6))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

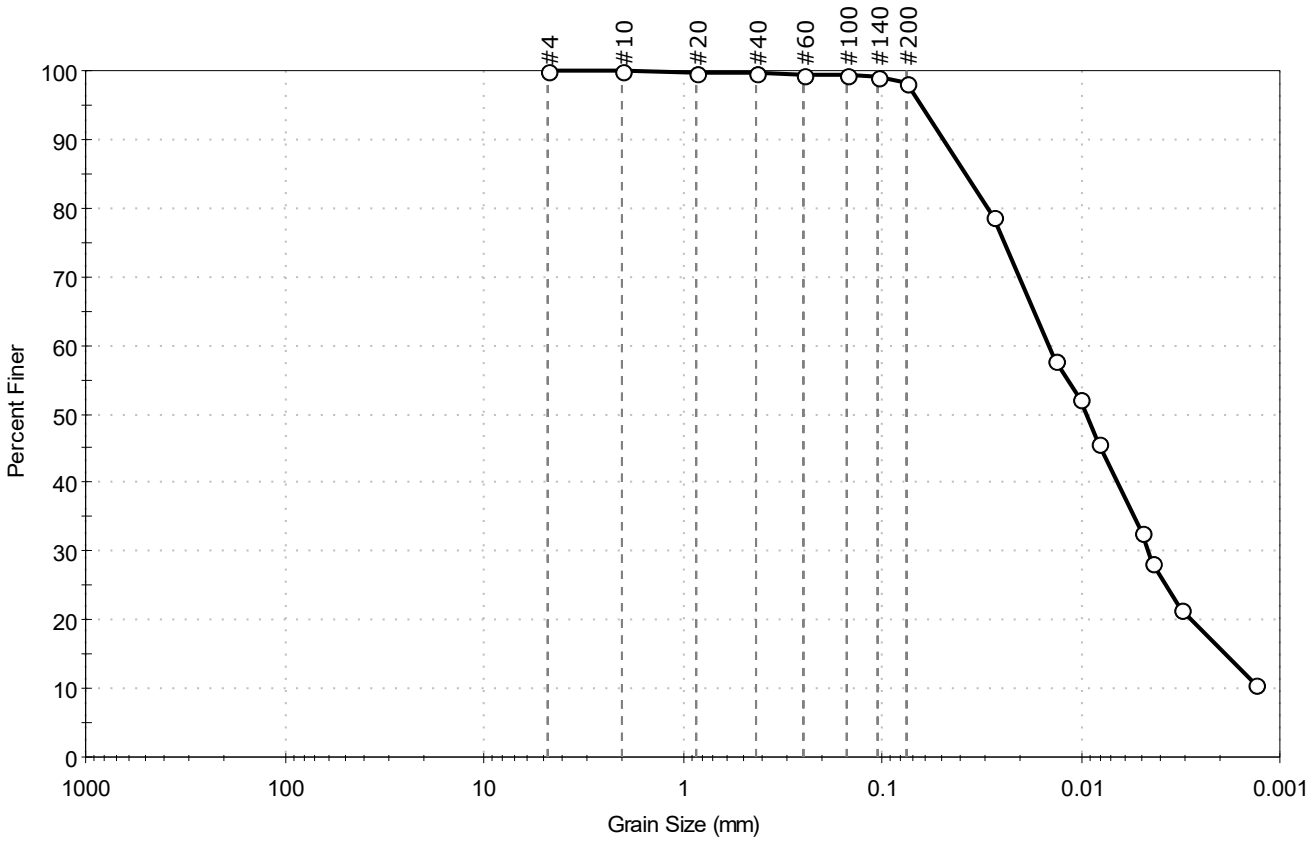
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor PO 004				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-201	Sample Type:	tube	Tested By:	ckg
Sample ID:	U-1	Test Date:	12/08/23	Checked By:	ank
Depth :	17-19'	Test Id:	748090		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	1.7	98.3

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	100		
#60	0.25	100		
#100	0.15	99		
#140	0.11	99		
#200	0.075	98		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0273	79		
---	0.0134	58		
---	0.0100	52		
---	0.0081	46		
---	0.0050	33		
---	0.0044	28		
---	0.0032	22		
---	0.0013	11		

Coefficients

D_{85} =0.0378 mm	D_{30} =0.0046 mm
D_{60} =0.0145 mm	D_{15} =0.0019 mm
D_{50} =0.0093 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Silty Soils (A-4 (9))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

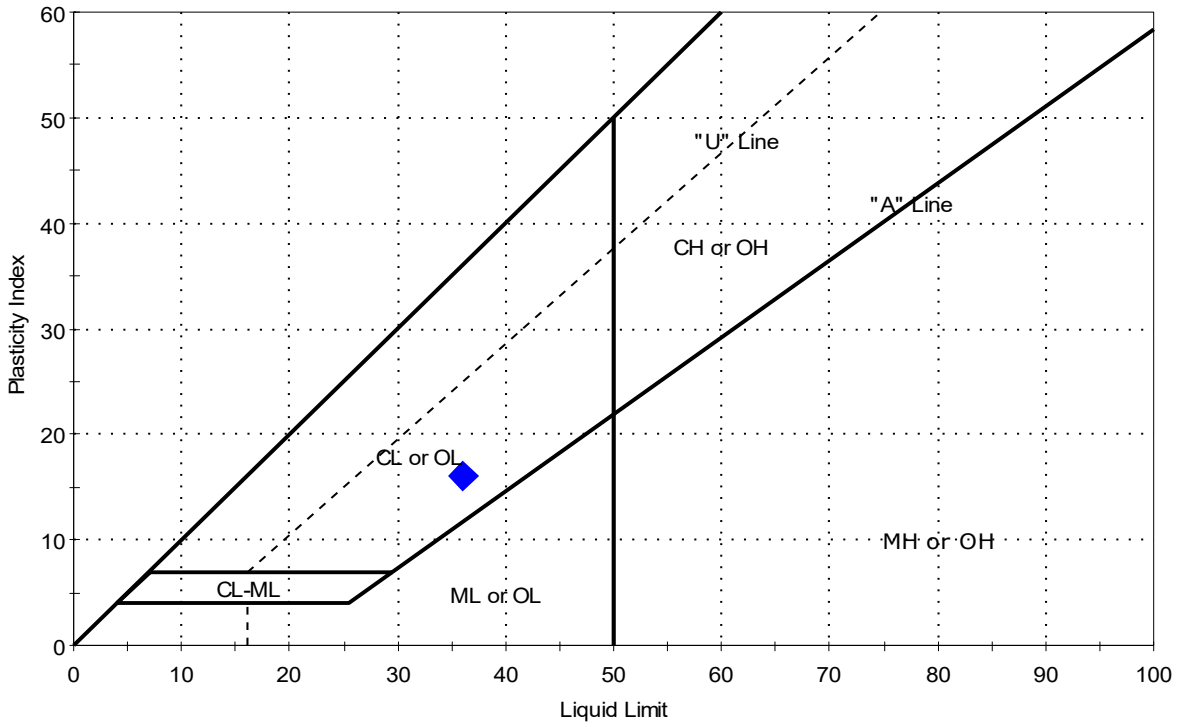
Dispersion Period : 1 minute

Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-201	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	11/02/23	Checked By:	ank
Depth :	5-7'	Test Id:	741400		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

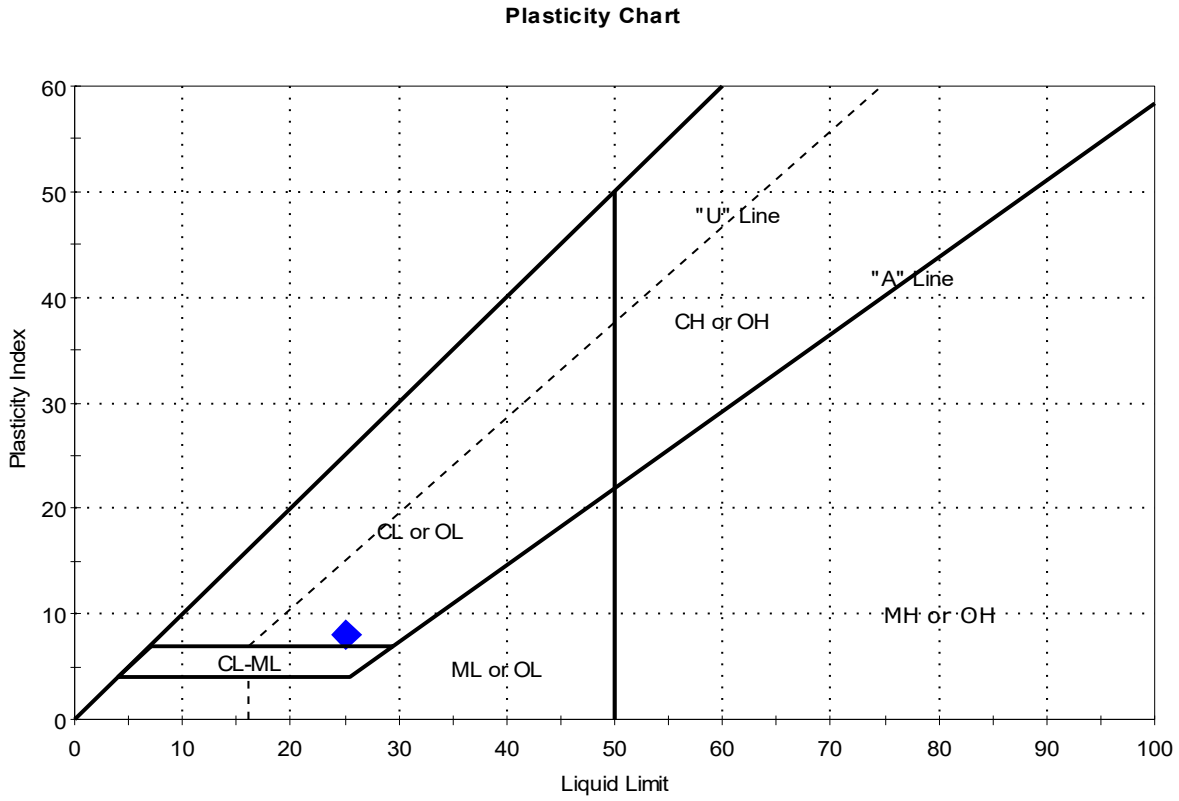


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	24	36	20	16	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-202	Sample Type:	jar	Tested By:	cam
Sample ID:	S-3	Test Date:	11/01/23	Checked By:	ank
Depth :	10-12'	Test Id:	741401		
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-3	B-BHR-20	10-12'	19	25	17	8	0.2	

Sample Prepared using the WET method

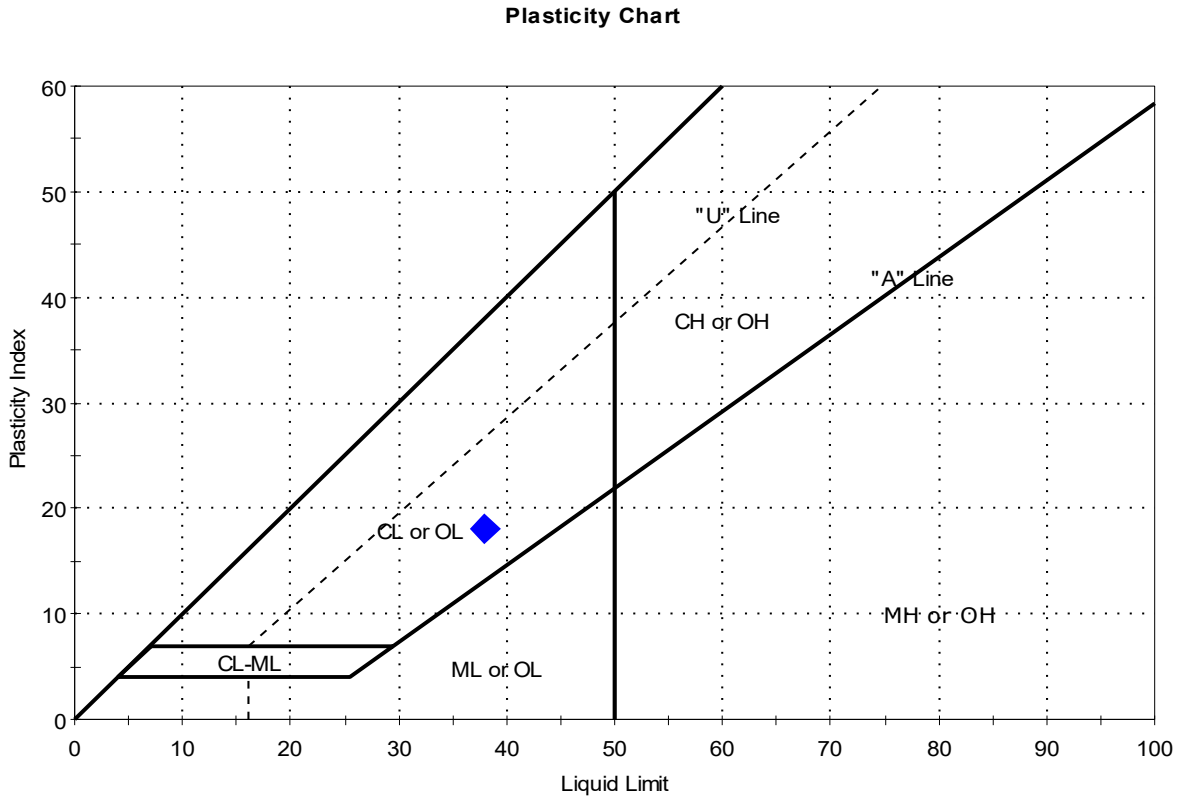
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-203	Sample Type:	jar	Tested By:	GA
Sample ID:	S-2	Test Date:	11/01/23	Checked By:	ank
Depth :	5-7'	Test Id:	741402		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

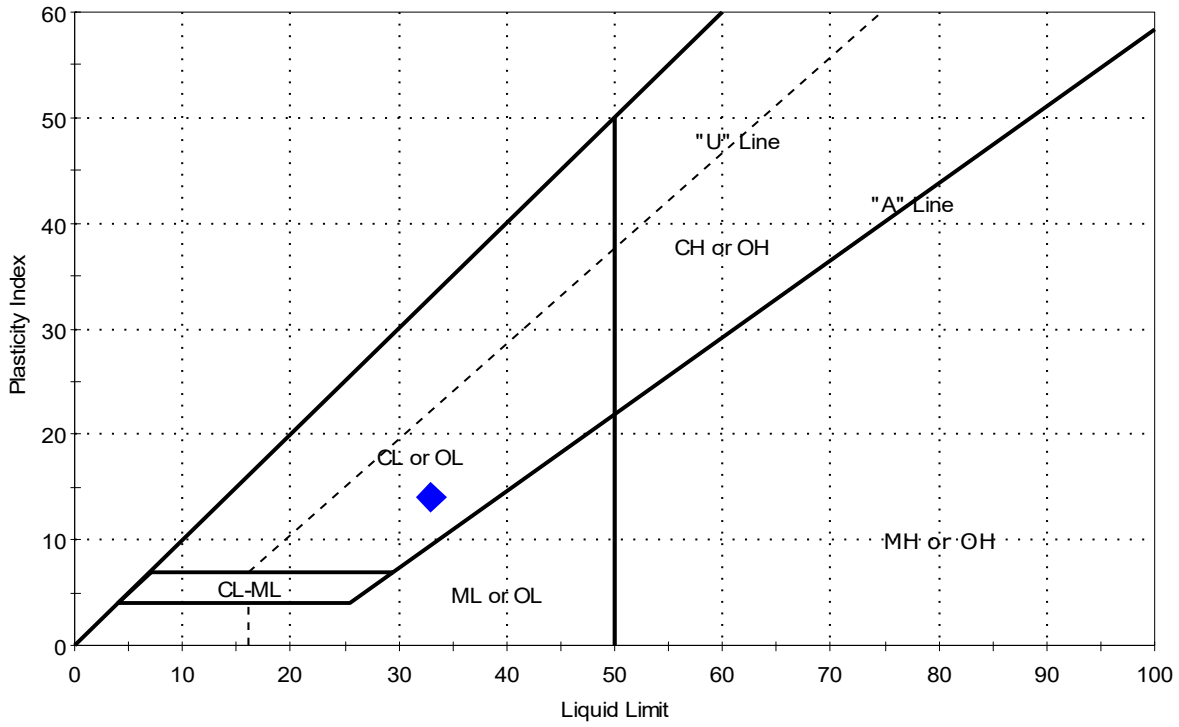


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	26	38	20	18	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-203	Sample Type:	jar	Tested By:	GA
Sample ID:	S-4	Test Date:	11/03/23	Checked By:	ank
Depth :	15-17'	Test Id:	741403		
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-20	15-17'	34	33	19	14	1.1	

Sample Prepared using the WET method

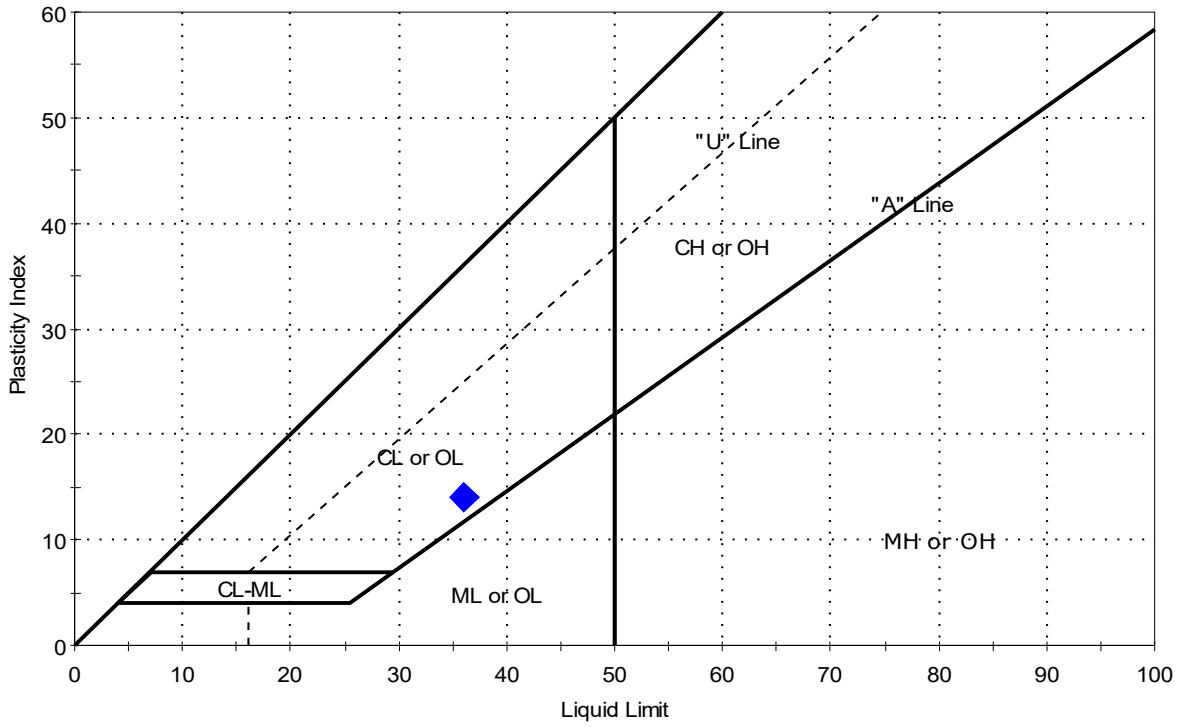
Dry Strength: HIGH

Dilatancy: SLOW

Toughness: MEDIUM

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-204	Sample Type:	jar	Tested By:	cam
Sample ID:	S-2	Test Date:	11/01/23	Checked By:	ank
Depth :	5-7'	Test Id:	741404		
Test Comment:	---				
Visual Description:	Moist, olive gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

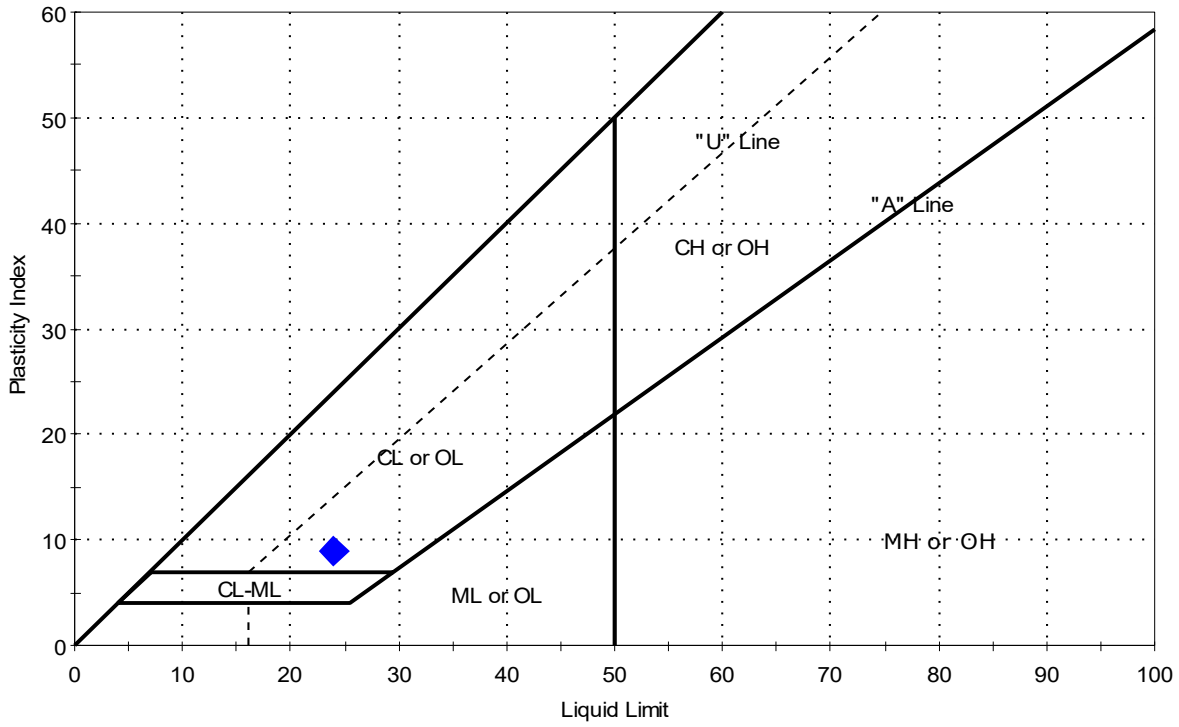


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	25	36	22	14	0.2	Lean CLAY (CL)

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-318065
Boring ID:	BB-BHR-204	Sample Type:	jar	Tested By:	cam
Sample ID:	S-5	Test Date:	10/31/23	Checked By:	ank
Depth :	17-19'	Test Id:	741405		
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-5	B-BHR-20	17-19'	19	24	15	9	0.4	

Sample Prepared using the WET method

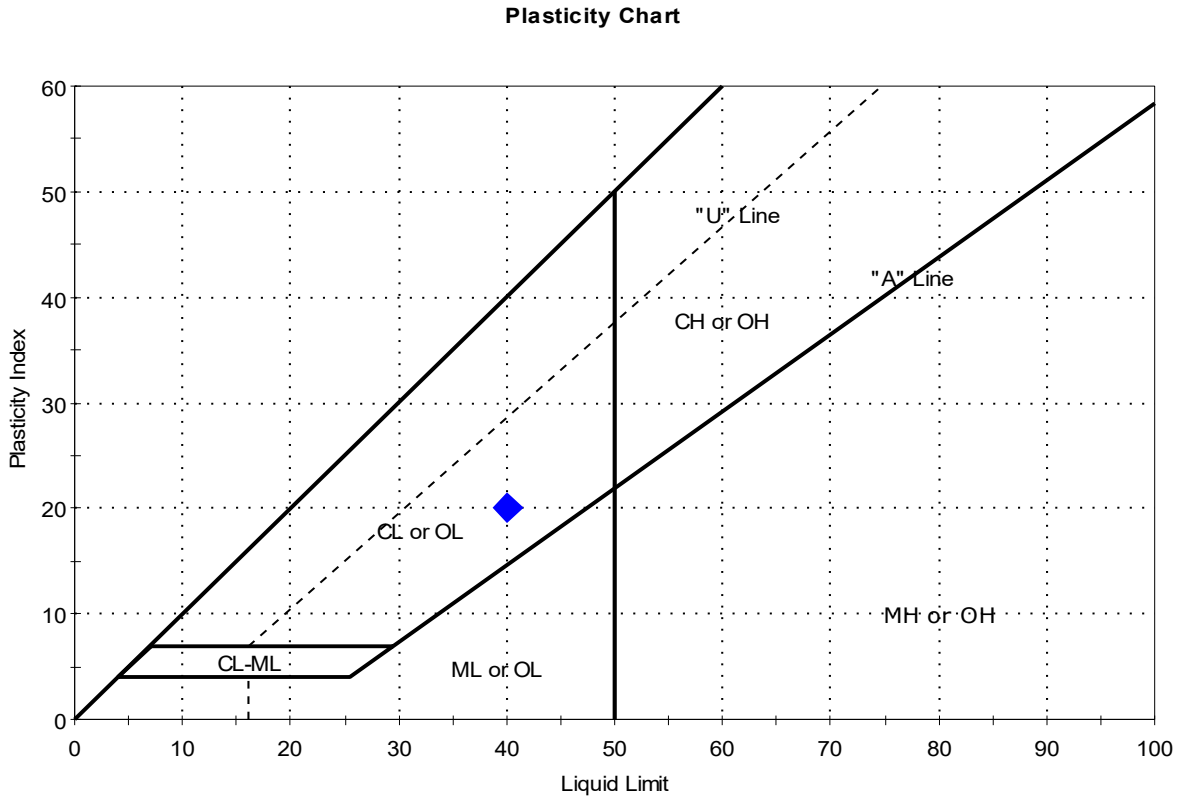
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-205	Sample Type:	jar	Tested By:	GA
Sample ID:	S-2	Test Date:	11/03/23	Checked By:	ank
Depth :	5-7'	Test Id:	741406		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

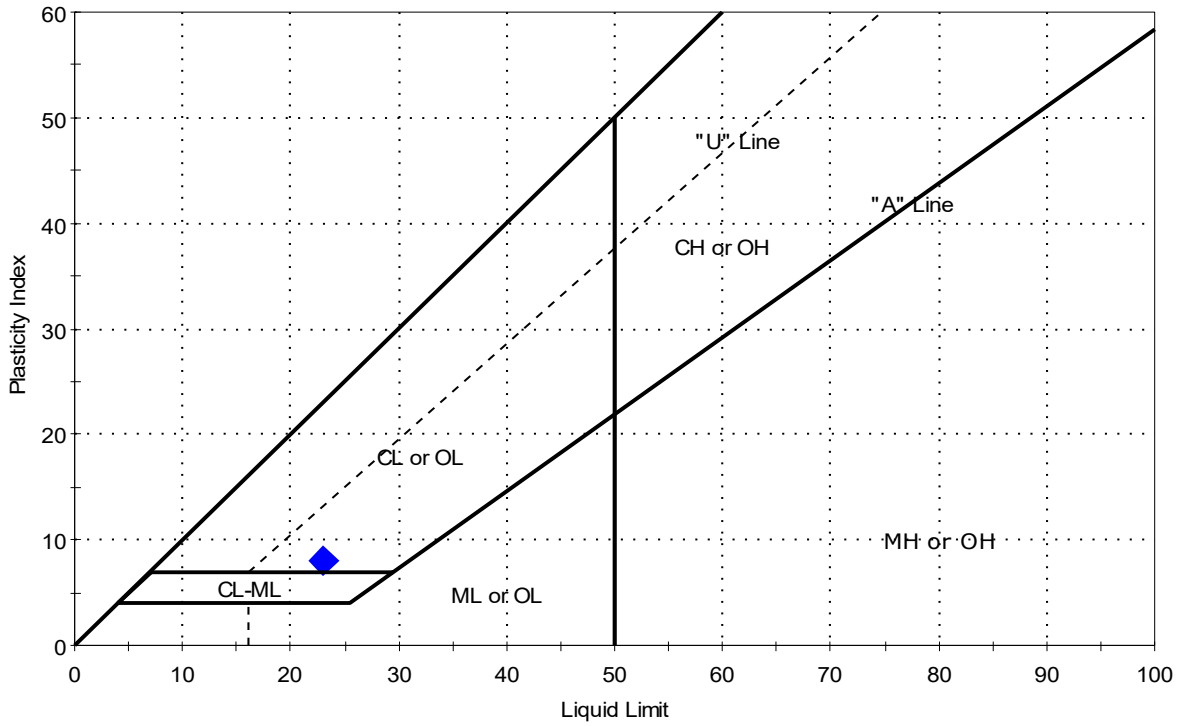


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	27	40	20	20	0.4	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: MEDIUM
Dilatancy: NONE
Toughness: MEDIUM

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-205	Sample Type:	jar	Tested By:	GA
Sample ID:	S-4A	Test Date:	11/06/23	Checked By:	ank
Depth :	15-16'	Test Id:	741407		
Test Comment:	---				
Visual Description:	Wet, 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-4A	B-BHR-20	15-16'	29	23	15	8	1.8	

Sample Prepared using the WET method

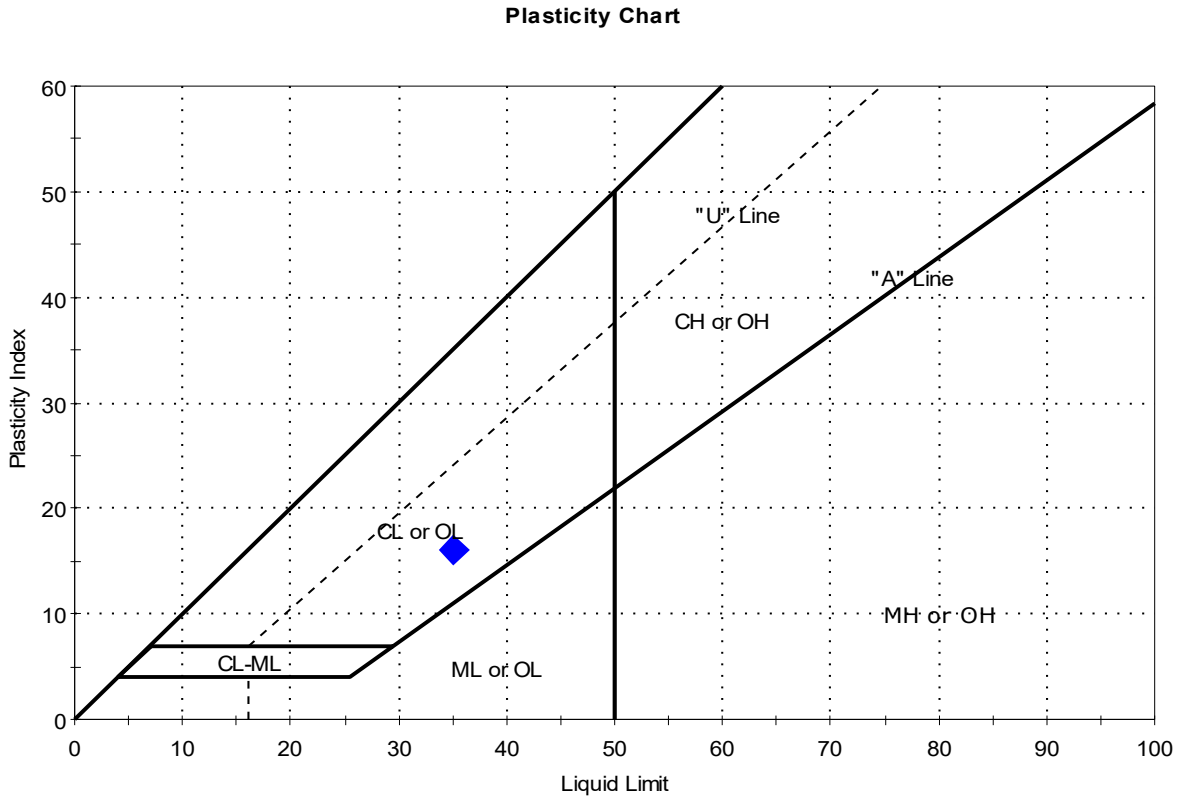
Dry Strength: MEDIUM

Dilatancy: SLOW

Toughness: MEDIUM

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-206	Sample Type:	jar	Tested By:	GA
Sample ID:	S-2	Test Date:	11/03/23	Checked By:	ank
Depth :	5-7'	Test Id:	741408		
Test Comment:	---				
Visual Description:	Moist, olive brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

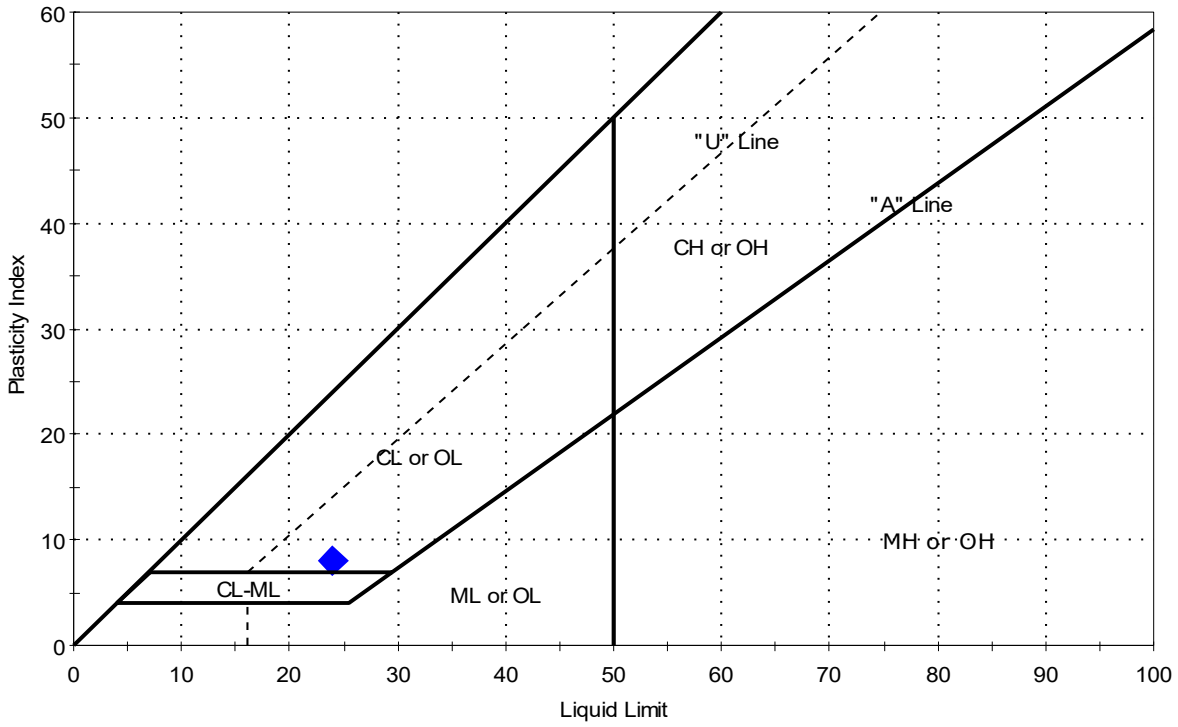


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-2	B-BHR-20	5-7'	26	35	19	16	0.5	Lean CLAY (CL)

Sample Prepared using the WET method
0% 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-318065
Boring ID:	BB-BHR-206	Sample Type:	jar	Tested By:	GA
Sample ID:	S-4	Test Date:	11/03/23	Checked By:	ank
Depth :	15-17'	Test Id:	741409		
Test Comment:	---				
Visual Description:	Moist, grayish brown clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

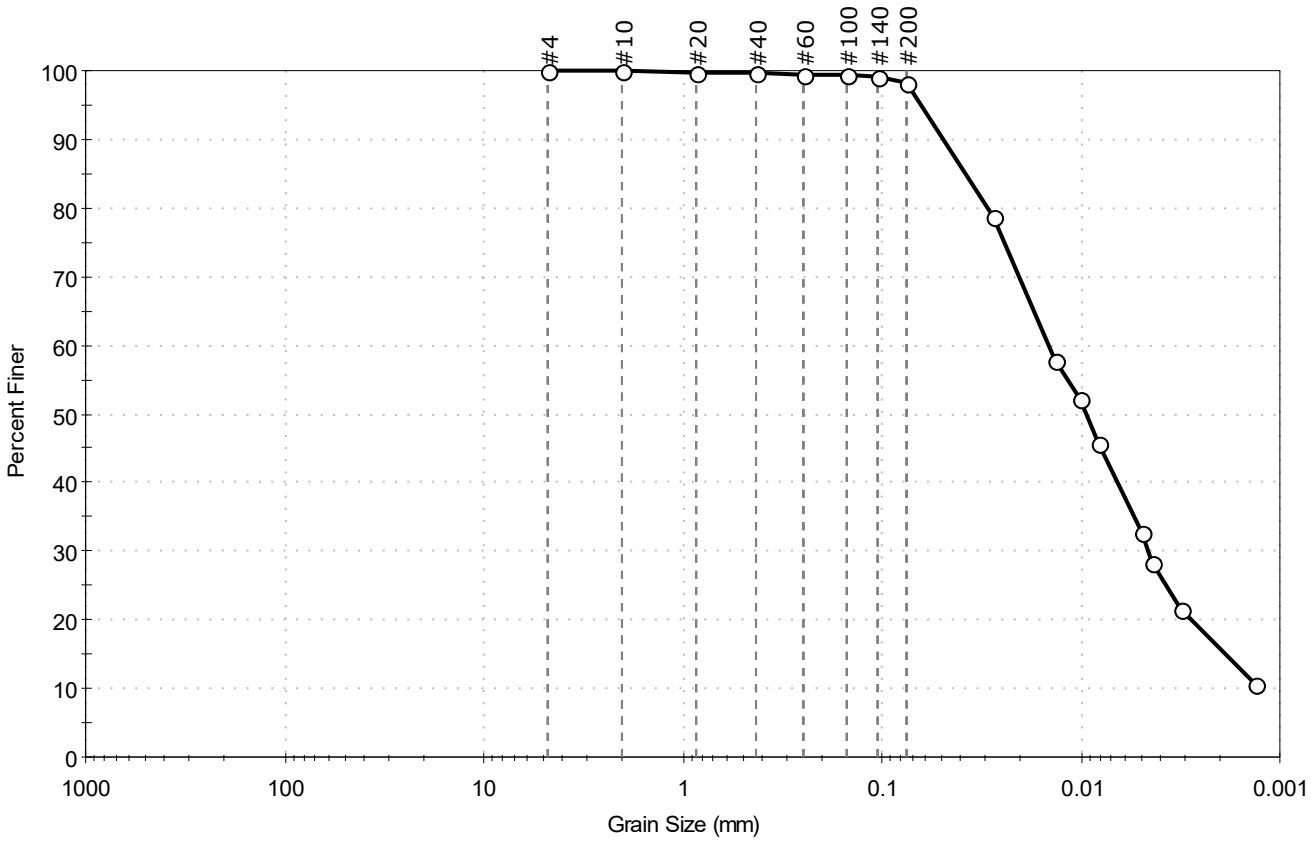


Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	S-4	B-BHR-20	15-17'	25	24	16	8	1.1	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: MEDIUM
Dilatancy: SLOW
Toughness: LOW

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor PO 004				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-201	Sample Type:	tube	Tested By:	ckg
Sample ID:	U-1	Test Date:	12/08/23	Checked By:	ank
Depth :	17-19'	Test Id:	748090		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	1.7	98.3

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	100		
#60	0.25	100		
#100	0.15	99		
#140	0.11	99		
#200	0.075	98		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0273	79		
---	0.0134	58		
---	0.0100	52		
---	0.0081	46		
---	0.0050	33		
---	0.0044	28		
---	0.0032	22		
---	0.0013	11		

Coefficients

D_{85} =0.0378 mm	D_{30} =0.0046 mm
D_{60} =0.0145 mm	D_{15} =0.0019 mm
D_{50} =0.0093 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM Lean CLAY (CL)

AASHTO Silty Soils (A-4 (9))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

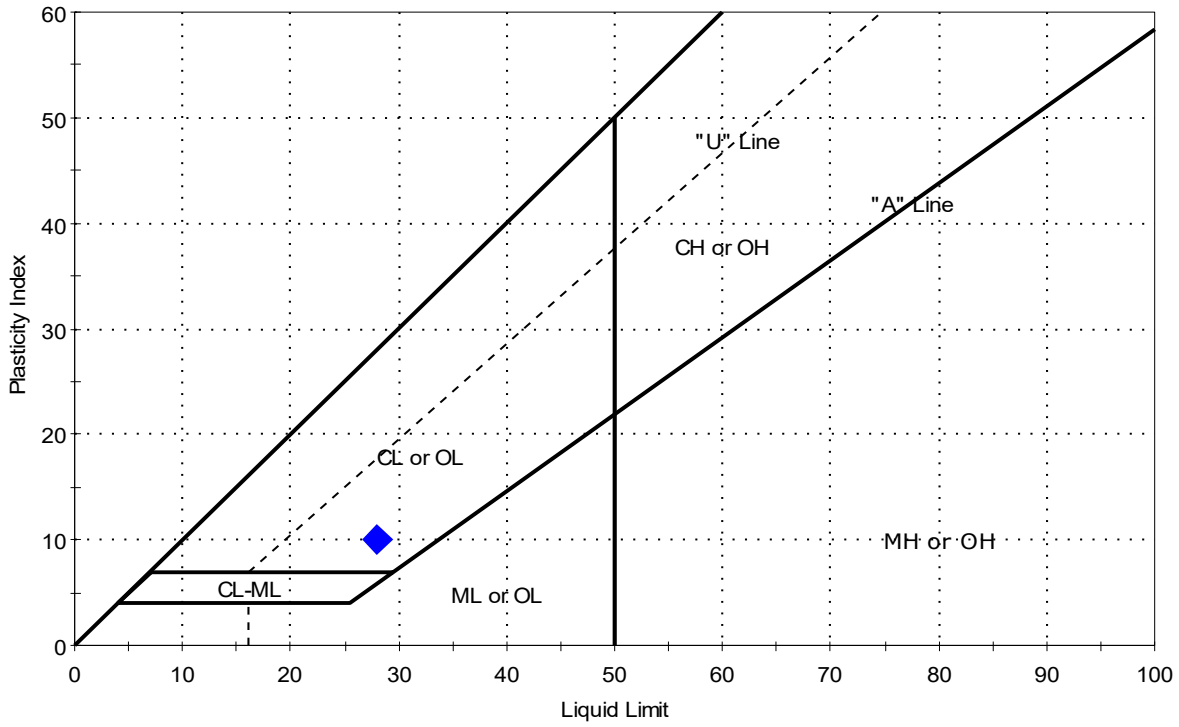
Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

GeoTesting EXPRESS

Client:	WSP USA, Inc.				
Project:	Hogan Road Bridge Bangor PO 004				
Location:	Bangor, ME			Project No:	GTX-318065
Boring ID:	BB-BHR-201	Sample Type:	tube	Tested By:	cam
Sample ID:	U-1	Test Date:	12/07/23	Checked By:	ank
Depth :	17-19'	Test Id:	748089		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

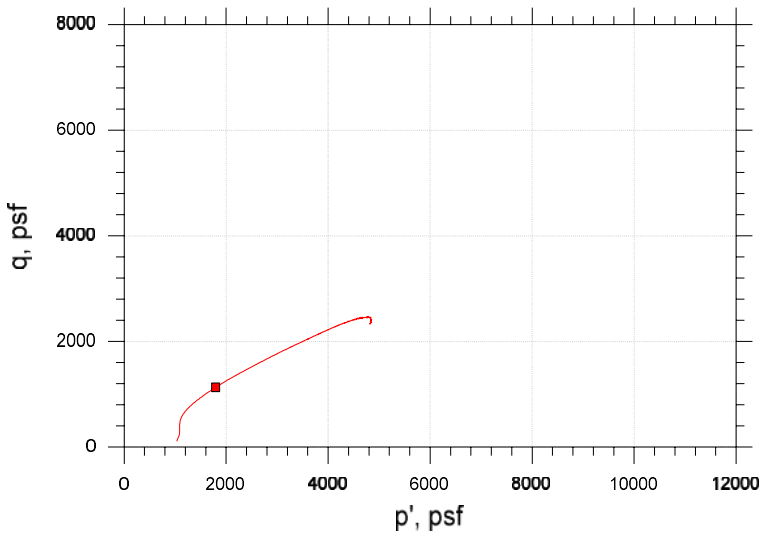


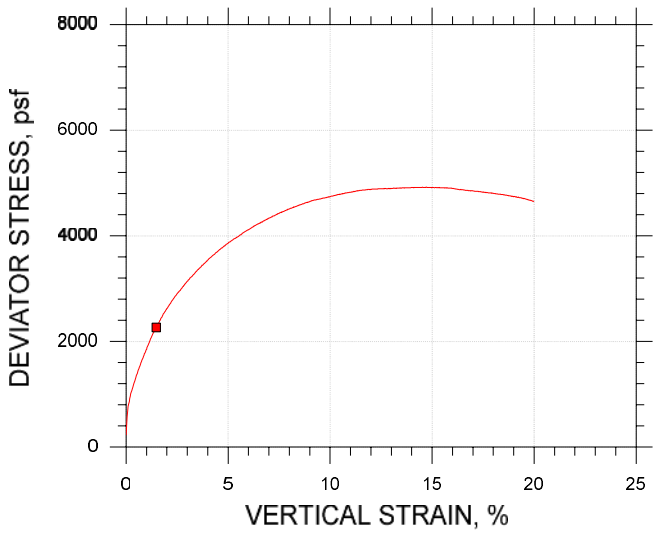
Symbol	Sample ID	Boring	Depth	Natural Moisture Content,%	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	U-1	B-BHR-20	17-19'	21	28	18	10	0.3	Lean CLAY (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client: WSP USA, Inc.	
Project Name: Hogan Road Bridge Bangor PO 004	
Project Location: Bangor, ME	
Project Number: GTX-318065	
Tested By: jlw	Checked By: njh
Boring ID: BB-BHR-201	
Preparation: intact	
Description: Moist, gray clay	
Classification: Lean CLAY	
Group Symbol: CL	
Liquid Limit: 28	Plastic Limit: 18
Plasticity Index: 10	Estimated Specific Gravity: 2.7

CONSOLIDATED UNDRAINED TRIAXIAL TEST - ANISOTROPIC CONSOLIDATION





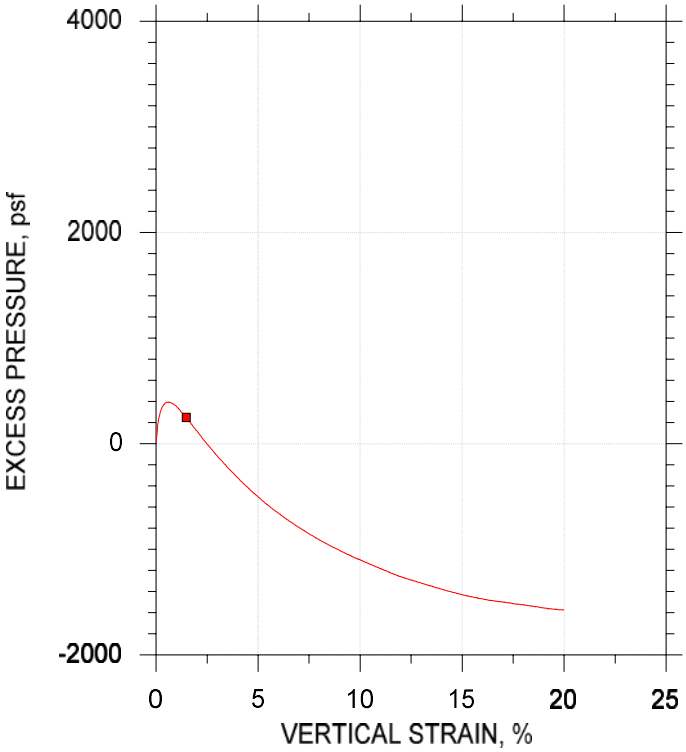
Symbol		■			
Sample ID		U-1			
Depth, ft		17-19'			
Test Number		CAU-1			
Initial	Height, in	5.890			
	Diameter, in	2.850			
	Moisture Content (from Cuttings), %	26.8			
	Dry Density, pcf	97.4			
	Saturation (Wet Method), %	99.1			
	Void Ratio	0.730			
Before Shear	Moisture Content, %	27.2			
	Dry Density, pcf	97.2			
	Cross-sectional Area (Method A), in²	6.399			
	Saturation, %	100.0			
	Void Ratio	0.734			
	Back Pressure, psf	1.080e+004			
Vertical Effective Consolidation Stress, psf		1139.			
Horizontal Effective Consolidation Stress, psf		915.6			
Vertical Strain after Consolidation, %		0.3175			
Volumetric Strain after Consolidation, %		0.5126			
Time to 50% Consolidation, min		---			
Shear Strength, psf		1132.			
Strain at Failure, %		1.48			
Strain Rate, %/min		0.01600			
Deviator Stress at Failure, psf		2265.			
Effective Minor Principal Stress at Failure, psf		662.4			
Effective Major Principal Stress at Failure, psf		2927.			
B-Value		0.95			

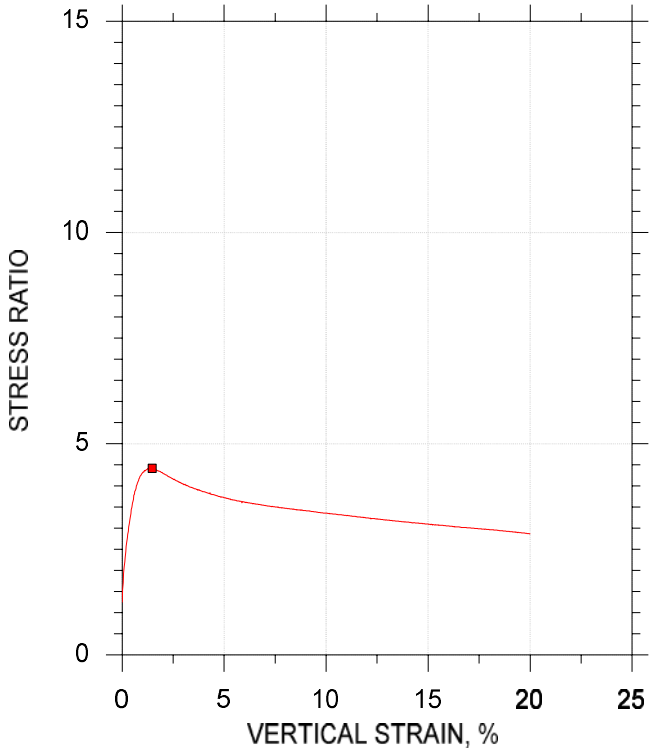
Notes:
- Before Shear Saturation set to 100% for phase calculation.
- Moisture Content determined by ASTM D2216.
- Atterberg Limits determined by ASTM D4318.
- Deviator Stress includes membrane correction.
- Values for c and φ determined from best-fit straight line for the specific test conditions. Actual strength parameters may vary and should be determined by an engineer for site conditions.

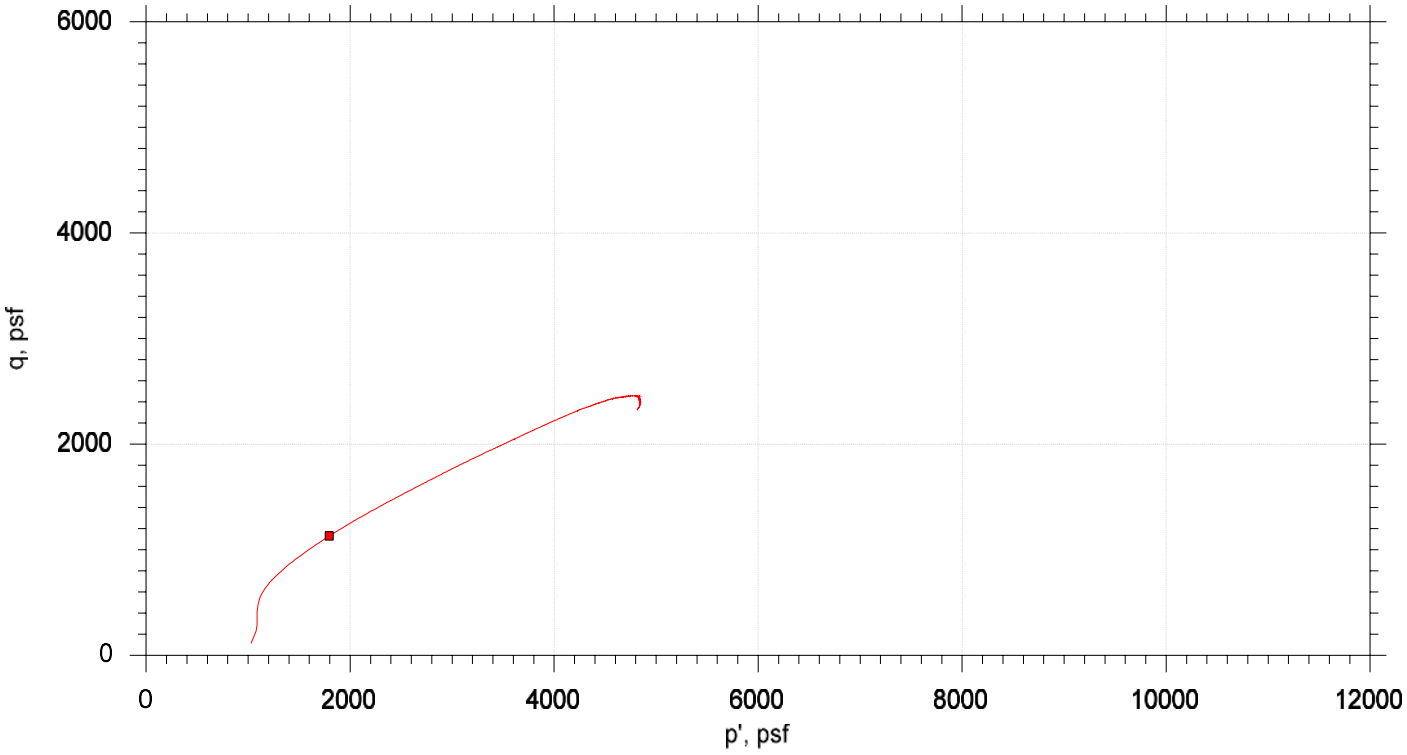
Remarks:

CONSOLIDATED UNDRAINED TRIAXIAL TEST

ANISOTROPIC CONSOLIDATION







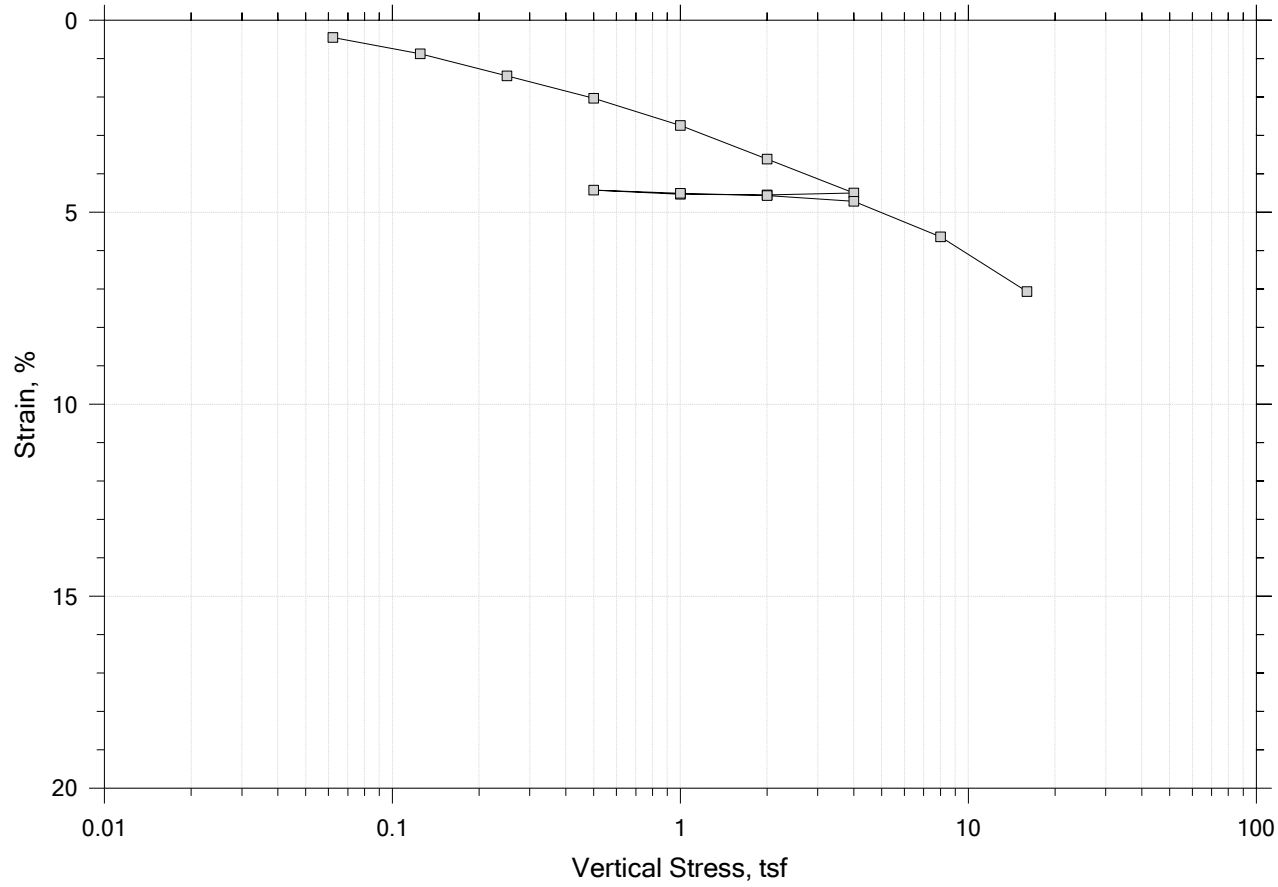
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
■	U-1	CAU-1	17-19'	jlw	12/13/23	njh	12/19/23	318065-CAU-1.dat

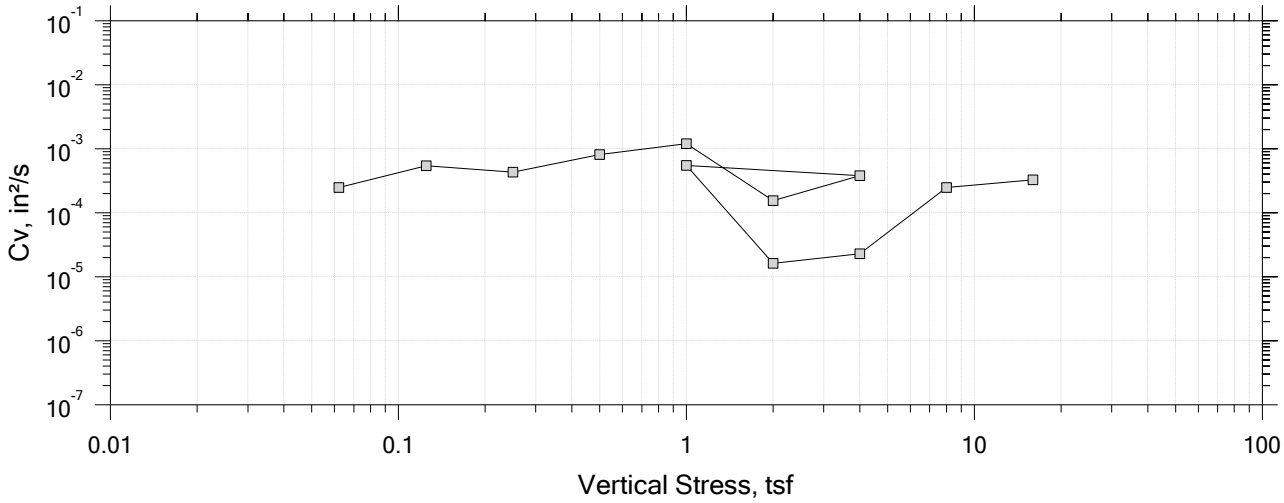
GeoTesting EXPRESS

Project: Hogan Road Bridge Bangor PO 004	Location: Bangor, ME	Project No.: GTX-318065
Boring No.: BB-BHR-201	Sample Type: intact	
Description: Moist, gray clay		
Remarks: TX-017		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

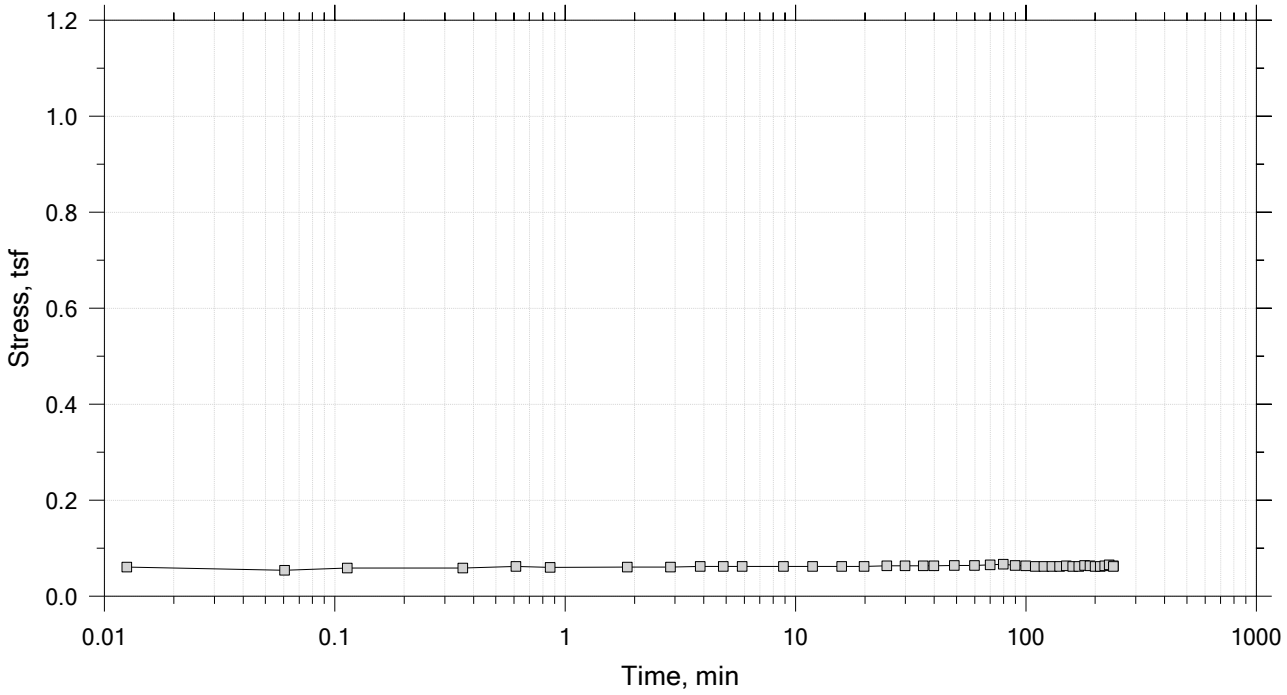


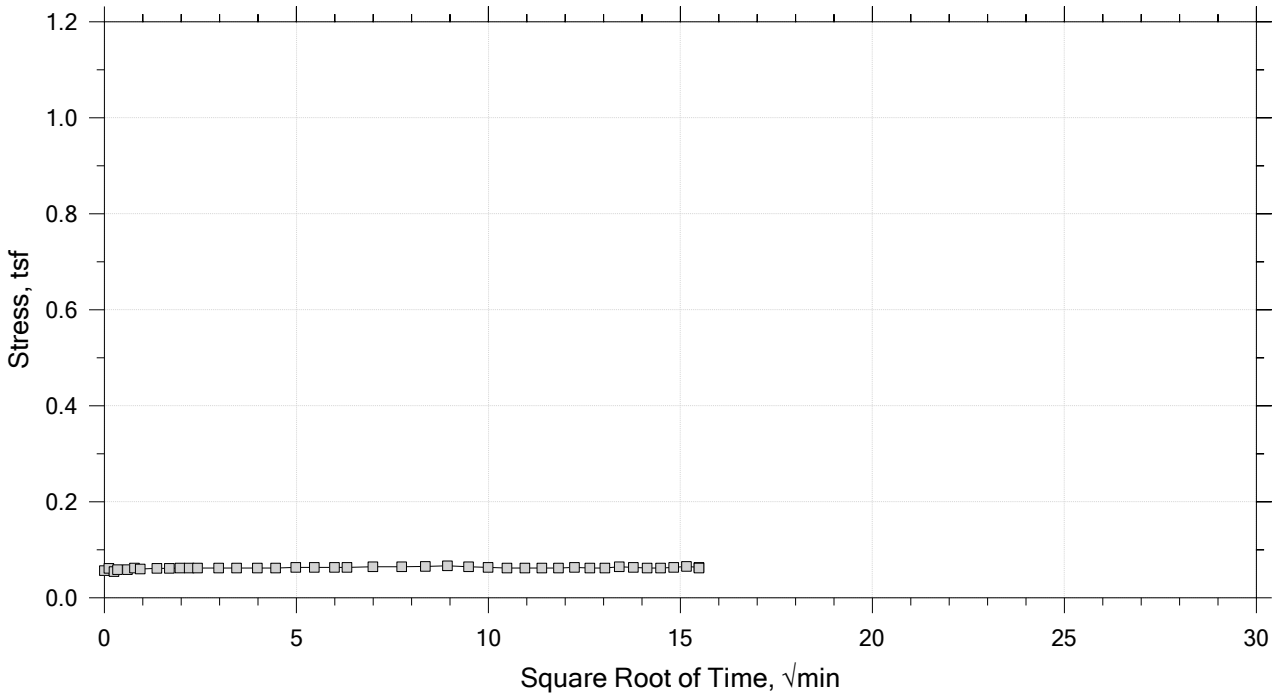


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 1 of 15
Constant Volume Step
Stress: 0.0621 tsf

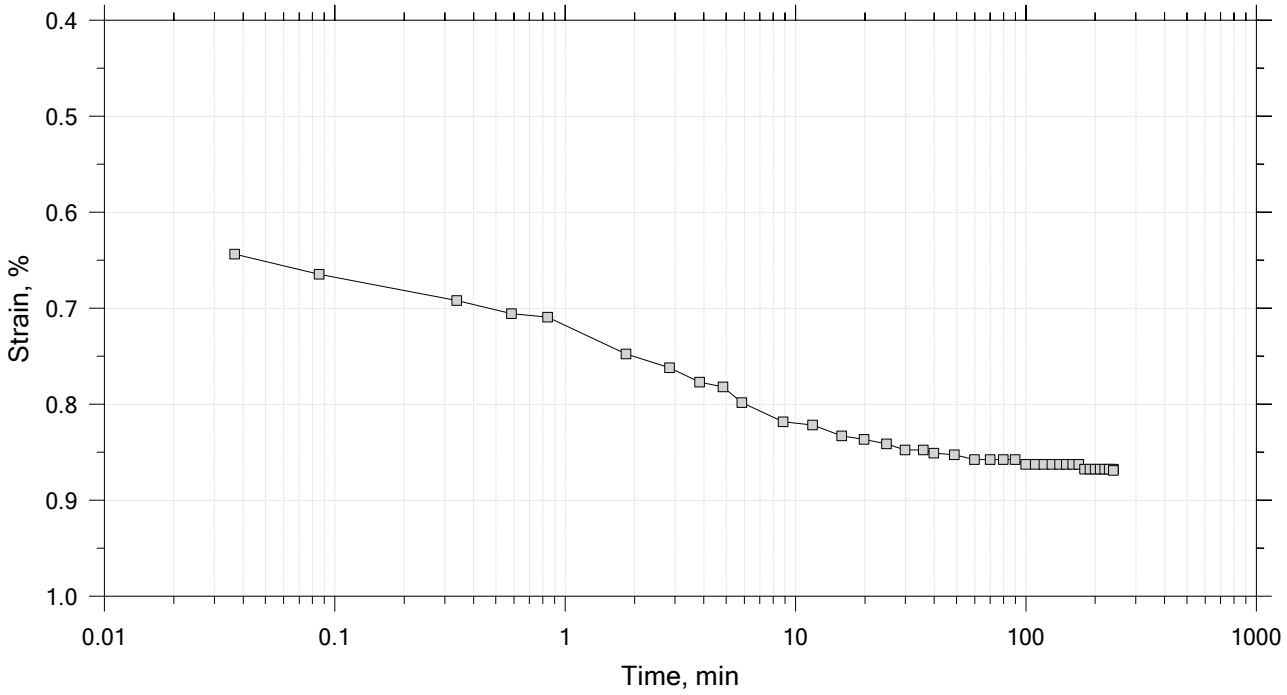


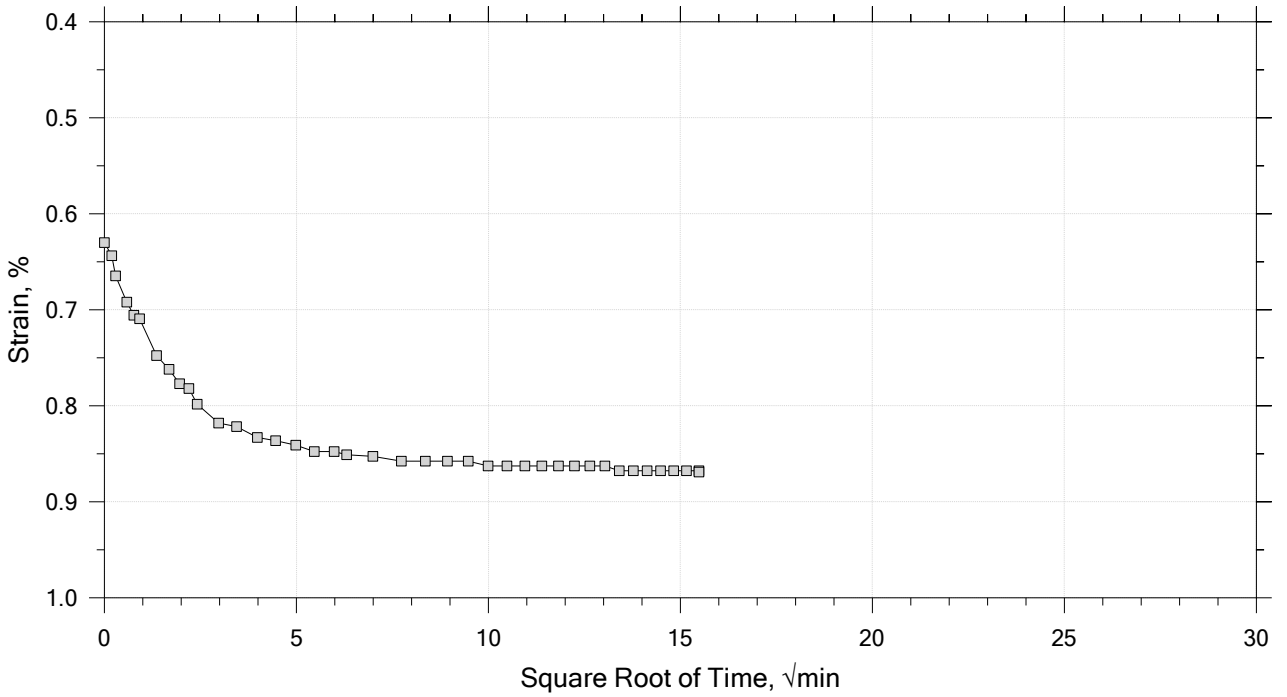


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 15
Constant Load Step
Stress: 0.125 tsf

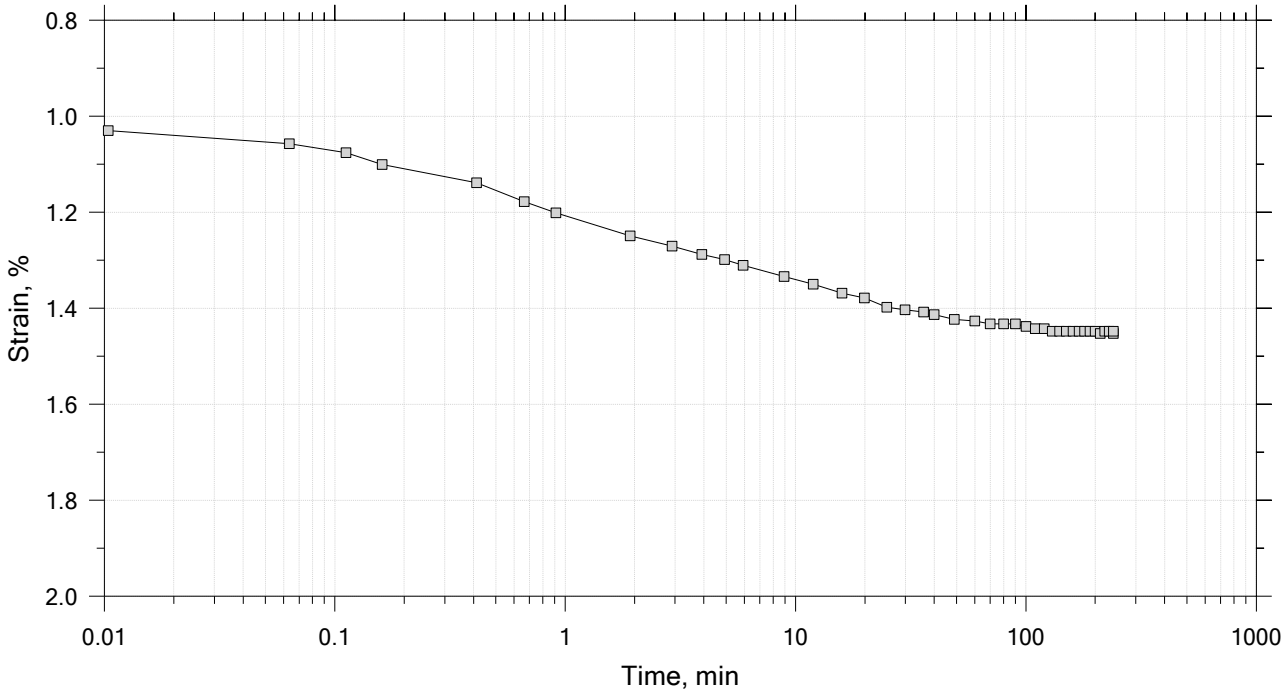


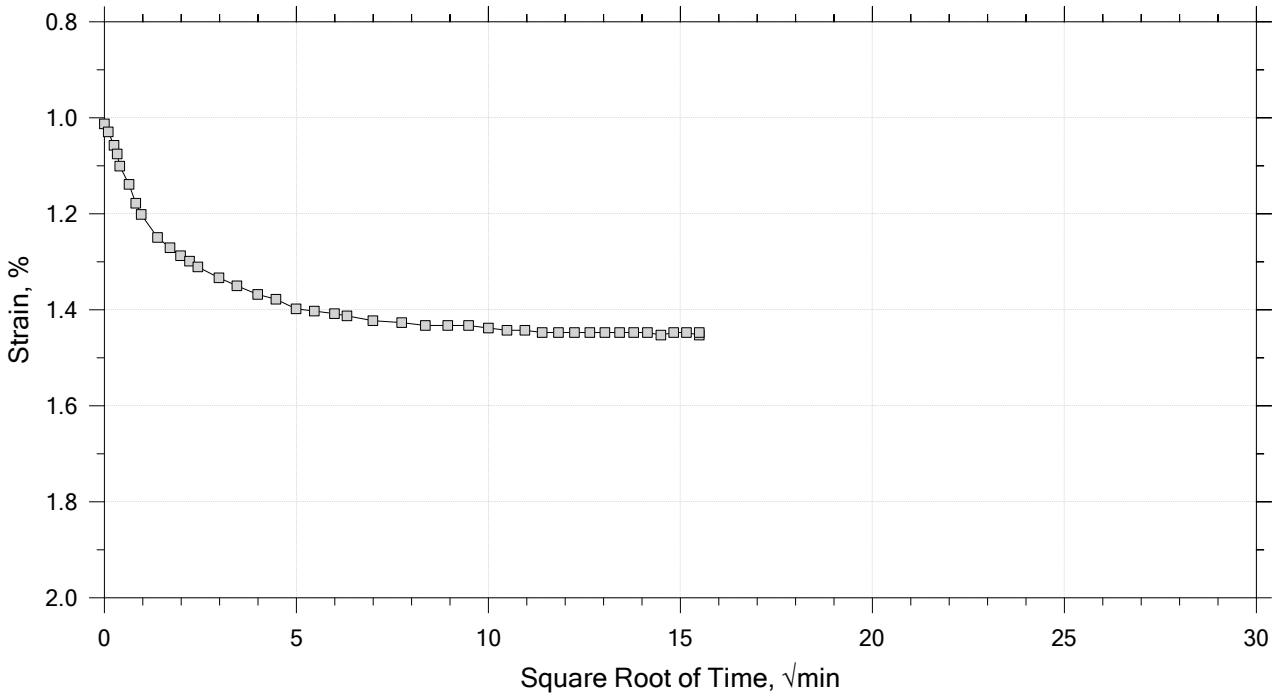


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 15
Constant Load Step
Stress: 0.25 tsf

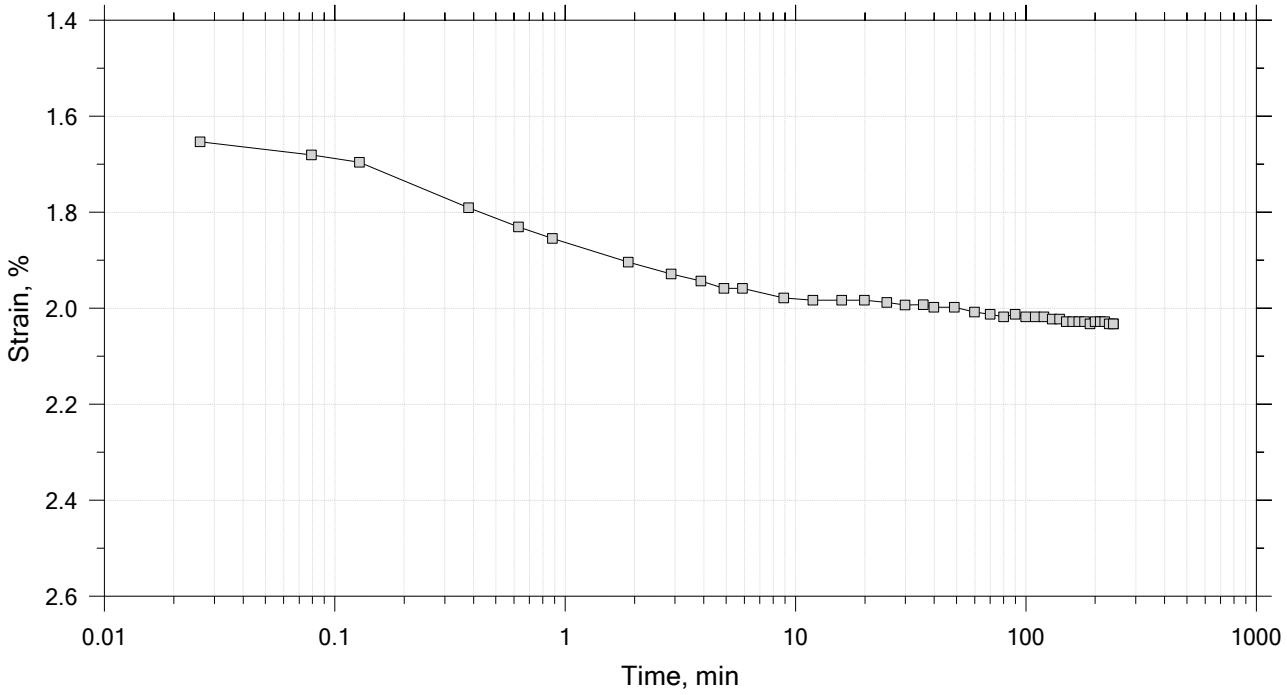


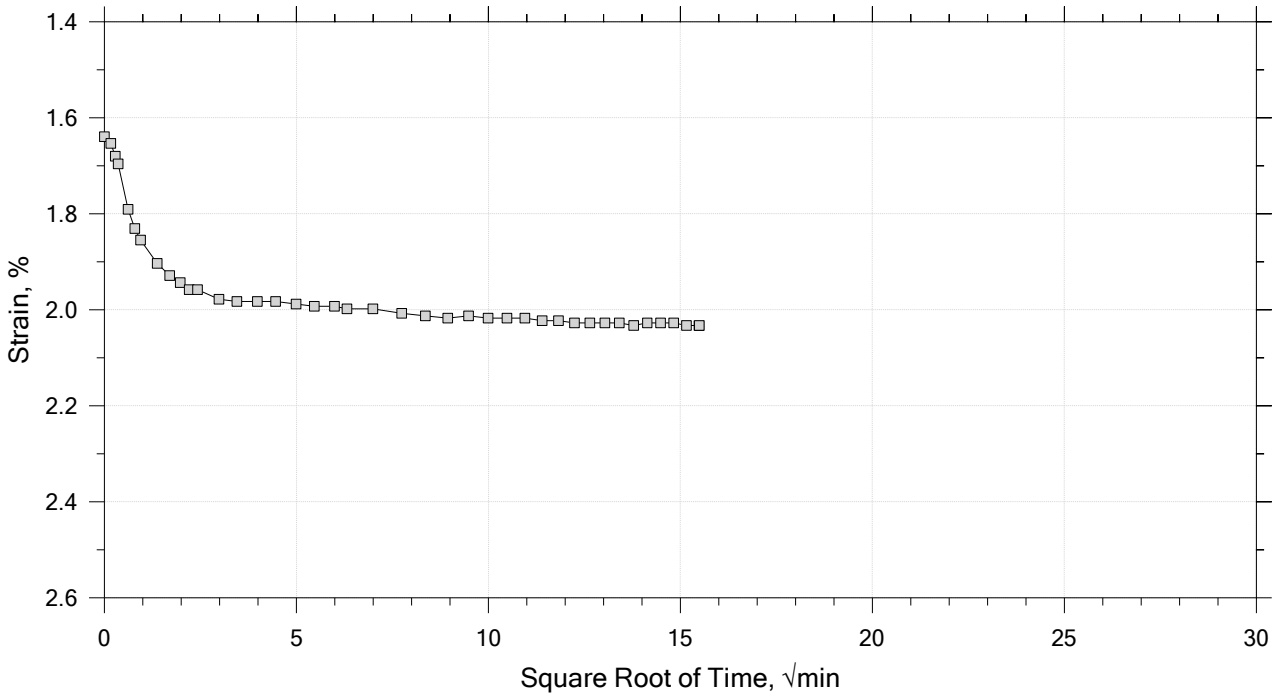


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 15
Constant Load Step
Stress: 0.5 tsf

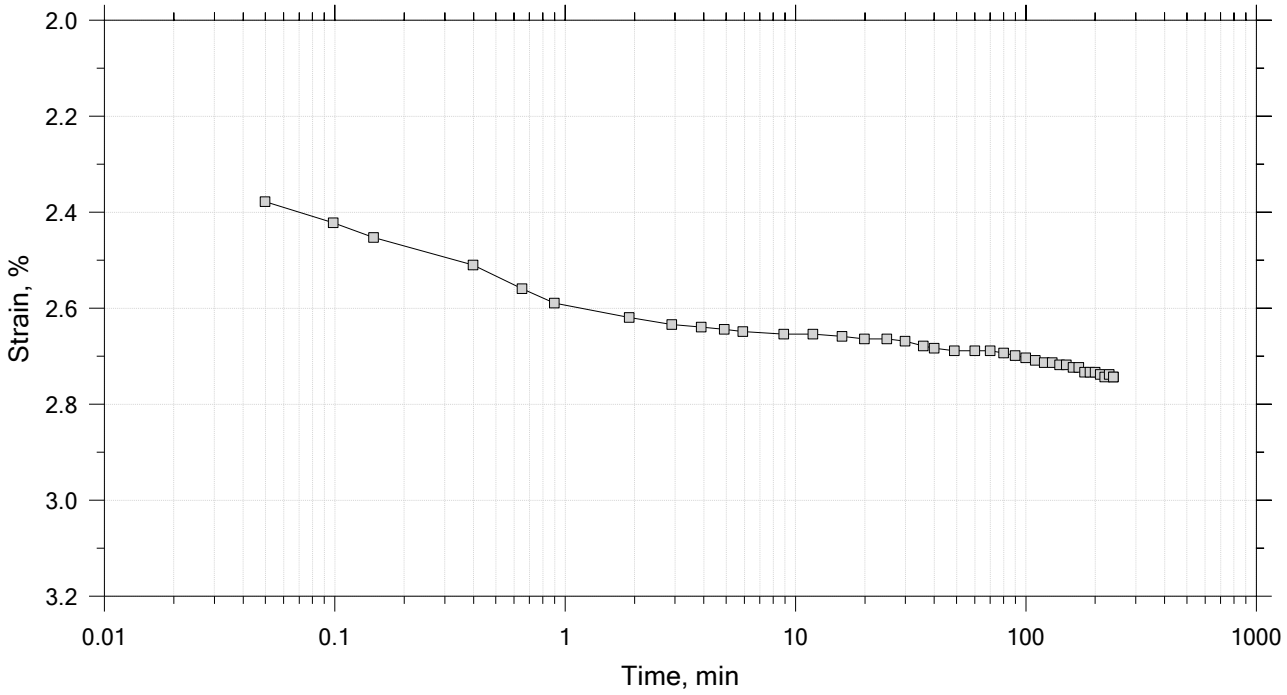


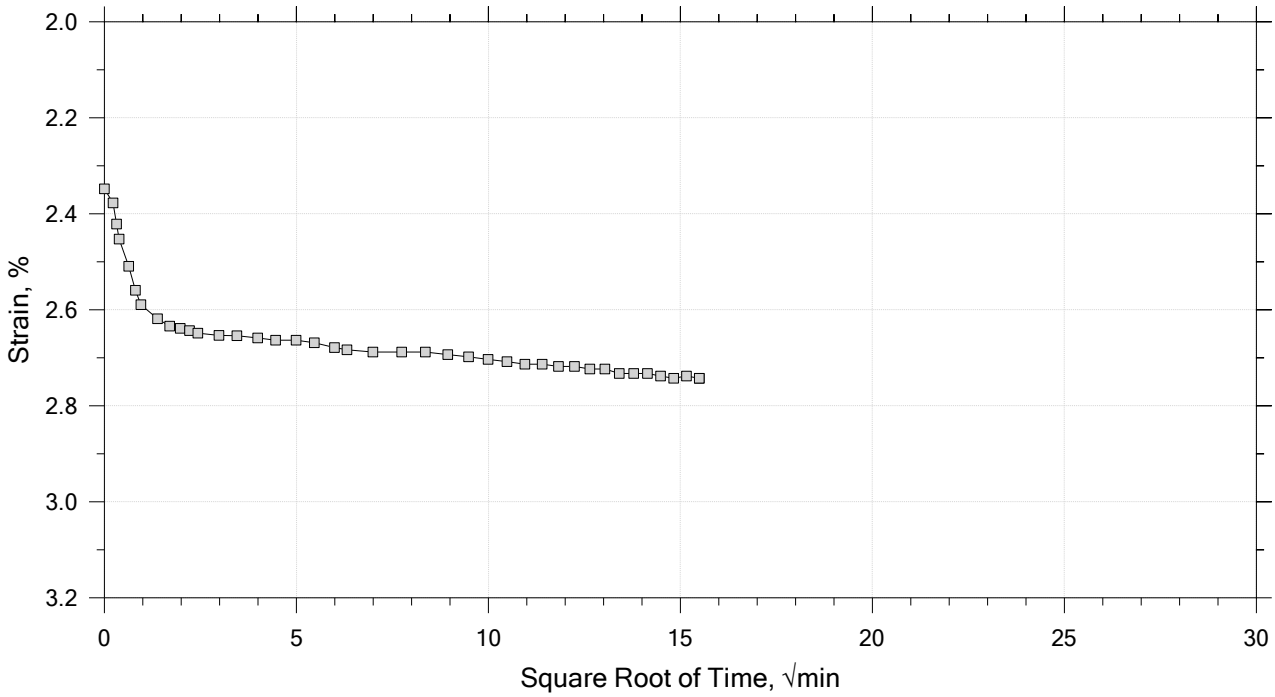


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 15
Constant Load Step
Stress: 1 tsf

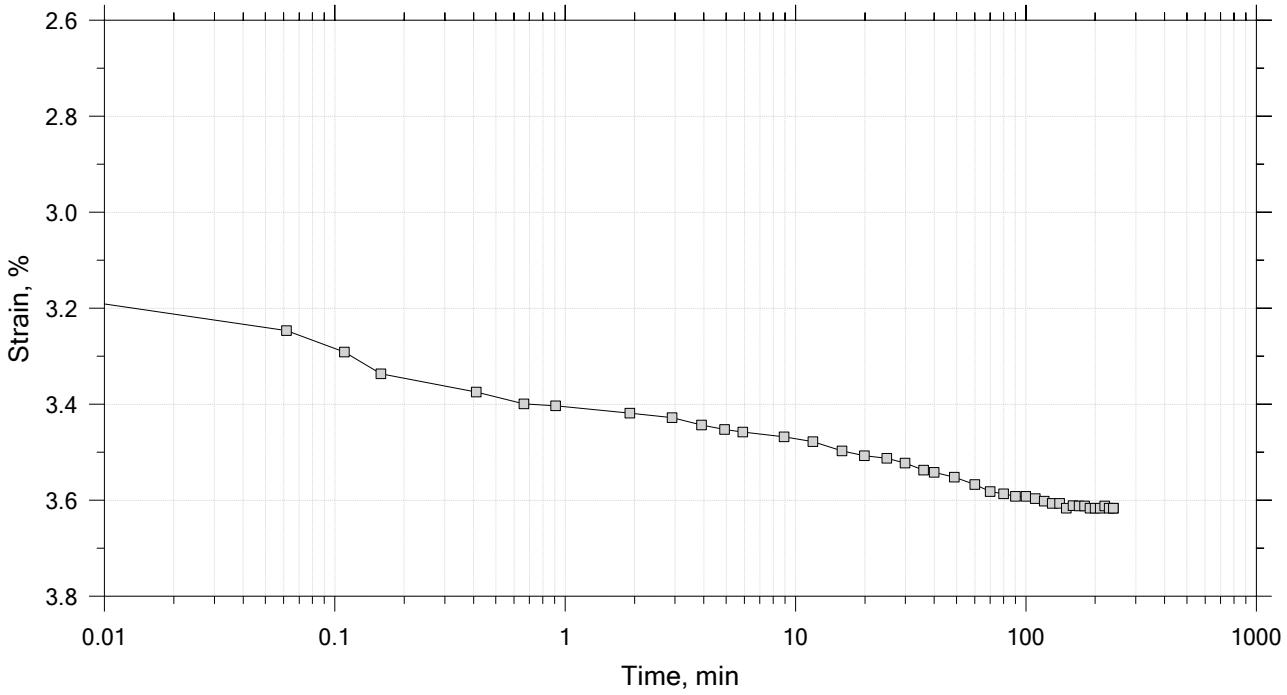


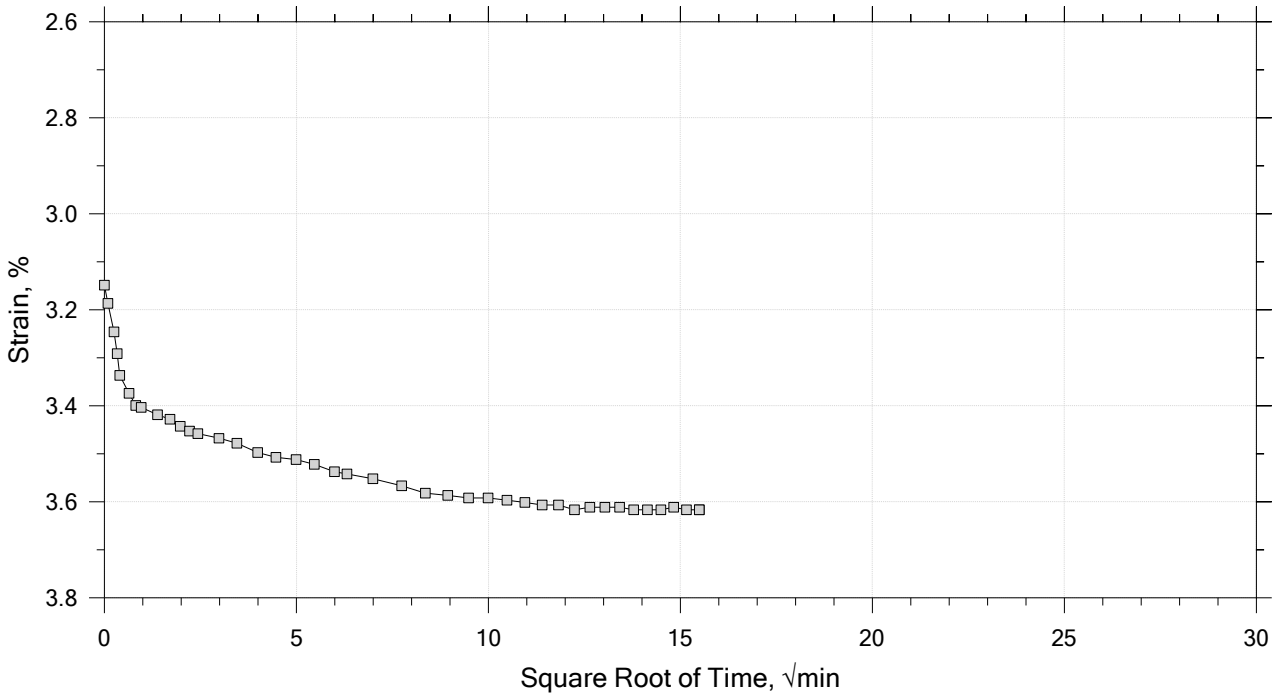


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 15
Constant Load Step
Stress: 2 tsf

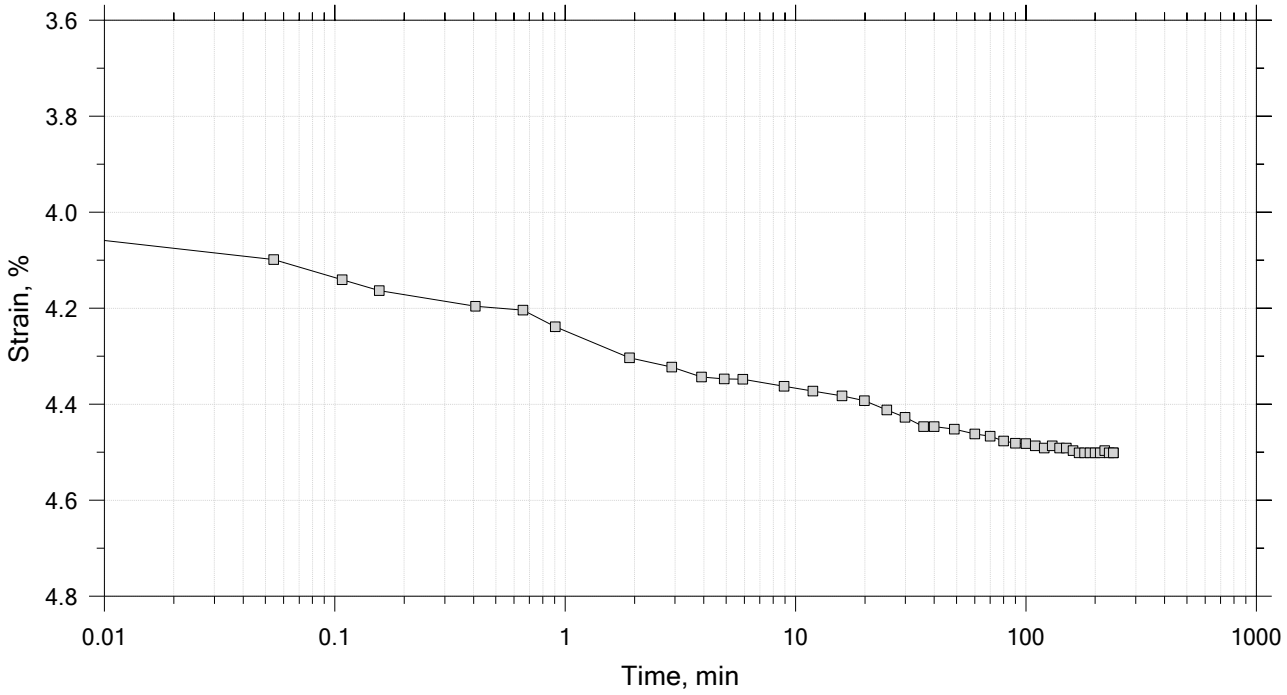


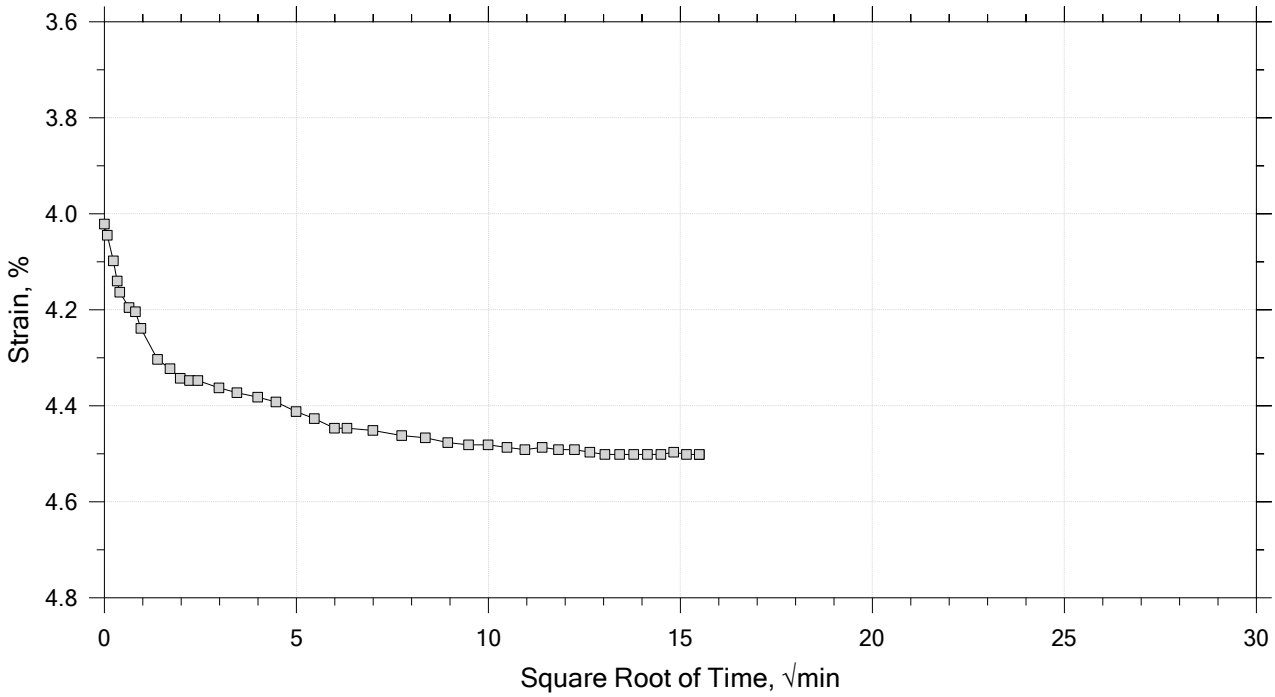


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 15
Constant Load Step
Stress: 4 tsf

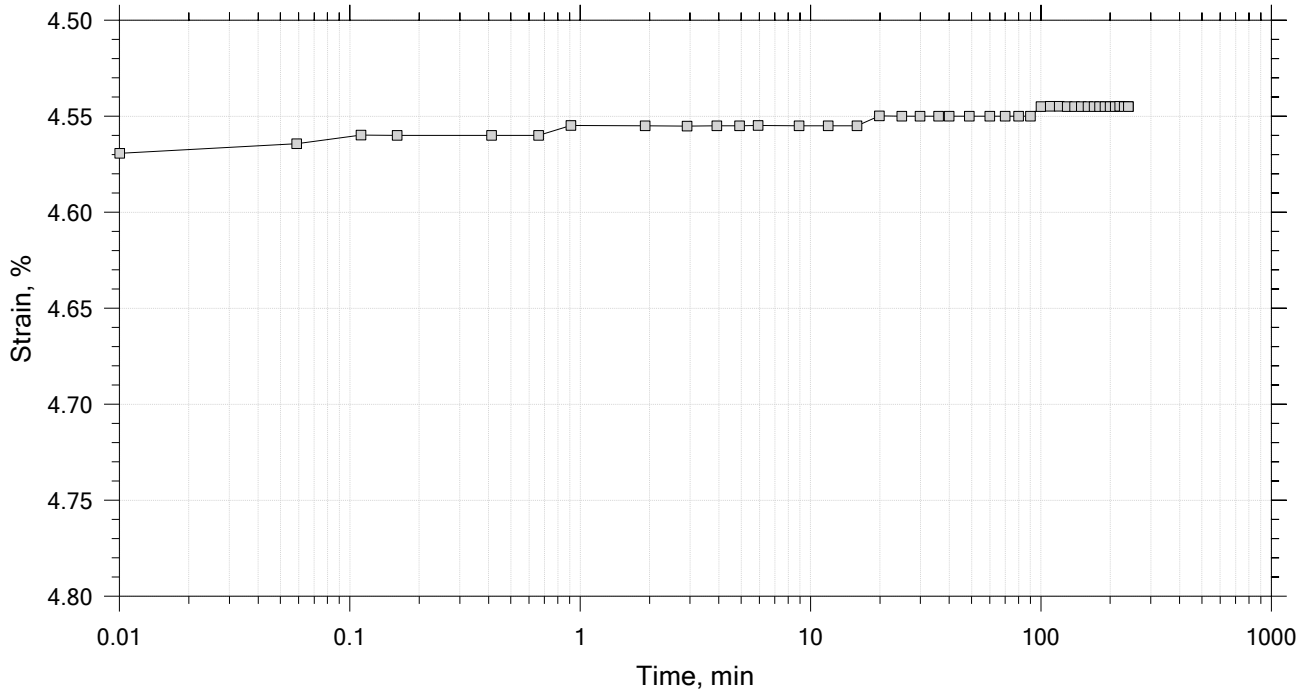


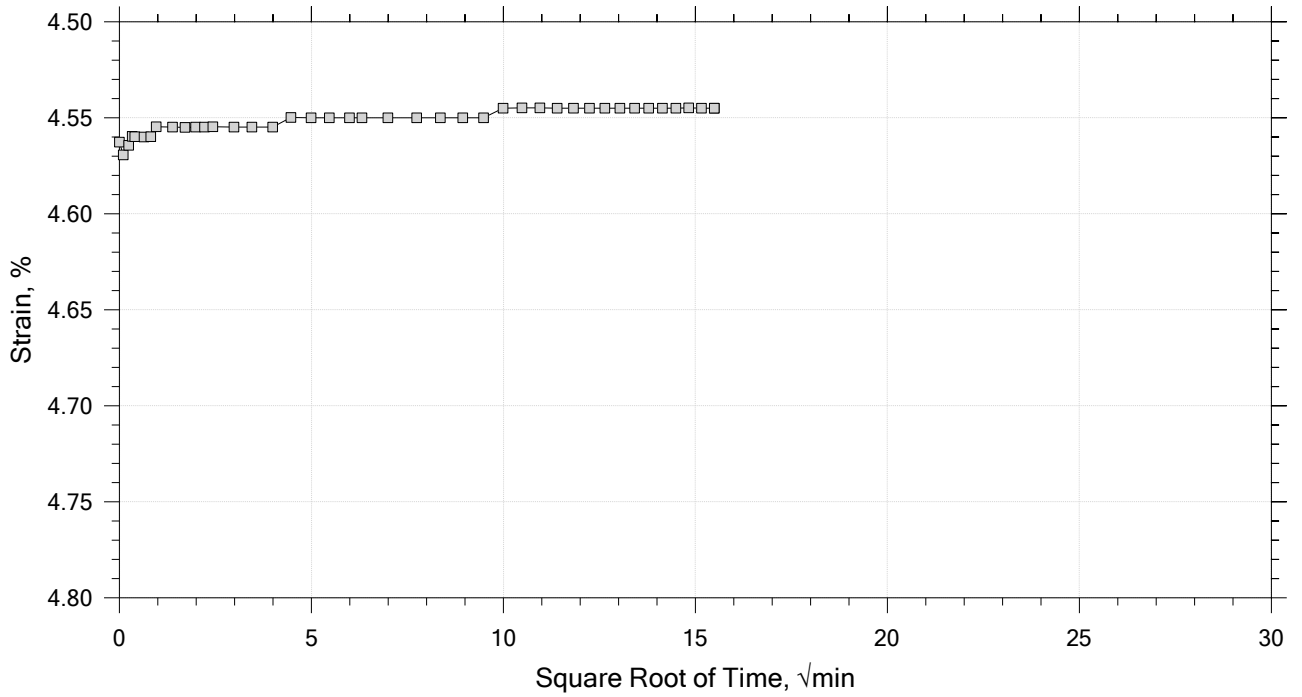


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 15
Constant Load Step
Stress: 2 tsf

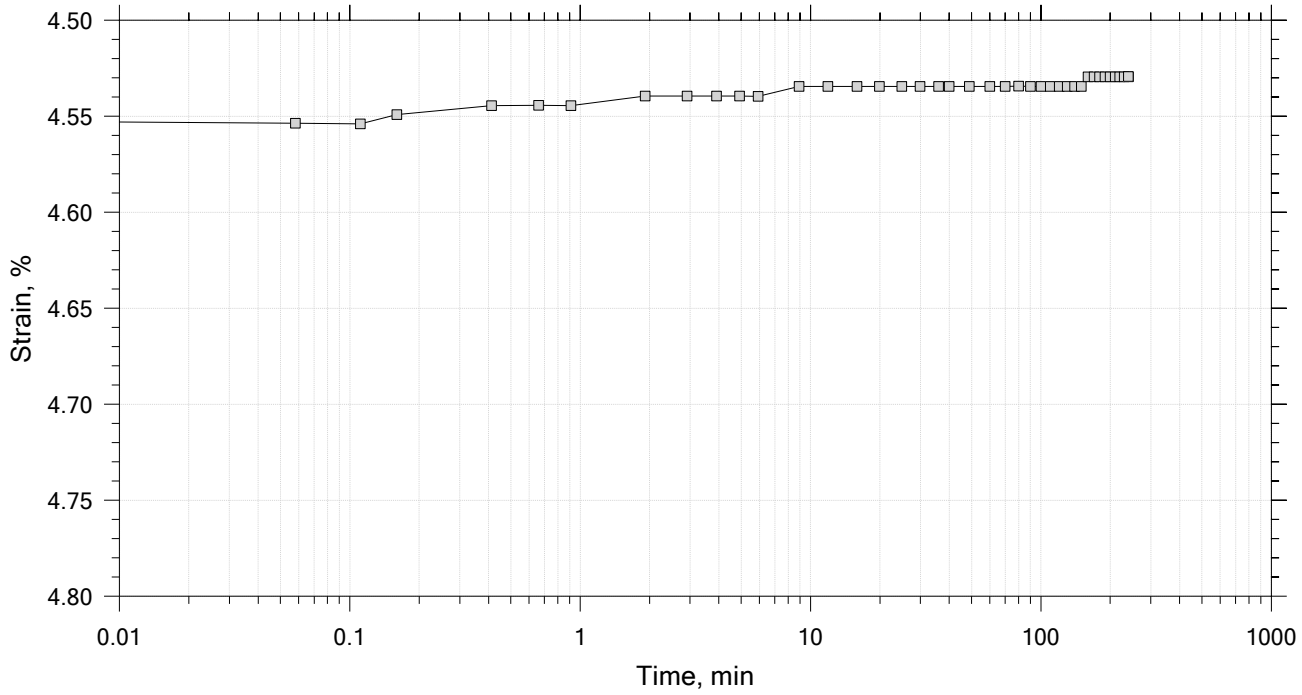


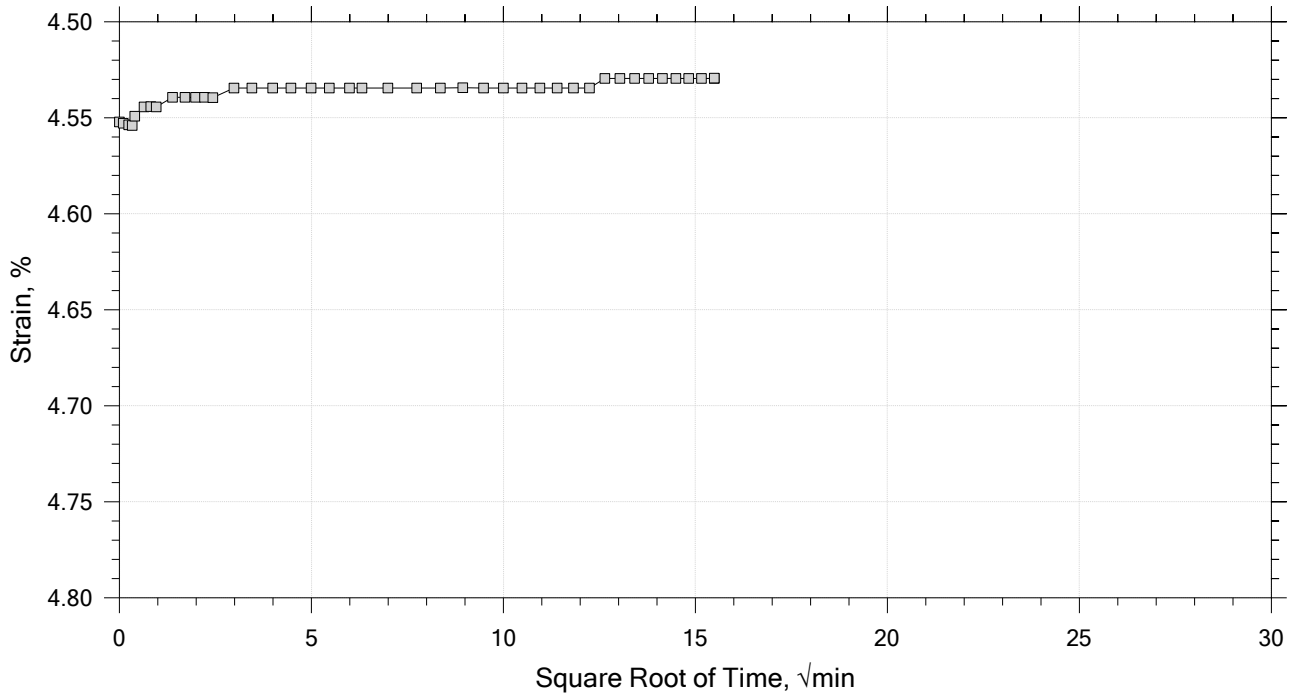


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 15
Constant Load Step
Stress: 1 tsf

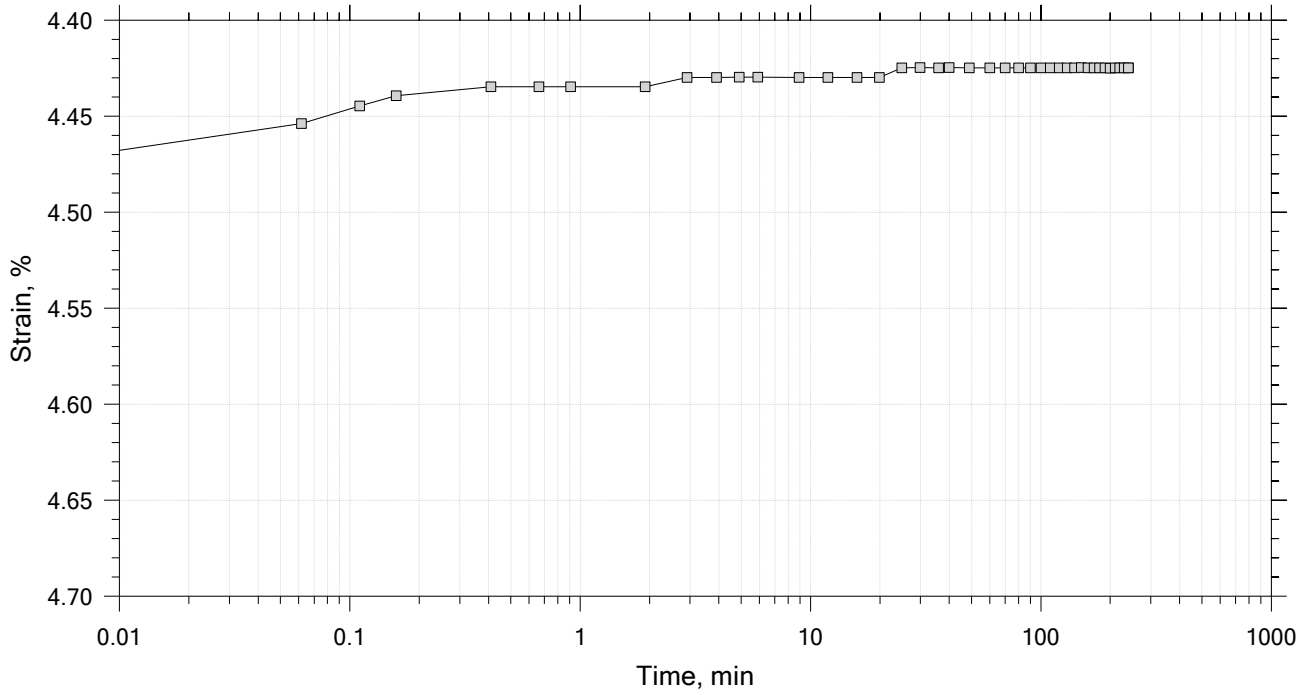


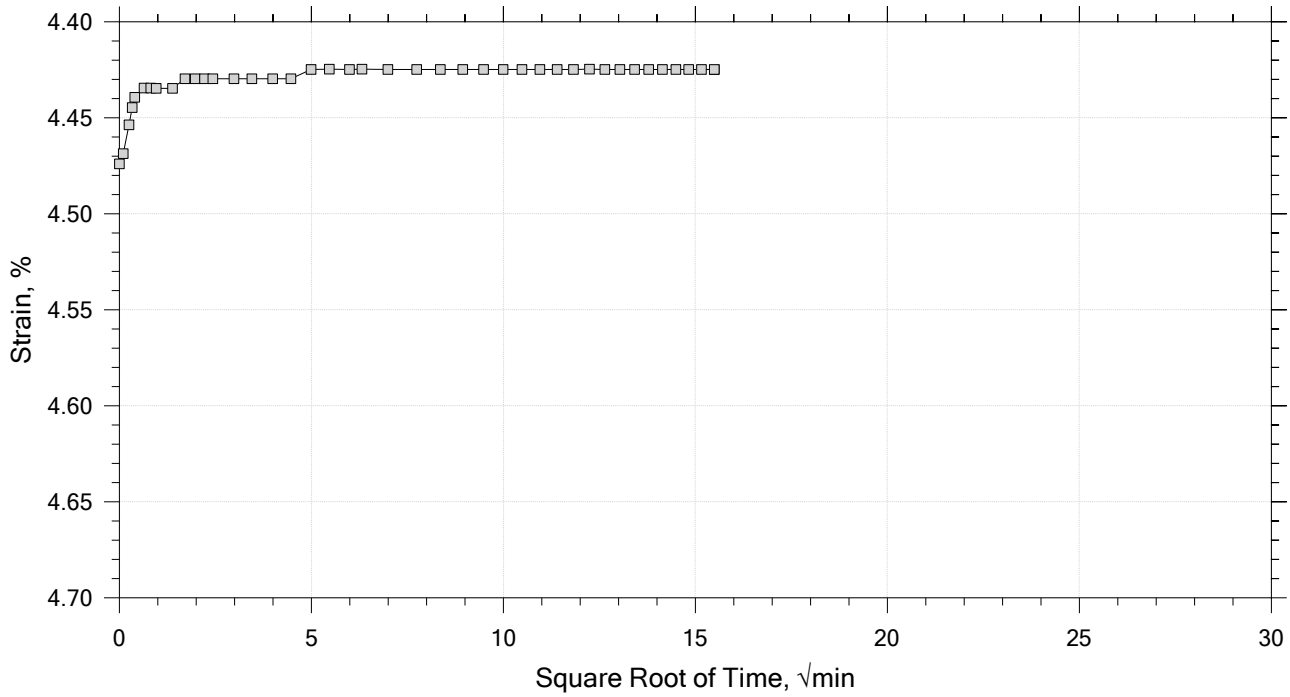


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 15
Constant Load Step
Stress: 0.5 tsf

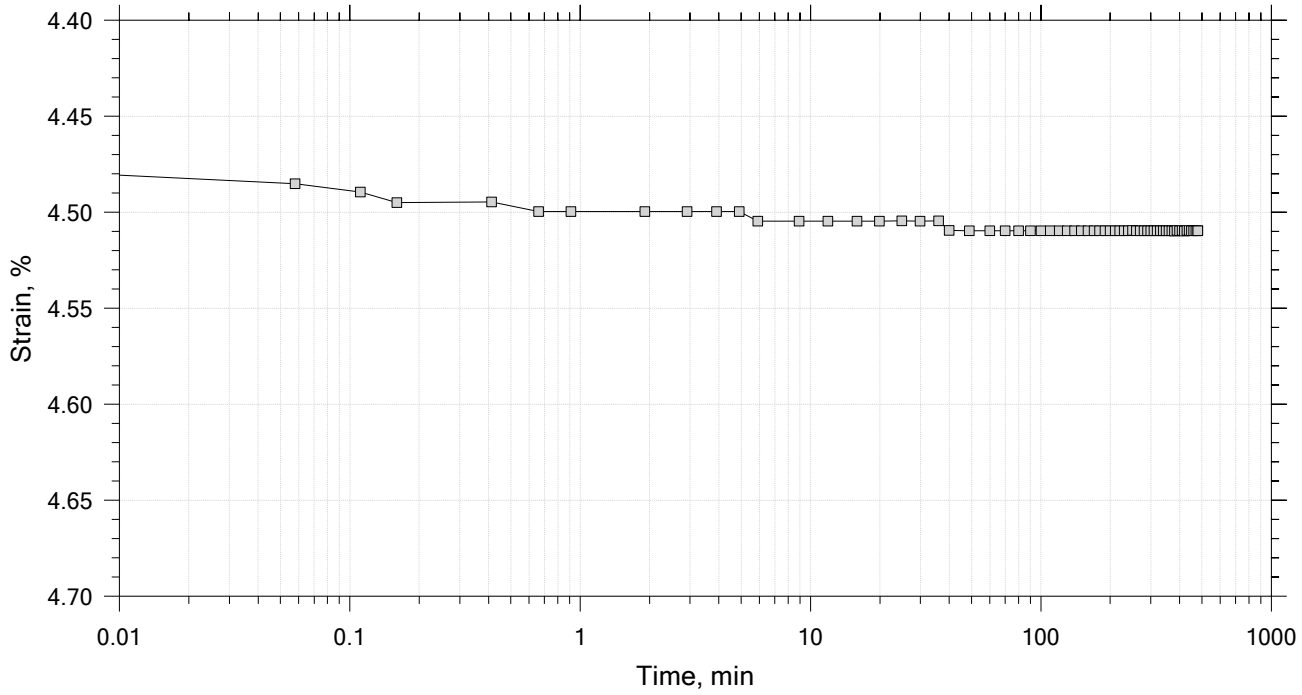


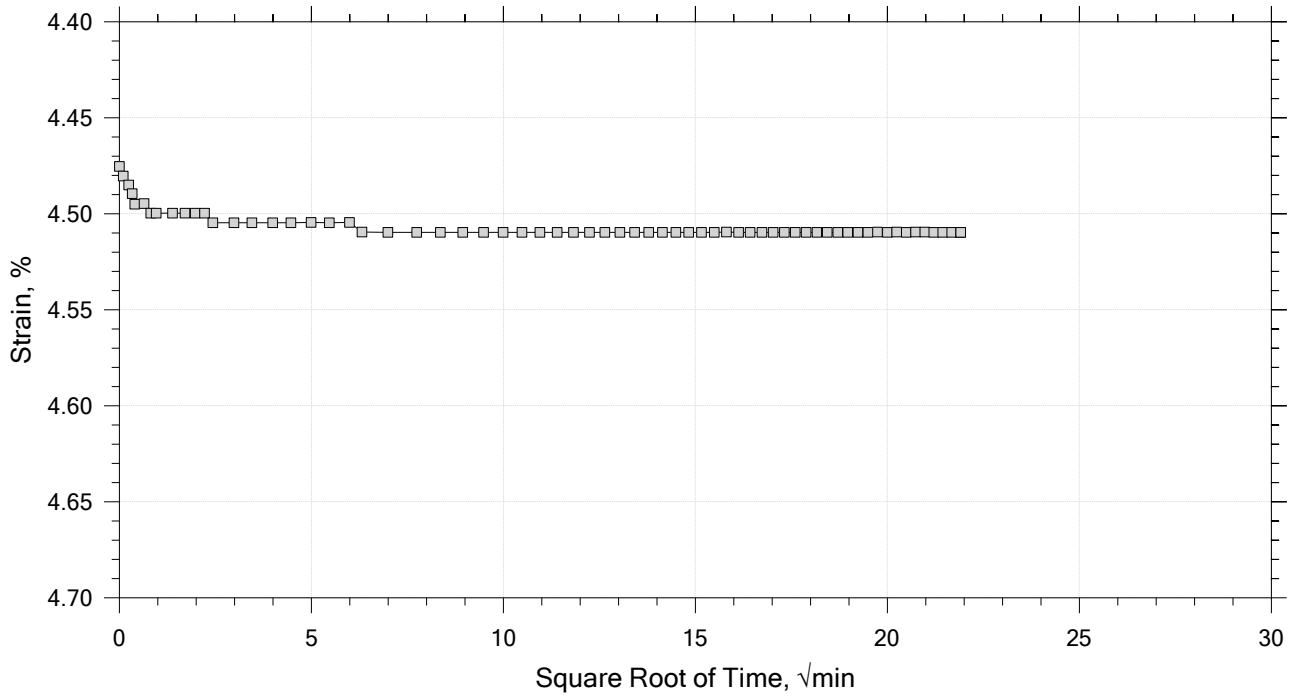


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 15
Constant Load Step
Stress: 1 tsf

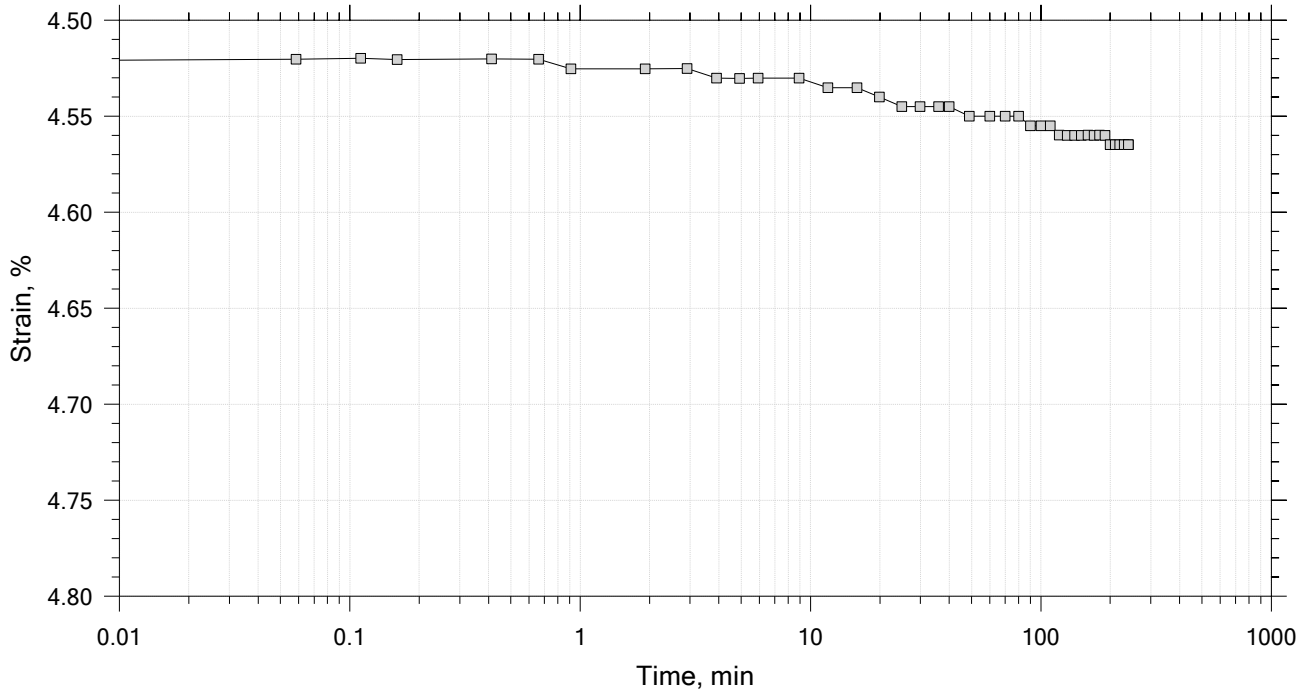


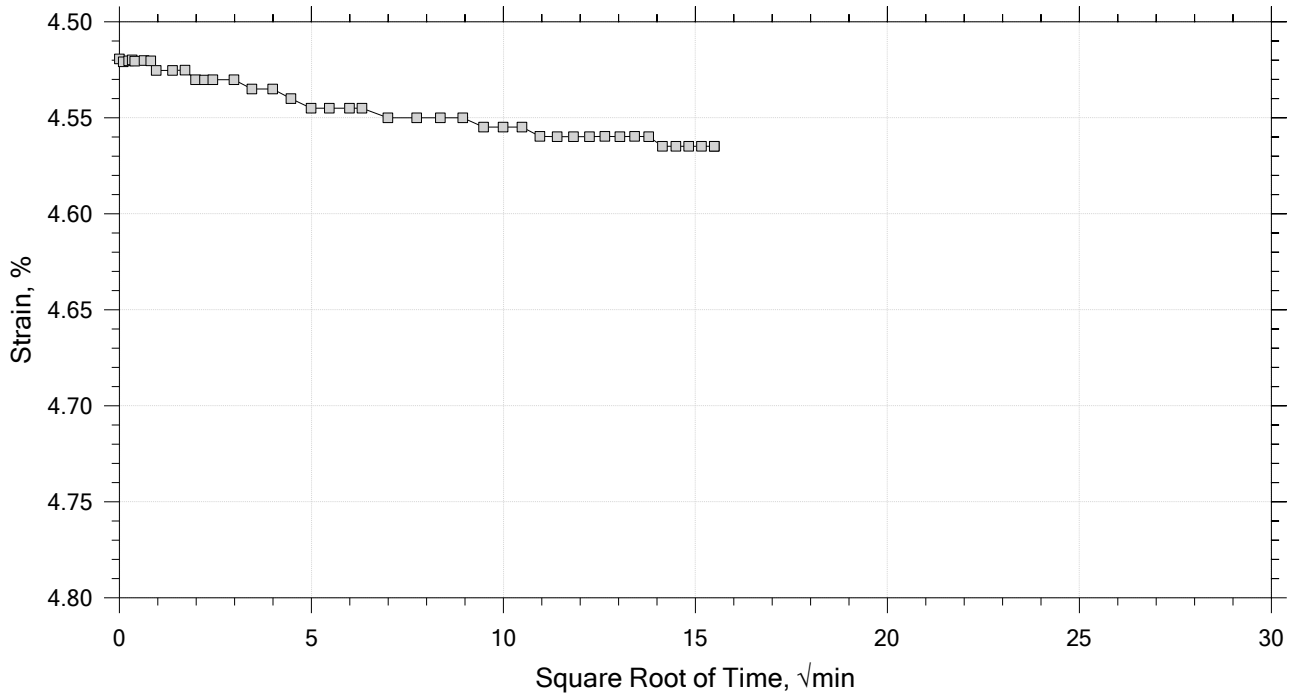


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 15
Constant Load Step
Stress: 2 tsf

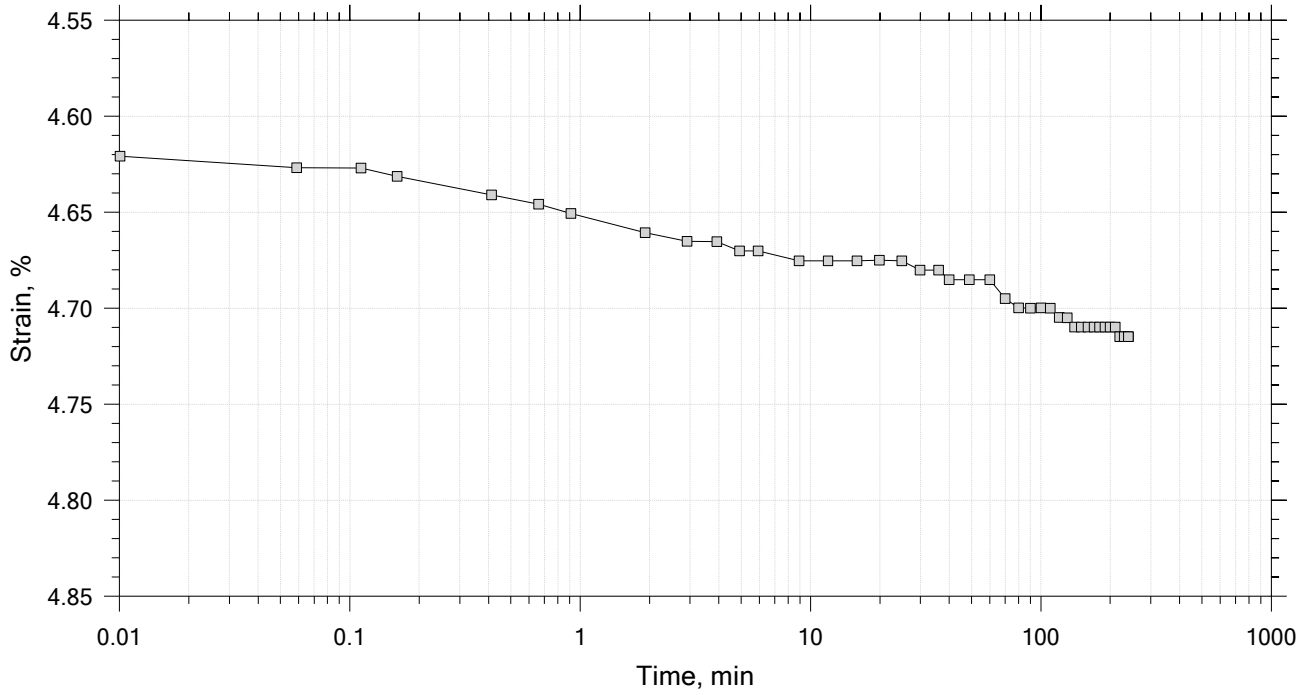


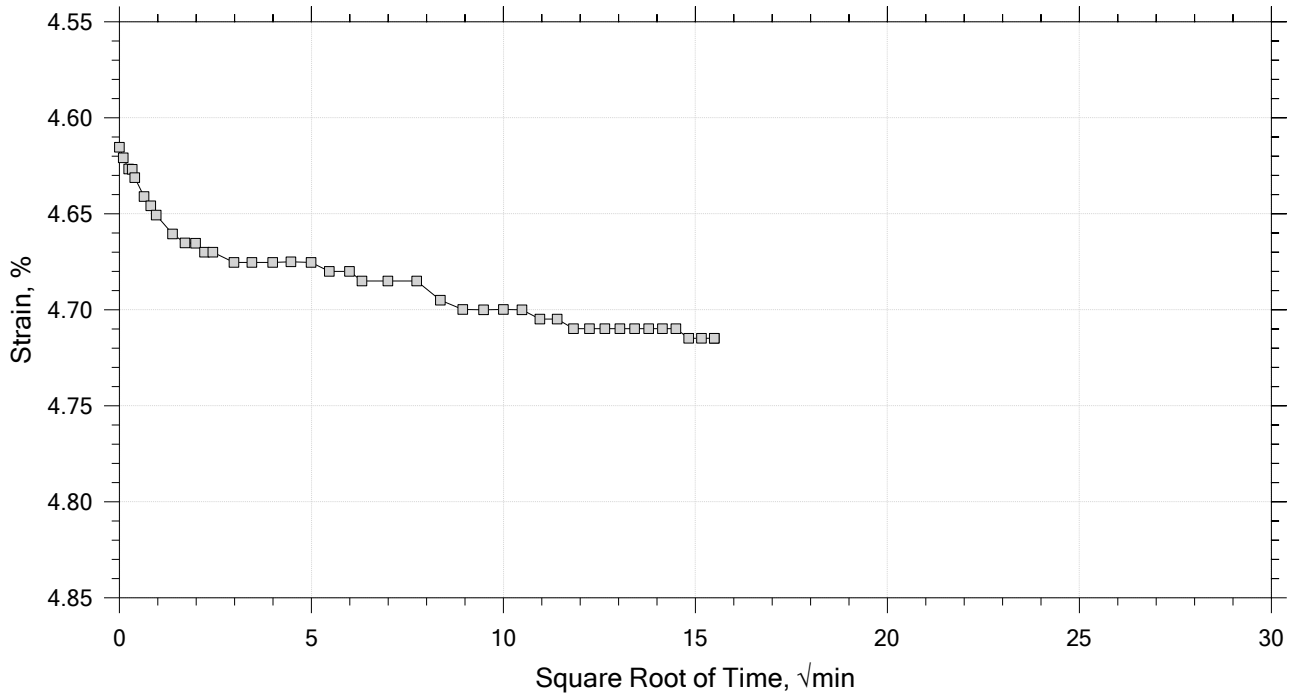


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 15
Constant Load Step
Stress: 4 tsf

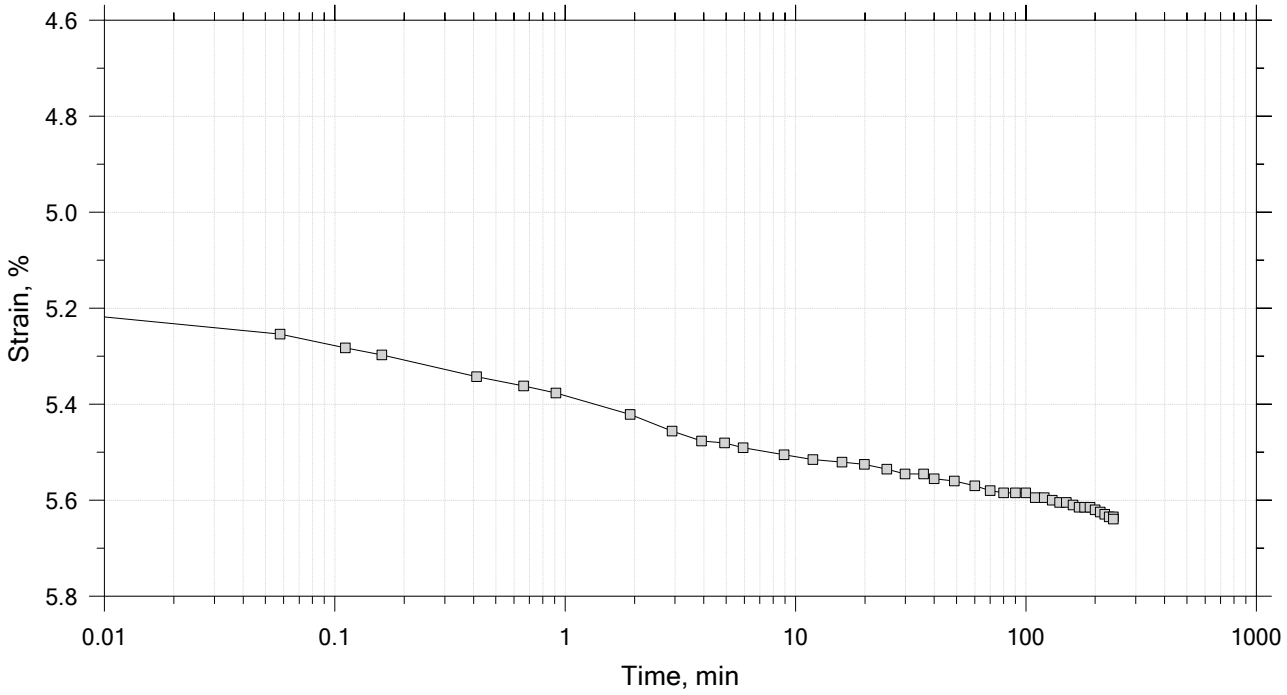


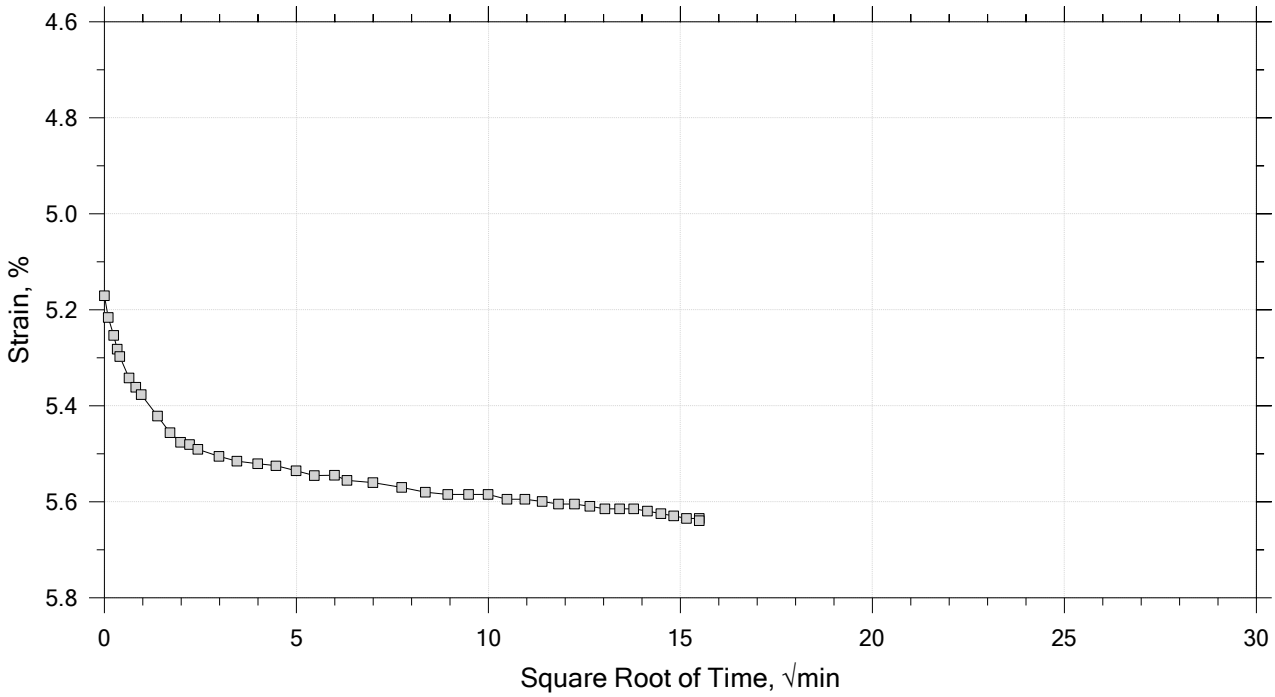


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 14 of 15
Constant Load Step
Stress: 8 tsf

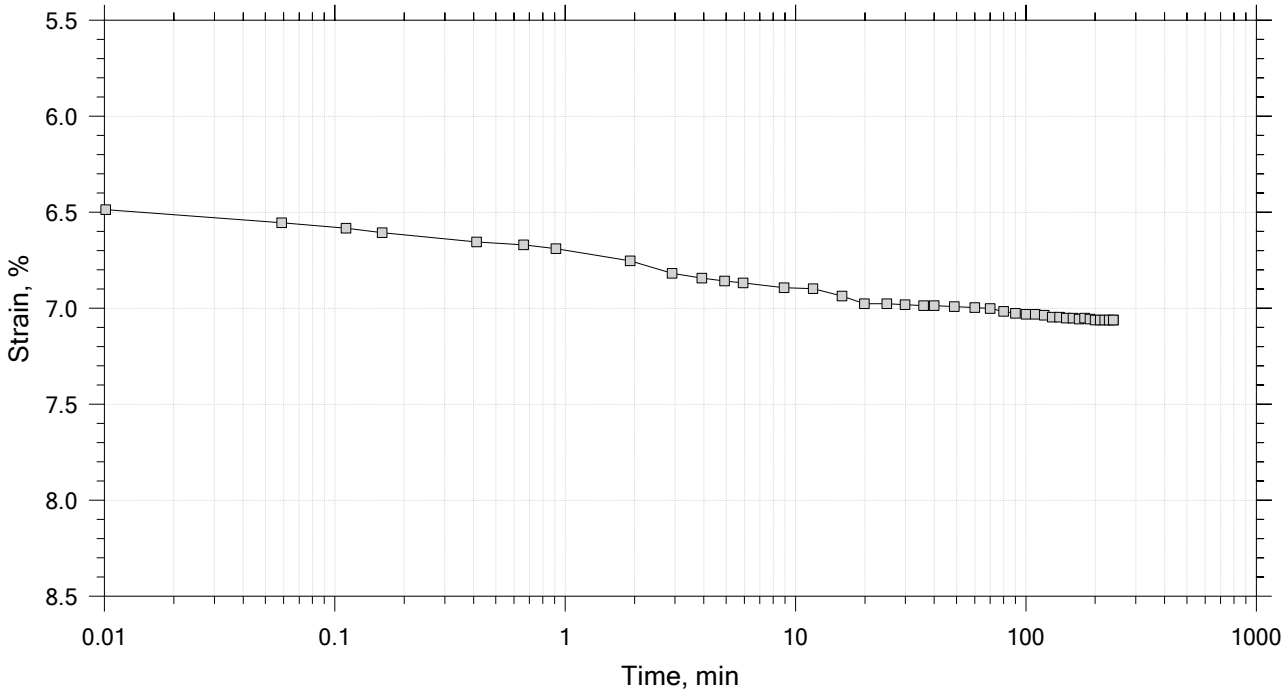


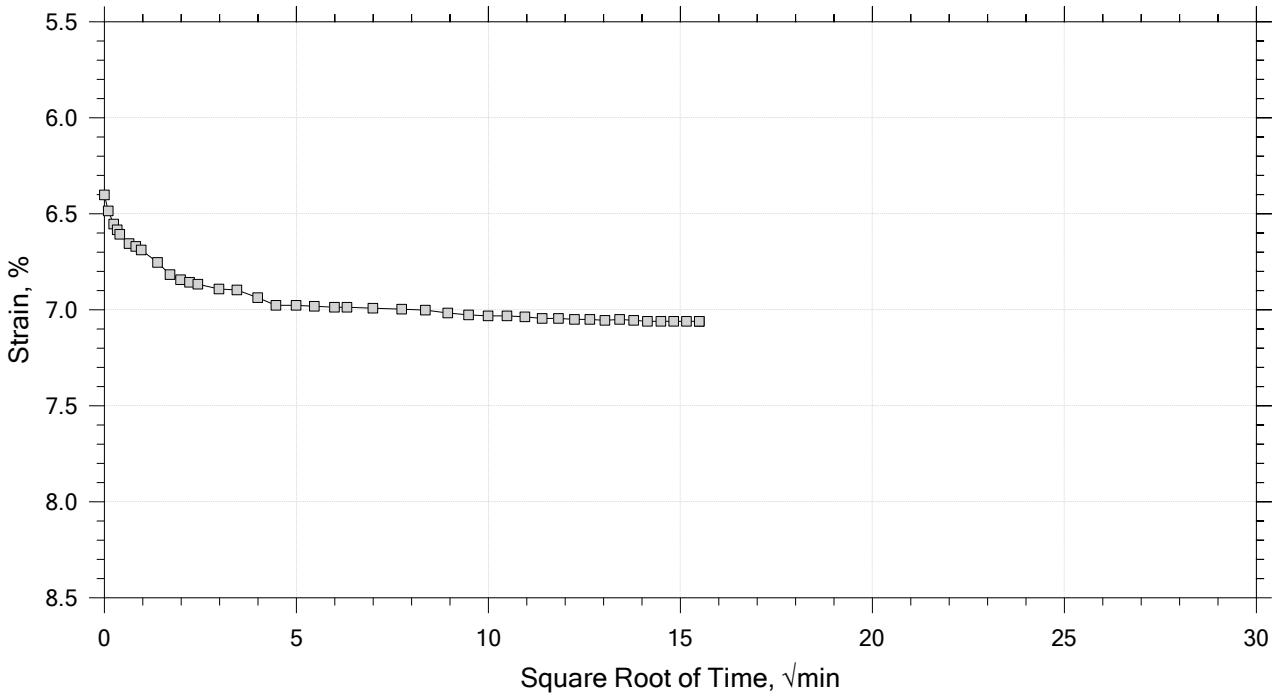


GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 15 of 15
Constant Load Step
Stress: 16 tsf





GeoTesting EXPRESS	Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

Specimen Diameter: 2.50 in	Estimated Specific Gravity: 2.76	Liquid Limit: 28
Initial Height: 1.00 in	Initial Void Ratio: 0.566	Plastic Limit: 18
Final Height: 0.91 in	Final Void Ratio: 0.426	Plasticity Index: 10

	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E3087	RING		E7788
Mass Container, gm	8.36	110.31	110.31	8.27
Mass Container + Wet Soil, gm	151.72	280.91	273.68	173.81
Mass Container + Dry Soil, gm	126.83	251.83	251.83	151.67
Mass Dry Soil, gm	118.47	141.52	141.52	143.4
Water Content, %	21.01	20.55	15.44	15.44
Void Ratio	---	0.57	0.43	---
Degree of Saturation, %	---	99.97	100.00	---
Dry Unit Weight, pcf	---	109.83	120.69	---

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 Bangor	Location: Bangor, ME	Project No.: GTX-318065
	Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
	Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, gray clay		
	Remarks: TX-011, Swell Pressure - 0.0621 tsf		

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	0.0621	0.004462	0.559	0.446	14.265	2.47e-04	7.18e-02	3.32e-04
2	0.125	0.008691	0.553	0.869	6.440	5.41e-04	6.72e-02	6.82e-04
3	0.250	0.01448	0.544	1.45	7.982	4.32e-04	4.63e-02	3.75e-04
4	0.500	0.02033	0.535	2.03	4.207	8.11e-04	2.34e-02	3.55e-04
5	1.00	0.02743	0.524	2.74	2.810	1.20e-03	1.42e-02	3.19e-04
6	2.00	0.03617	0.510	3.62	21.312	1.55e-04	8.74e-03	2.54e-05
7	4.00	0.04501	0.496	4.50	8.609	3.78e-04	4.42e-03	3.13e-05
8	2.00	0.04545	0.495	4.54	99.909	3.22e-05	-2.18e-04	-1.31e-07
9	1.00	0.04529	0.496	4.53	158.404	2.03e-05	1.56e-04	5.95e-08
10	0.500	0.04425	0.497	4.42	0.402	8.03e-03	2.09e-03	3.14e-04
11	1.00	0.04510	0.496	4.51	5.908	5.46e-04	1.70e-03	1.74e-05
12	2.00	0.04565	0.495	4.56	199.908	1.61e-05	5.52e-04	1.67e-07
13	4.00	0.04715	0.493	4.71	139.908	2.30e-05	7.50e-04	3.23e-07
14	8.00	0.05639	0.478	5.64	12.795	2.48e-04	2.31e-03	1.07e-05
15	16.0	0.07061	0.456	7.06	9.451	3.28e-04	1.78e-03	1.09e-05

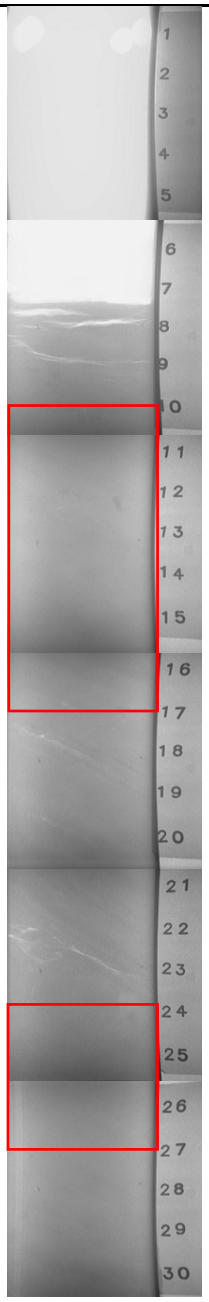
GeoTesting EXPRESS

Project: Hogan Road Bridge Bangor	Location: Bangor, ME	Project No.: GTX-318065
Boring No.: BB-BHR-201	Tested By: jlw	Checked By: njh
Sample No.: U-1	Test Date: 12/7/23	Depth: 17-19'
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, gray clay		
Remarks: TX-011, Swell Pressure - 0.0621 tsf		
Displacement at End of Increment		

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube

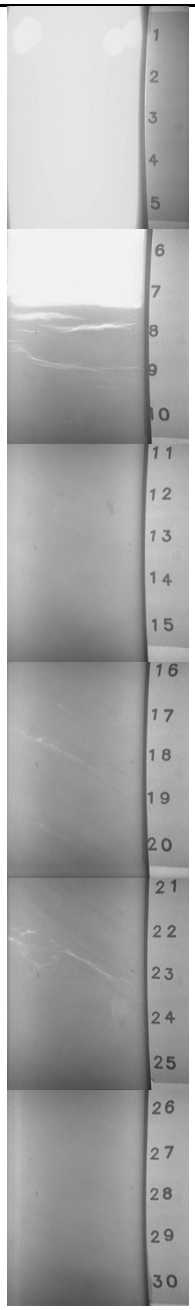


Section 1

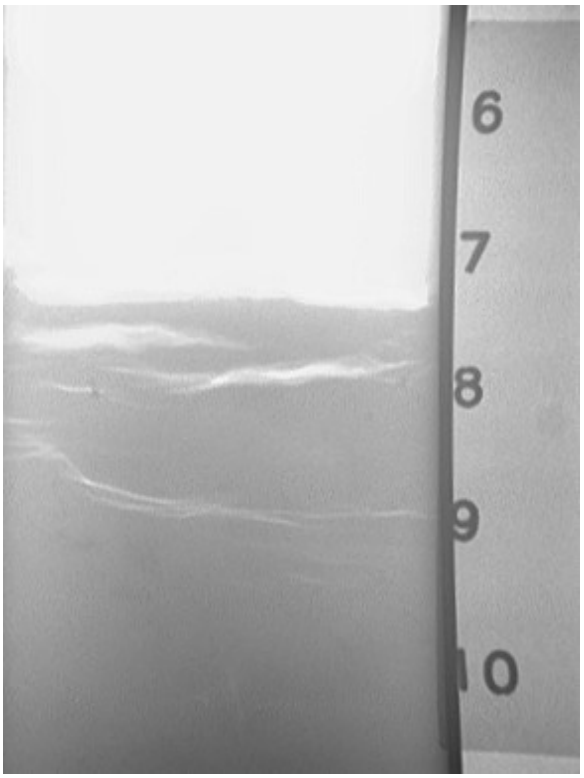
Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



Section 2



Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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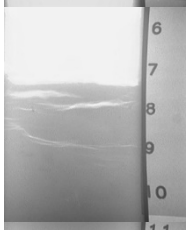
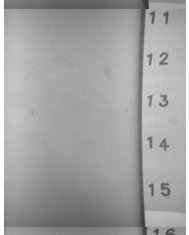
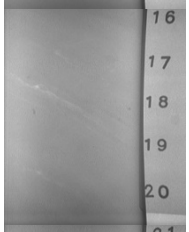
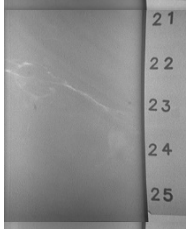
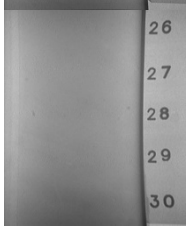
2	
3	
4	
5	
6	
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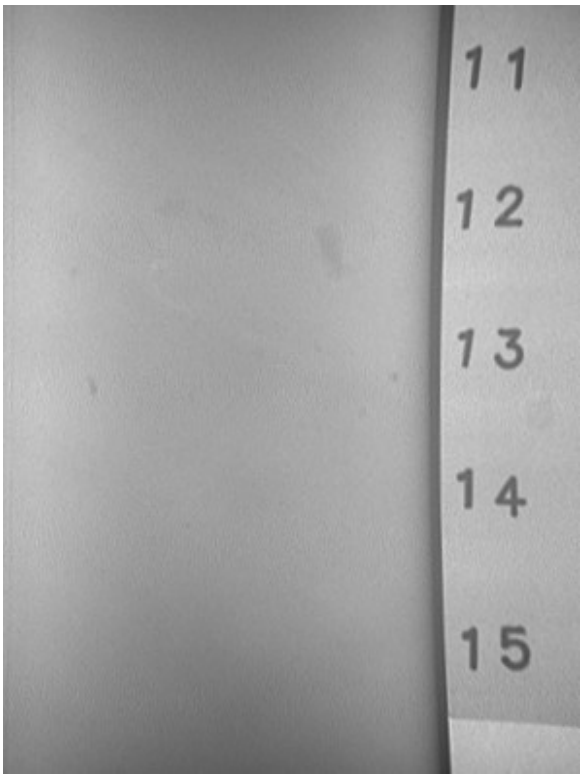
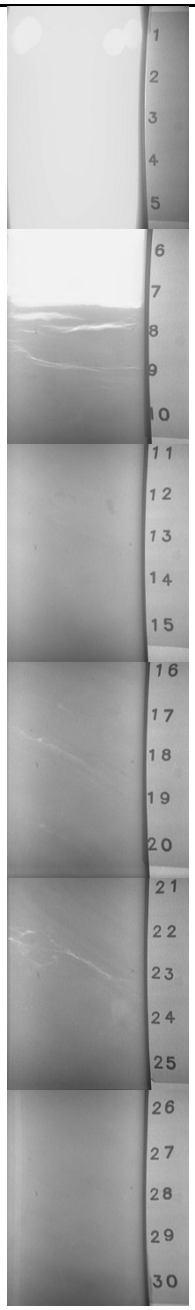
Section 3



Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

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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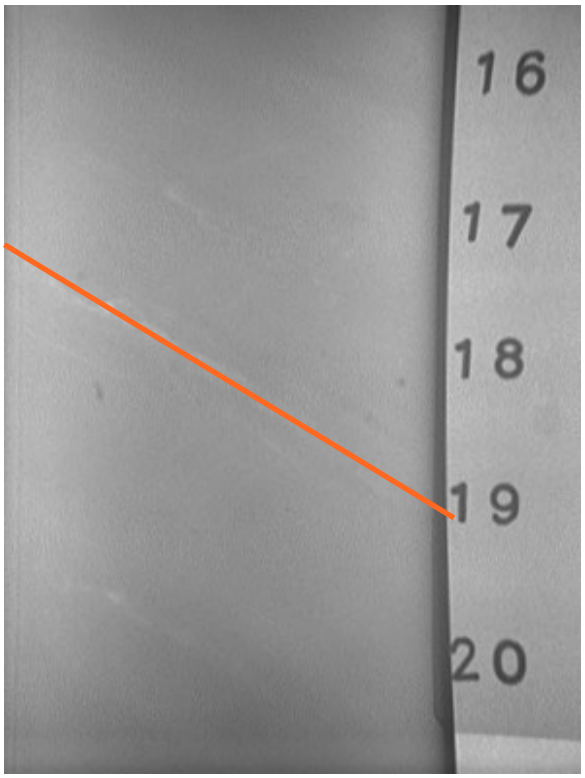
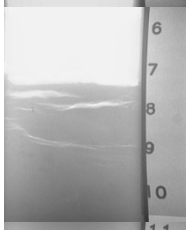
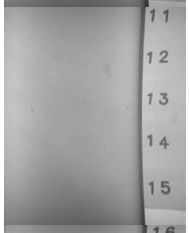
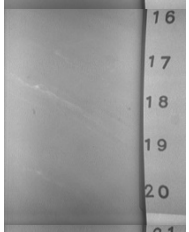
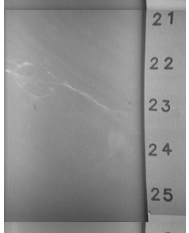
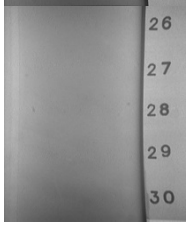
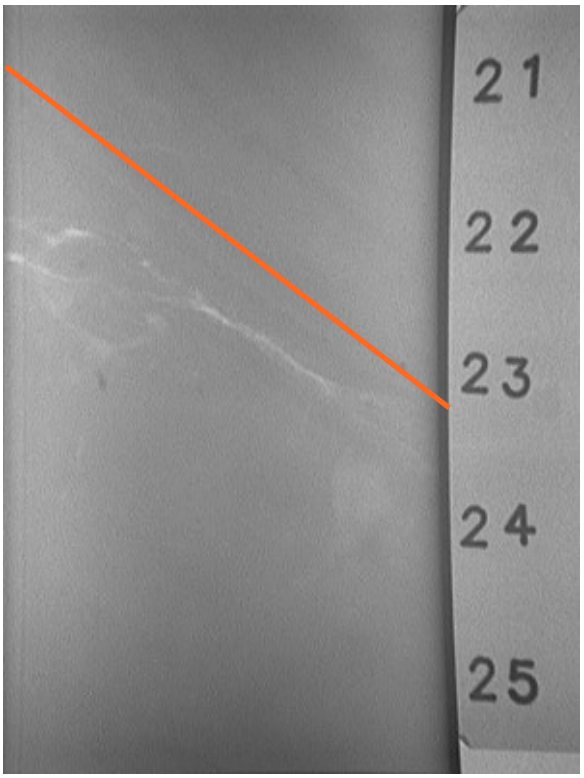


Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

X-Ray of Soil Sample by ASTM D4452

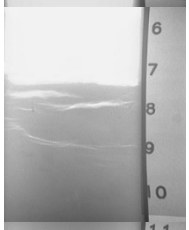
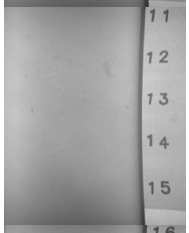
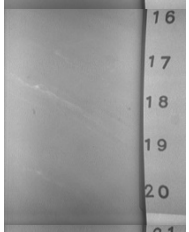
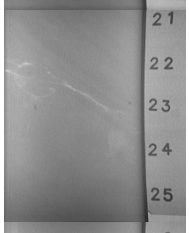
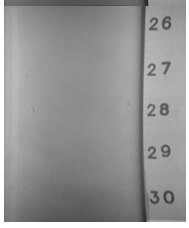
Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube

Section 5

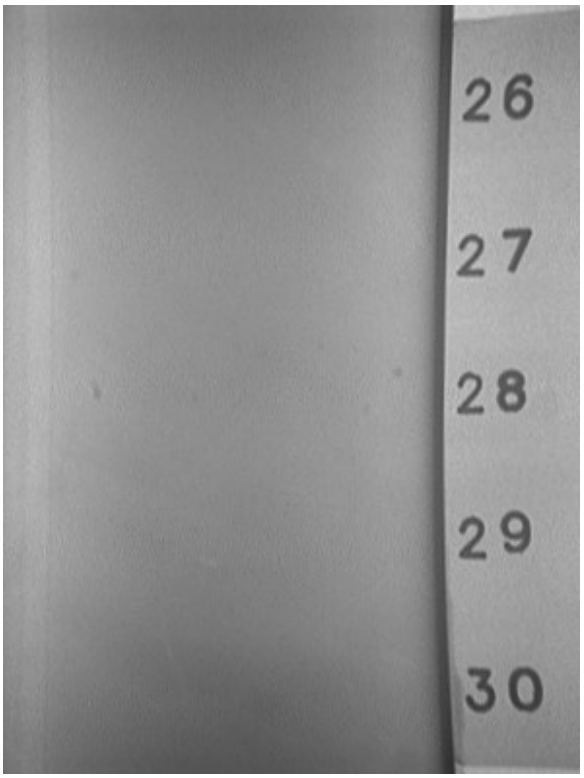


Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	317984
Test Date:	10/23/23
Tested By:	md
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	U1
Depth:	17-19'

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube

Section 6



Client:	WSP USA, Inc
Project Name:	Hogan Road Bridge Bangor PO 004
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	01/03/24
Tested By:	slv
Checked By:	ank

pH by AASHTO T 289

Boring ID	Sample ID	Depth, ft	Description	pH
BB-BHR-202	1D	0-2 ft	Moist, grayish brown clay	7.80
BB-BHR-202	2D	5-7 ft	Moist, grayish brown clay	8.20
BB-BHR-203	1D	0-2 ft	Moist, grayish brown clay	8.17
BB-BHR-203	3D	10-12 ft	Moist, grayish brown clay	7.51
BB-BHR-204	3D	10-12 ft	Moist, grayish brown clay	7.68
BB-BHR-205	3D	10-12 ft	Moist, gray clay	7.54

Client:	WSP USA, Inc.
Project:	Hogan Road Bridge Bangor PO 004
Location:	Bangor, ME
GTX#:	318065
Test Date:	12/30/23
Due Date:	01/03/24
Tested By:	slv
Checked By:	ank

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

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(ohm\text{-}cm)^{-1}$
BB-BHR-202	1D	0-2 ft	Moist, grayish brown clay	1,516	6.59E-04
BB-BHR-203	1D	0-2 ft	Moist, grayish brown clay	1,212	8.25E-04
BB-BHR-202	2D	5-7 ft	Moist, grayish brown clay	2,473	4.04E-04
BB-BHR-203	3D	10-12 ft	Moist, grayish brown clay	2,212	4.52E-04
BB-BHR-204	3D	10-12 ft	Moist, grayish brown clay	2,603	3.84E-04
BB-BHR-205	3D	10-12 ft	Moist, gray clay	1,038	9.63E-04

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box

Water added to sample to create a thick slurry prior to testing (saturated condition).

Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)

Test conducted in standard laboratory atmosphere: 68-73 F

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

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

Analysis No.	TS-A2311600
Report Date	28 December 2023
Date Sampled	18 December 2023
Date Received	26 December 2023
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Soil: Project: Hogan Road Bridge Bangor PO 004; Site Location: Bangor, ME; Job Number: GTX-318065

When examined to the applicable requirements of:

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

Results:

AASHTO T 291 - Chloride Method B

Sample		Results		Minimum Detection Limit
		ppm (mg/kg)	%[1]	
BB-BHR-202		37.	0.0037	10.
2D	5.0 – 7.0'			
BB-BHR-203		78.	0.0078	
1D	0.0 – 2.0'			
BB-BHR-204		18.	0.0018	
1D	0.0 – 2.0'			
BB-BHR-205		93.	0.0093	
3D	10.0 –12.0'			
BB-BHR-301		< 10.	< 0.0010	
2D	2.0 – 2.6'			
BB-BHR-302		112.	0.0112	
4D	4.0 – 4.6'			
BB-BHR-303		< 10.	< 0.0010	
3D	3.0 – 3.6'			

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

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

AASHTO T 290 – Sulfates (Soluble)

Sample		Results		Minimum Detection Limit
		ppm (mg/kg)	%[1]	
BB-BHR-202		< 10.	< 0.0010	10.
2D	5.0 – 7.0’			
BB-BHR-203		< 10.	< 0.0010	
1D	0.0 – 2.0’			
BB-BHR-204		< 10.	< 0.0010	
1D	0.0 – 2.0’			
BB-BHR-205		10.	0.0010	
3D	10.0 –12.0’			
BB-BHR-301		30.	0.0030	
2D	2.0 – 2.6’			
BB-BHR-302		10.	0.0010	
4D	4.0 – 4.6’			
BB-BHR-303		20.	0.0020	
3D	3.0 – 3.6’			

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

END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

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Client:	WSP USA, Inc
Project:	Hogan Road Bridge Bangor PO 004
Location:	Bangor, ME
GTX#:	318065
Test Date:	01/11/24
Due Date:	01/03/24
Tested By:	slv
Checked By:	ank

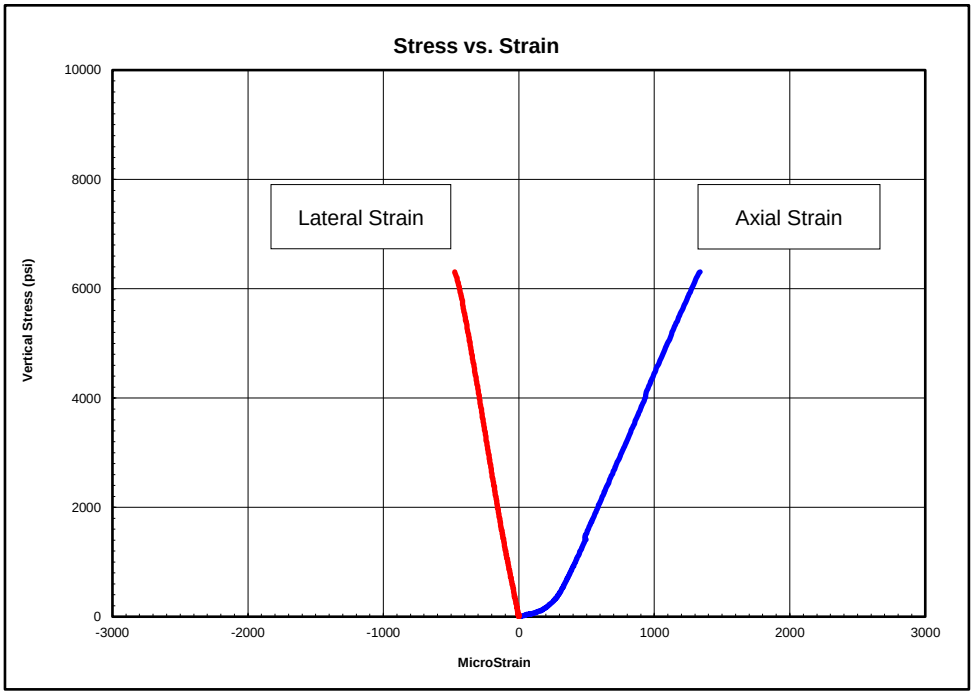
Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, ft.	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(ohm\text{-}cm)^{-1}$
BB-BHR-301	1D, 2D	1-2.6 ft	Moist brown sand with gravel and organics	7,666	1.30E-04
BB-BHR-303	1D, 2D, 3D	1-3.6 ft	Moist brown sand with gravel and organics	7,523	1.33E-04

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

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	jsc
Boring ID:	BB-BHR-201
Sample ID:	R4 (31.4-35.0')
Depth, ft:	32.94-33.31
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

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



Peak Compressive Stress: 6,308 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
600-2300	5,850,000	0.45
2300-4000	5,700,000	0.38
4000-5700	5,720,000	0.41

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

Client:	WSP USA, Inc.	Test Date:	11/3/2023
Project Name:	Hogan Road Bridge Bangor	Tested By:	te
Project Location:	Bangor, ME	Checked By:	smd
GTX #:	318605		
Boring ID:	BB-BHR-201		
Sample ID:	R4 (31.4-35.0')		
Depth:	32.94-33.31 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.26	4.26	4.26
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	598.86		
Bulk Density, lb/ft^3	172	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

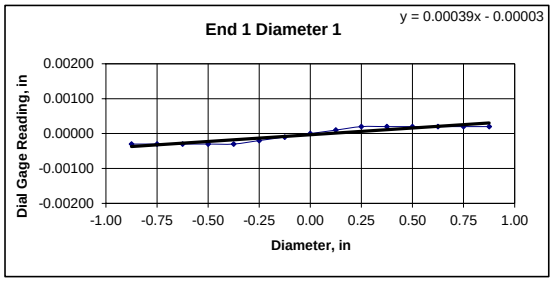
END FLATNESS AND PARALLELISM (Procedure FP1)

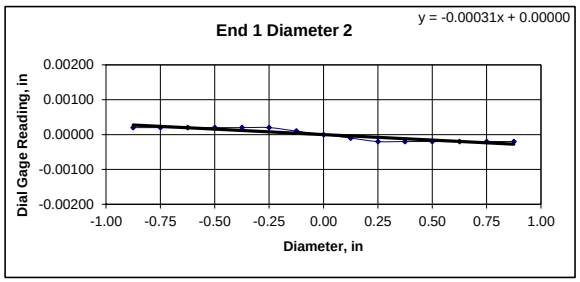
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00020	0.00020	0.00020	0.00020	0.00020
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00020	0.00020	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020

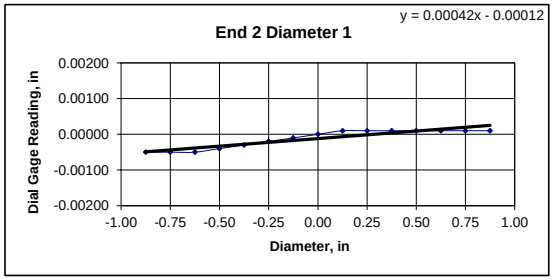
Difference between max and min readings, in:
0° = 0.00050 90° = 0.00040

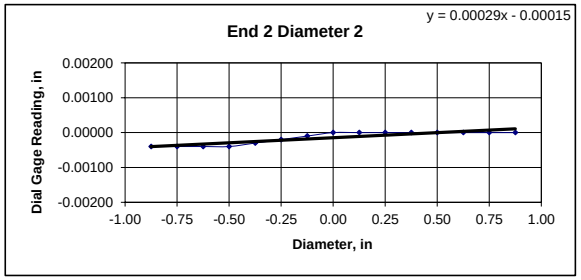
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00050	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00040	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0006 90° = 0.0004
Maximum difference must be < 0.0020 in. Difference = ± 0.00030
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00039
Angle of Best Fit Line: 0.02210

End 2:
Slope of Best Fit Line 0.00042
Angle of Best Fit Line: 0.02423

Maximum Angular Difference: 0.00213

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00031
Angle of Best Fit Line: 0.01801

End 2:
Slope of Best Fit Line 0.00029
Angle of Best Fit Line: 0.01670

Maximum Angular Difference: 0.00131

Parallelism Tolerance Met? YES
Spherically Seated

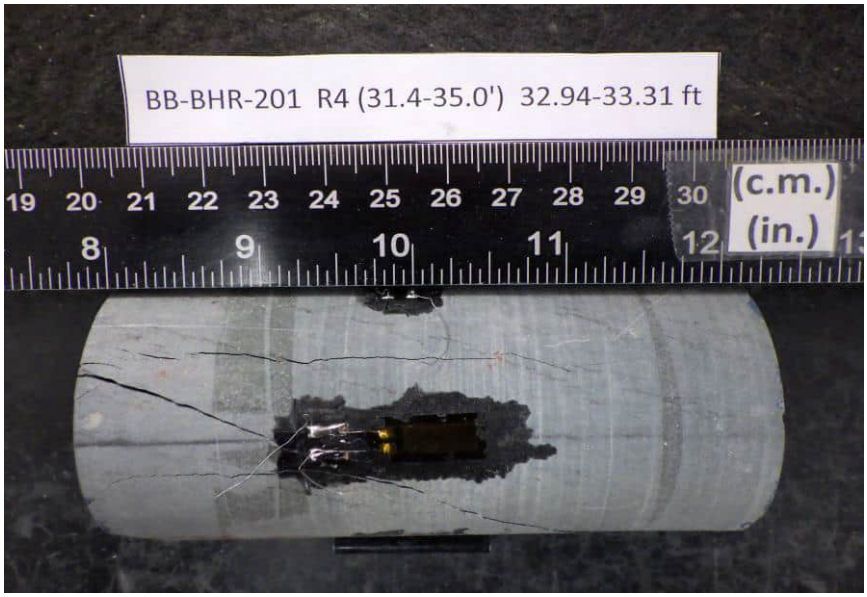
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00050	1.990	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00060	1.990	0.00030	0.017	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	smd
Boring ID:	BB-BHR-201
Sample ID:	R4 (31.4-35.0')
Depth, ft:	32.94-33.31

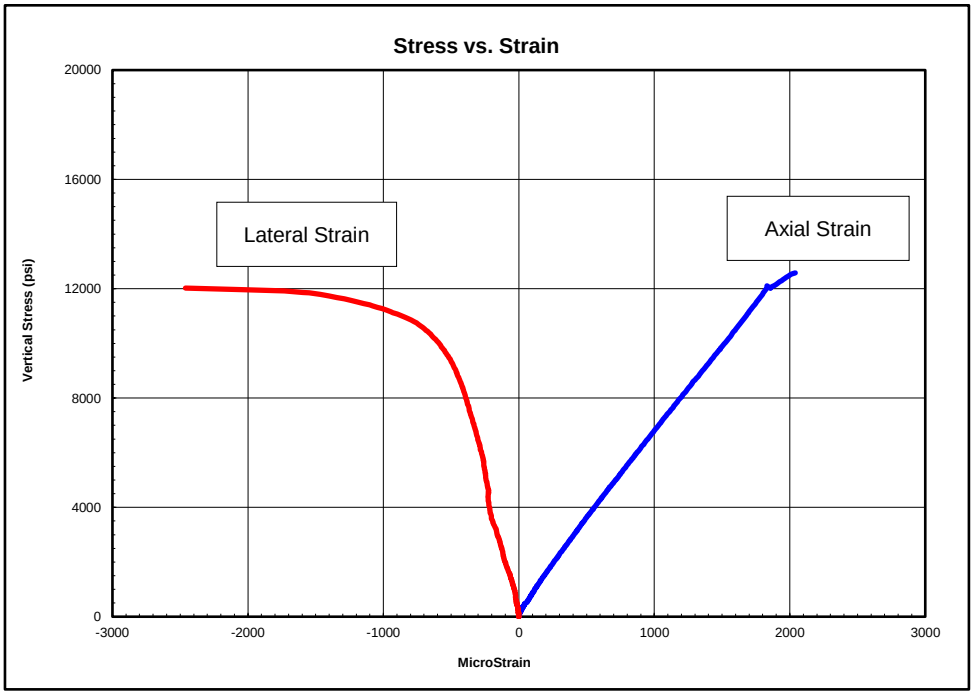
After cutting and grinding



After break

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	jsc
Boring ID:	BB-BHR-206
Sample ID:	R1 (18.8-22.3')
Depth, ft:	19.09-19.47
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 12,579 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1300-4600	6,720,000	0.40
4600-8000	6,280,000	0.31
8000-11300	6,190,000	---

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

Client:	WSP USA, Inc.	Test Date:	11/6/2023
Project Name:	Hogan Road Bridge Bangor	Tested By:	te
Project Location:	Bangor, ME	Checked By:	smd
GTX #:	318065		
Boring ID:	BB-BHR-206		
Sample ID:	R1 (18.8-22.3')		
Depth:	19.09-19.47 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.22	4.22	4.22
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	578.96		
Bulk Density, lb/ft^3	169	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

END FLATNESS AND PARALLELISM (Procedure FP1)

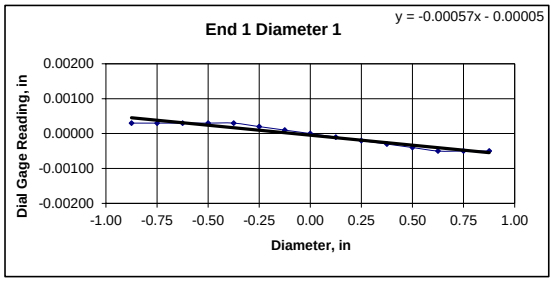
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00030	0.00030	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050	-0.00050
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00030	-0.00030	-0.00030	-0.00030

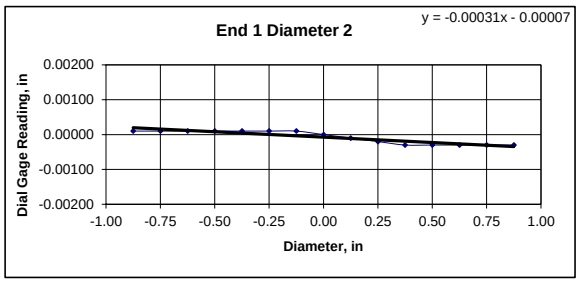
Difference between max and min readings, in:
0° = 0.00080 90° = 0.00040

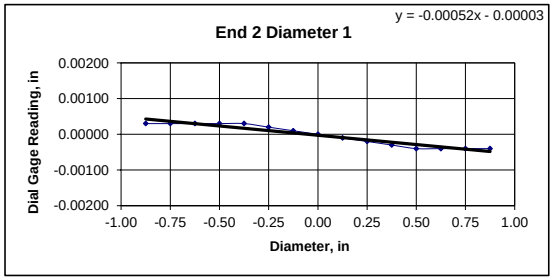
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00030	0.00030	0.00020	0.00010	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00040	-0.00040	-0.00040
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010

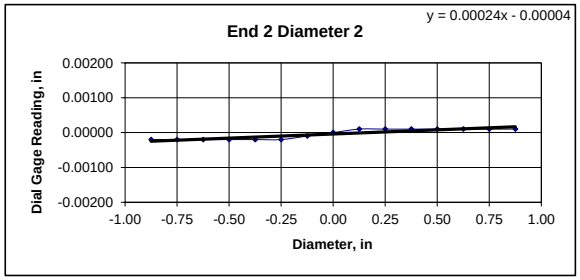
Difference between max and min readings, in:
0° = 0.0007 90° = 0.0003

Maximum difference must be < 0.0020 in. Difference = ± 0.00040
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00057
Angle of Best Fit Line: 0.03274

End 2:
Slope of Best Fit Line 0.00052
Angle of Best Fit Line: 0.02979

Maximum Angular Difference: 0.00295

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00031
Angle of Best Fit Line: 0.01768

End 2:
Slope of Best Fit Line 0.00024
Angle of Best Fit Line: 0.01359

Maximum Angular Difference: 0.00409

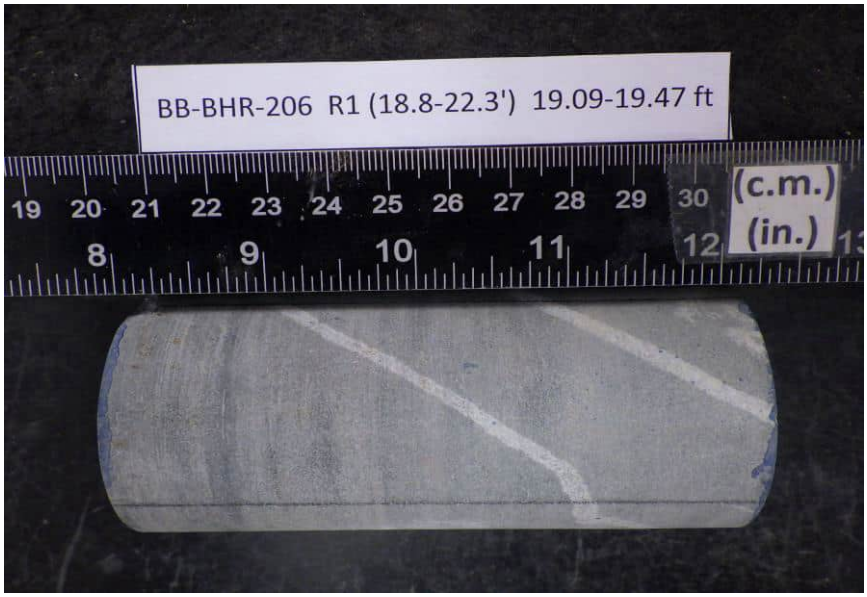
Parallelism Tolerance Met? YES
Spherically Seated

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

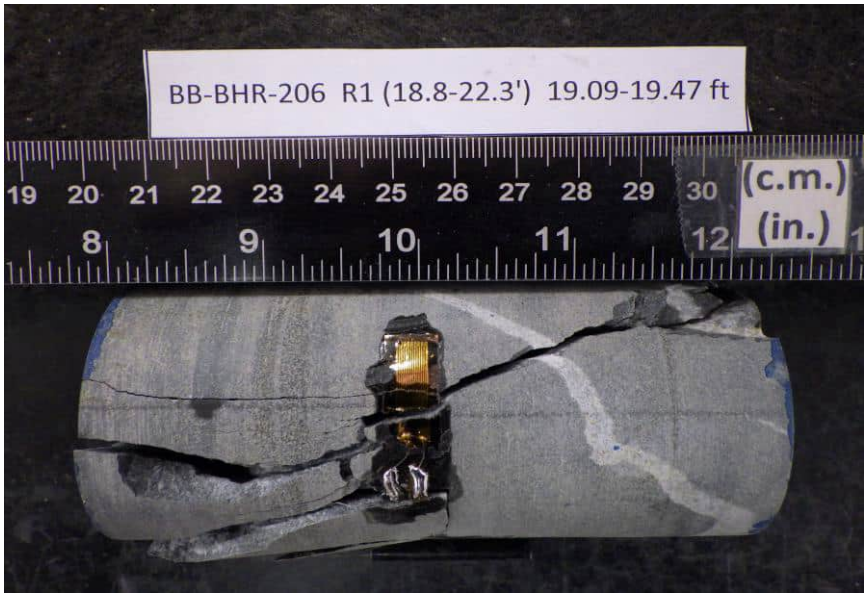
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00080	1.980	0.00040	0.023	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00070	1.980	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	smd
Boring ID:	BB-BHR-206
Sample ID:	R1 (18.8-22.3')
Depth, ft:	19.09-19.47



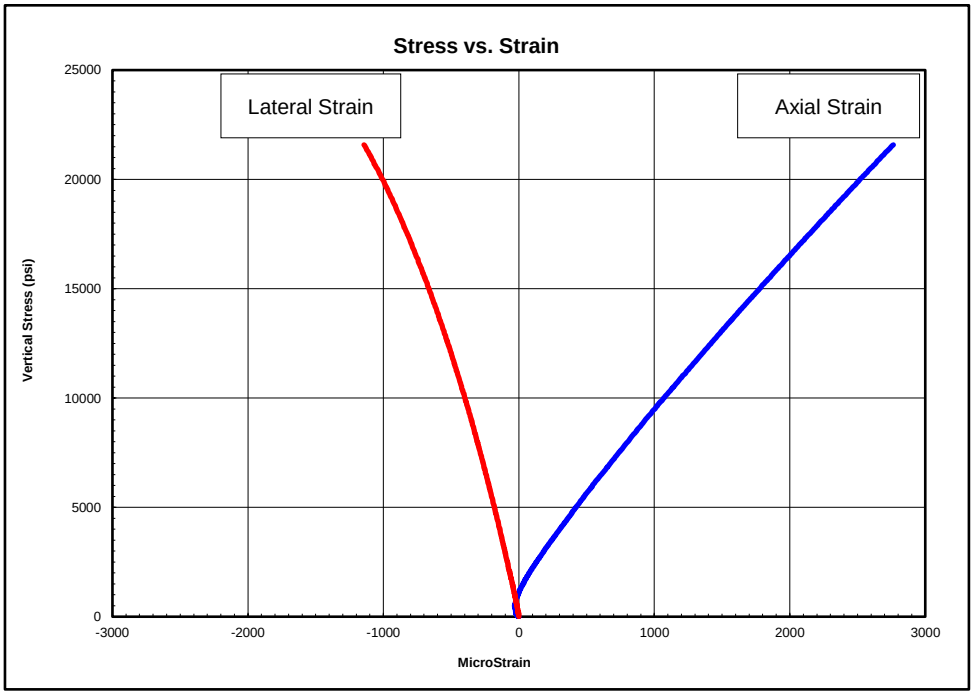
After cutting and grinding



After break

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	jsc
Boring ID:	BRP-BHR-202
Sample ID:	R1 (23.0-28.0)
Depth, ft:	25.57-25.95
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

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



Peak Compressive Stress: 21,578 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2200-7900	8,230,000	0.33
7900-13700	7,250,000	0.36
13700-19400	6,780,000	0.44

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

Client:	WSP USA, Inc.	Test Date:	11/6/2023
Project Name:	Hogan Road Bridge Bangor	Tested By:	te
Project Location:	Bangor, ME	Checked By:	smd
GTX #:	318065		
Boring ID:	BRP-BHR-202		
Sample ID:	R1 (23.0-28.0')		
Depth:	25.57-25.95 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.32	4.32	4.32
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	596.79		
Bulk Density, lb/ft^3	171	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

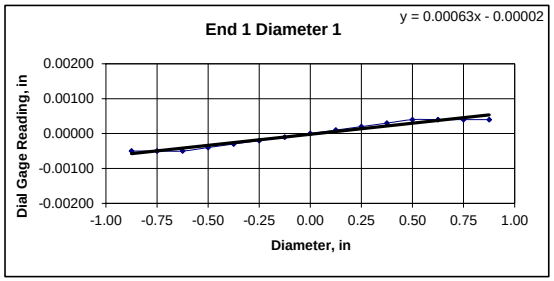
Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

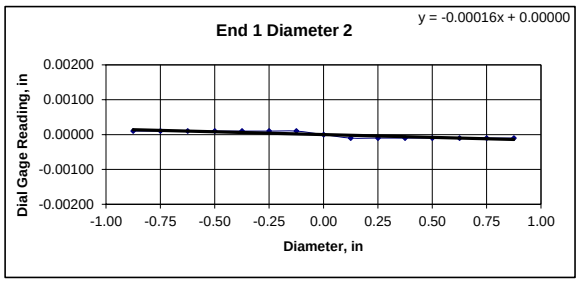
END FLATNESS AND PARALLELISM (Procedure FP1)

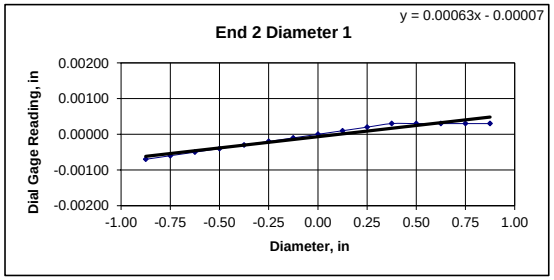
	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
END 1															
Diameter 1, in	-0.00050	-0.00050	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00040	0.00040	0.00040	0.00040
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010
Difference between max and min readings, in:															
0° =	0.00090												90° =	0.00020	

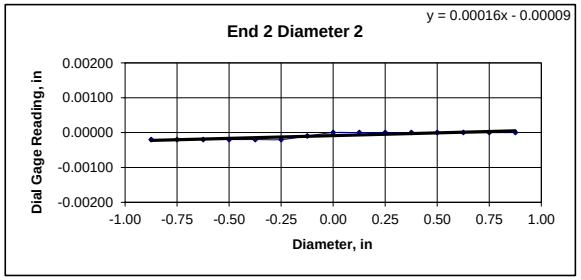
	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
END 2															
Diameter 1, in	-0.00070	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00010	0.00020	0.00030	0.00030	0.00030	0.00030	0.00030
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° =	0.001												90° =	0.0002	

Maximum difference must be < 0.0020 in. Difference = ± 0.00050
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00063
Angle of Best Fit Line: 0.03634

End 2:
Slope of Best Fit Line 0.00063
Angle of Best Fit Line: 0.03601

Maximum Angular Difference: 0.00033

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00016
Angle of Best Fit Line: 0.00917

End 2:
Slope of Best Fit Line 0.00016
Angle of Best Fit Line: 0.00900

Maximum Angular Difference: 0.00016

Parallelism Tolerance Met? YES
Spherically Seated

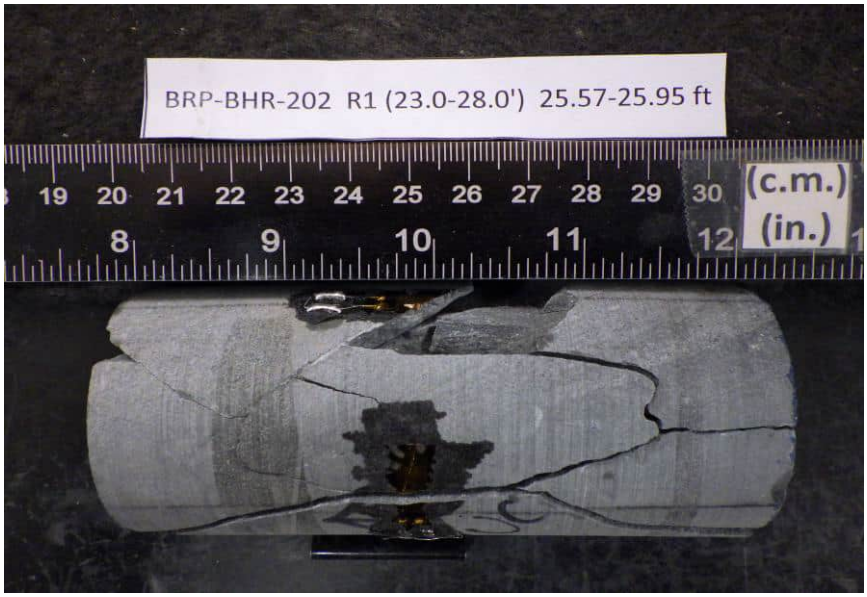
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00090	1.980	0.00045	0.026	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00100	1.980	0.00051	0.029	YES
Diameter 2, in (rotated 90°)	0.00020	1.980	0.00010	0.006	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	smd
Boring ID:	BRP-BHR-202
Sample ID:	R1 (23.0-28.0)
Depth, ft:	25.57-25.95

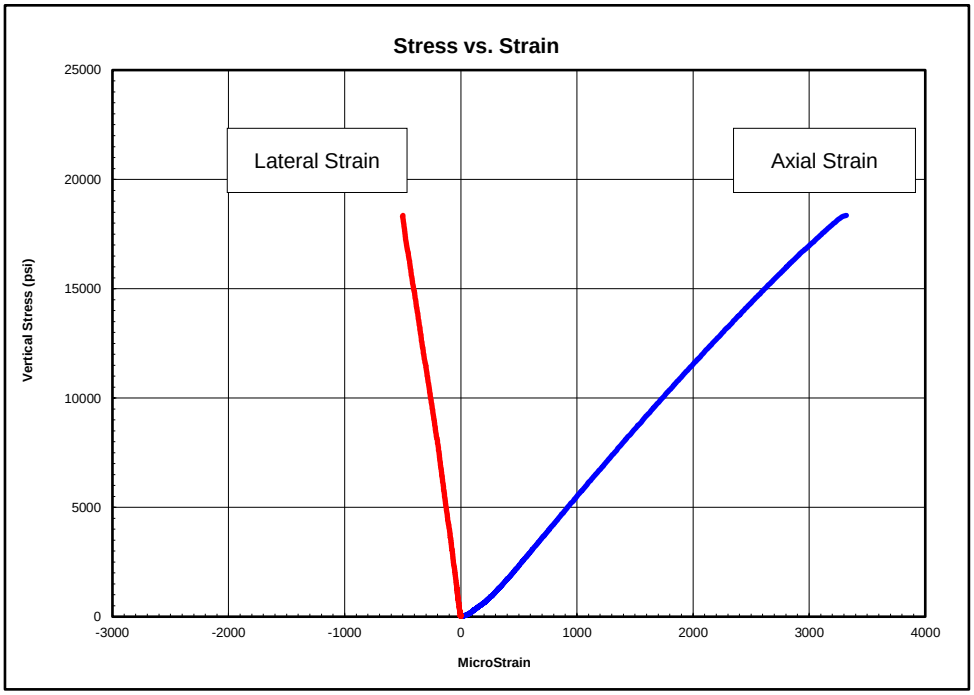
After cutting and grinding



After break

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	jsc
Boring ID:	BRP-BHR-203
Sample ID:	R1 (14.2-19.2')
Depth, ft:	16.88-17.26
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

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



Peak Compressive Stress: 18,354 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6700	6,320,000	0.16
6700-11600	6,010,000	0.17
11600-16500	5,500,000	0.16

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

Client:	WSP USA, Inc.	Test Date:	11/3/2023
Project Name:	Hogan Road Bridge Bangor	Tested By:	te
Project Location:	Bangor, ME	Checked By:	smd
GTX #:	318065		
Boring ID:	BRP-BHR-203		
Sample ID:	R1 (14.2-19.2')		
Depth:	16.88-17.26 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.30	4.30	4.30
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	596.6		
Bulk Density, lb/ft^3	171	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

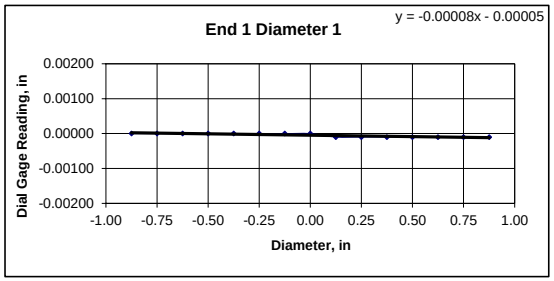
END FLATNESS AND PARALLELISM (Procedure FP1)

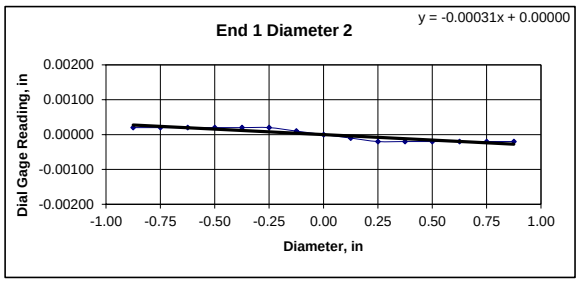
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00020	0.00020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

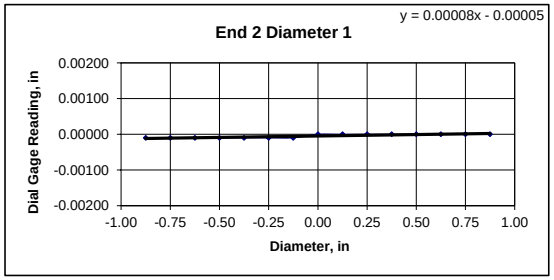
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00040

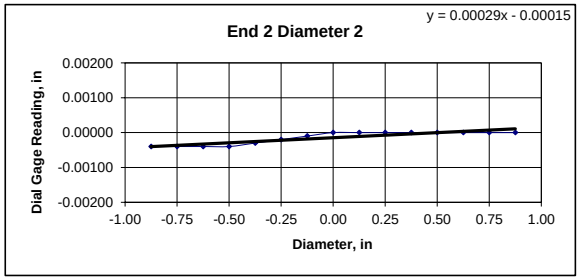
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00040	-0.00040	-0.00040	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0001 90° = 0.0004
Maximum difference must be < 0.0020 in. Difference = ± 0.00020
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00008
Angle of Best Fit Line: 0.00458

End 2:
Slope of Best Fit Line 0.00008
Angle of Best Fit Line: 0.00458

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00031
Angle of Best Fit Line: 0.01801

End 2:
Slope of Best Fit Line 0.00029
Angle of Best Fit Line: 0.01670

Maximum Angular Difference: 0.00131

Parallelism Tolerance Met? YES
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00010	1.980	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00040	1.980	0.00020	0.012	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	WSP USA, Inc.
Project Name:	Hogan Road Bridge Bangor
Project Location:	Bangor, ME
GTX #:	318065
Test Date:	11/16/2023
Tested By:	te
Checked By:	smd
Boring ID:	BRP-BHR-203
Sample ID:	R1 (14.2-19.2')
Depth, ft:	16.88-17.26

After cutting and grinding

After break

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.

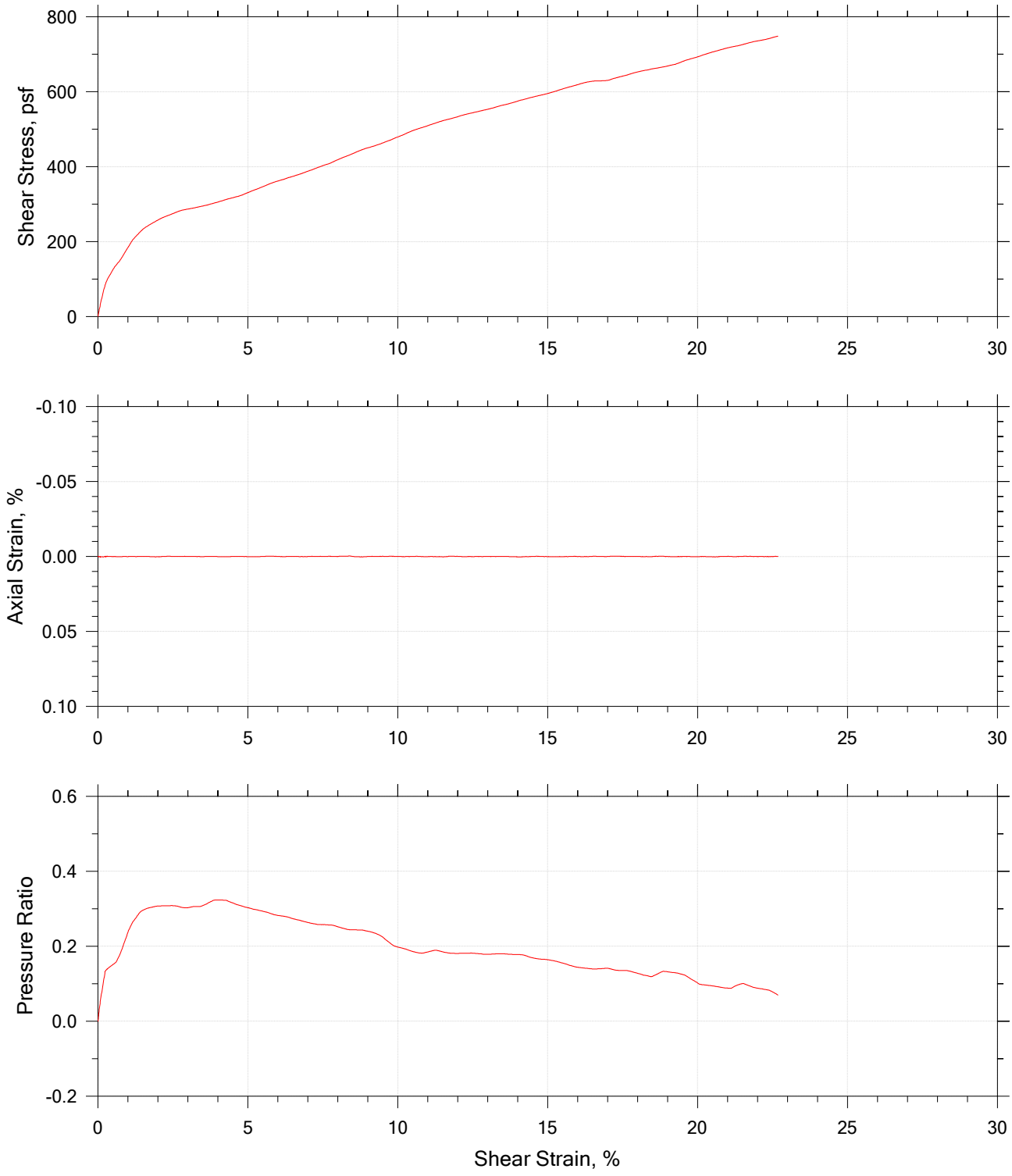
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-3				
Initial Moisture Content, %	26.9				
Initial Dry Density, pcf	95.4				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	925				
Vertical Consolidation Stress at time of shear, psf	925				
Final Moisture 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

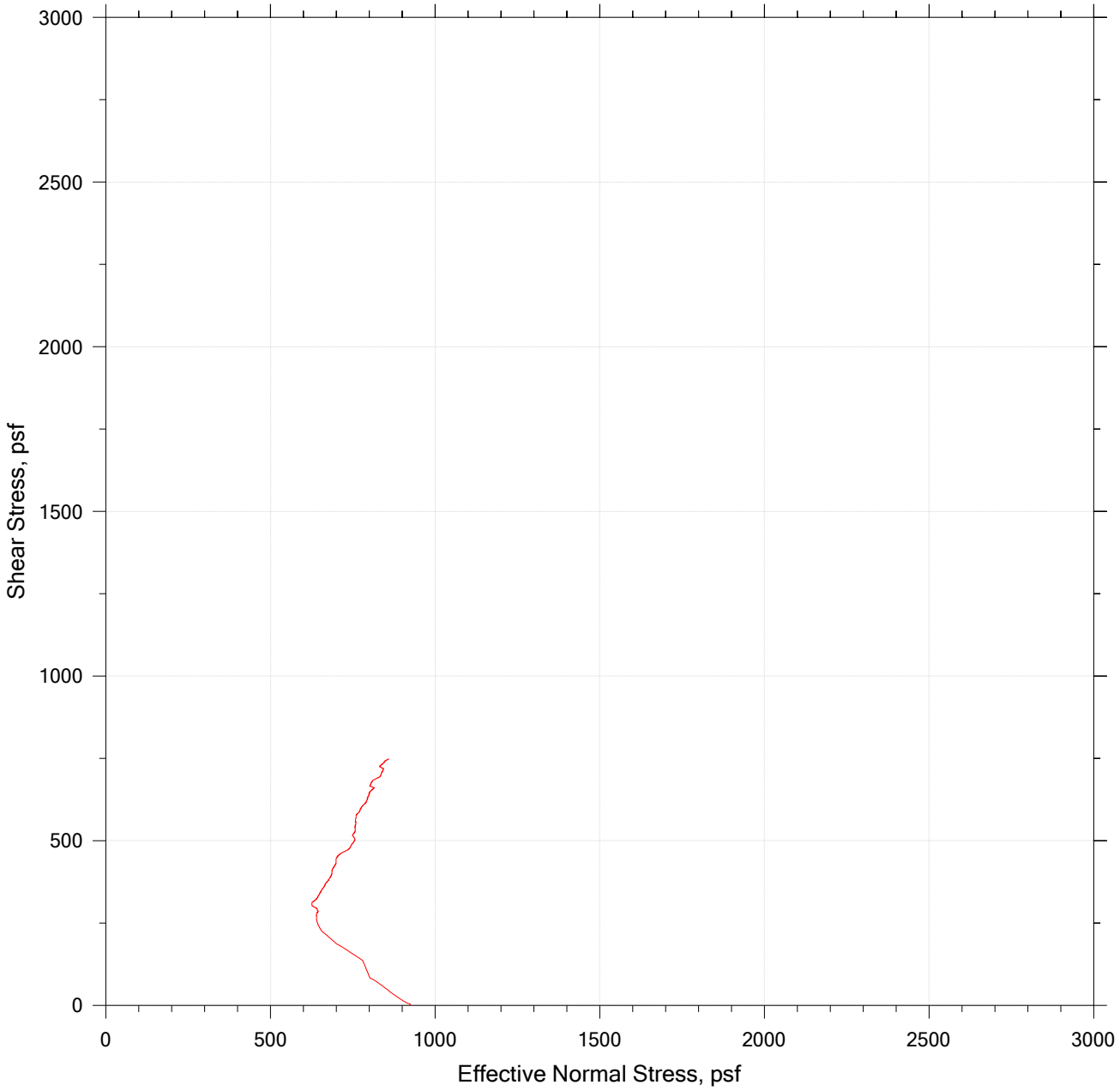
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

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.

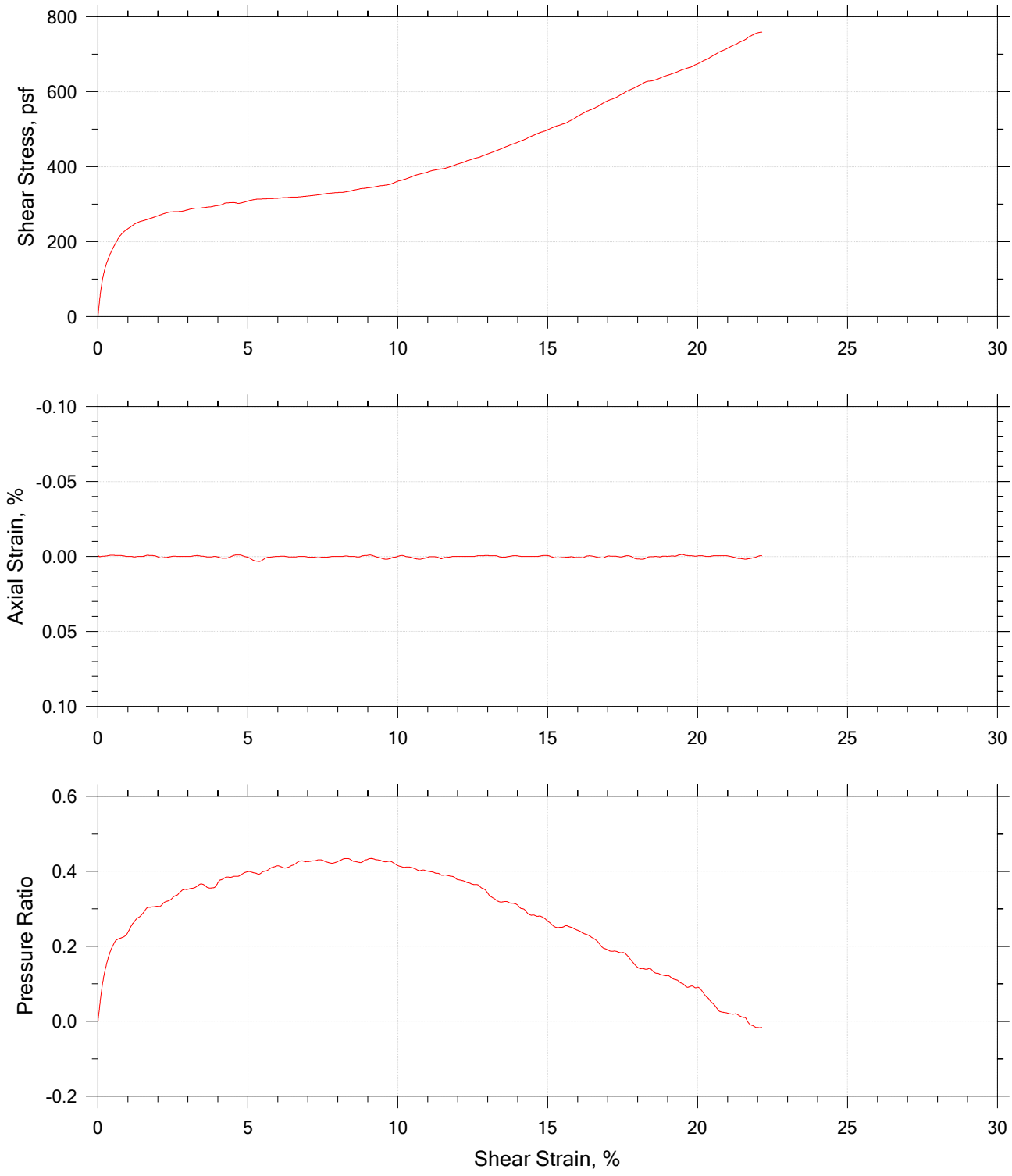
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-2				
Initial Moisture Content, %	25.3				
Initial Dry Density, pcf	98.9				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	1,050				
Vertical Consolidation Stress at time of shear, psf	1,050				
Final 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

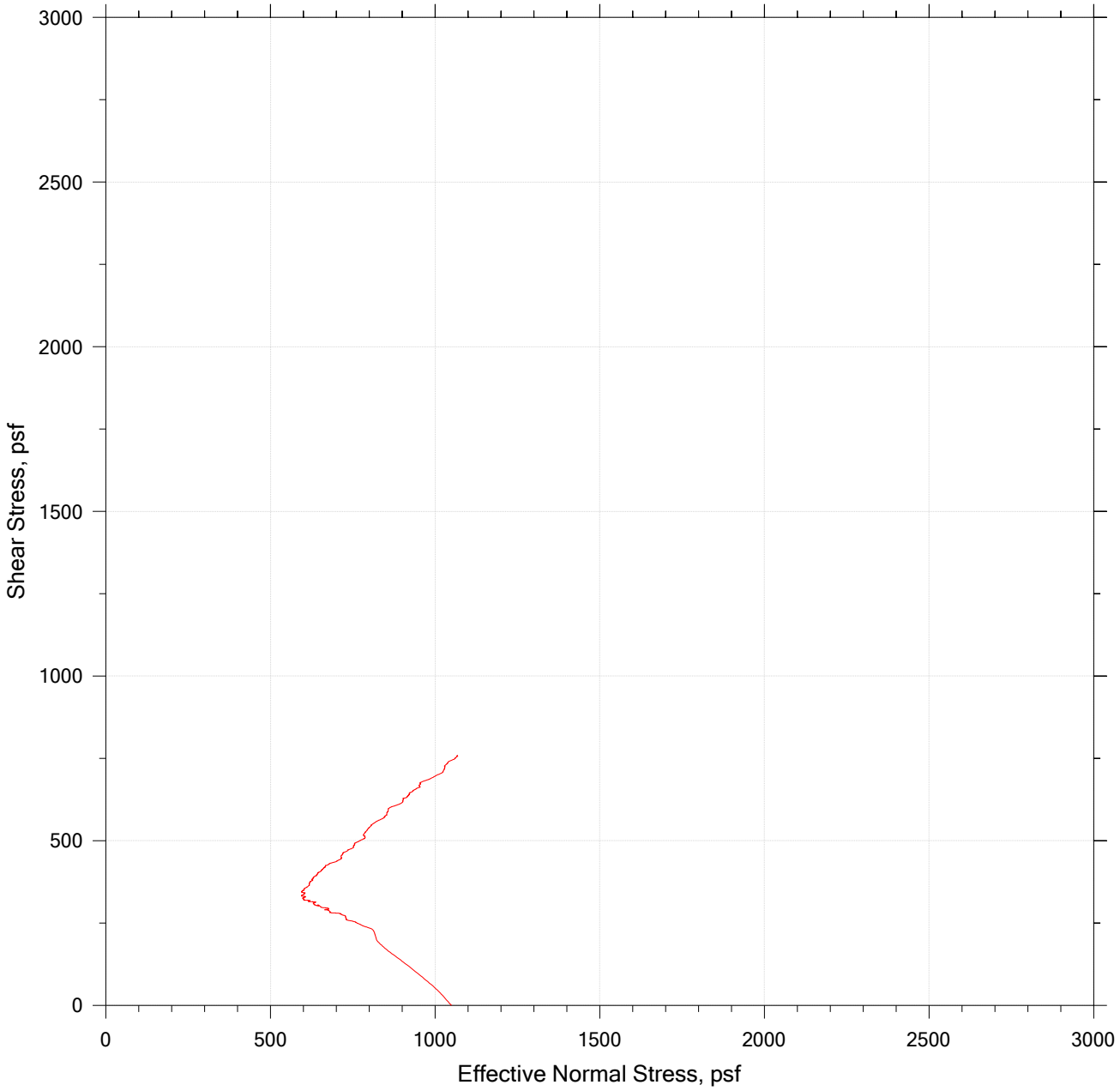
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

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.

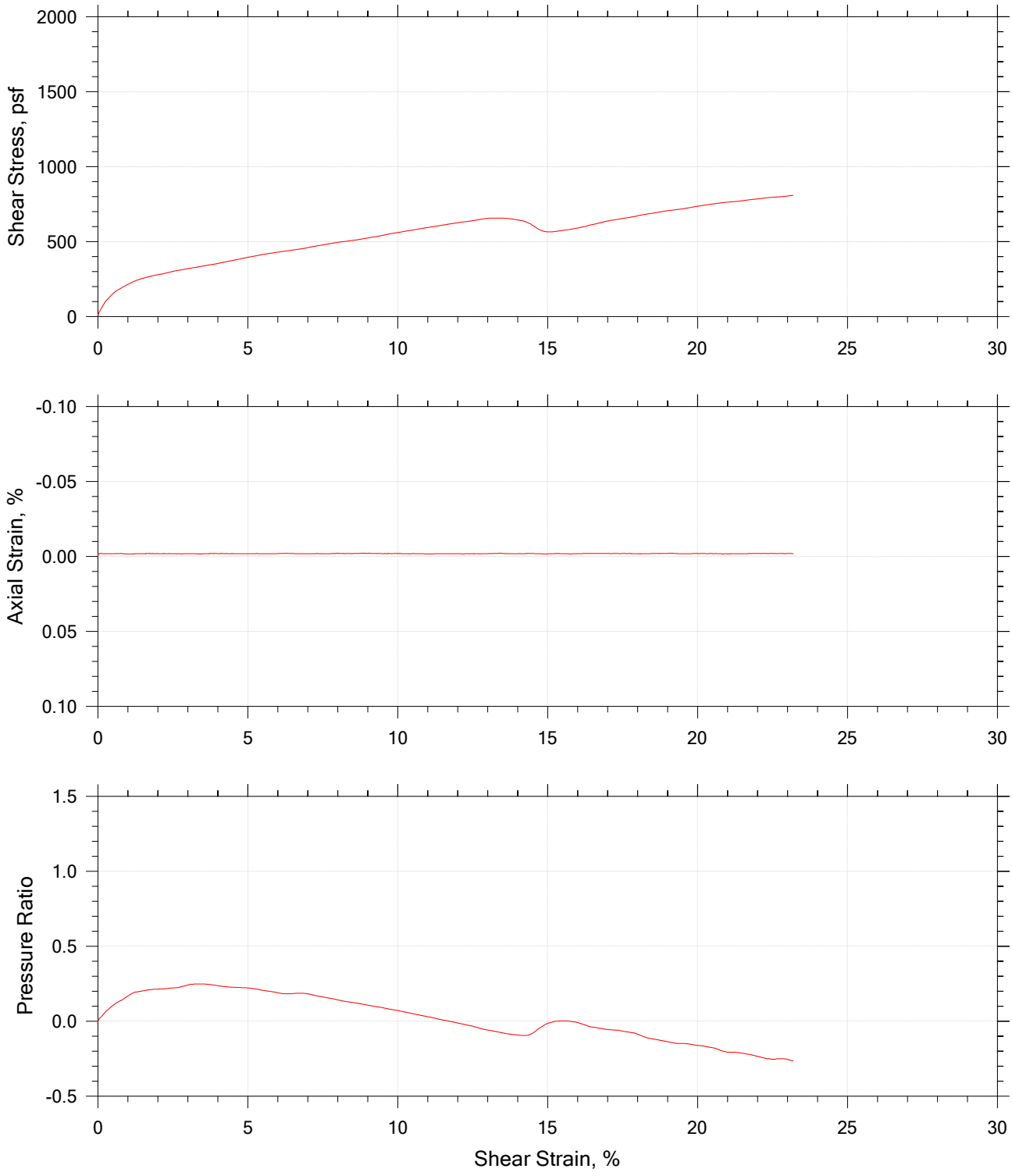
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	27.8				
Initial Dry Density, pcf	95.4				
Nominal Rate of Shear Strain, %/hr	5.0				
Maximum Vertical Consolidation Stress, psf	900				
Vertical Consolidation Stress at time of shear, psf	900				
Final Moisture 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

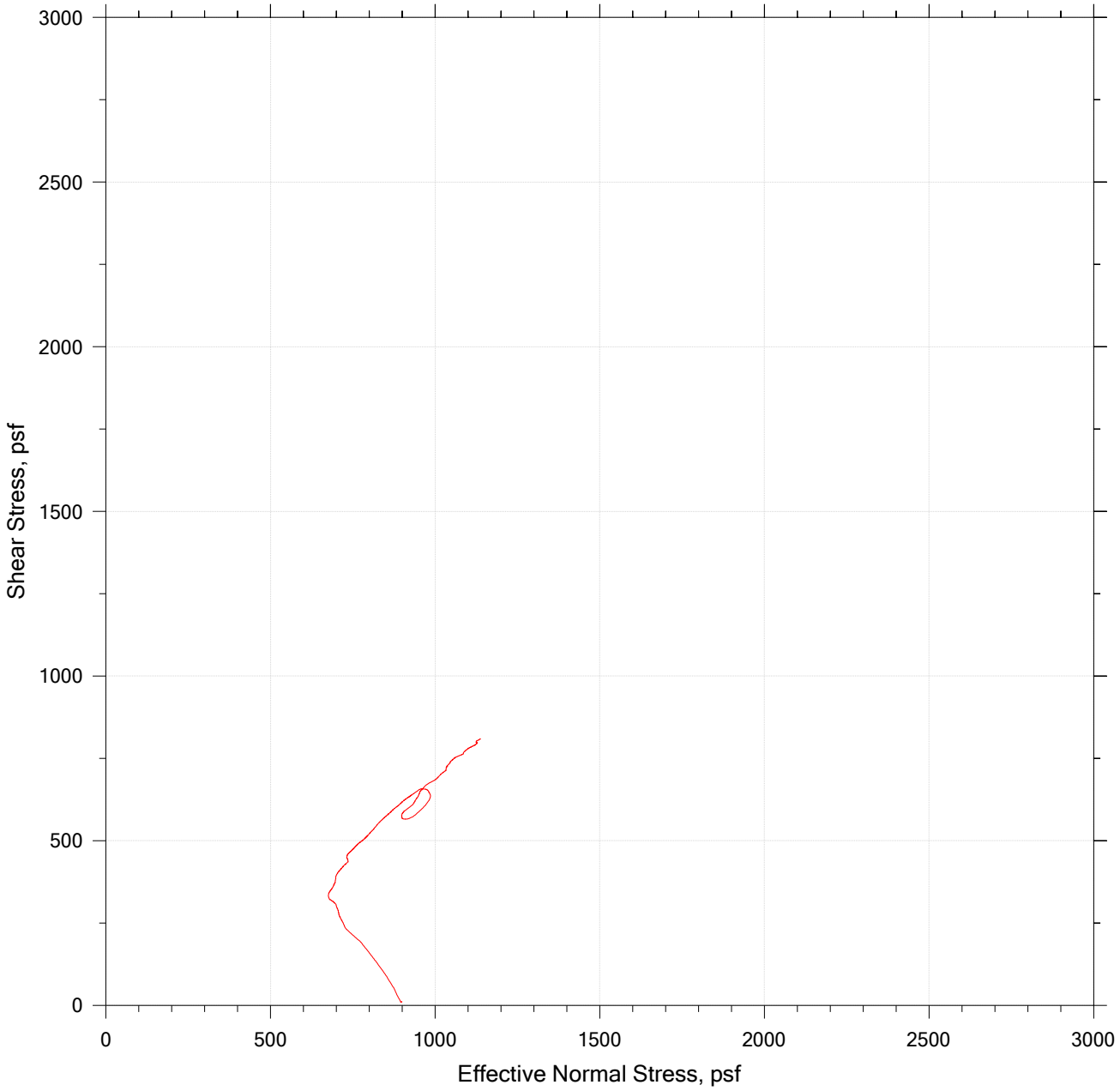
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

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:		

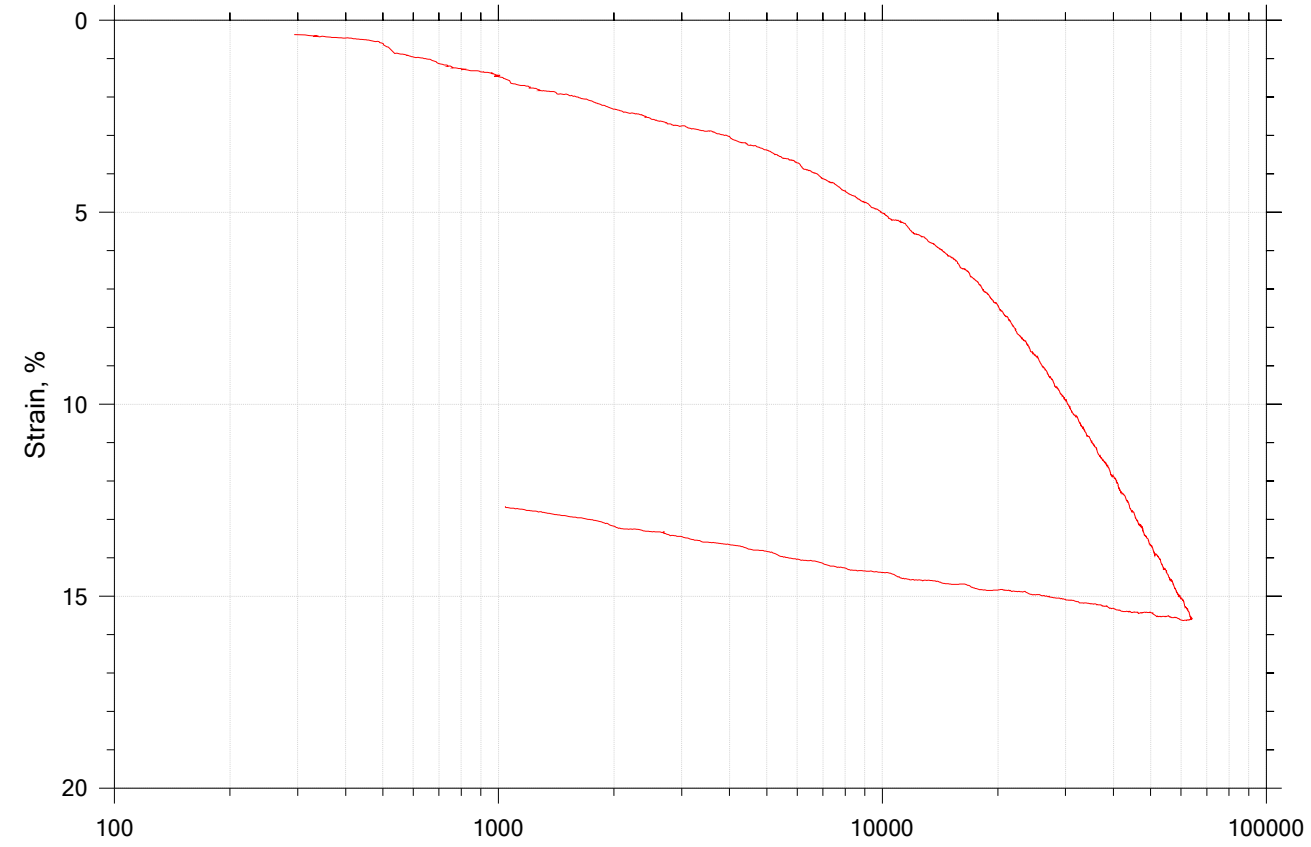
Direct Simple Shear Test by ASTM D6528

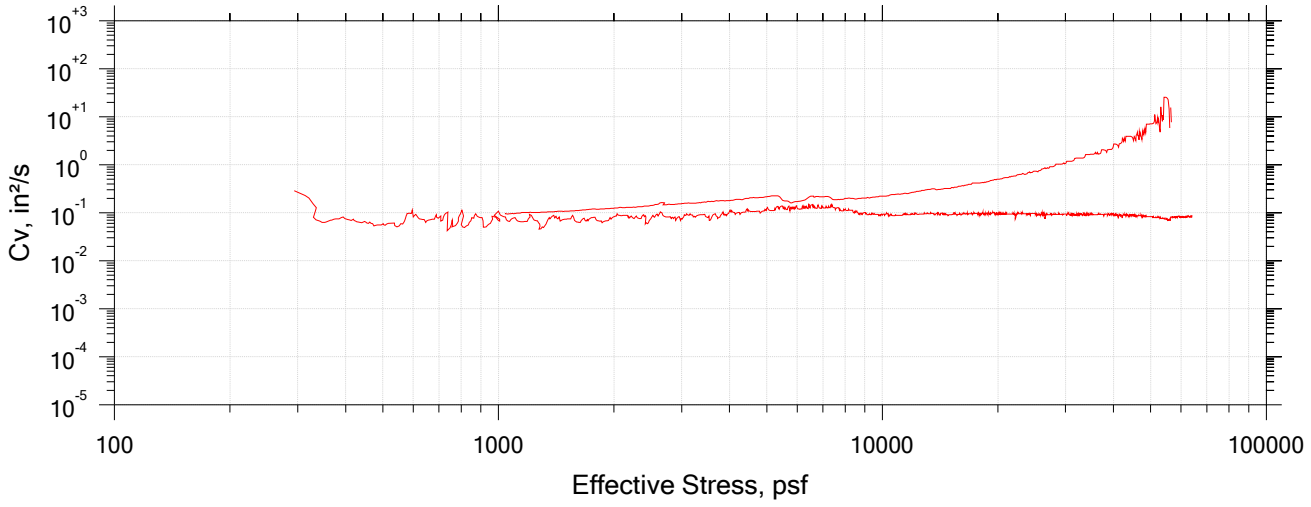


GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: DSS-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks:		

CRC Test

Summary

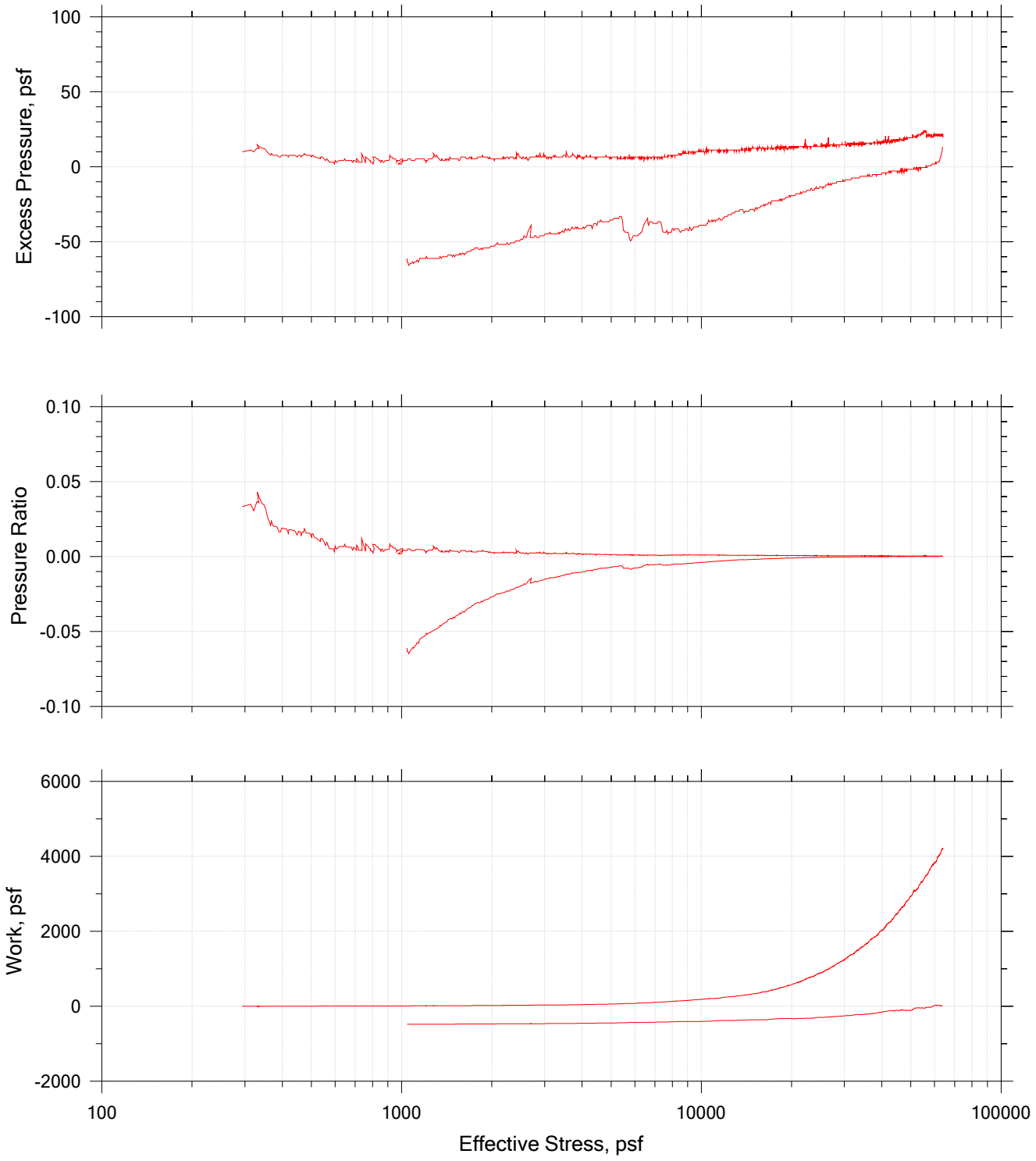




GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-101	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test Number: CRC-3	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: ---		

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

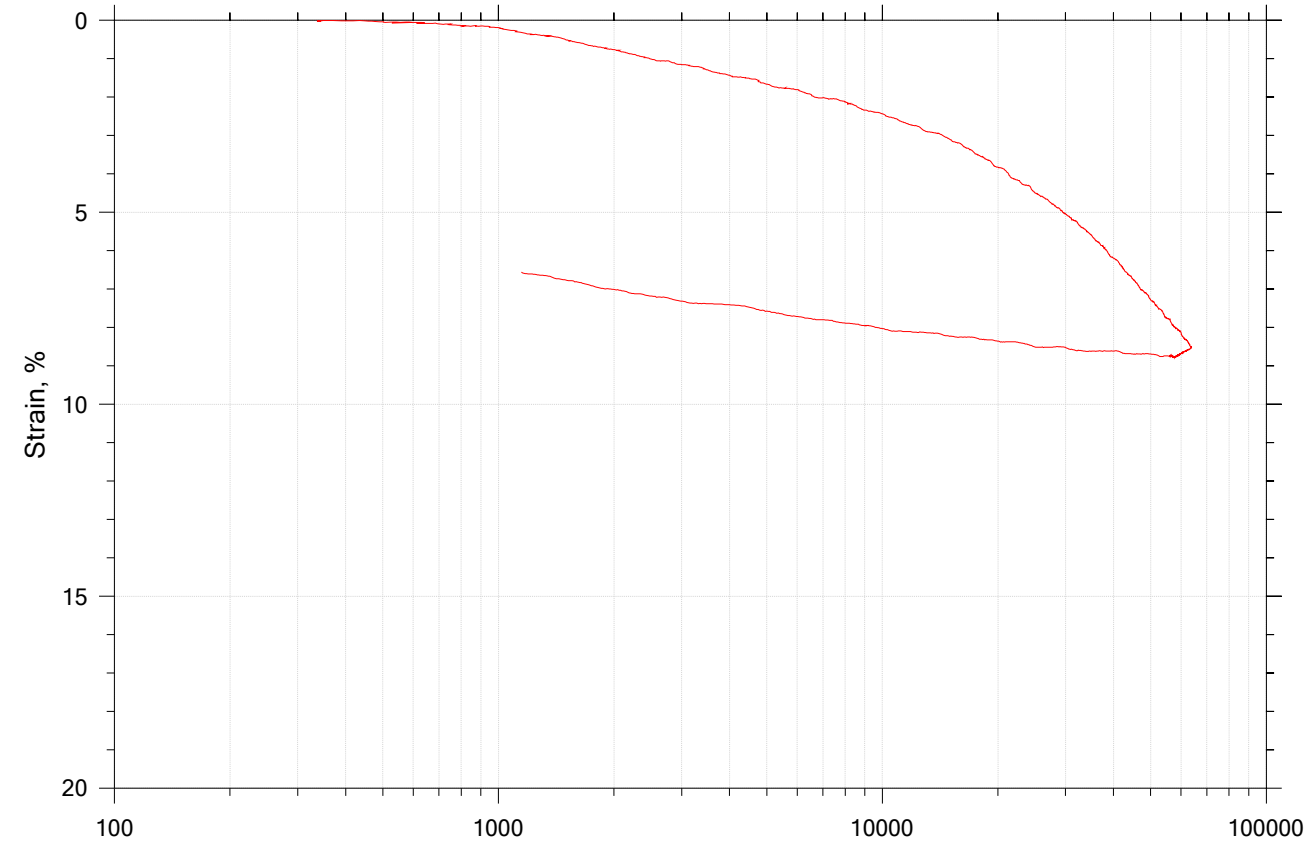
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	A-1859	---		E4886
Mass Container, gm	8.61	108.88	108.88	8.05
Mass Container + Wet Soil, gm	74.09	265.01	255.14	157.65
Mass Container + Dry Soil, gm	58.93	228.16	228.16	130.05
Mass Dry Soil, gm	50.32	119.28	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	---

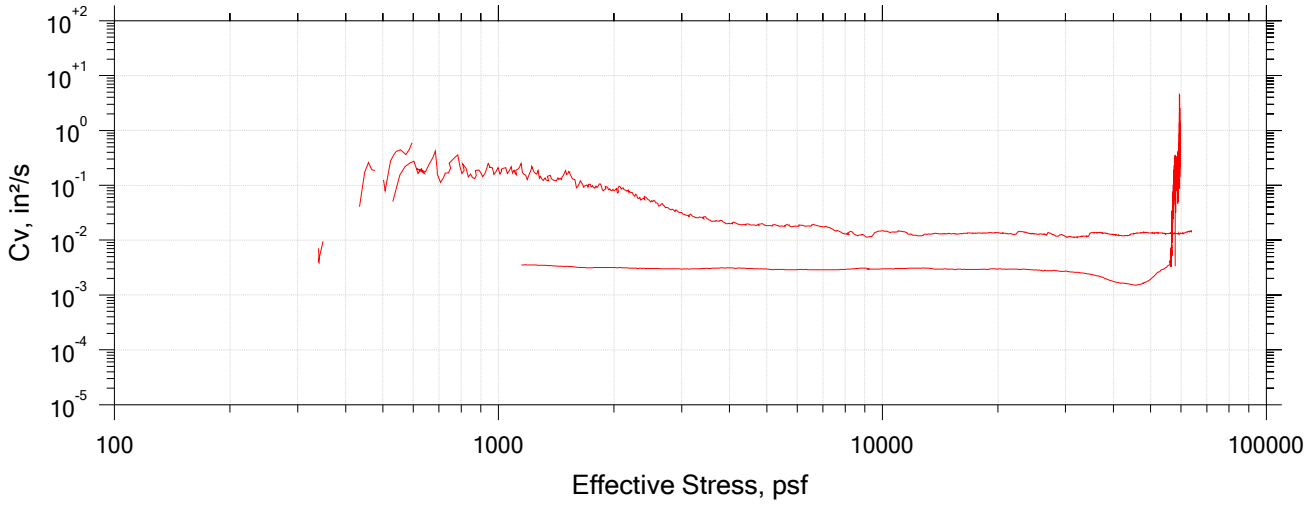
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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

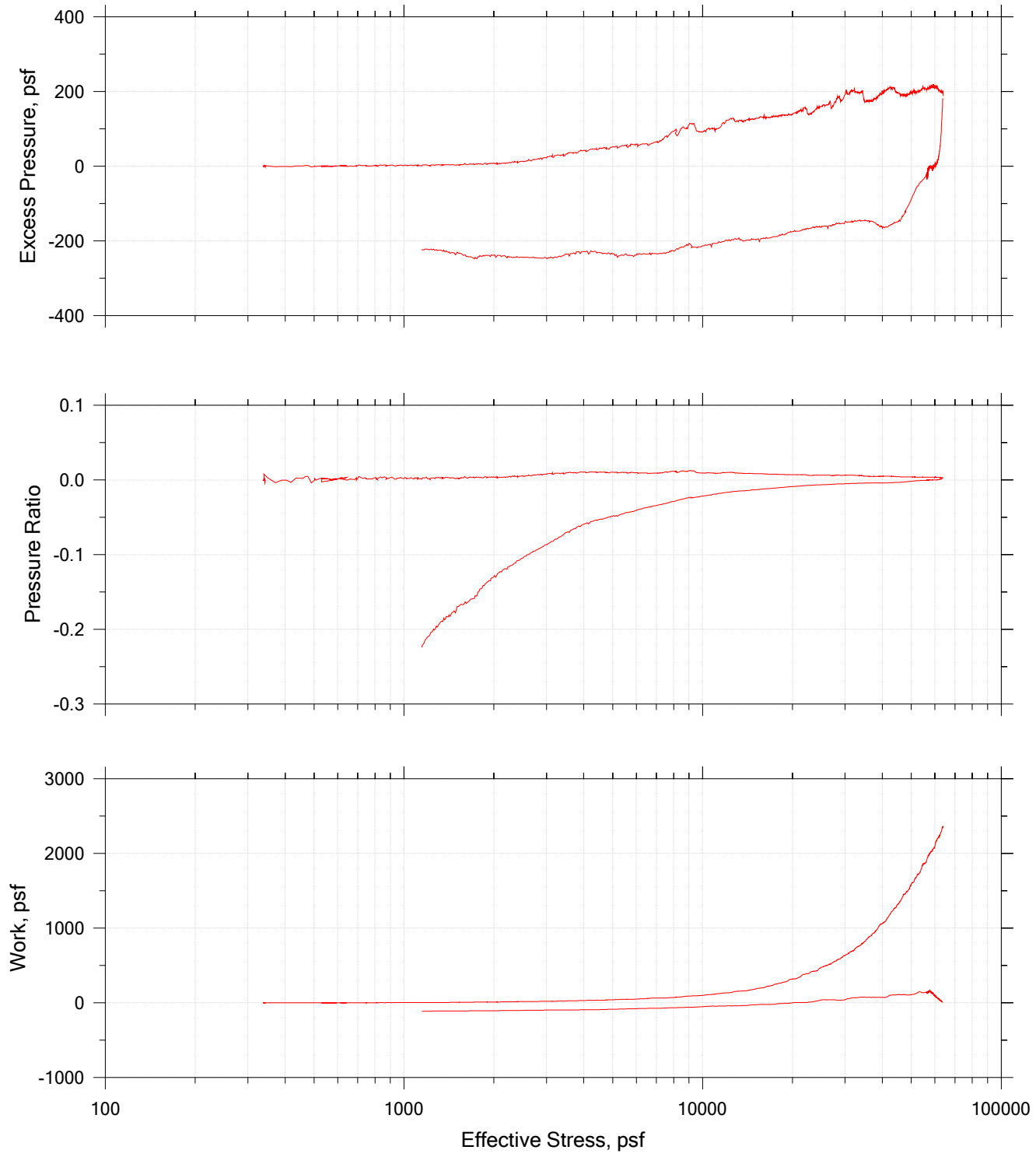




GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-102	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test Number: CRC-2	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: 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

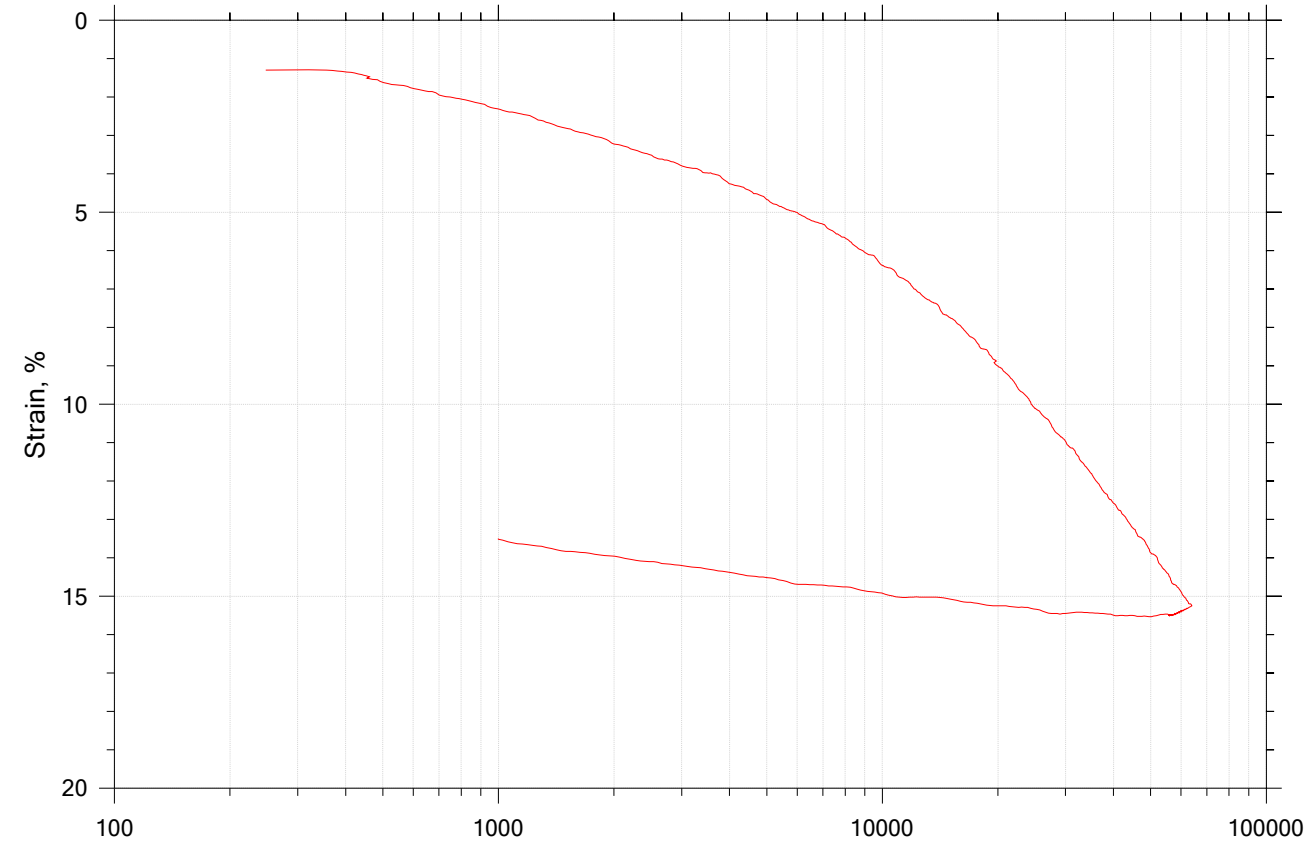
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-2107	---		E1885
Mass Container, gm	8.7	110.07	110.07	8.5
Mass Container + Wet Soil, gm	163.2	271.82	265.69	167.49
Mass Container + Dry Soil, gm	132.66	238.35	238.35	139.56
Mass Dry Soil, gm	123.96	128.28	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	---

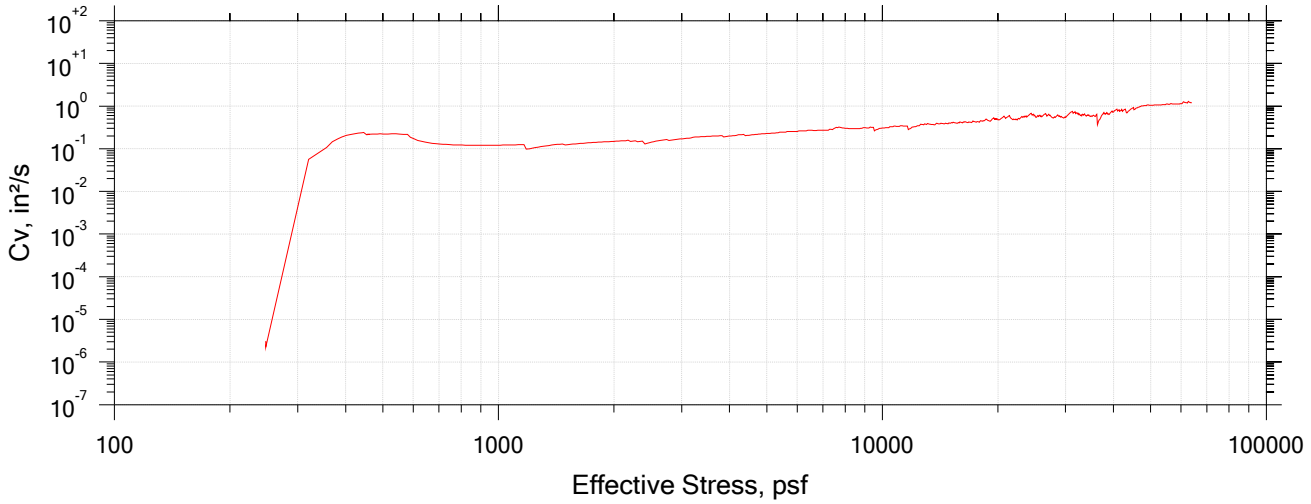
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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

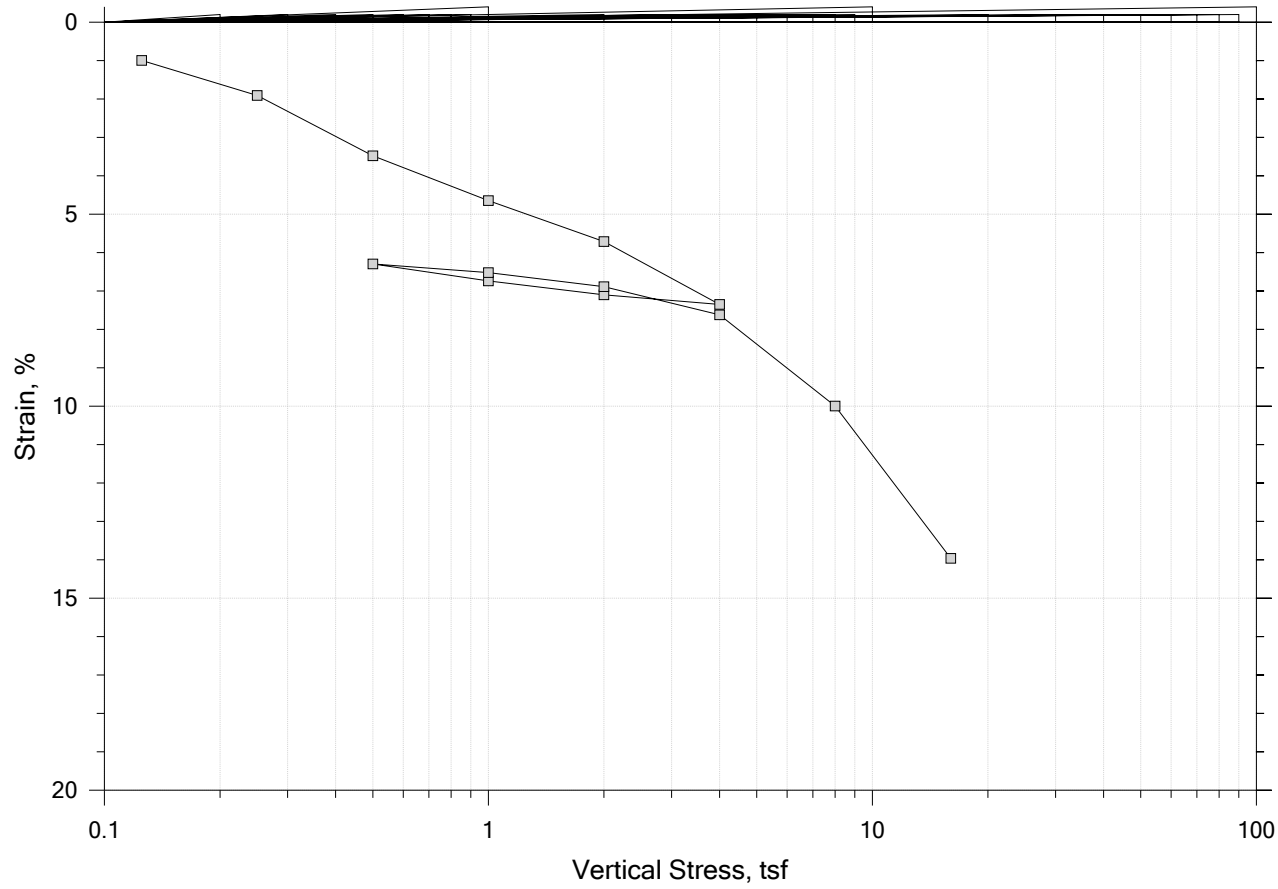


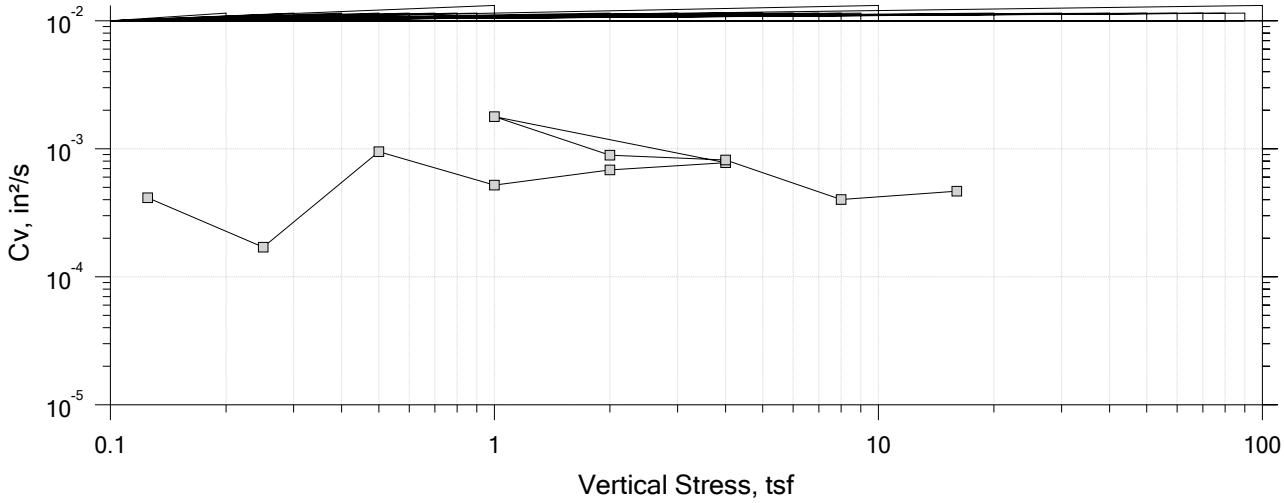


GeoTesting EXPRESS	Project Name: Hogan Road Highway	Location: Bangor, ME	Project Number: GTX-315483
	Boring Number: HB-BHR-108	Tester: md	Checker: njh
	Sample Number: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test Number: CRC-1	Preparation: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System JJ		

One-Dimensional Consolidation by ASTM D2435 - Method B

Summary Report

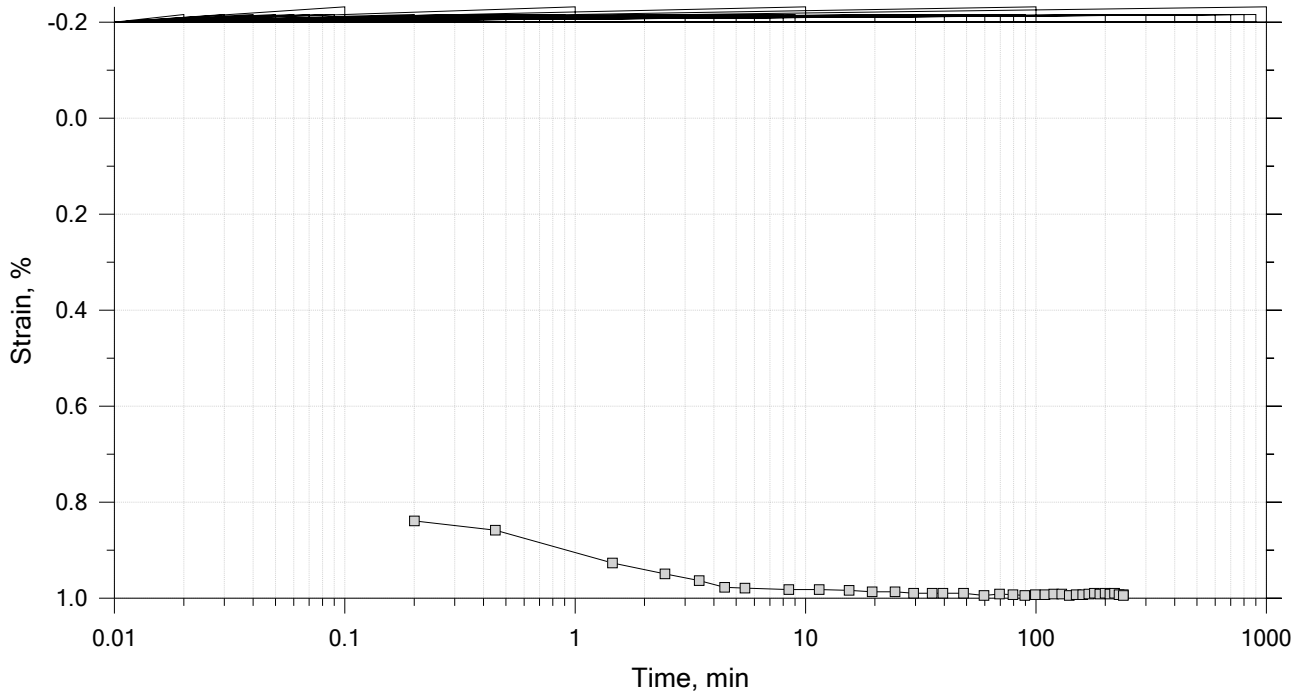


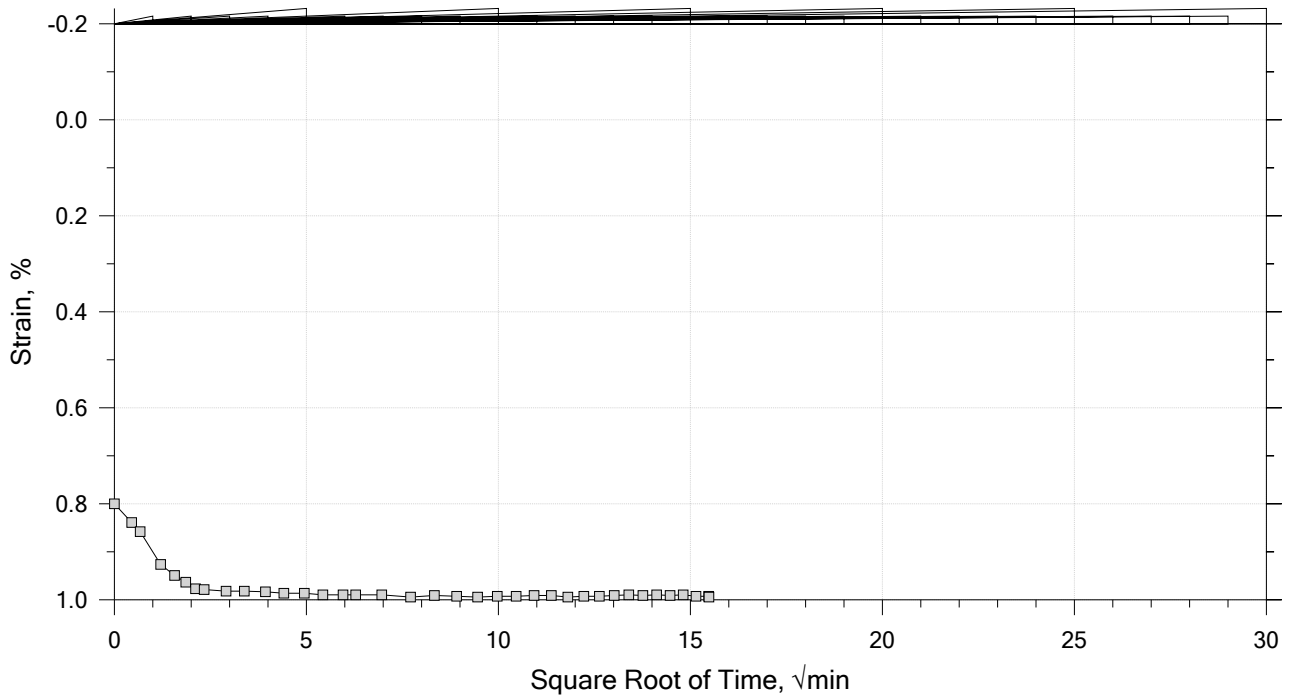


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		
	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

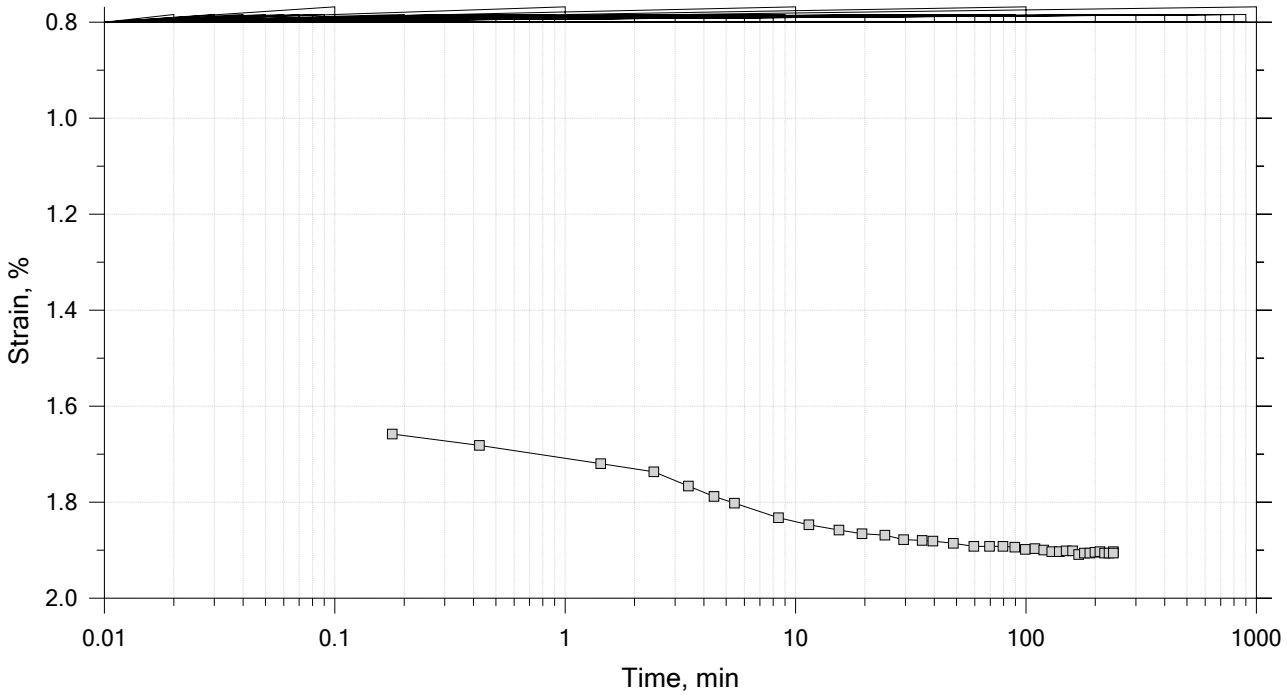


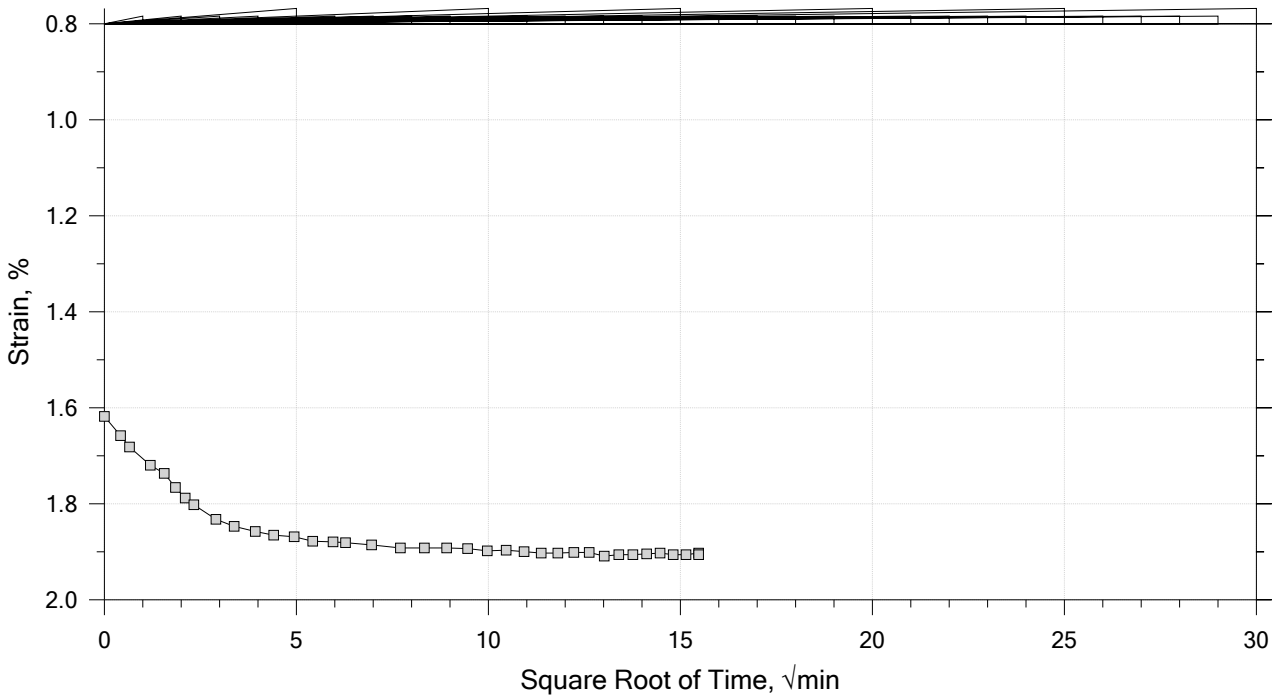


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf

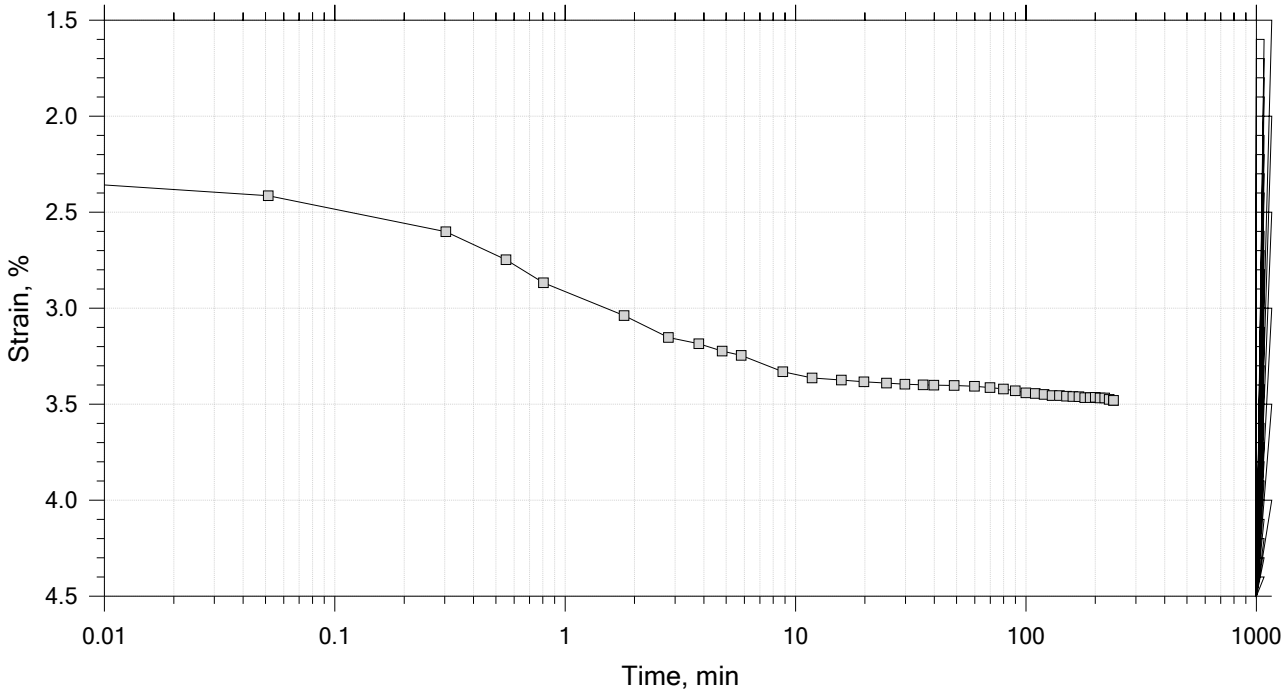


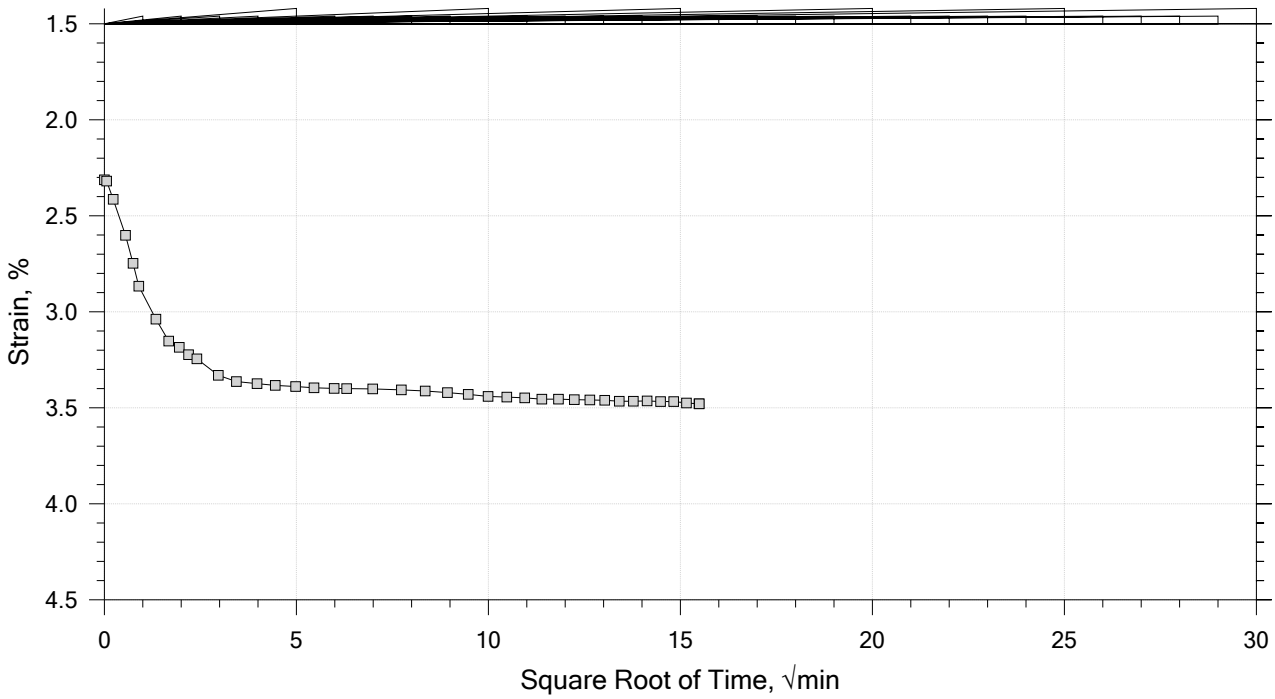


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf

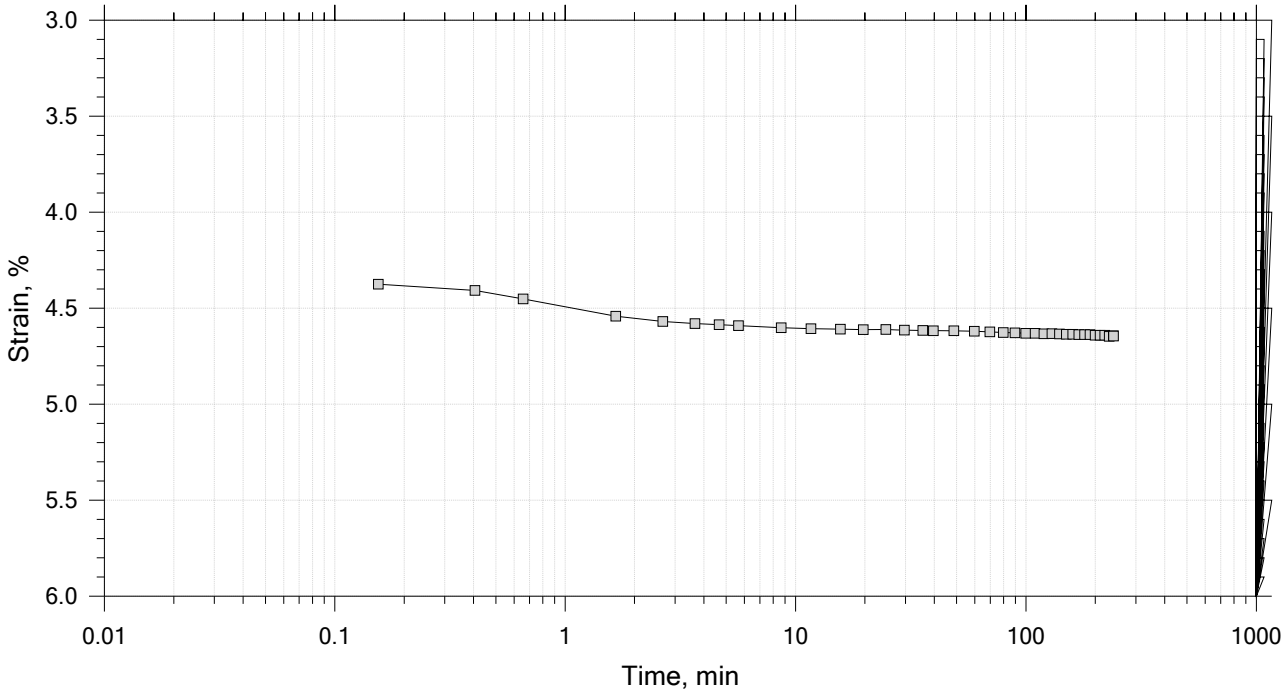


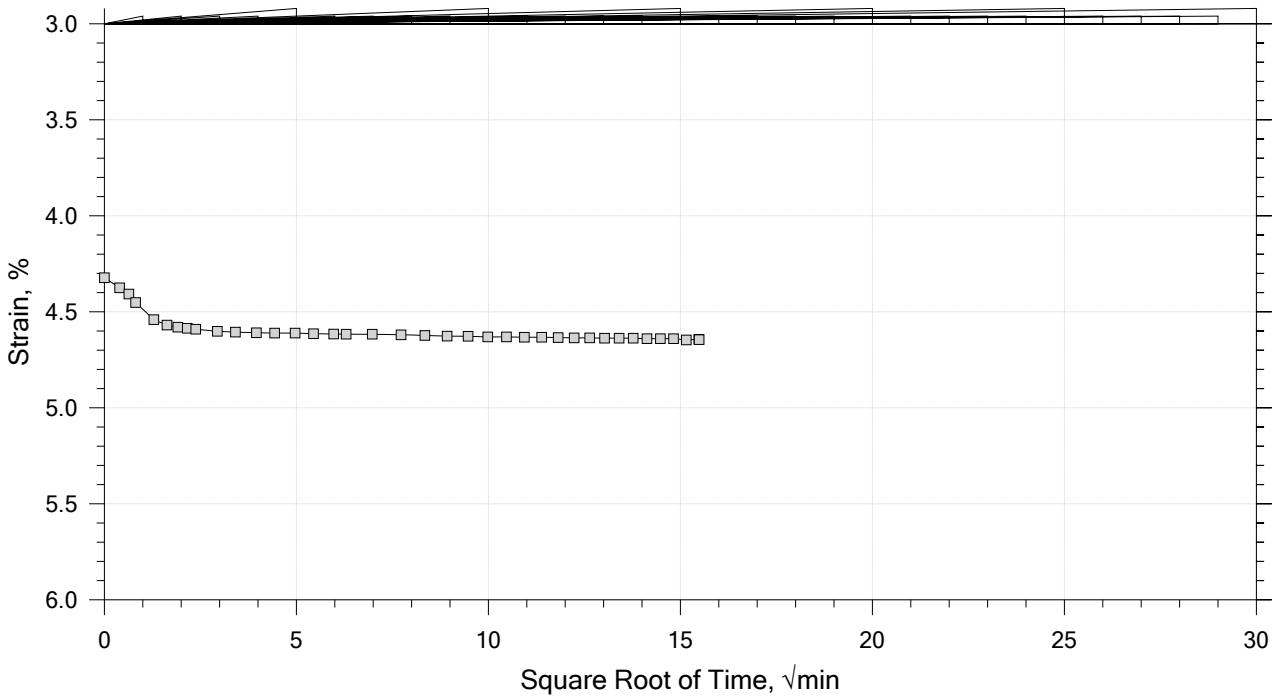


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14
Constant Load Step
Stress: 1 tsf





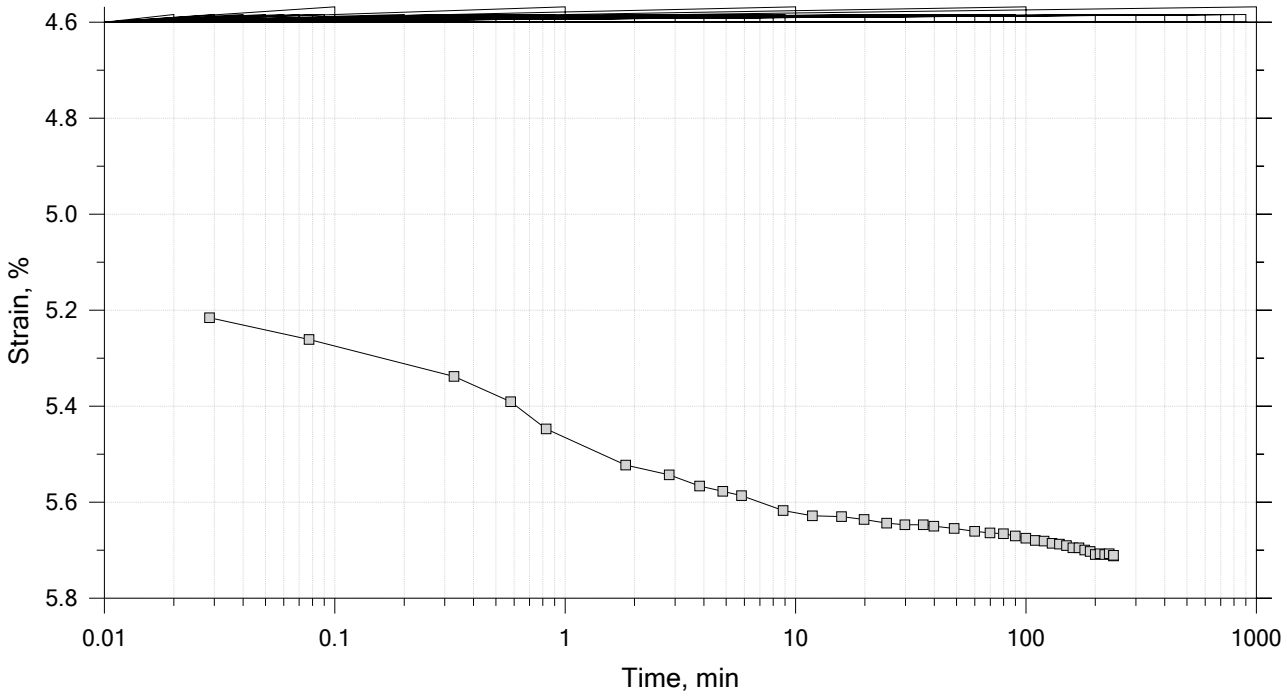
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

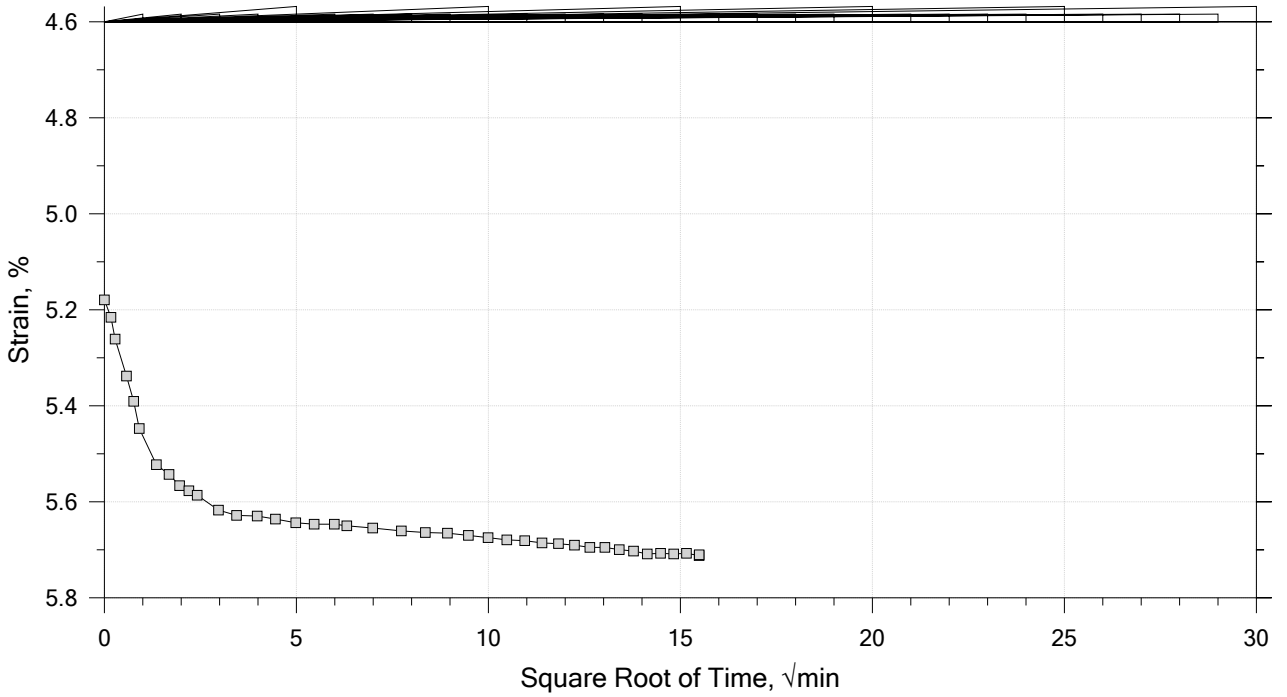
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14

Constant Load Step

Stress: 2 tsf

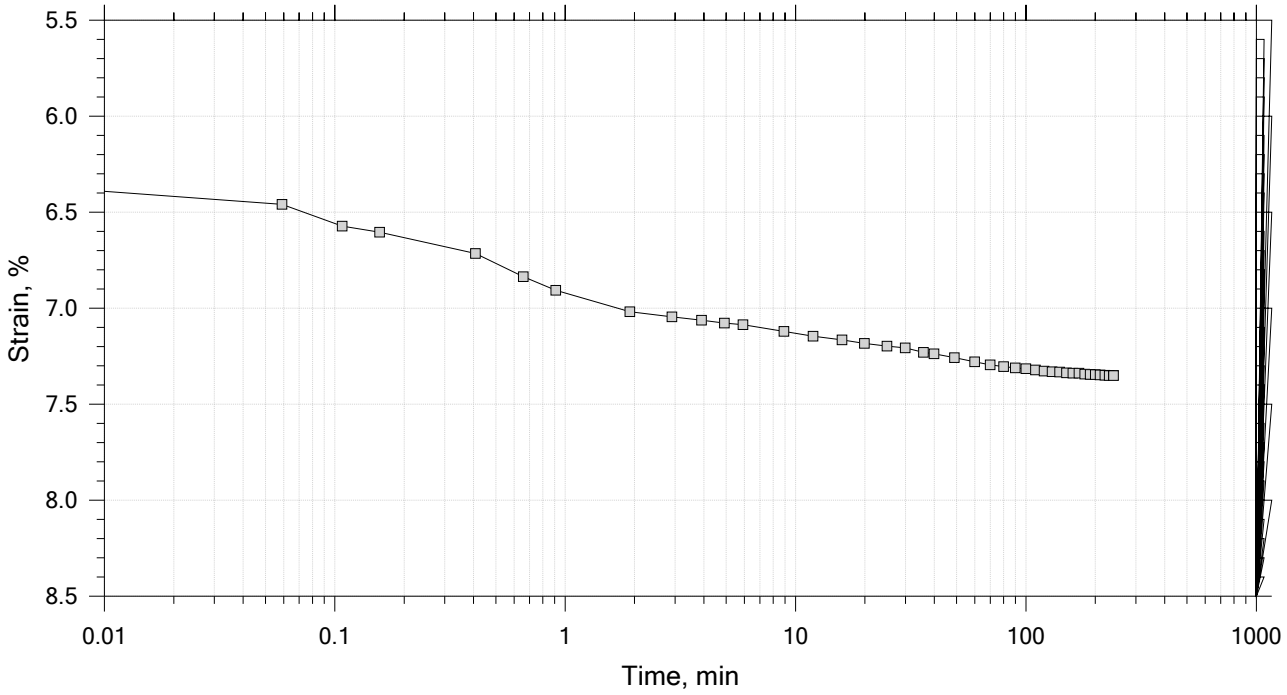


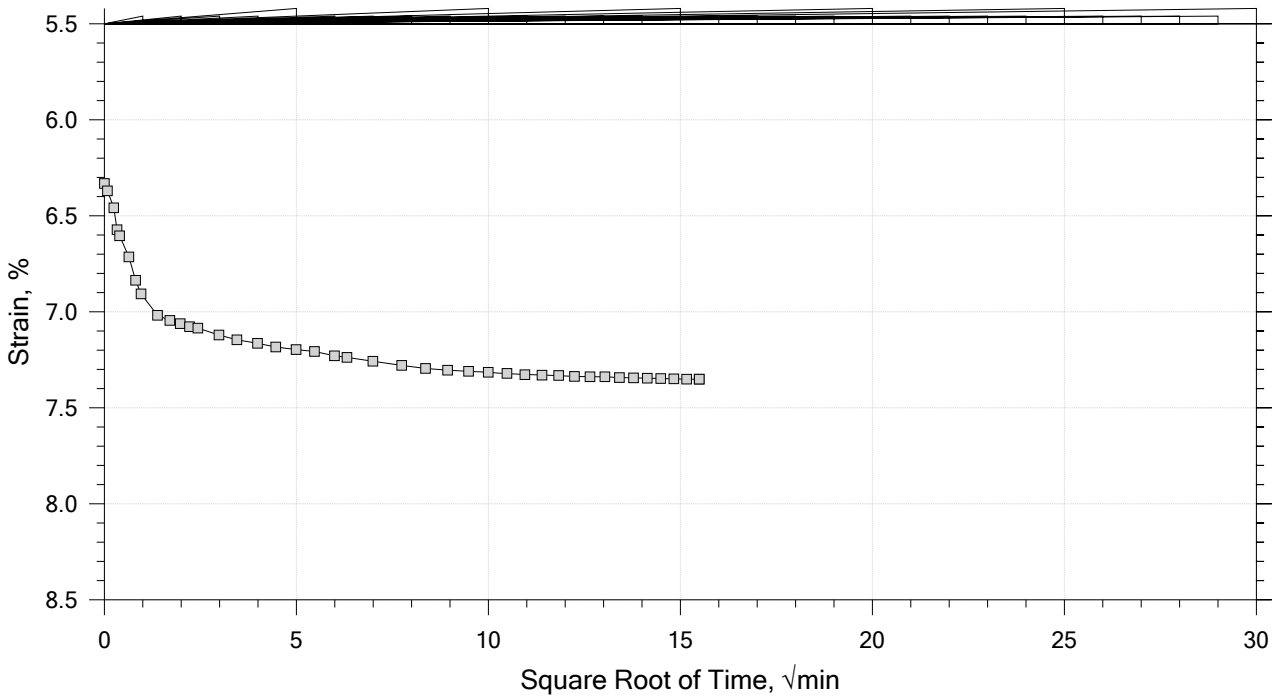


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14
Constant Load Step
Stress: 4 tsf

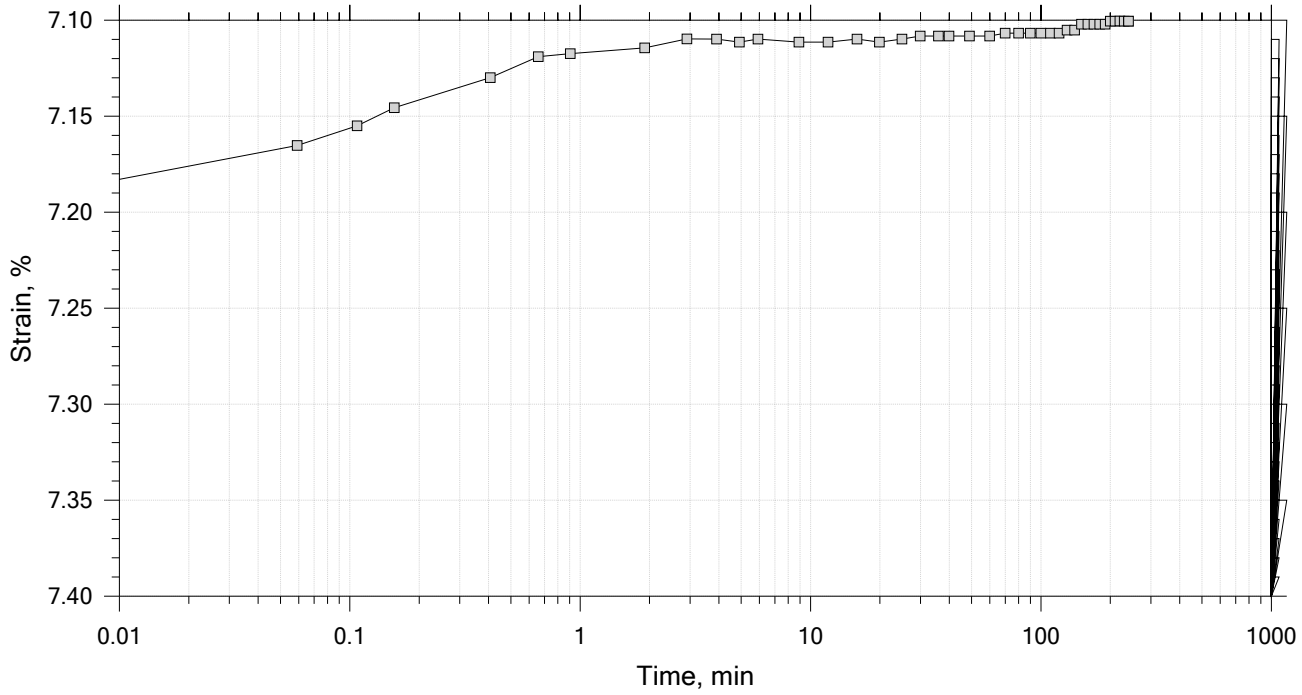


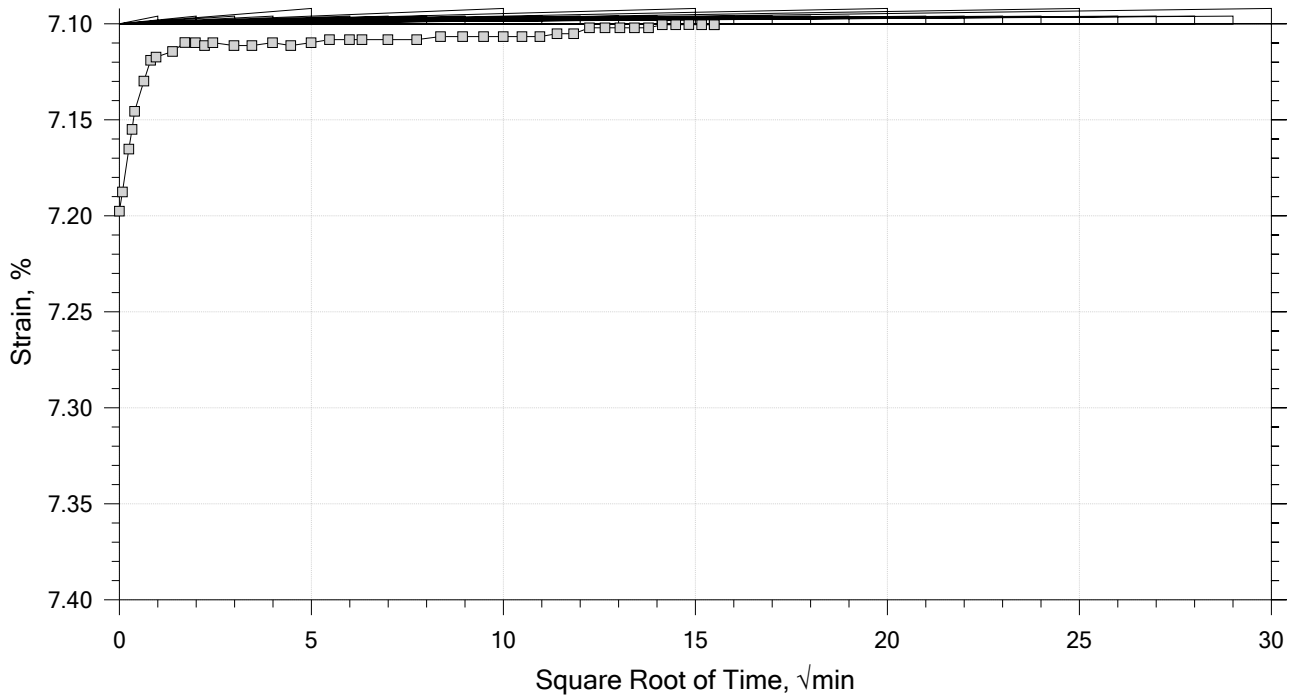


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14
Constant Load Step
Stress: 2 tsf





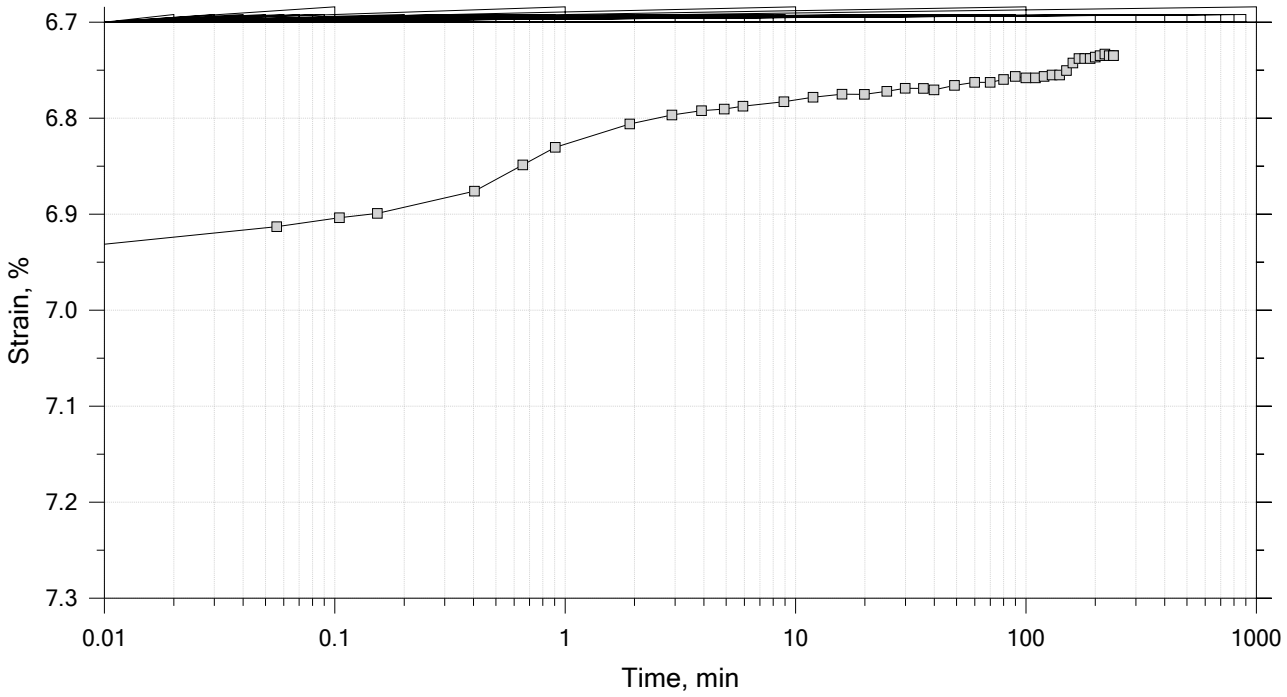
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

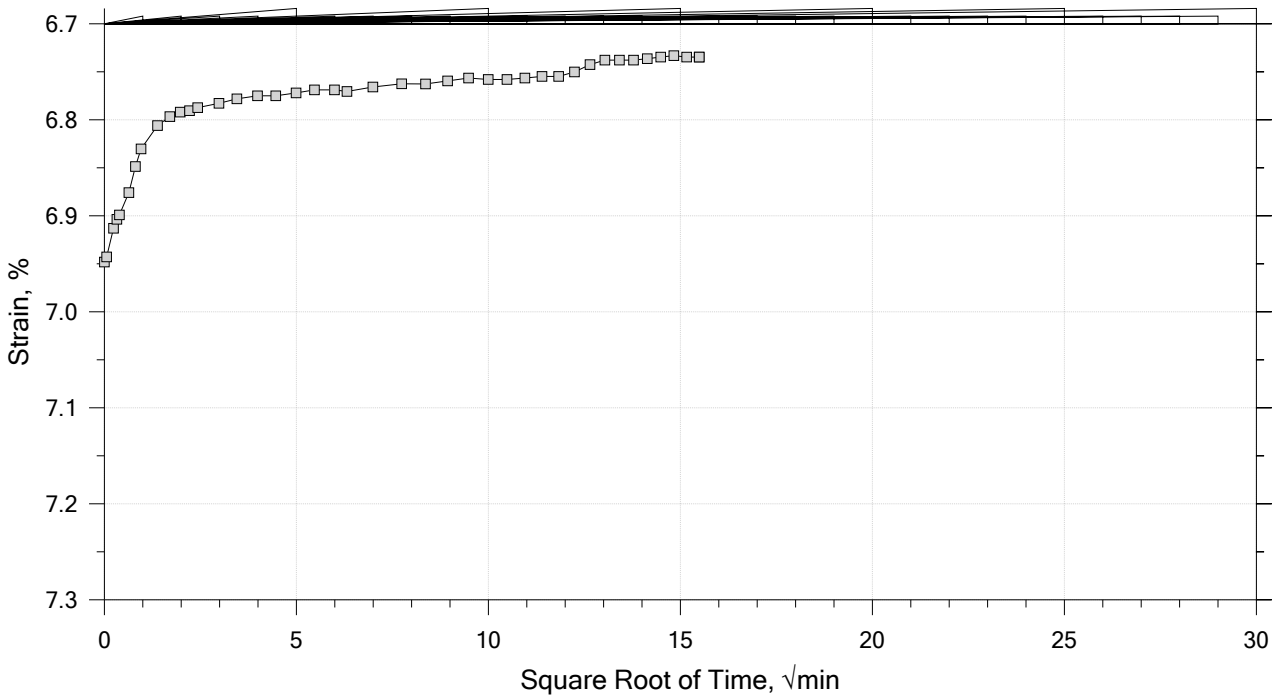
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf

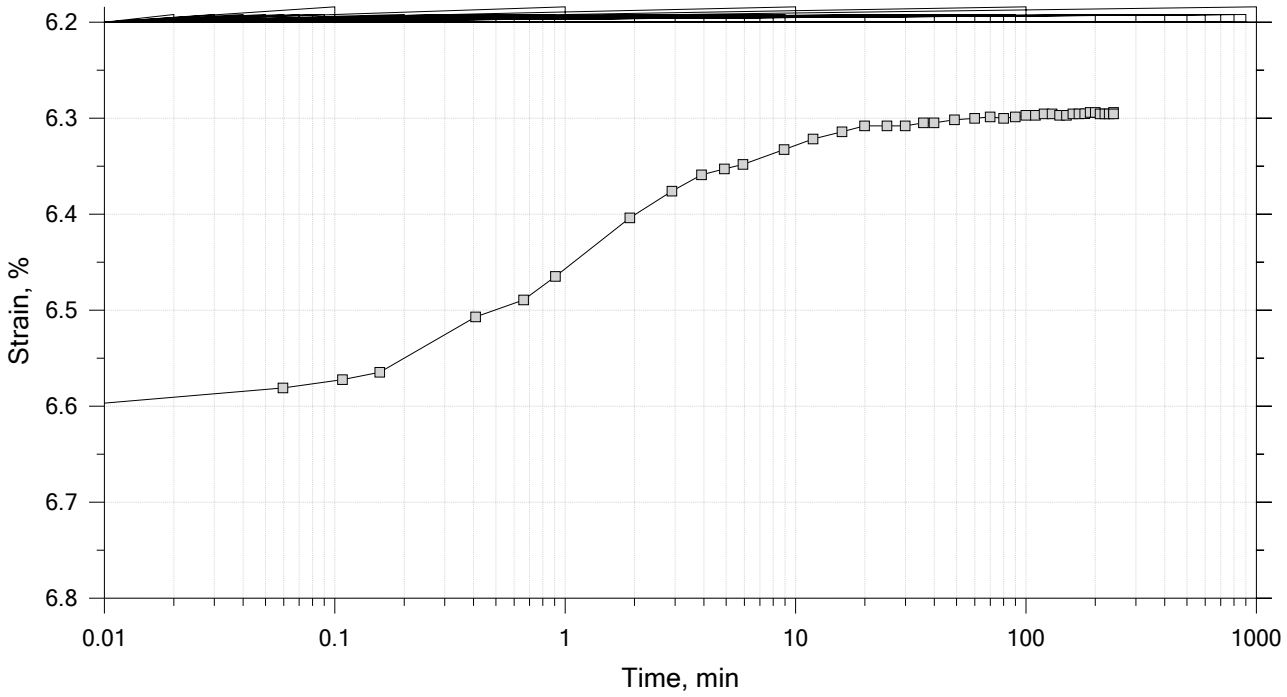


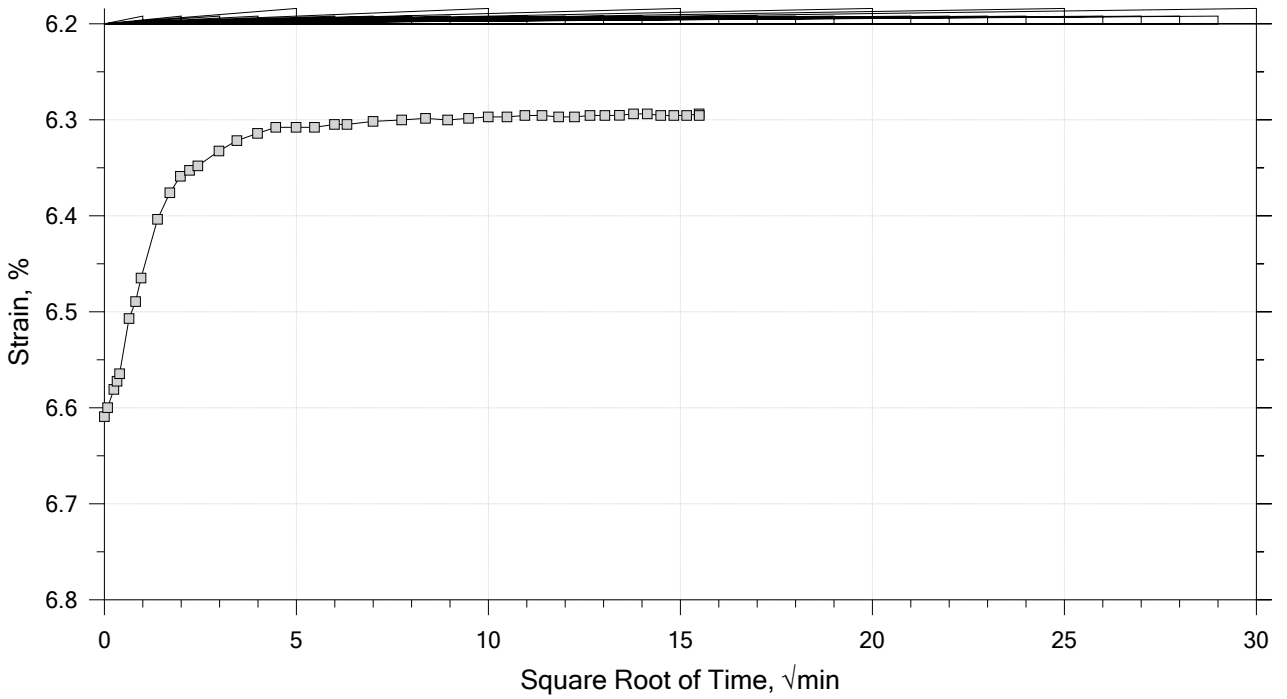


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

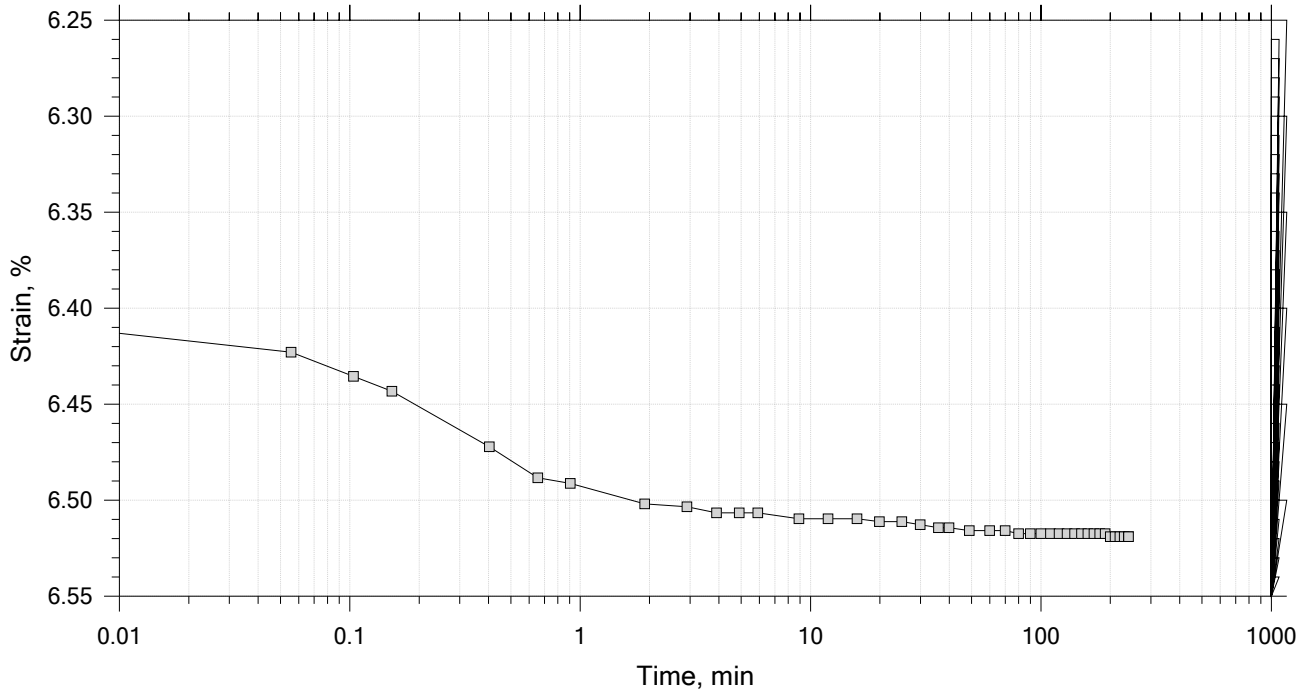


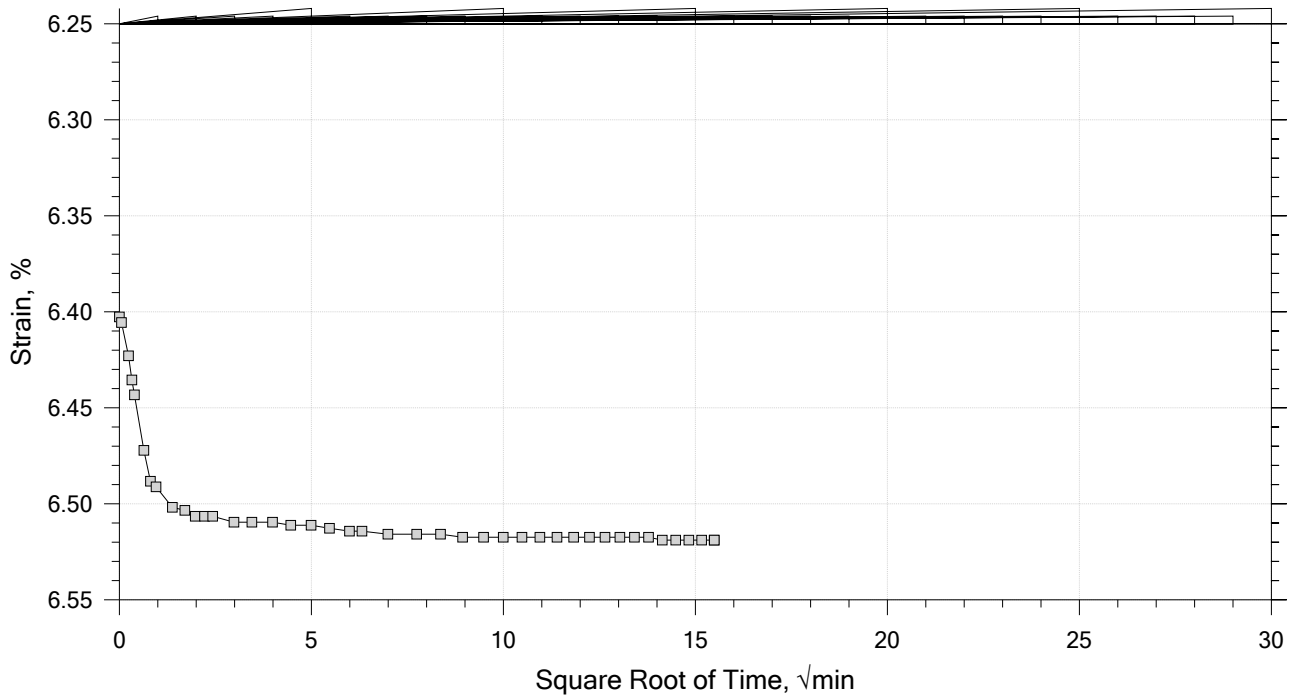


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

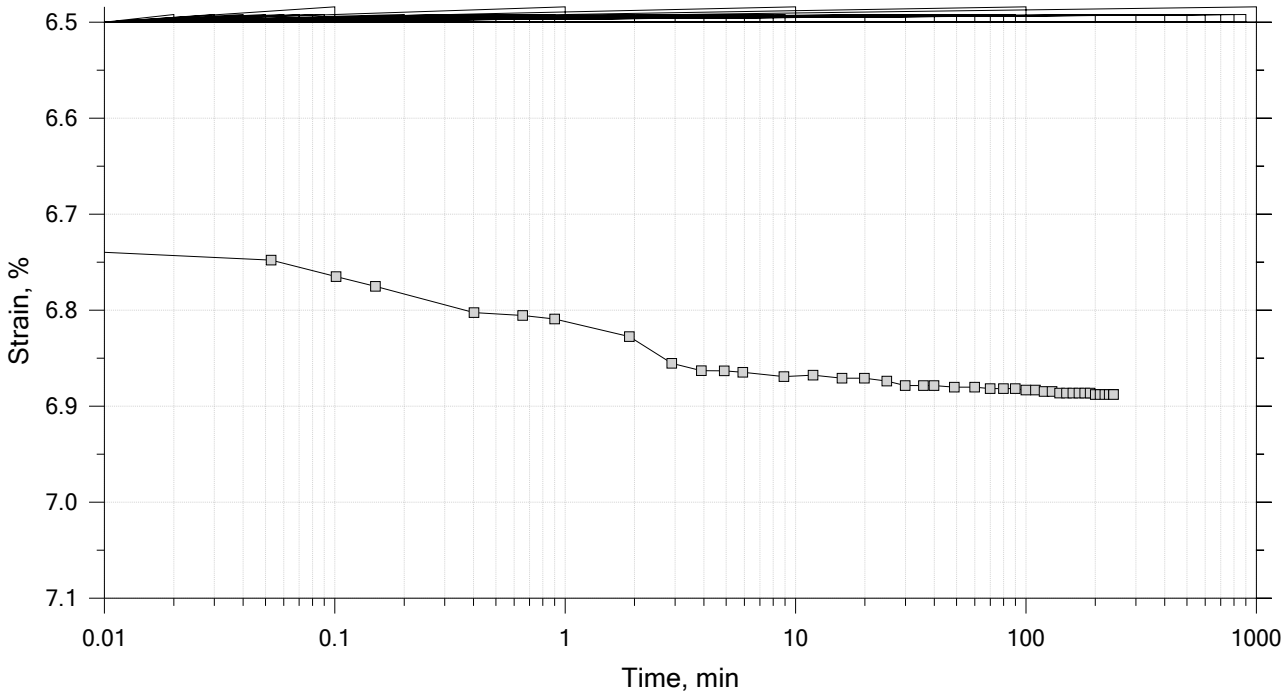


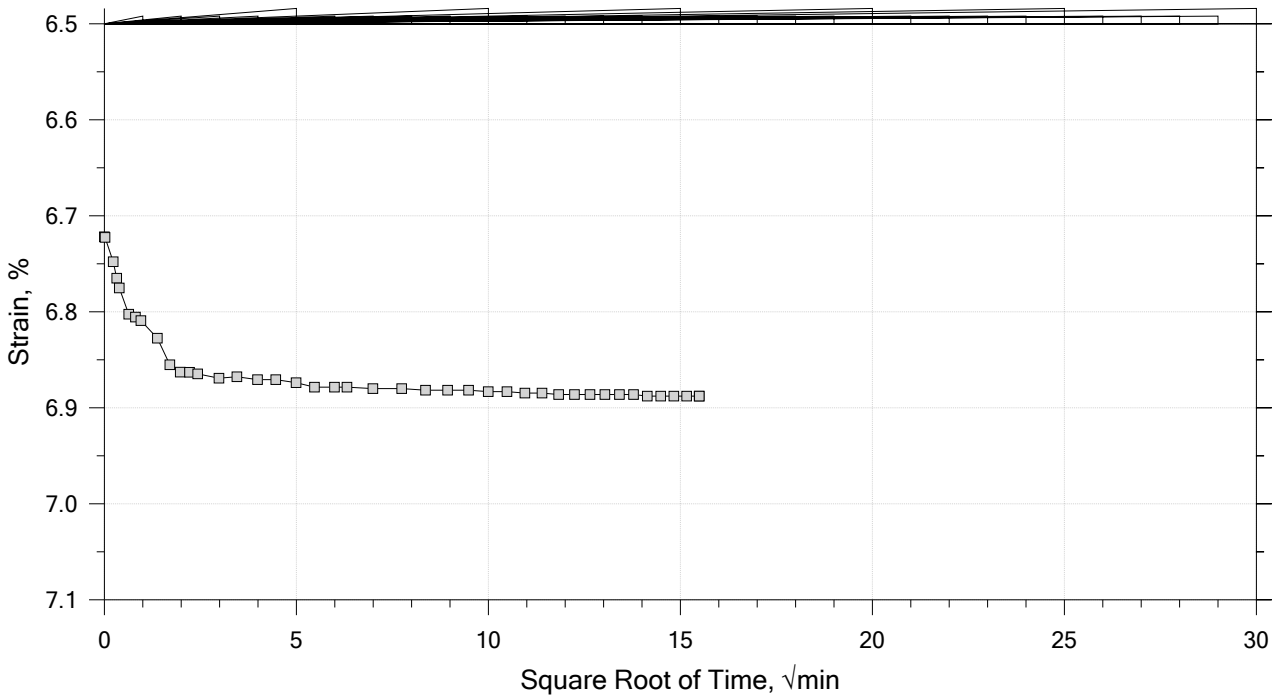


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf

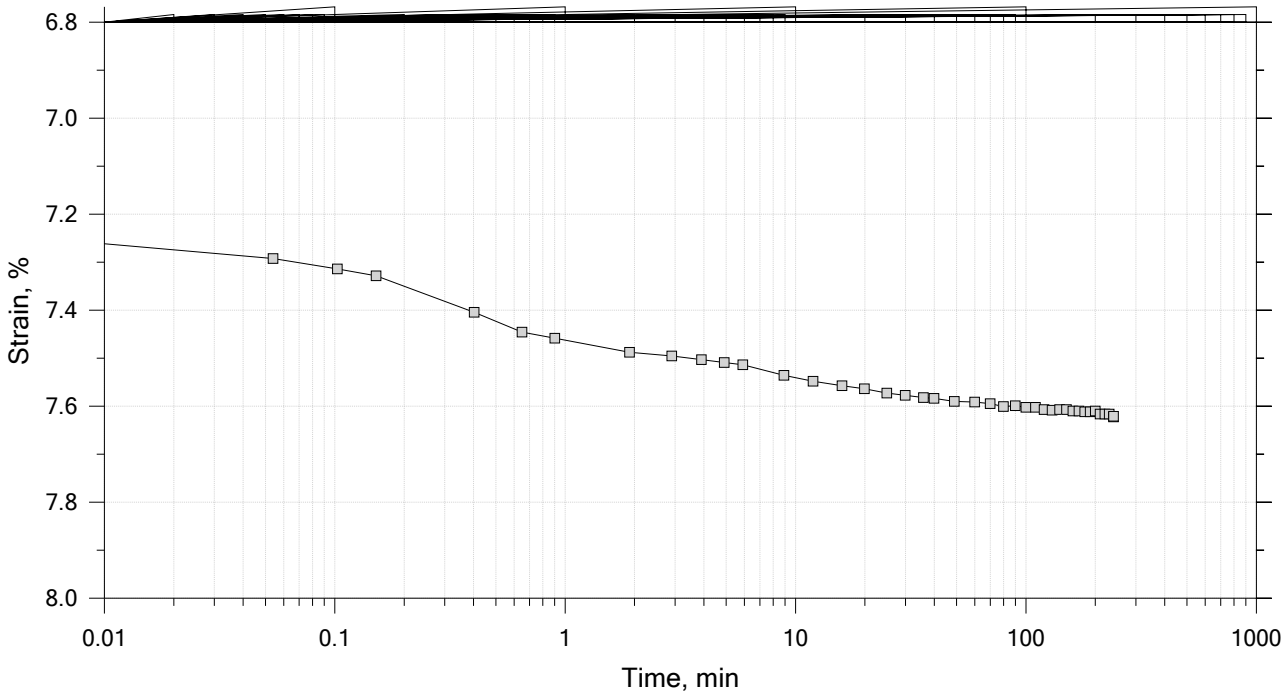


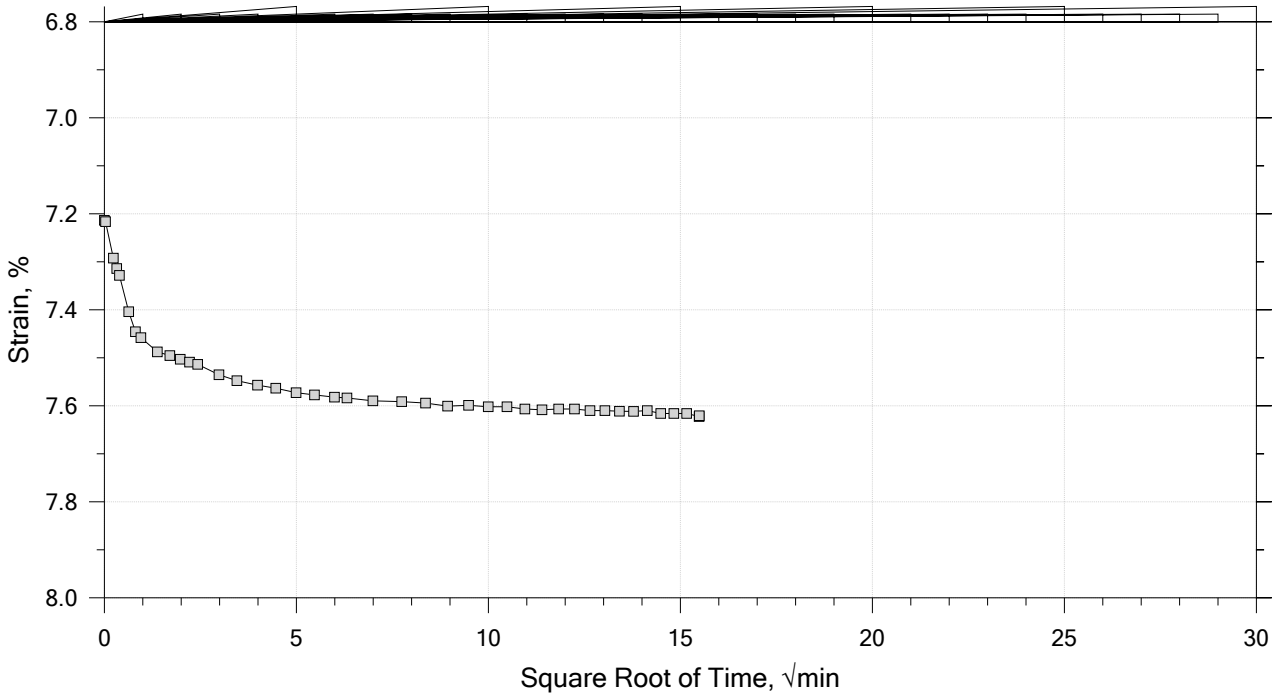


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14
Constant Load Step
Stress: 4 tsf

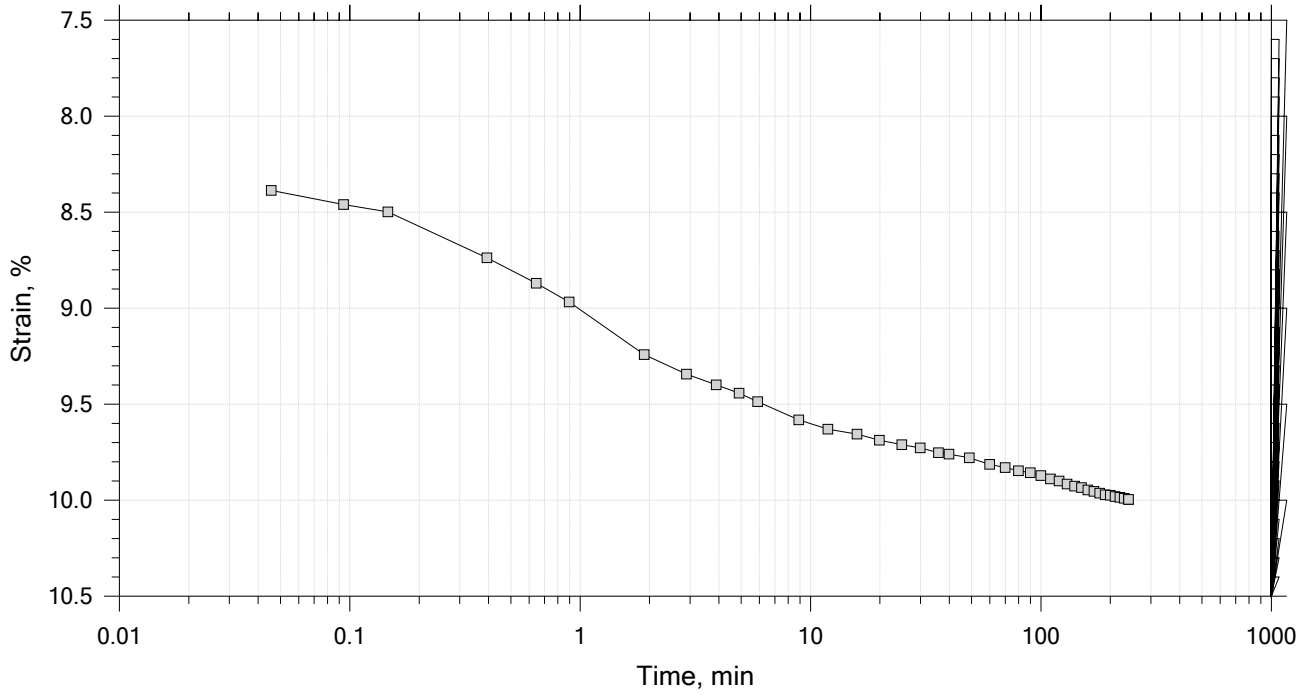


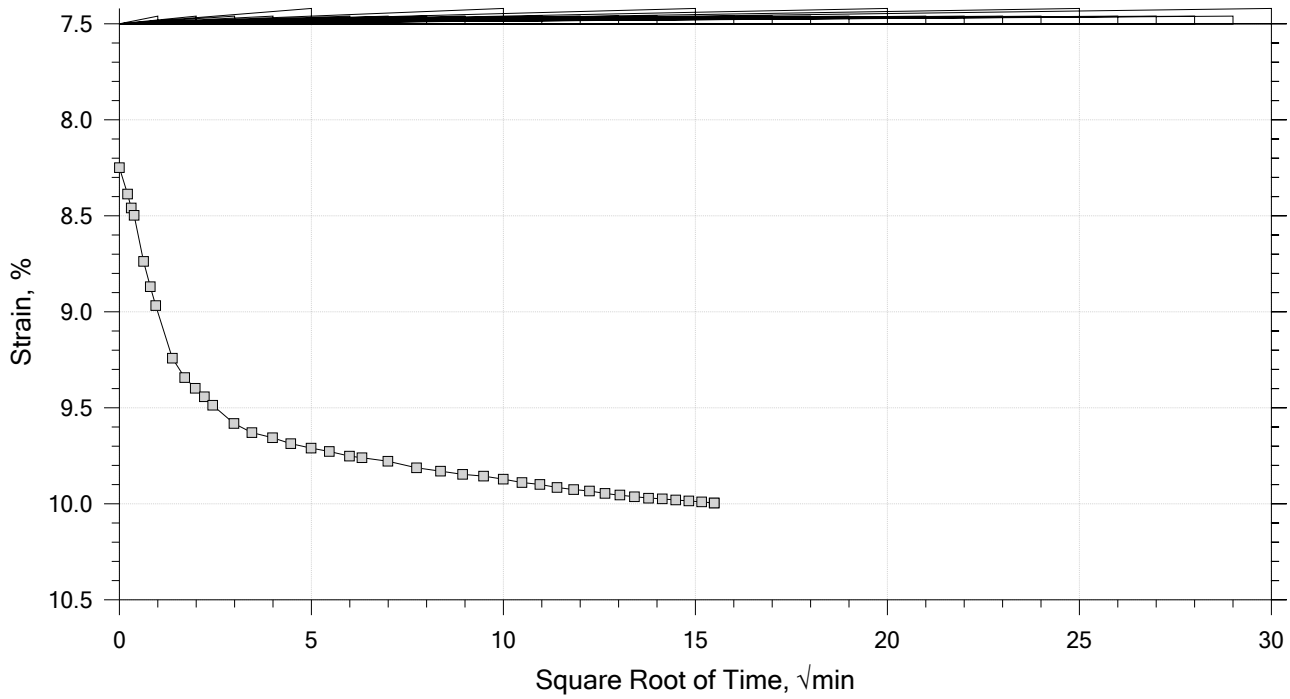


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14
Constant Load Step
Stress: 8 tsf

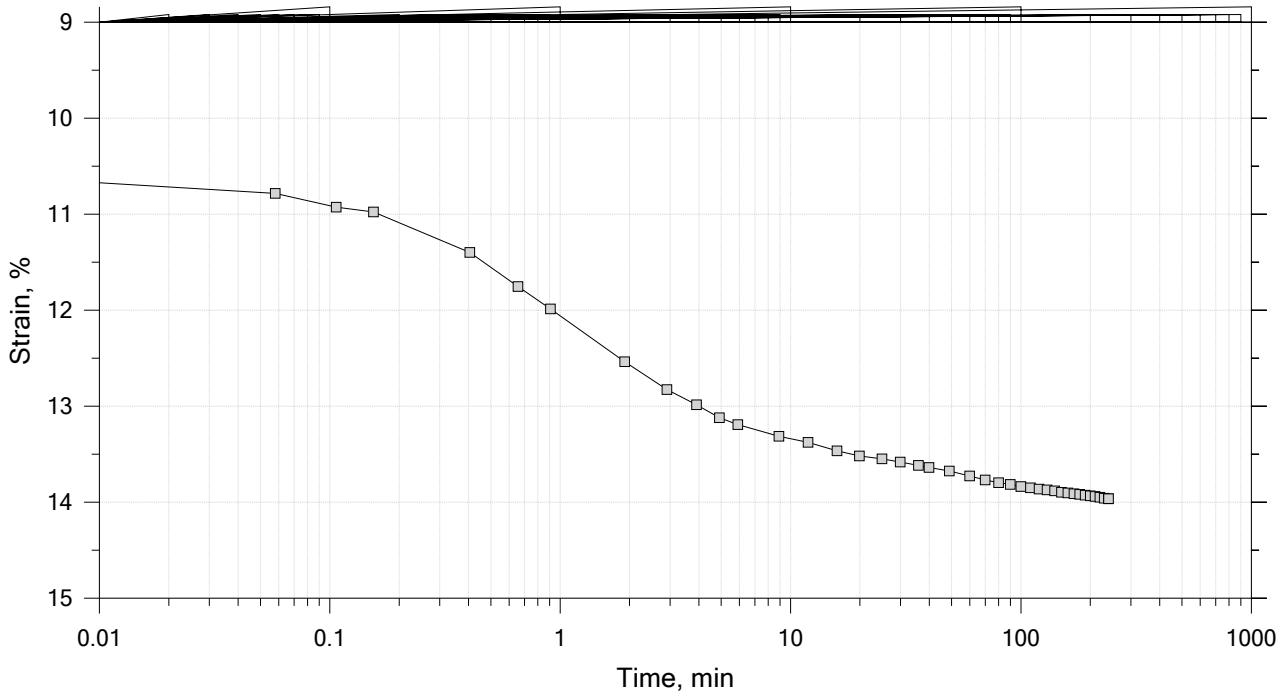


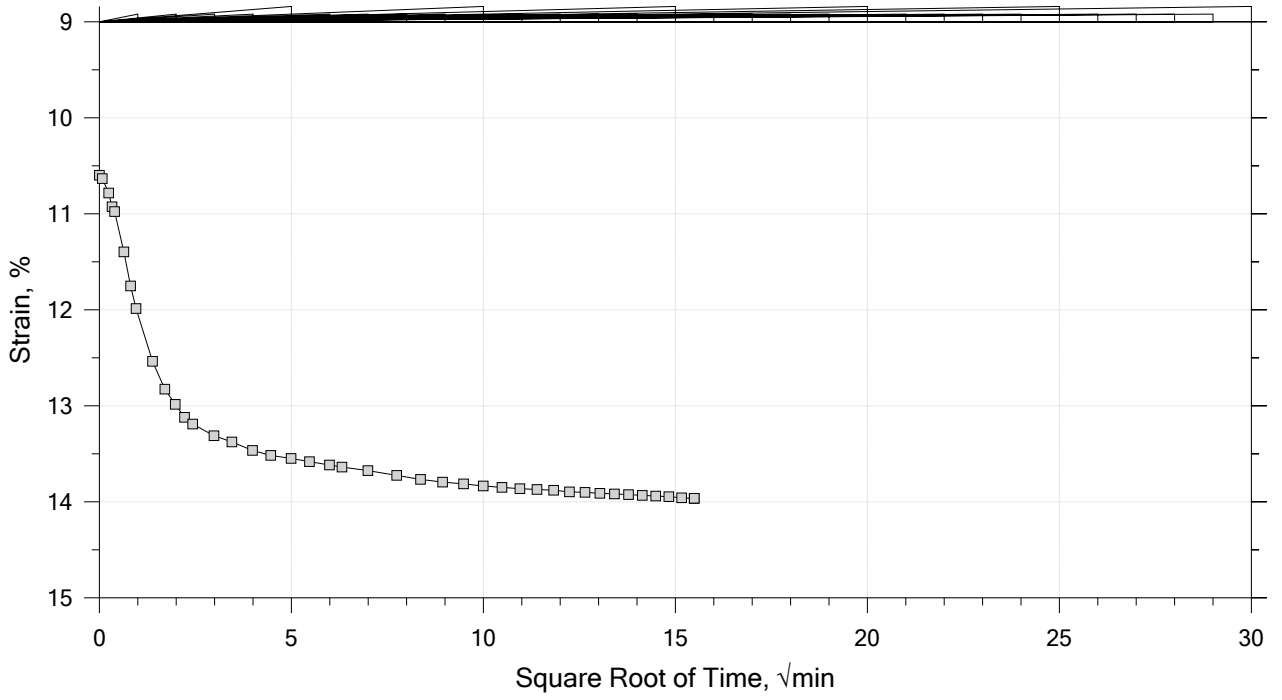


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-101	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/25/22	Depth: 10-12 ft
	Test No.: IP-3	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-G		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 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

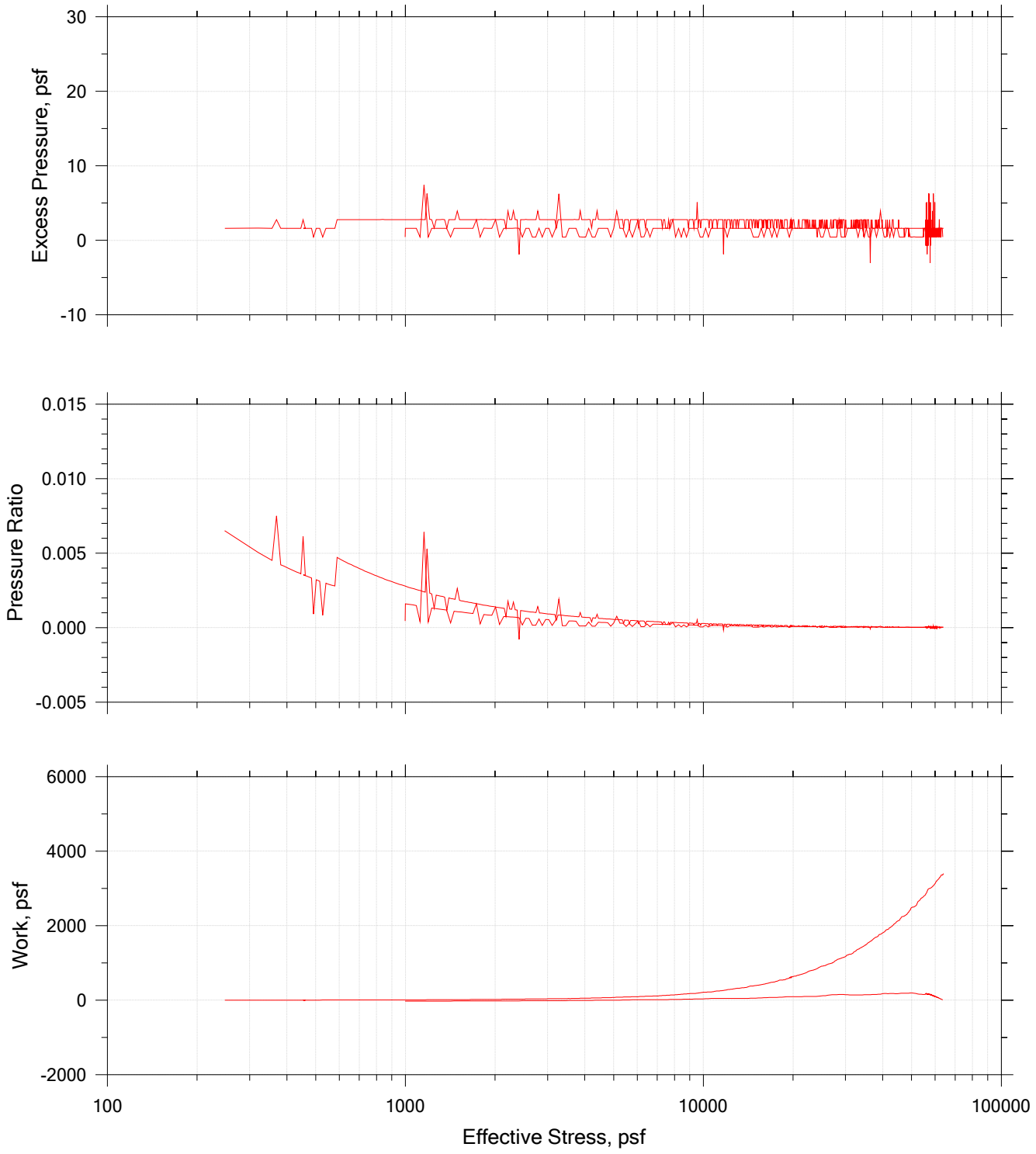
Square Root of Time Coefficients

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

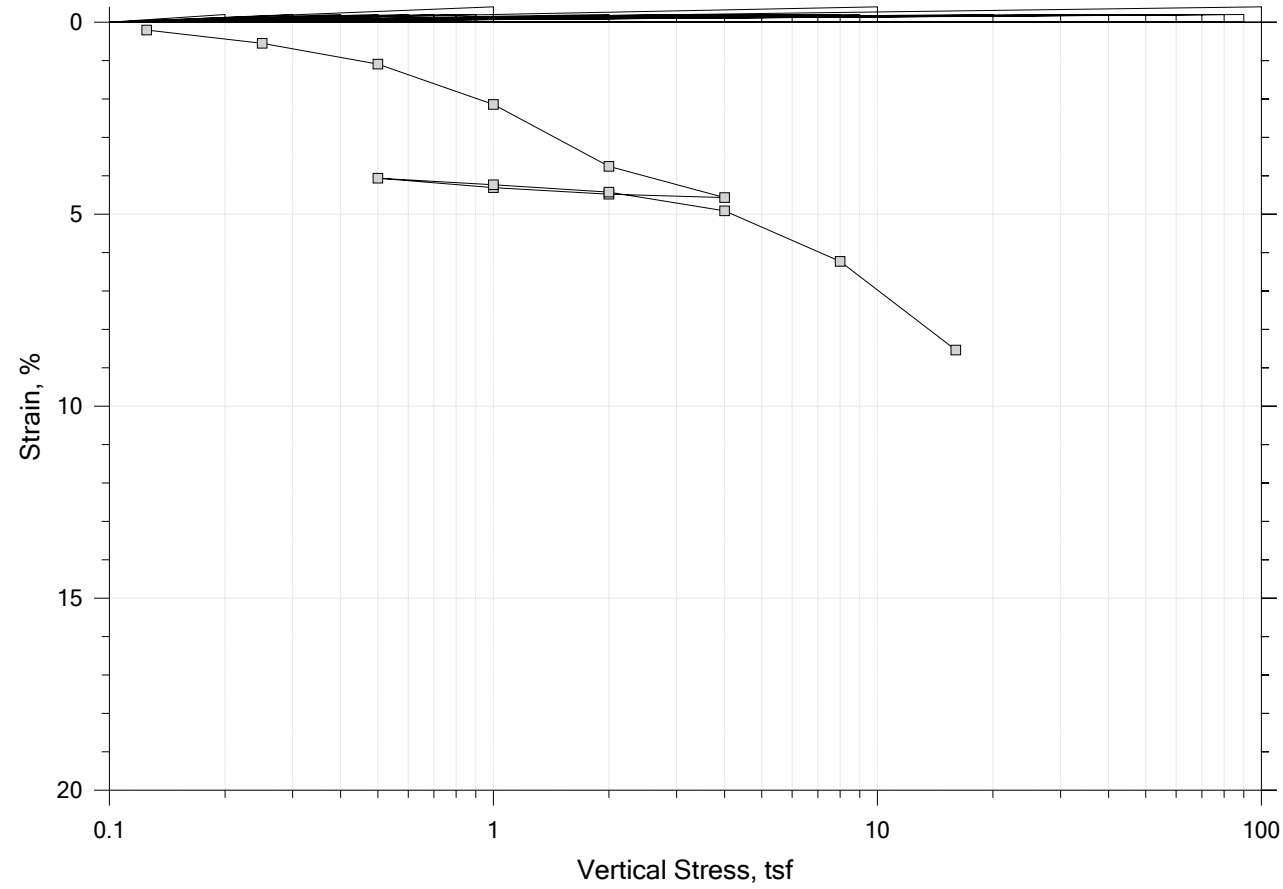
	Before Test Trimmings	Before Test Specimen	After Test Specimen	After Test Trimmings
Container ID	E-3892	---		E5211
Mass Container, gm	8.43	111.05	111.05	8.46
Mass Container + Wet Soil, gm	217.8	266.79	255.62	157.31
Mass Container + Dry Soil, gm	171.21	229.29	229.29	130.2
Mass Dry Soil, gm	162.78	118.24	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	---

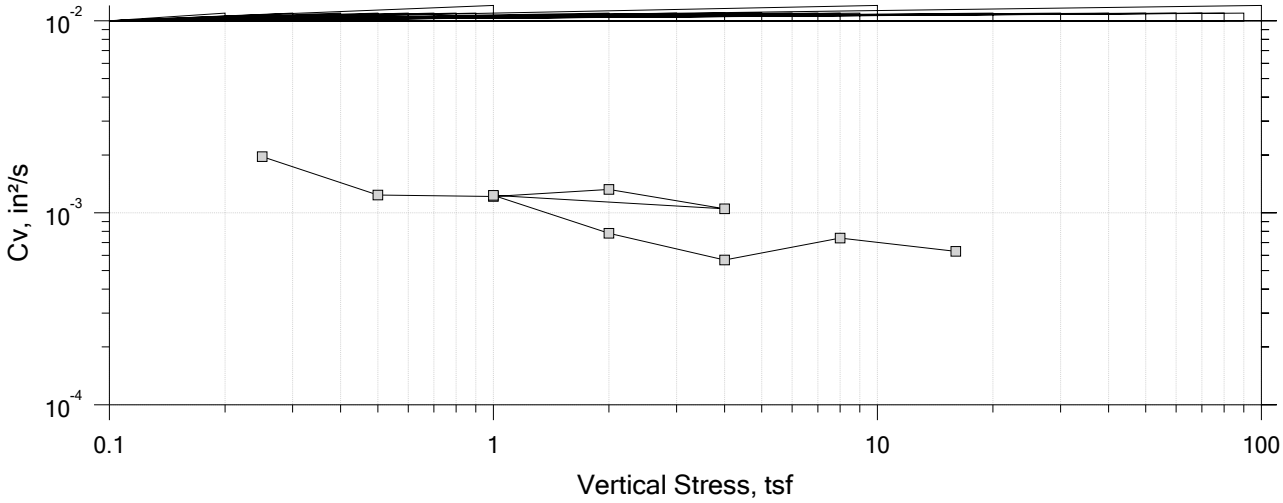
Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

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

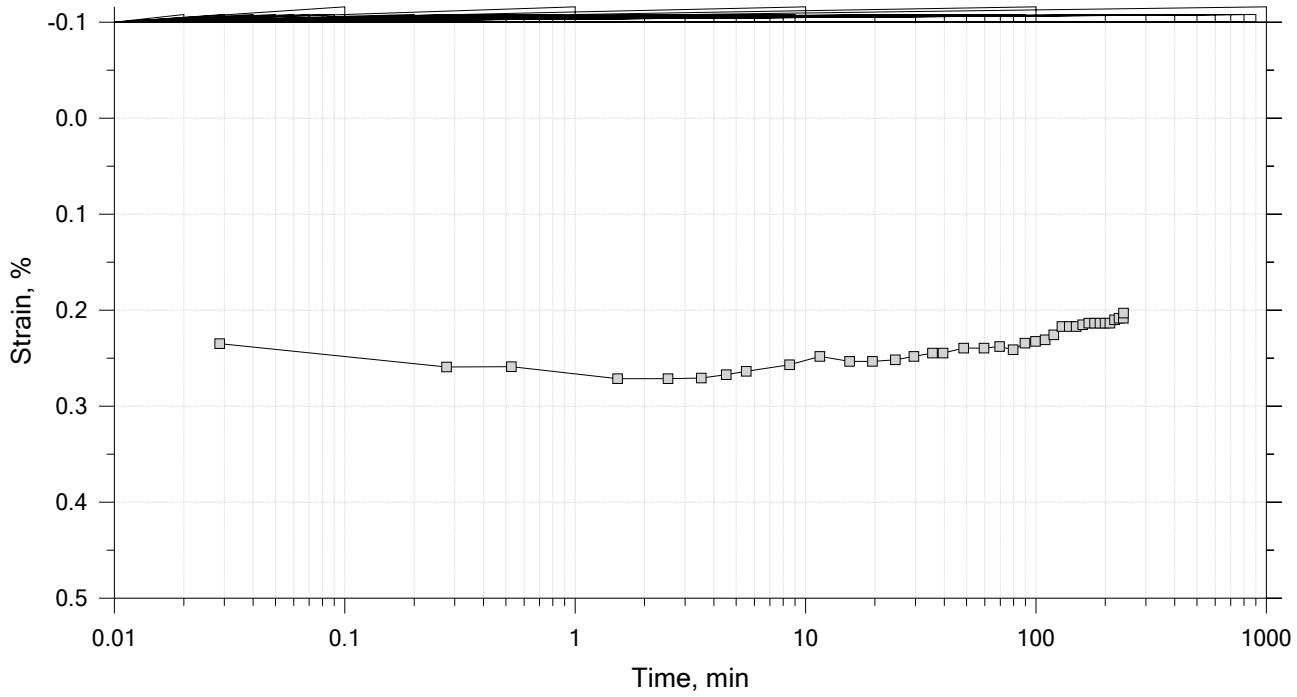


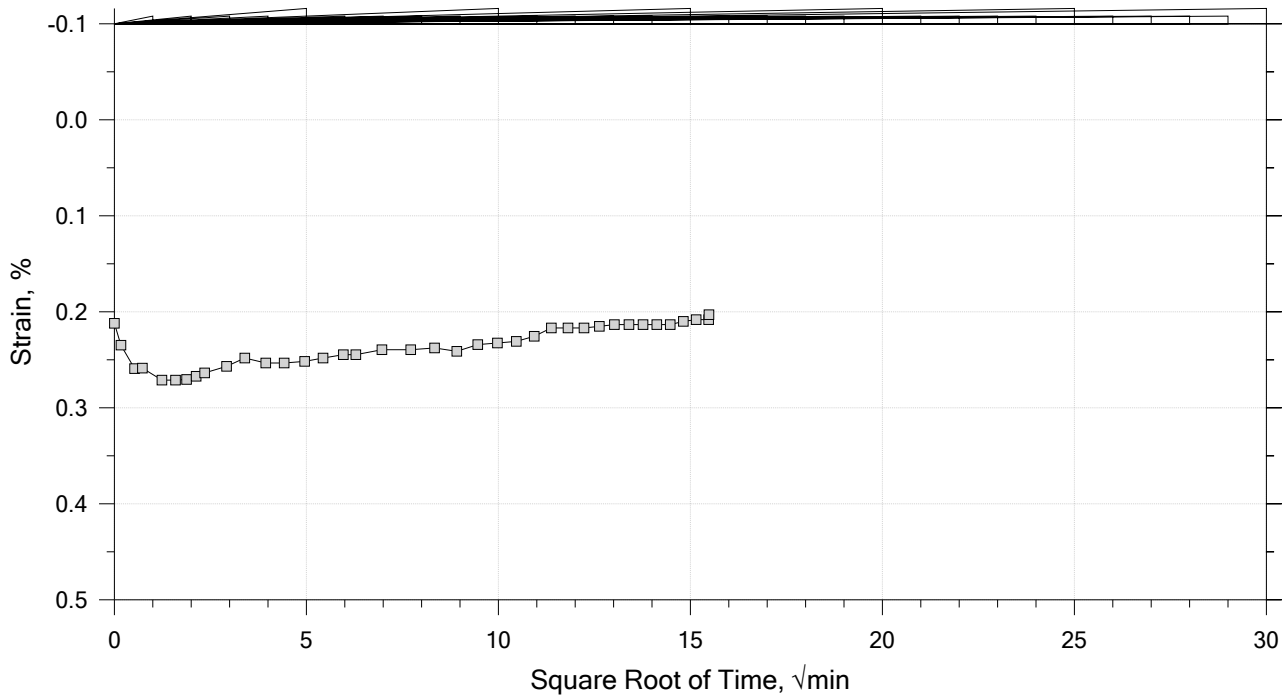


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System 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

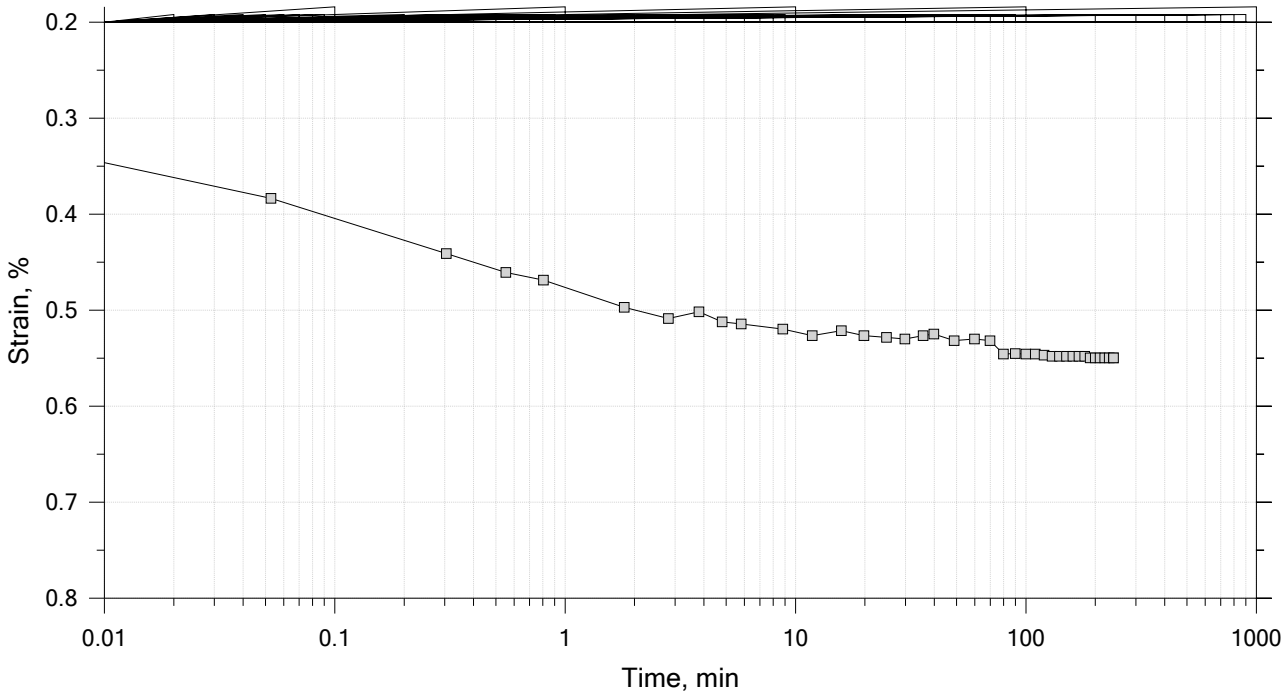


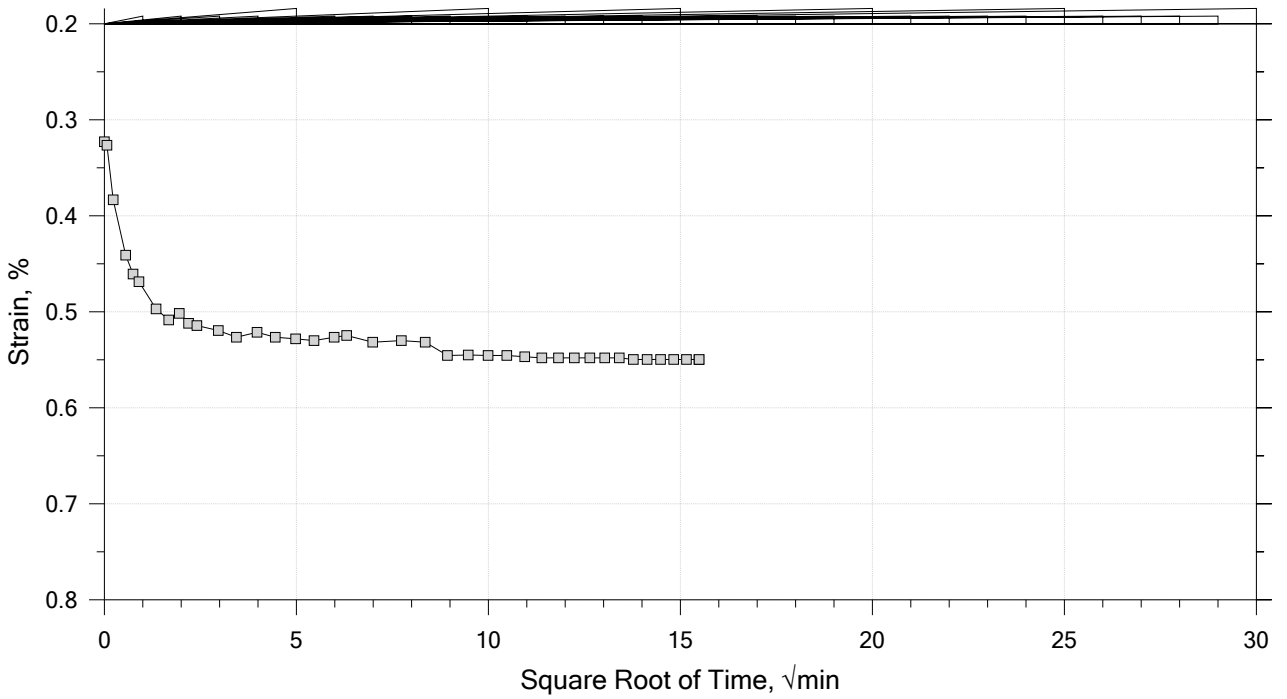


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf





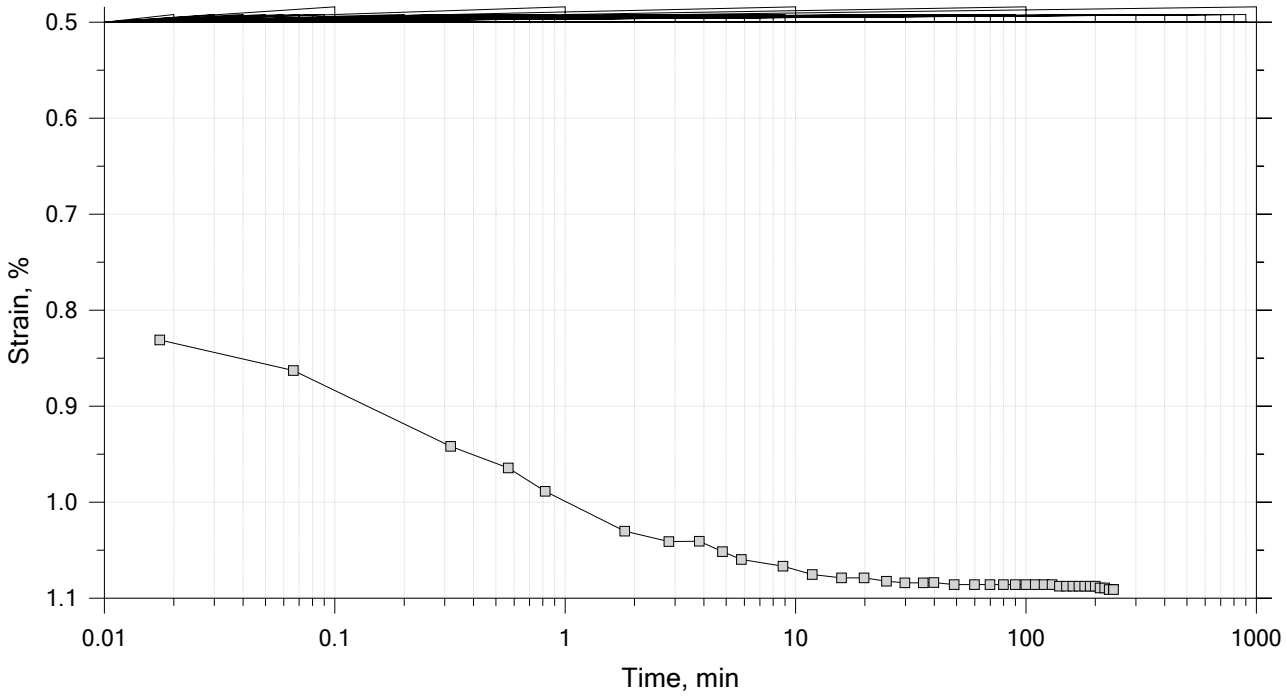
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

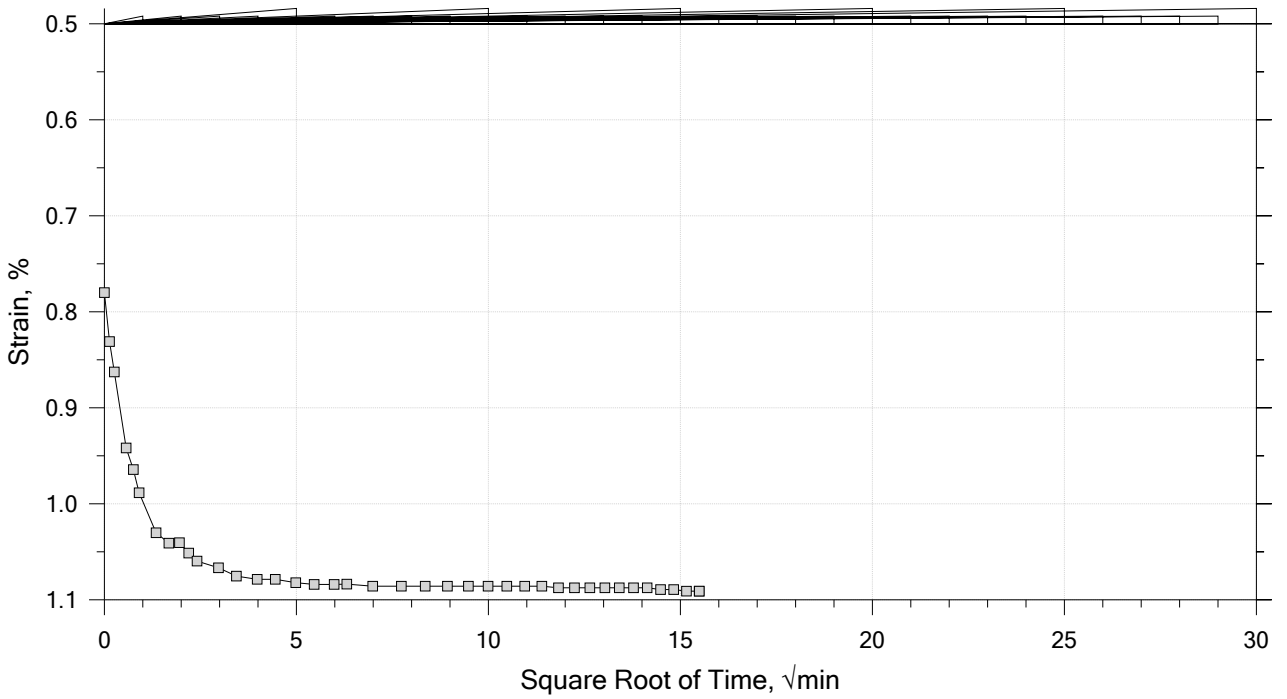
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14

Constant Load Step

Stress: 0.5 tsf





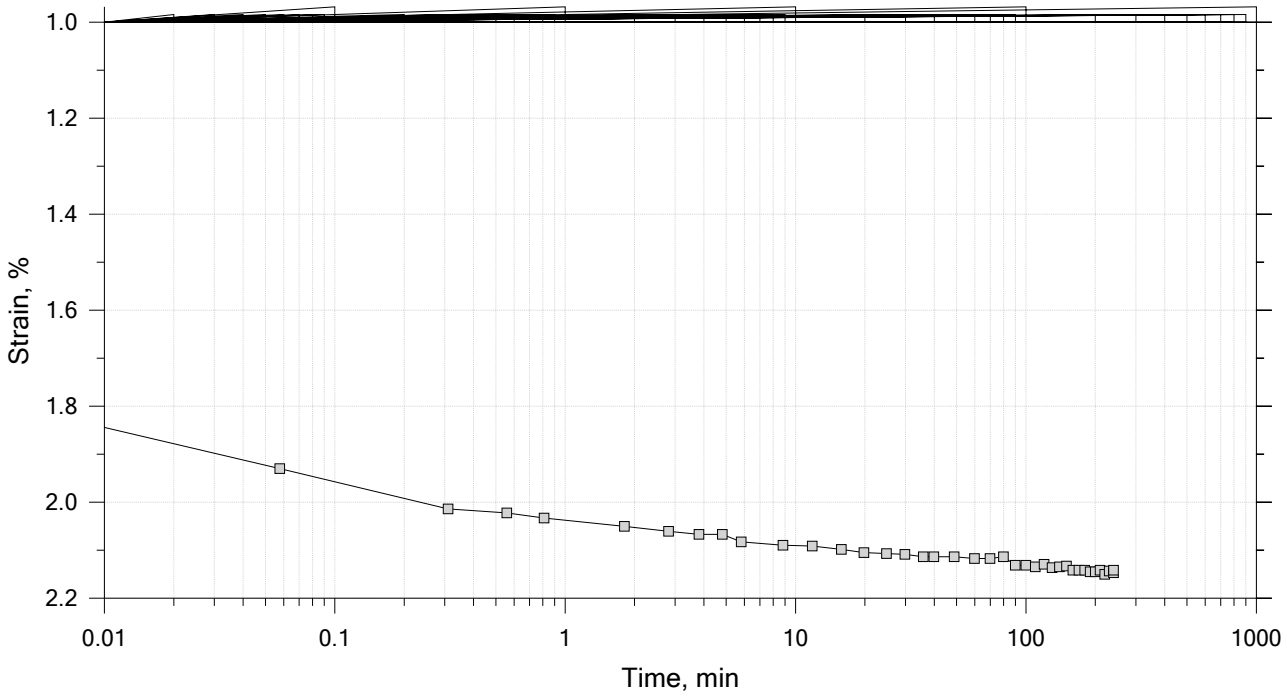
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

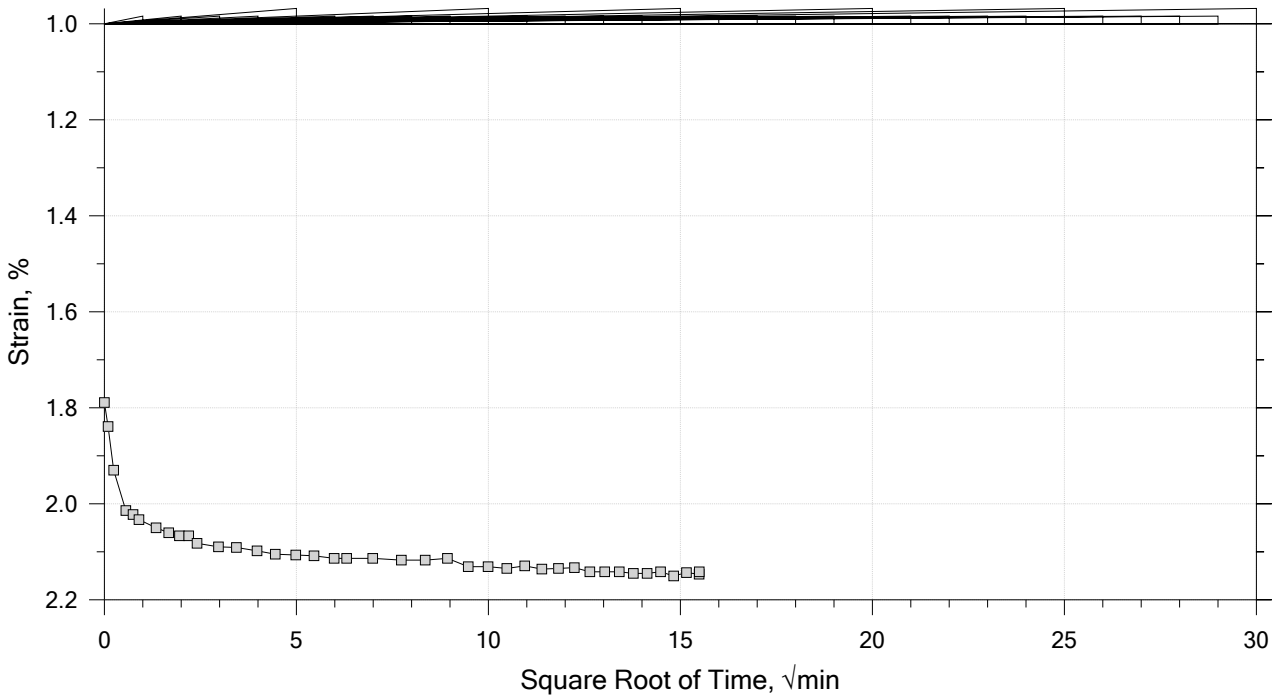
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 1 tsf

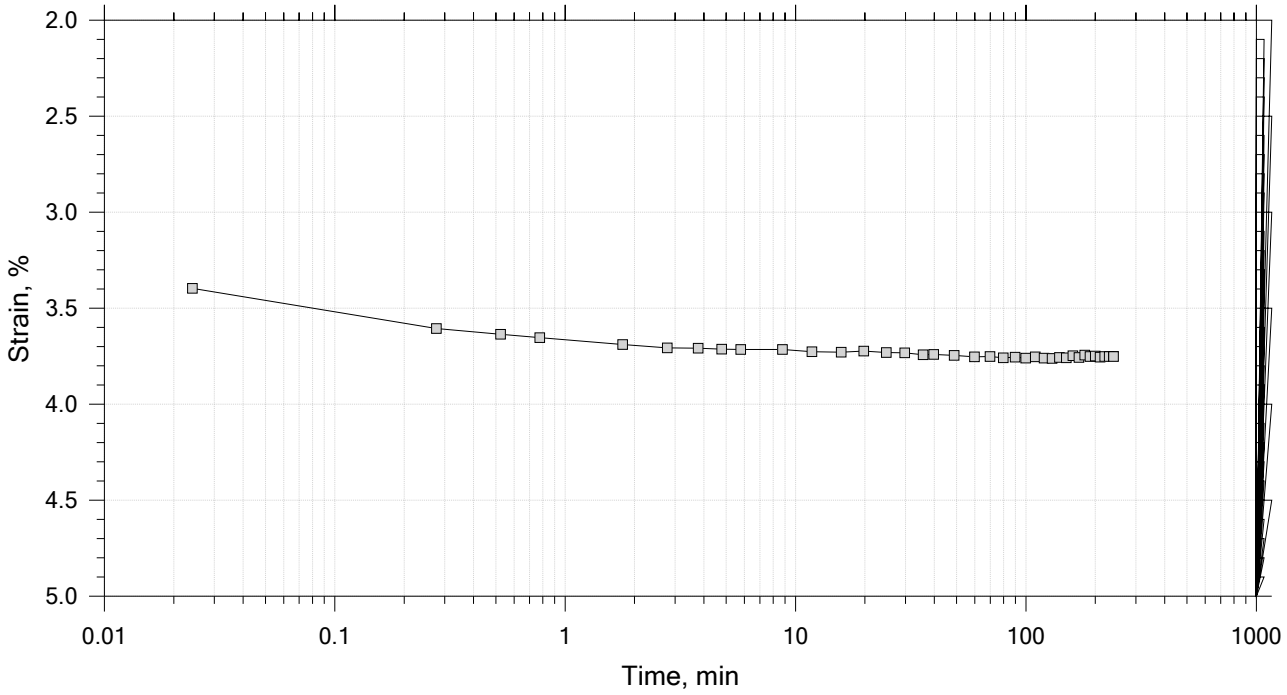


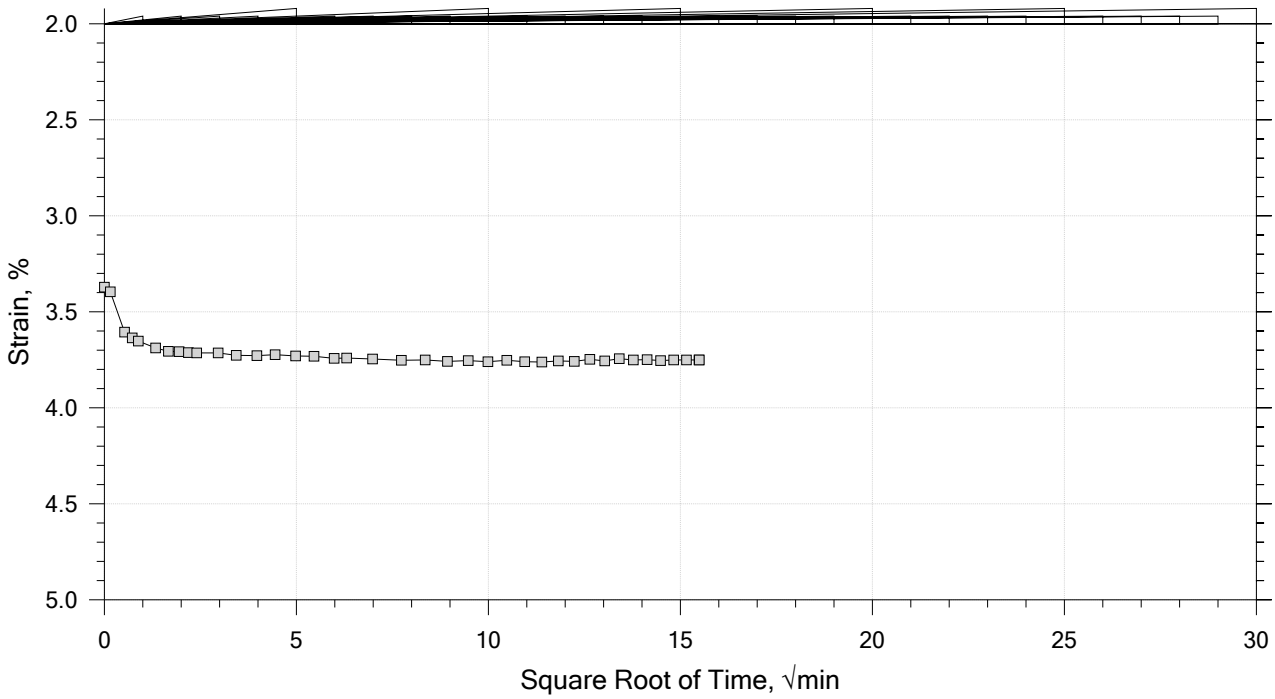


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14
Constant Load Step
Stress: 2 tsf





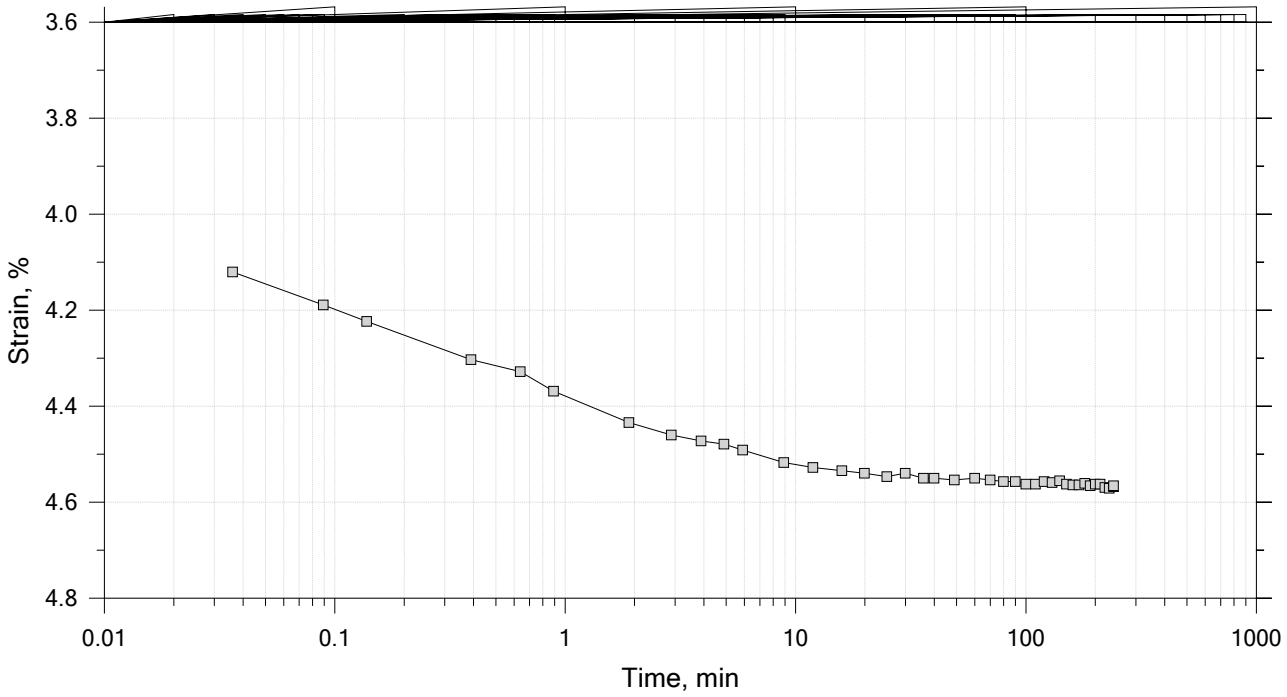
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

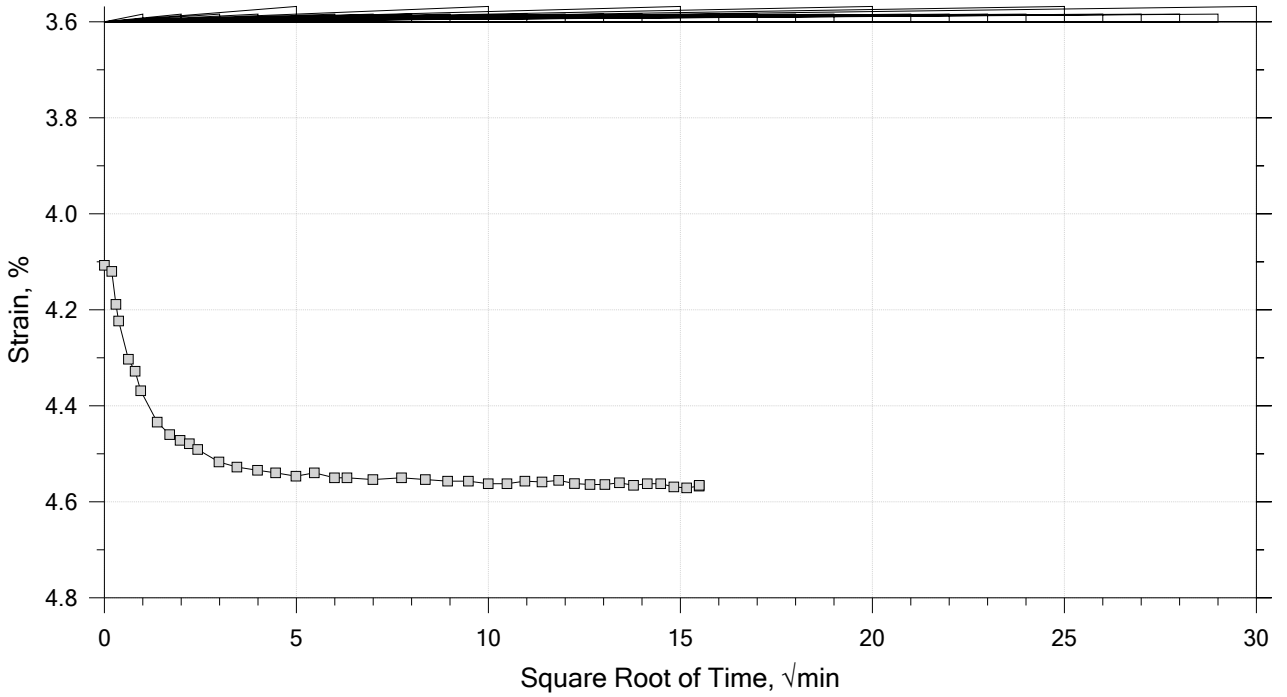
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14

Constant Load Step

Stress: 4 tsf

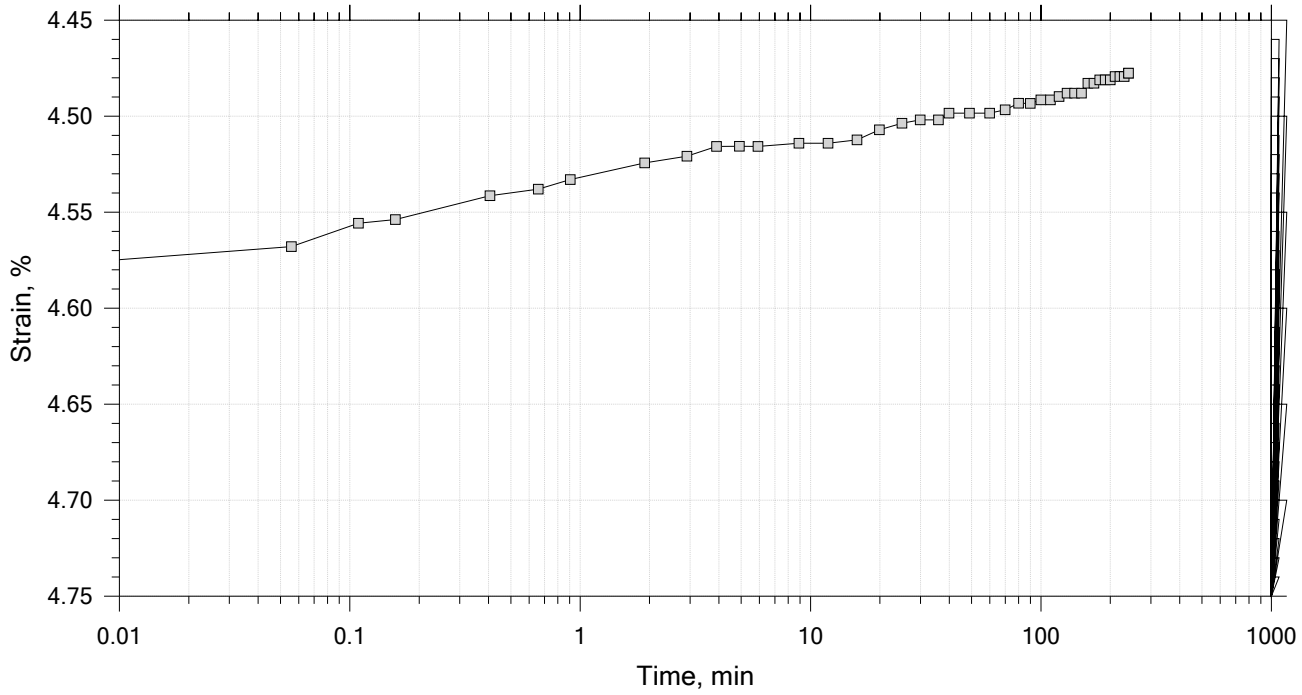


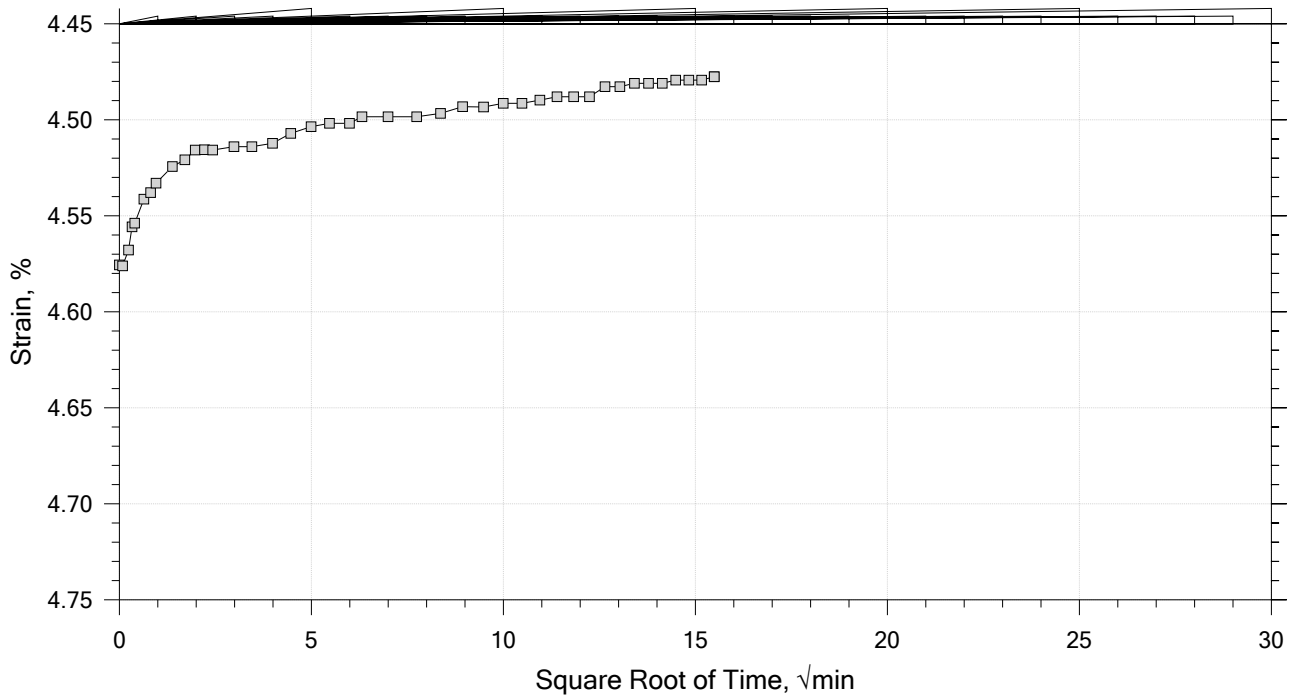


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14
Constant Load Step
Stress: 2 tsf

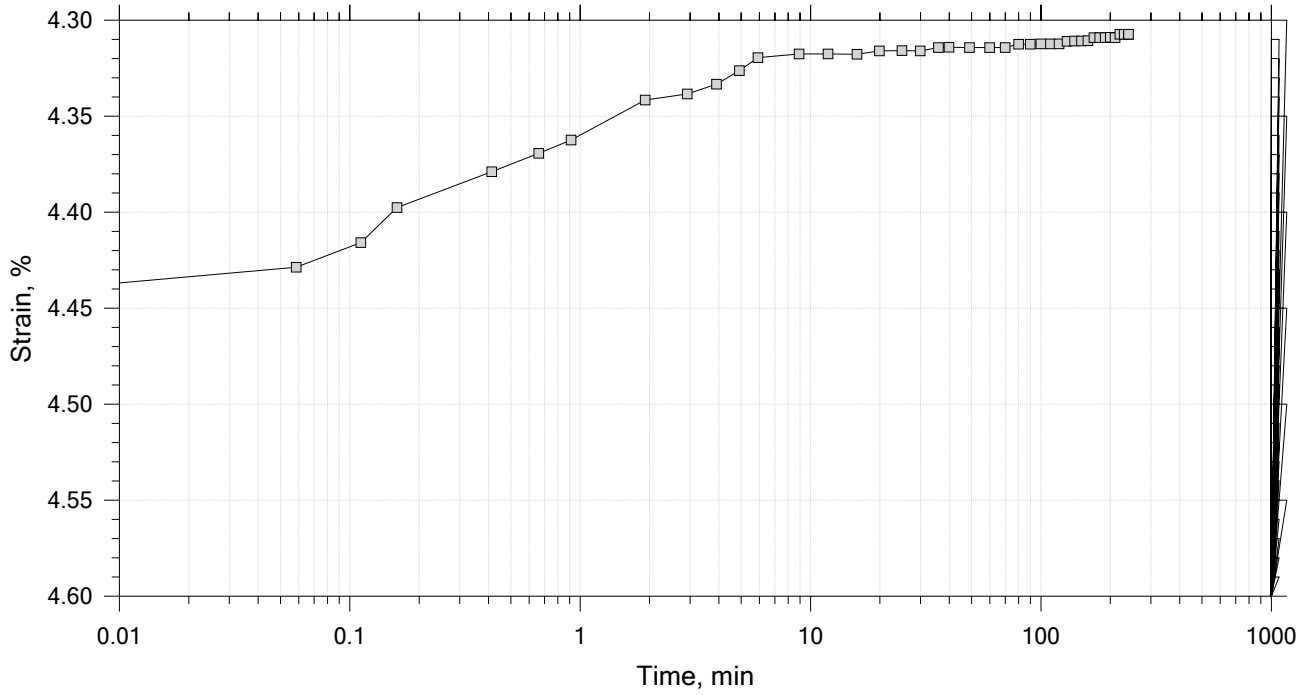


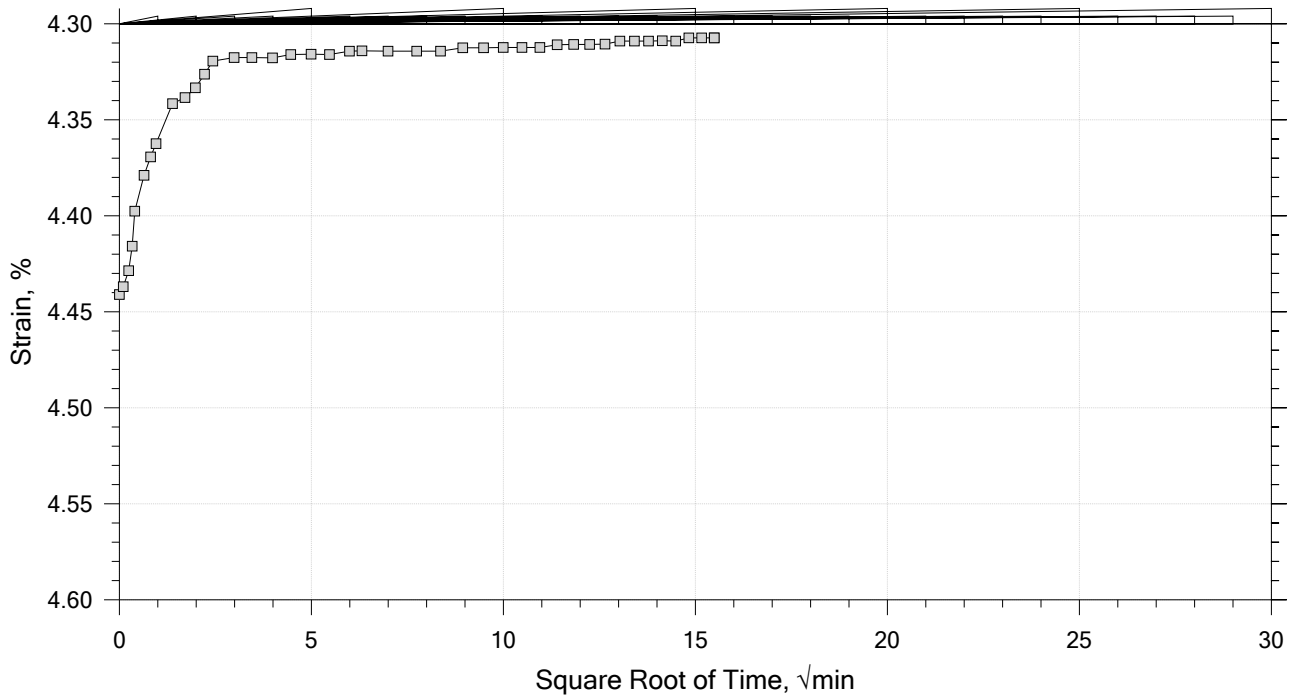


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
Test No.: IP-2	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14
Constant Load Step
Stress: 1 tsf

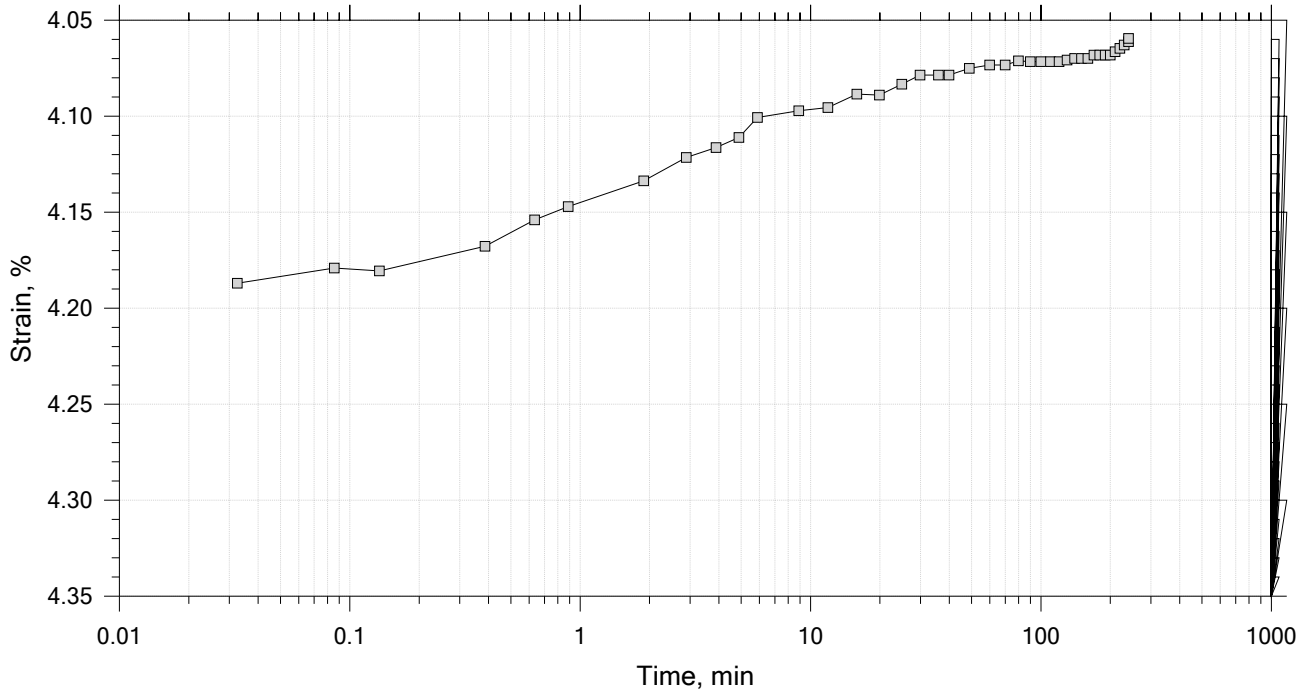


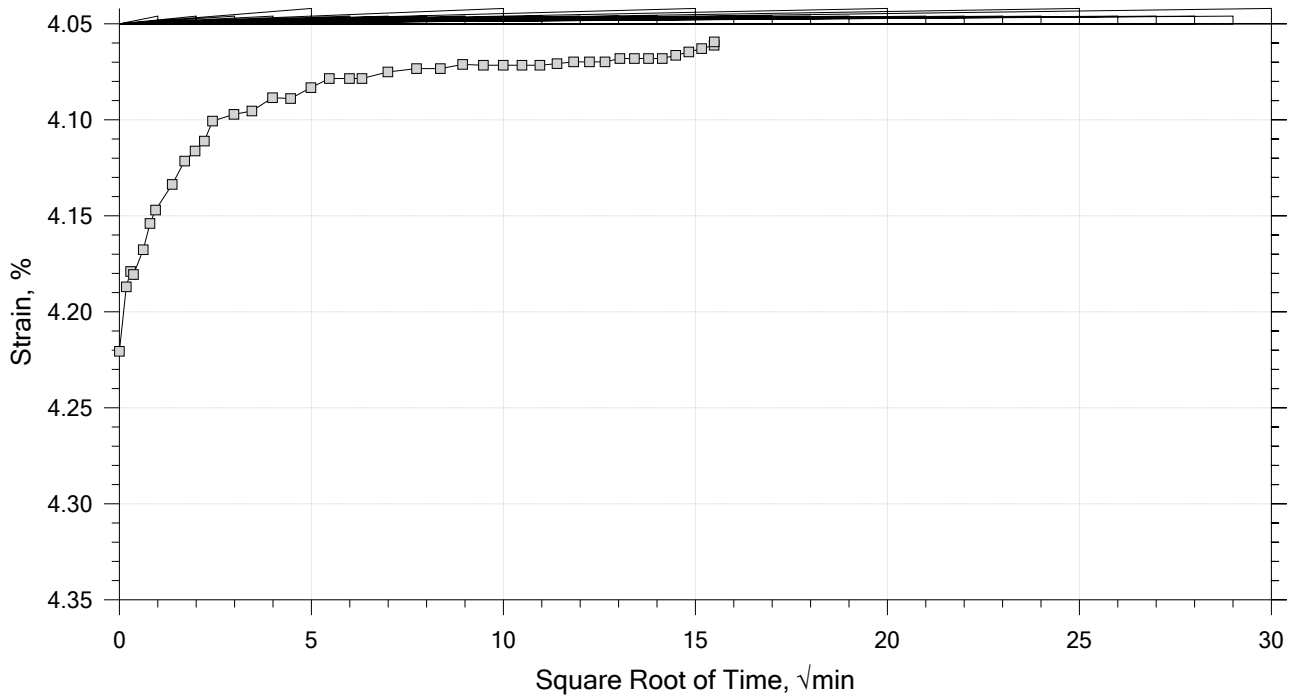


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

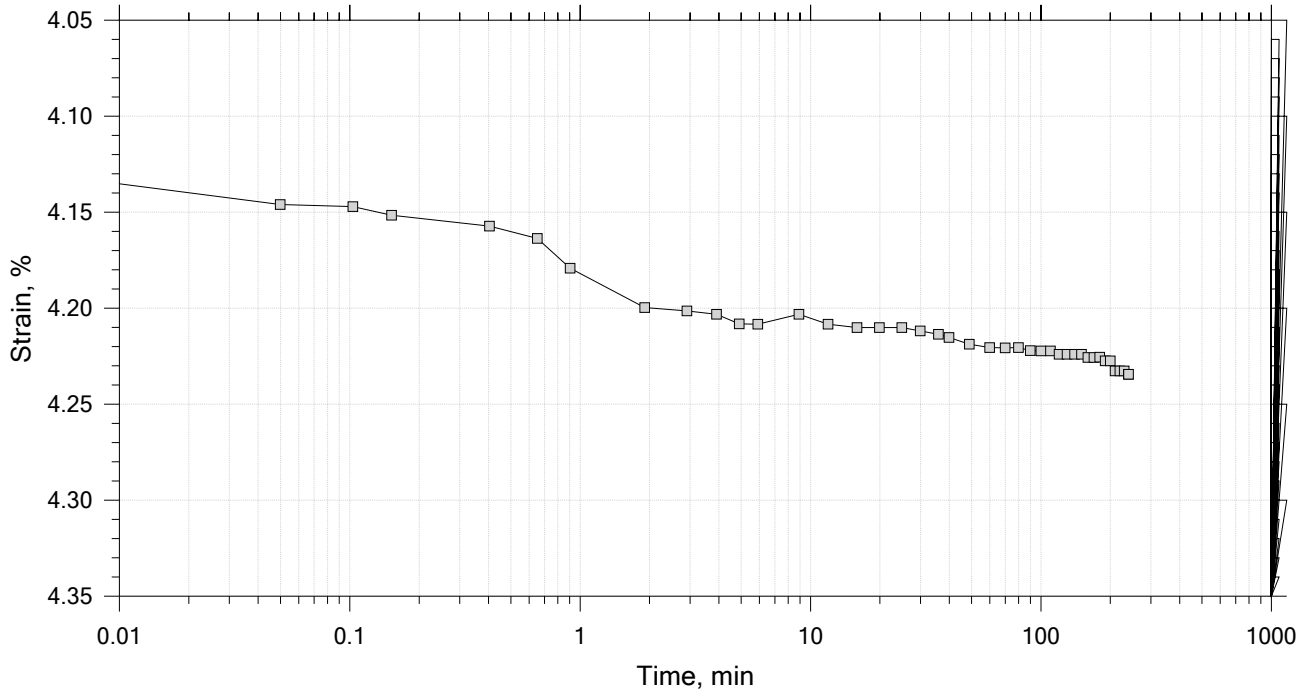


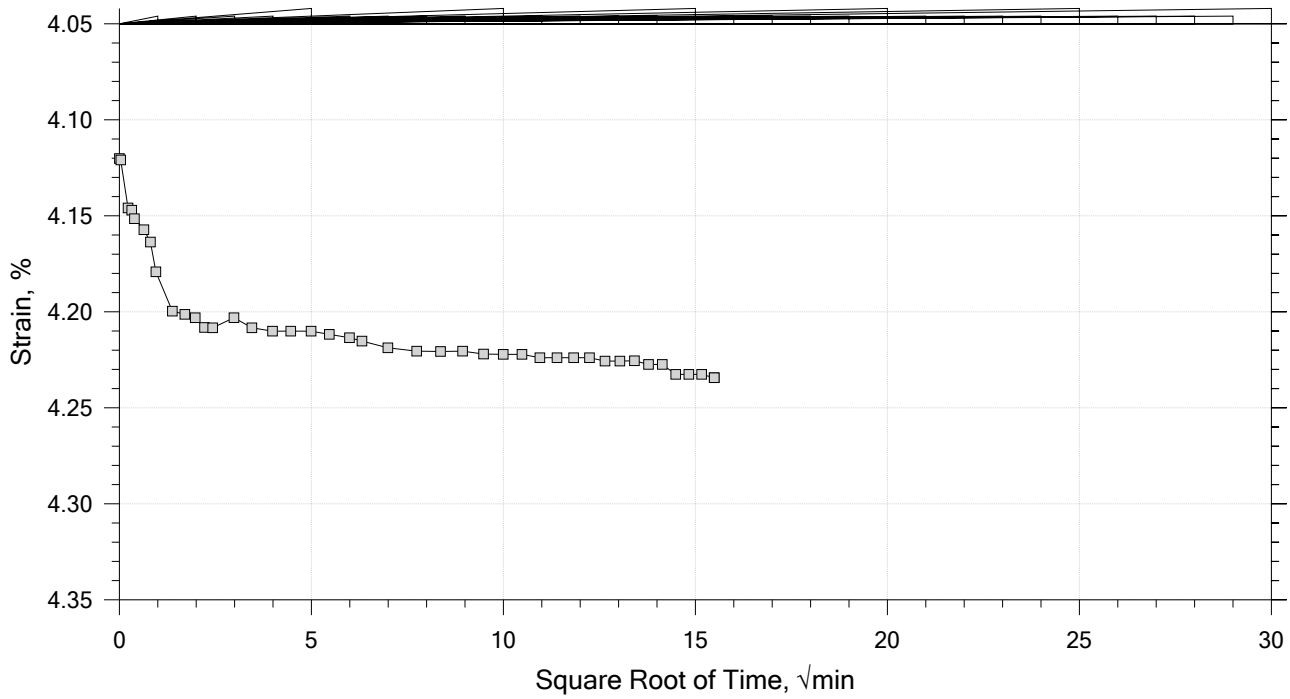


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

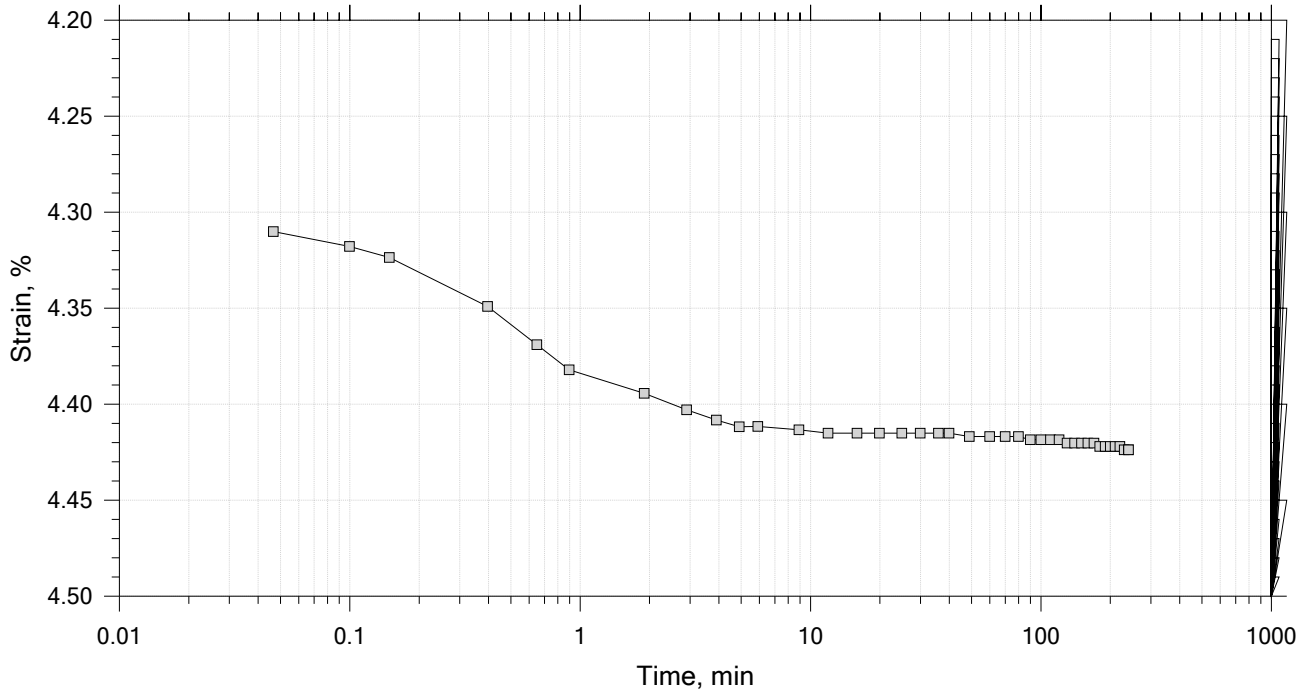


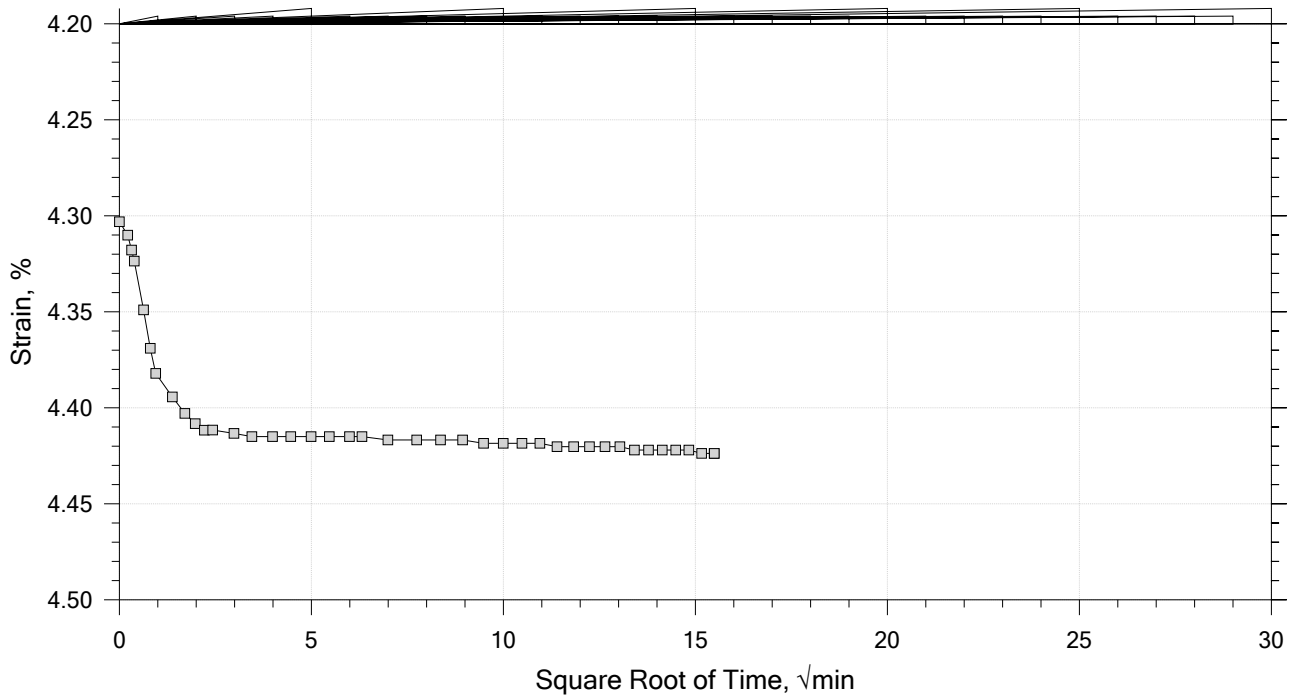


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf





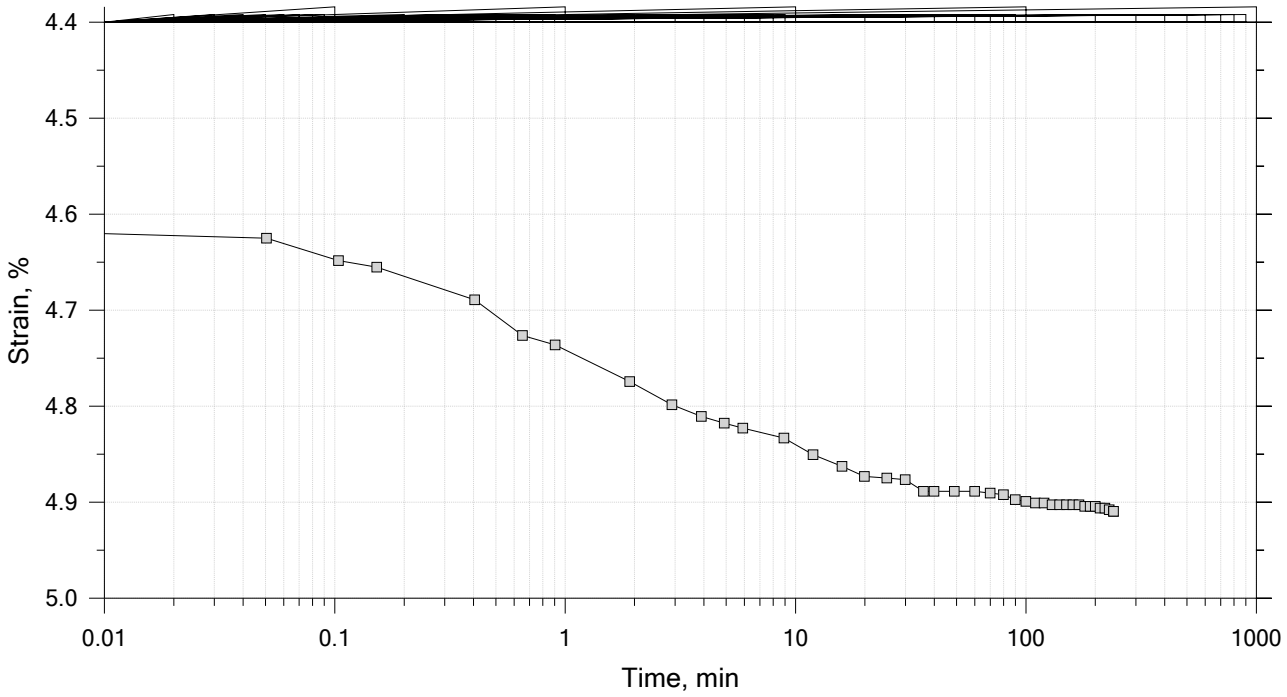
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

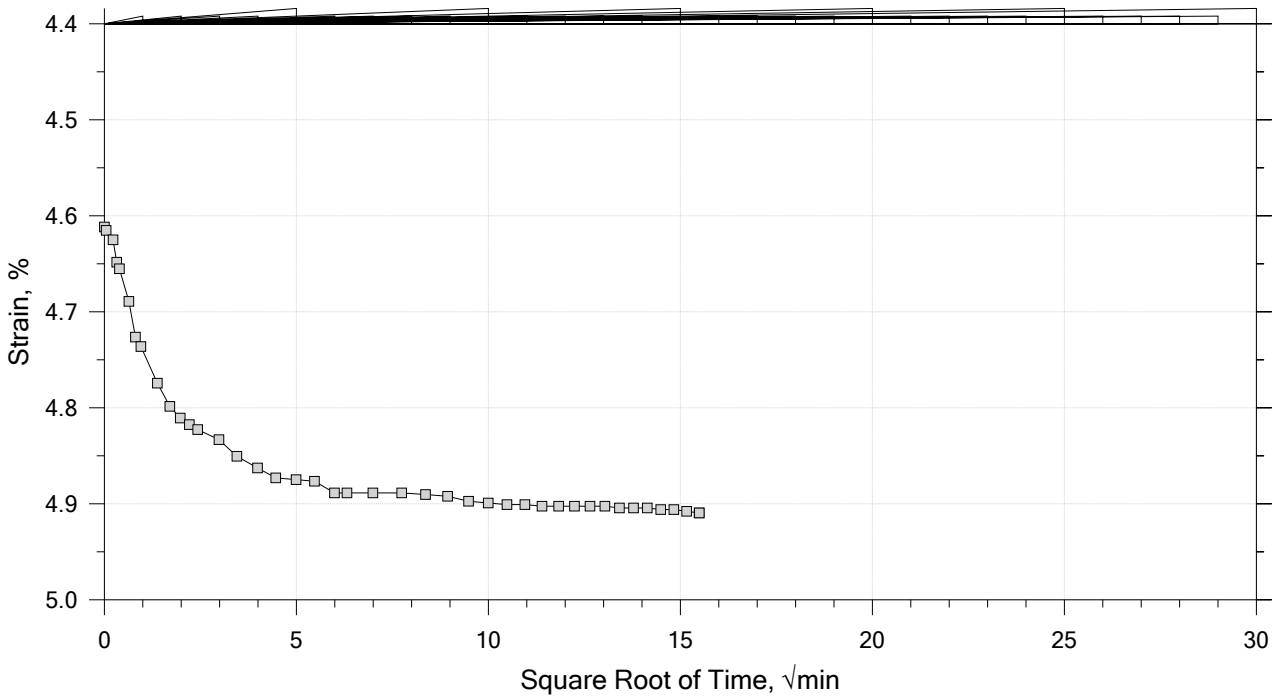
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14

Constant Load Step

Stress: 4 tsf





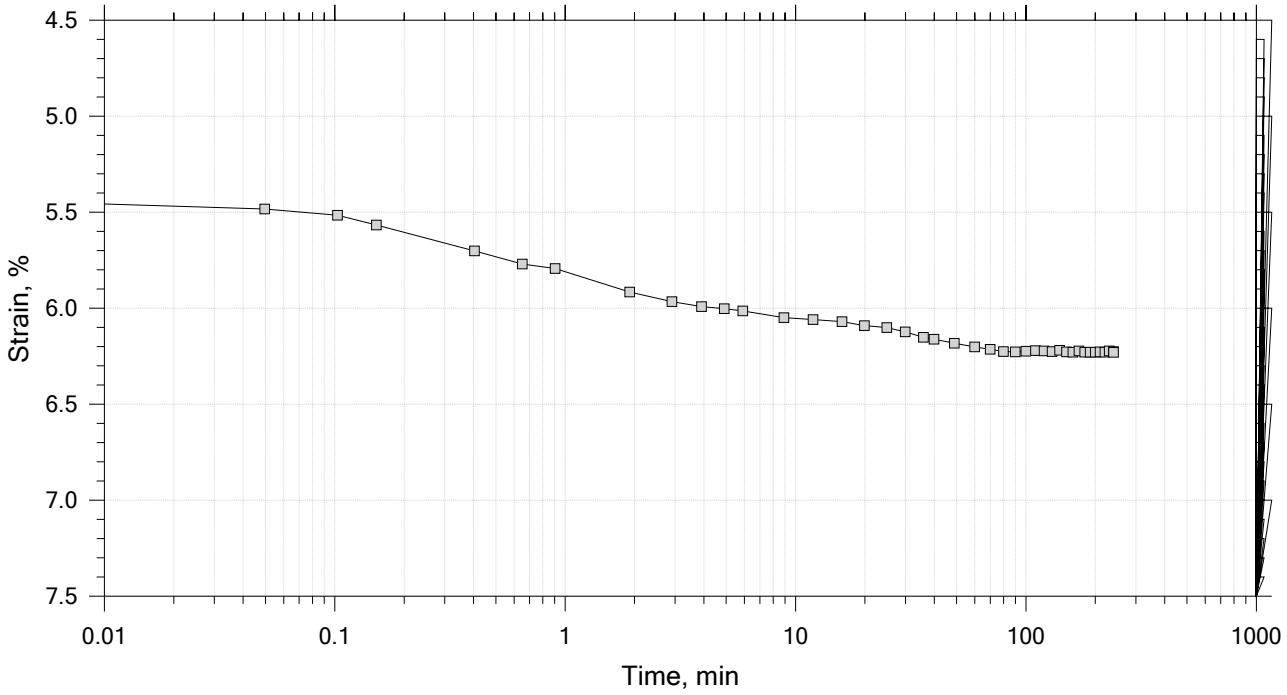
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-F		

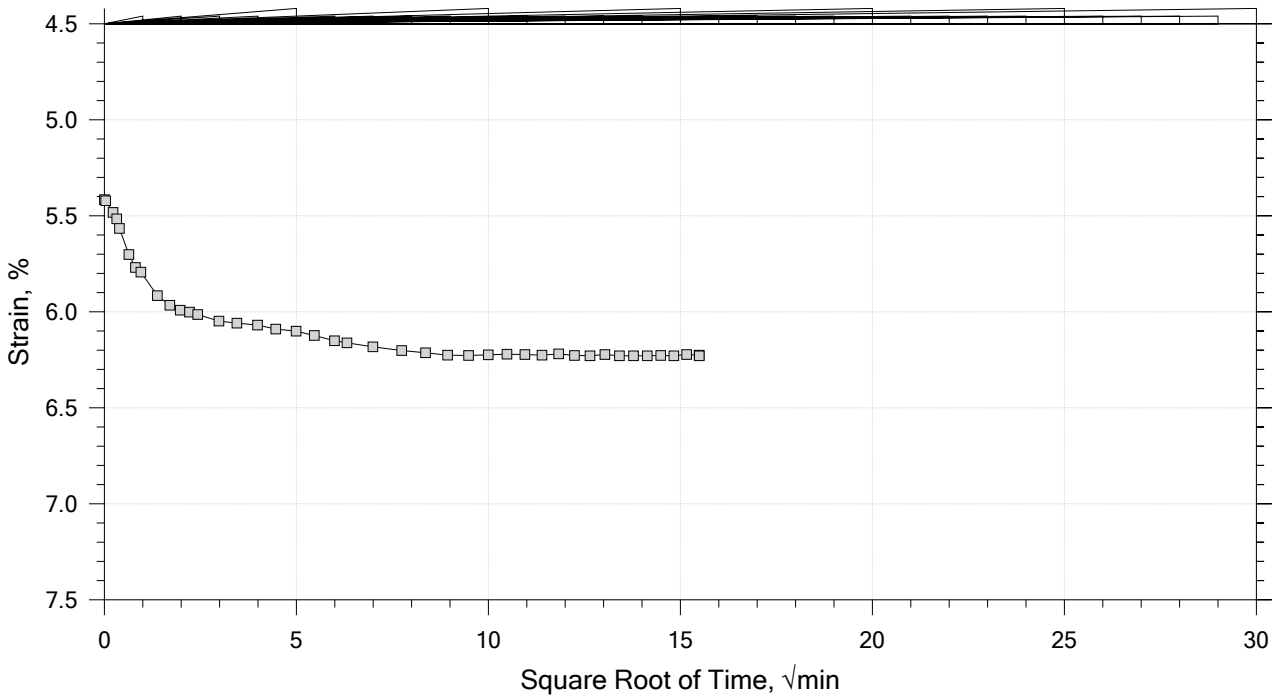
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 8 tsf

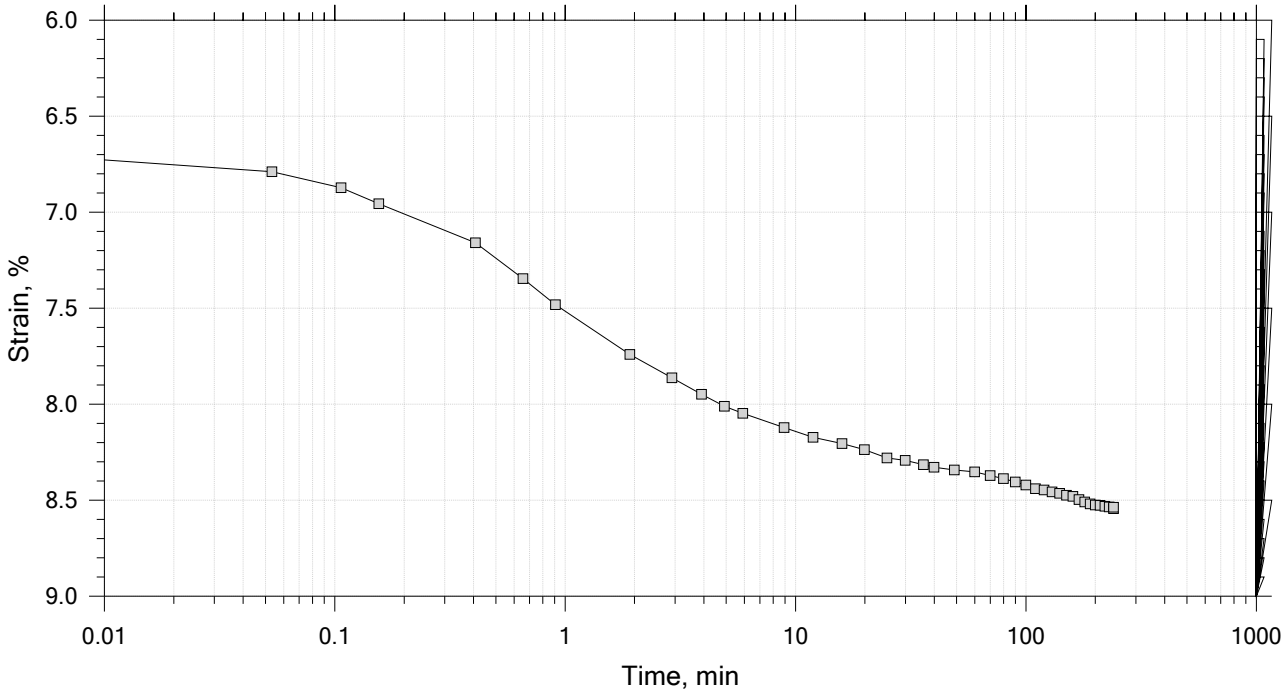


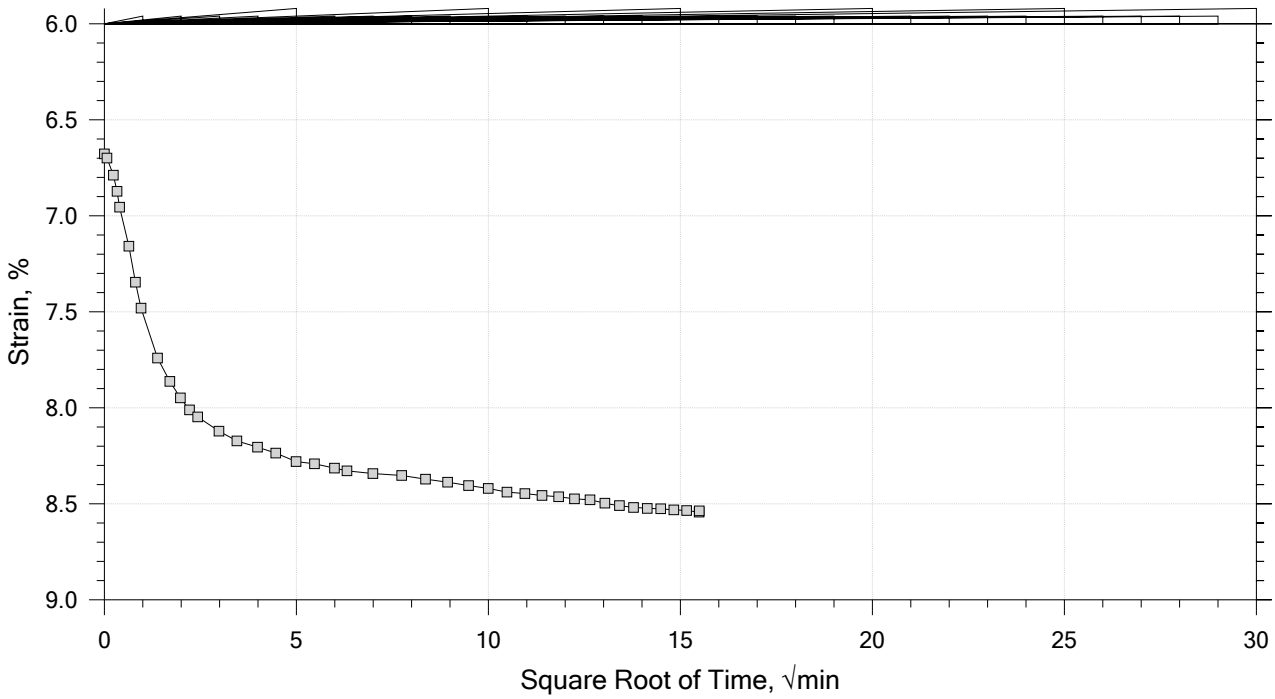


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-102	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 10.5-12.5 ft
	Test No.: IP-2	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System 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

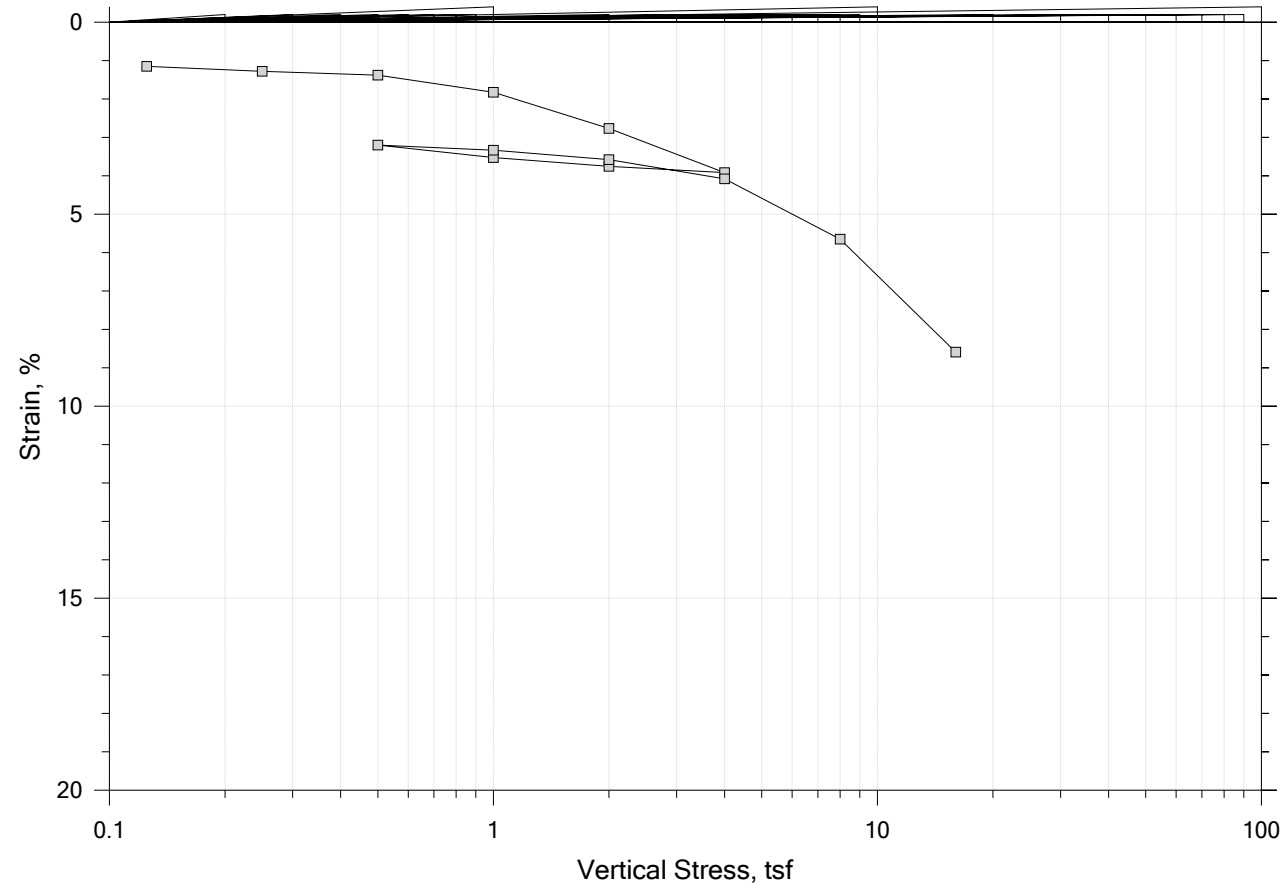
Square Root of Time Coefficients

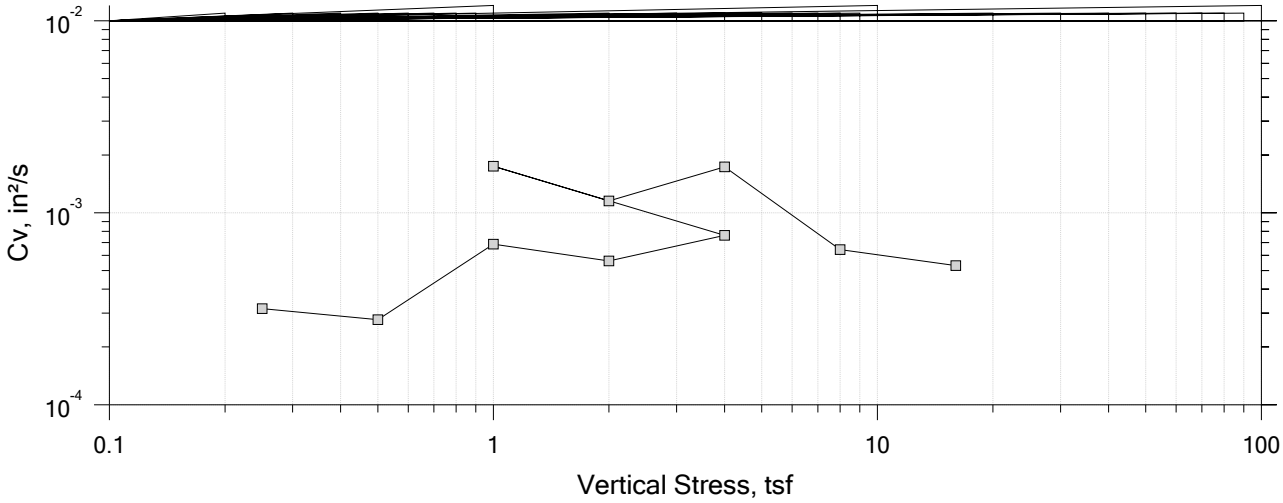
Step	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt. T90 min	Cv in²/s	Mv 1/tsf	k ft/day
1	0.125	0.002030	0.758	0.203	---	---	1.62e-02	---
2	0.250	0.005497	0.752	0.550	1.789	1.96e-03	2.77e-02	1.02e-03
3	0.500	0.01091	0.743	1.09	2.810	1.24e-03	2.17e-02	5.02e-04
4	1.00	0.02142	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

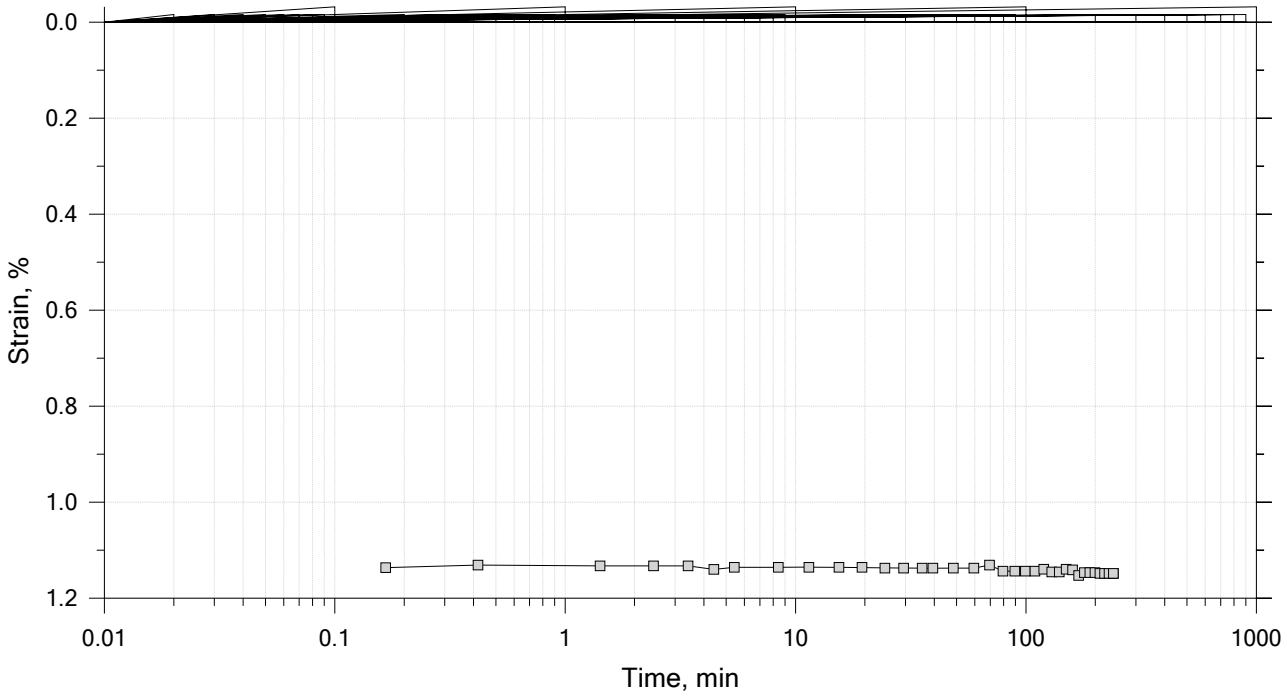


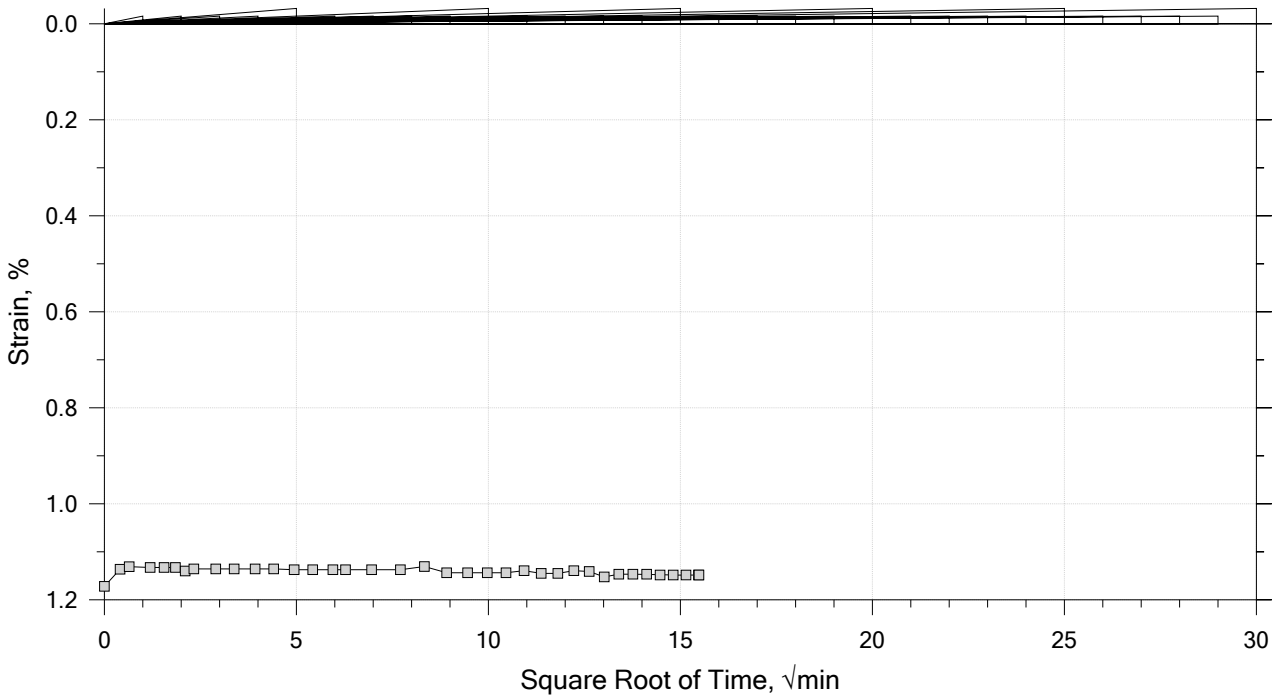


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		
	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

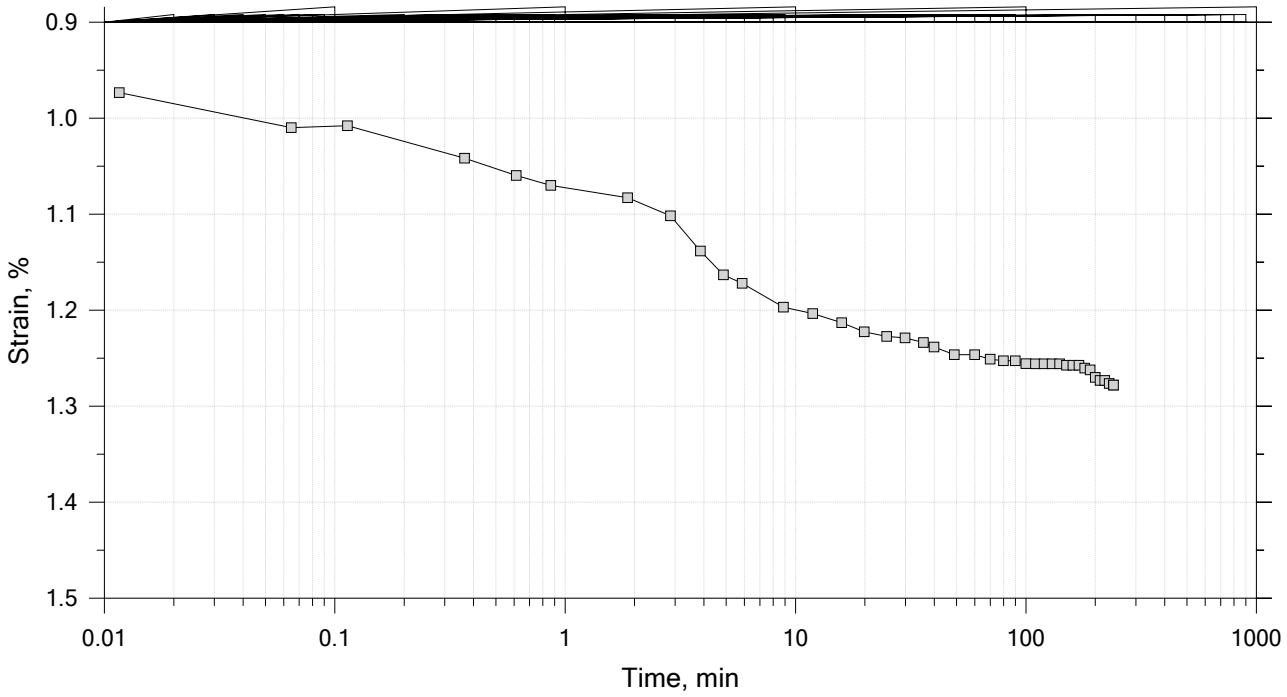


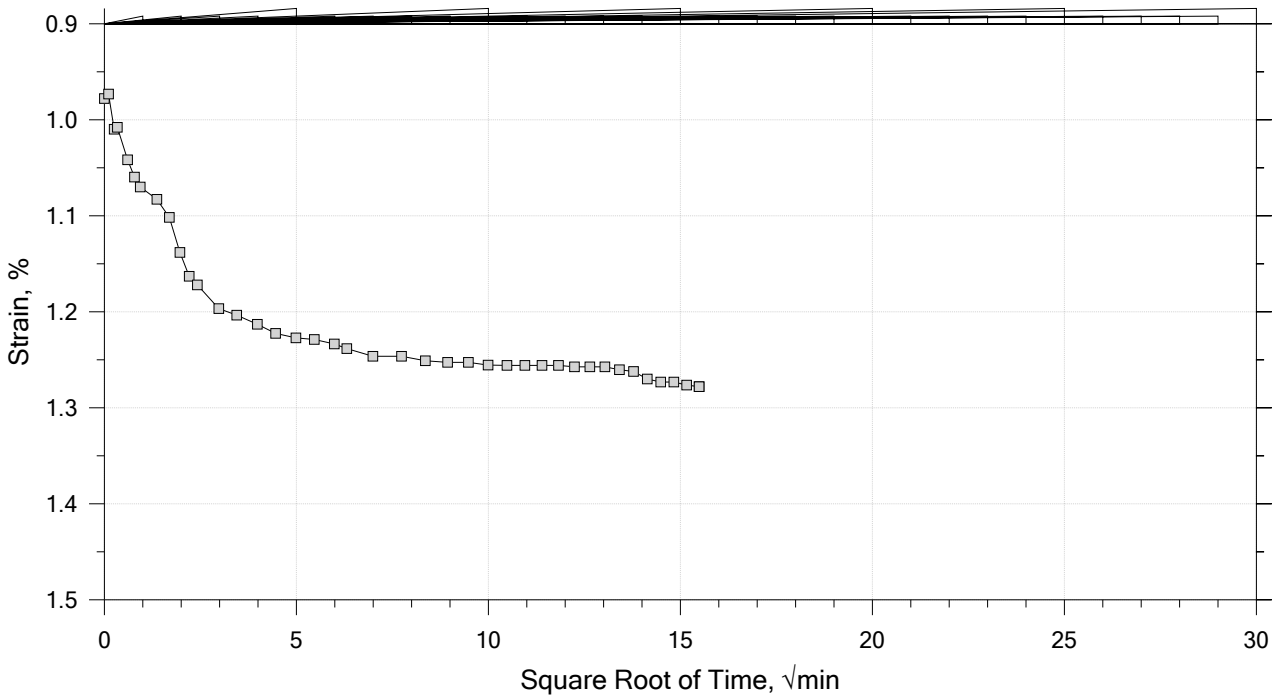


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 2 of 14
Constant Load Step
Stress: 0.25 tsf

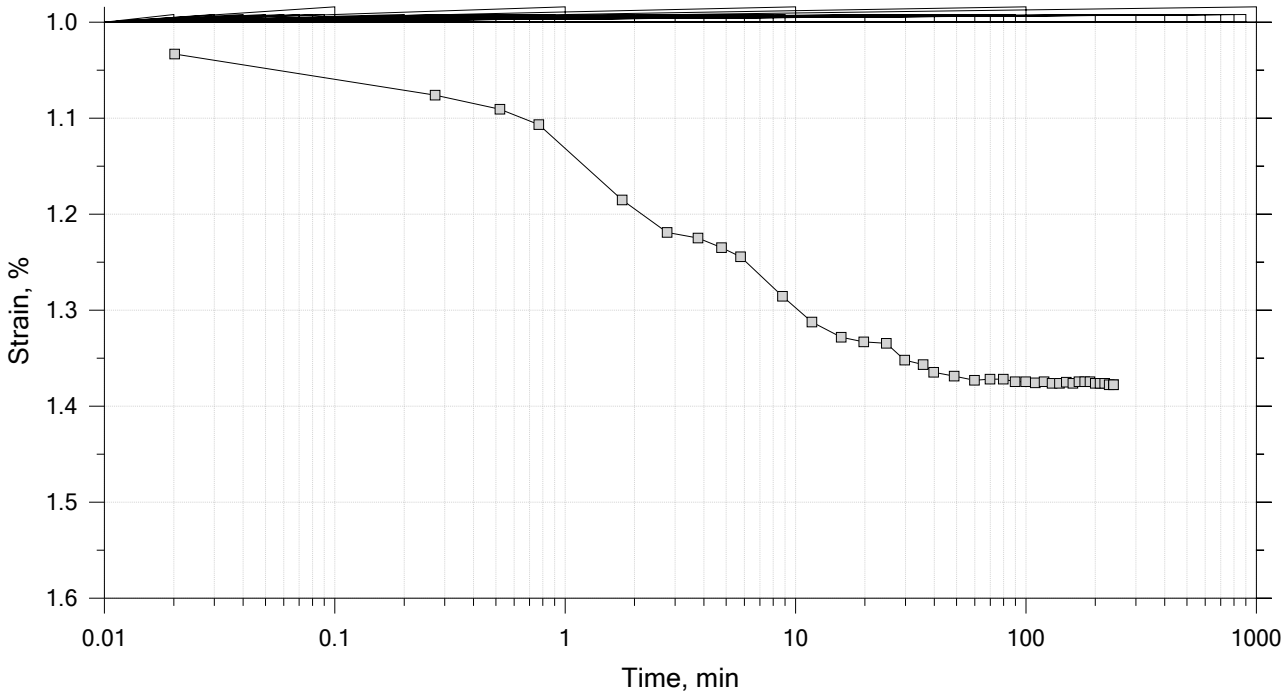


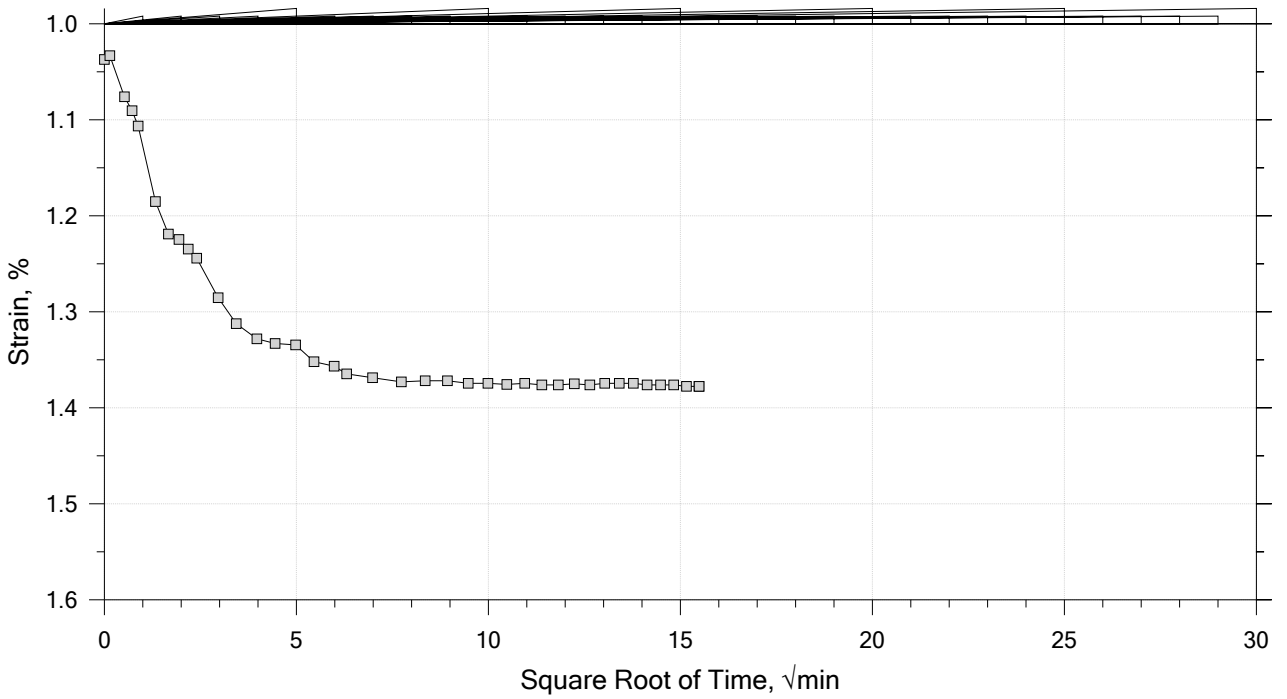


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 3 of 14
Constant Load Step
Stress: 0.5 tsf





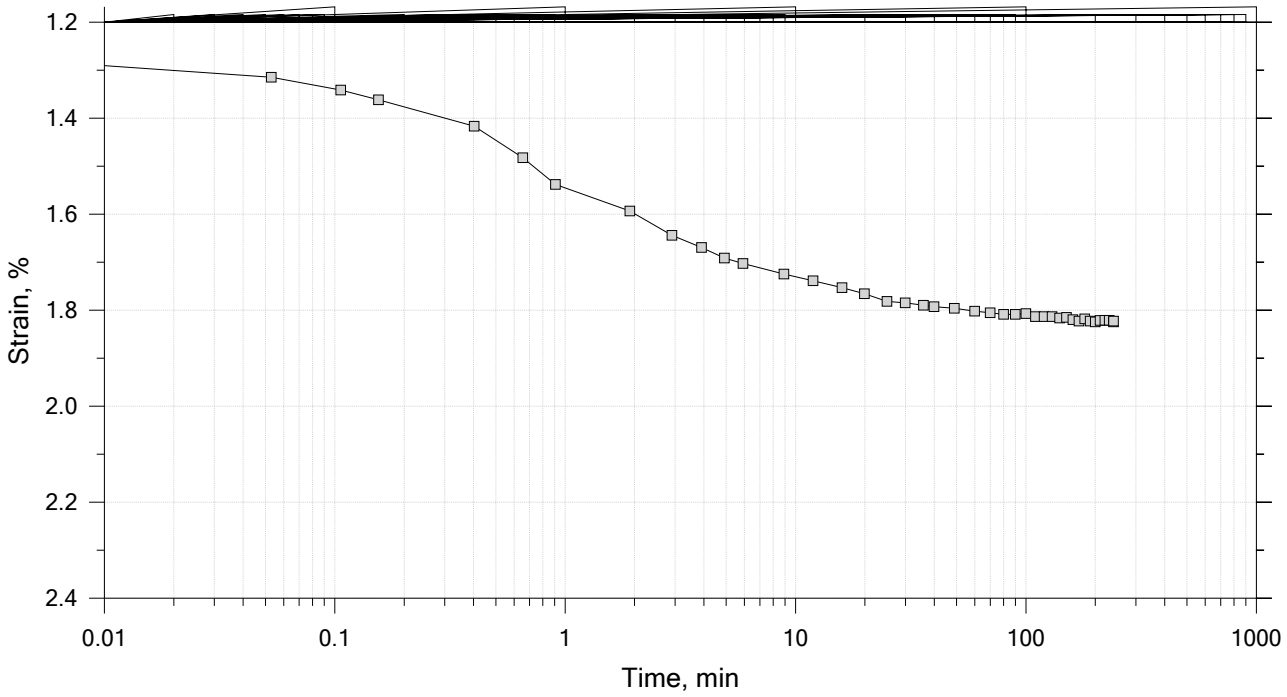
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

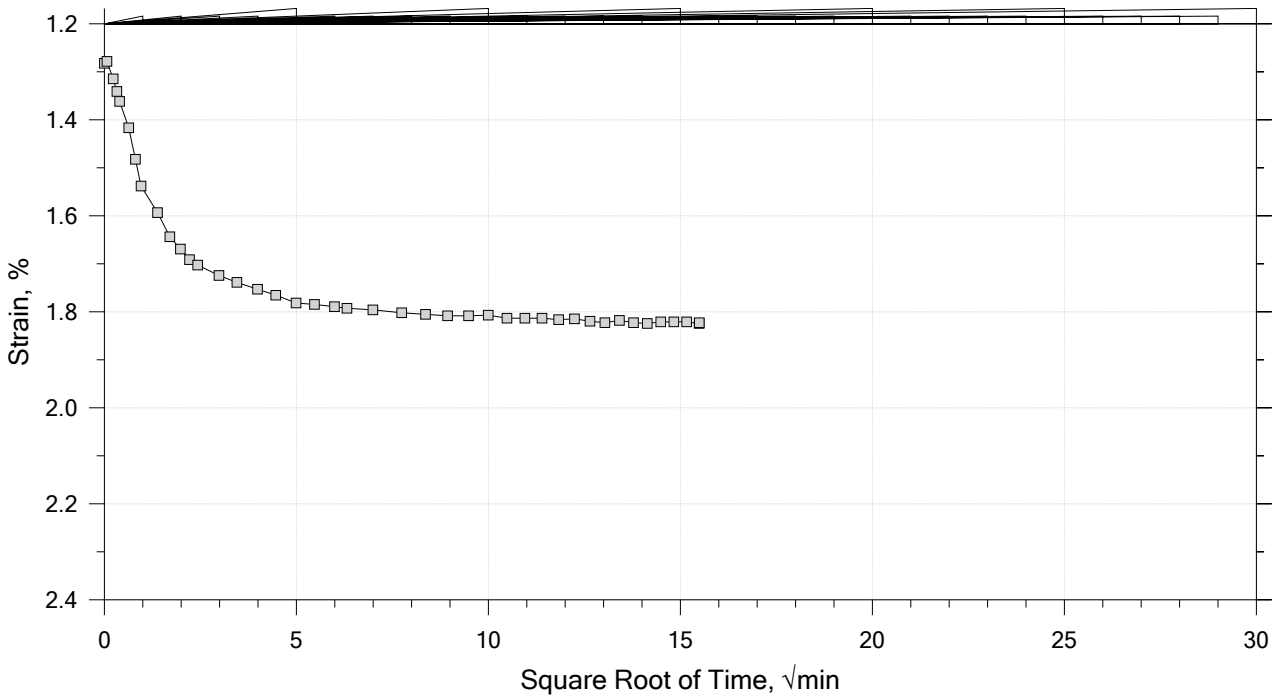
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 4 of 14

Constant Load Step

Stress: 1 tsf

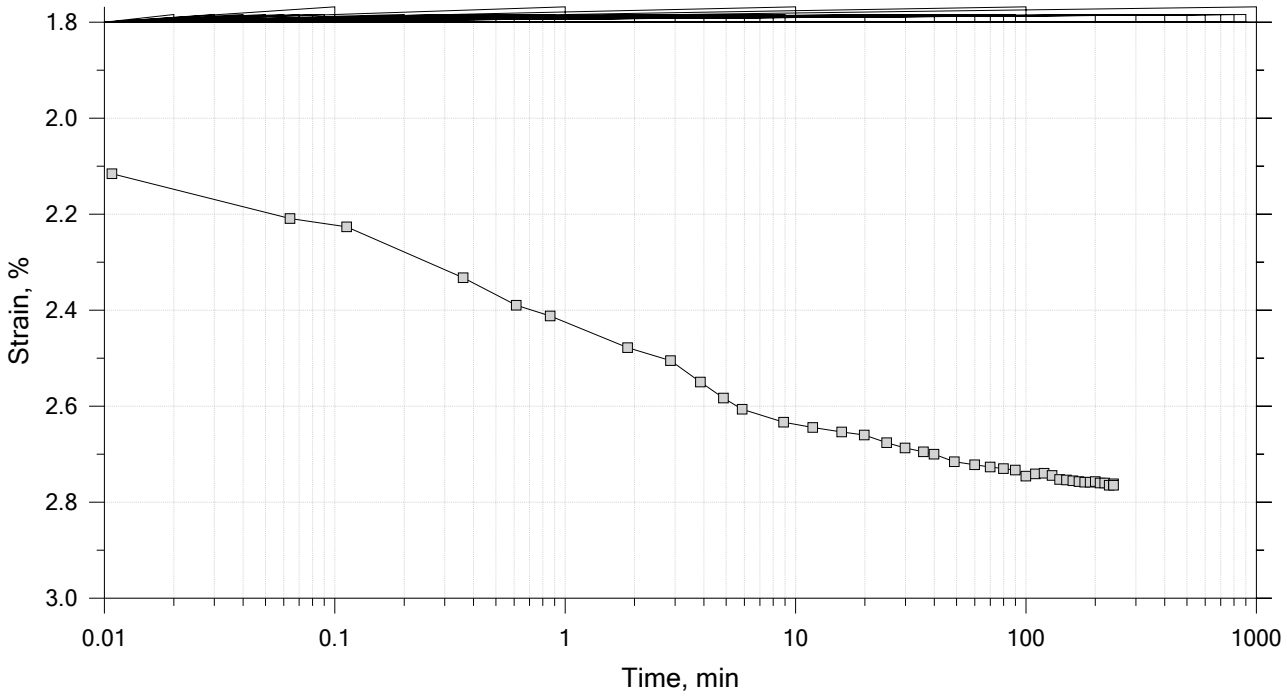


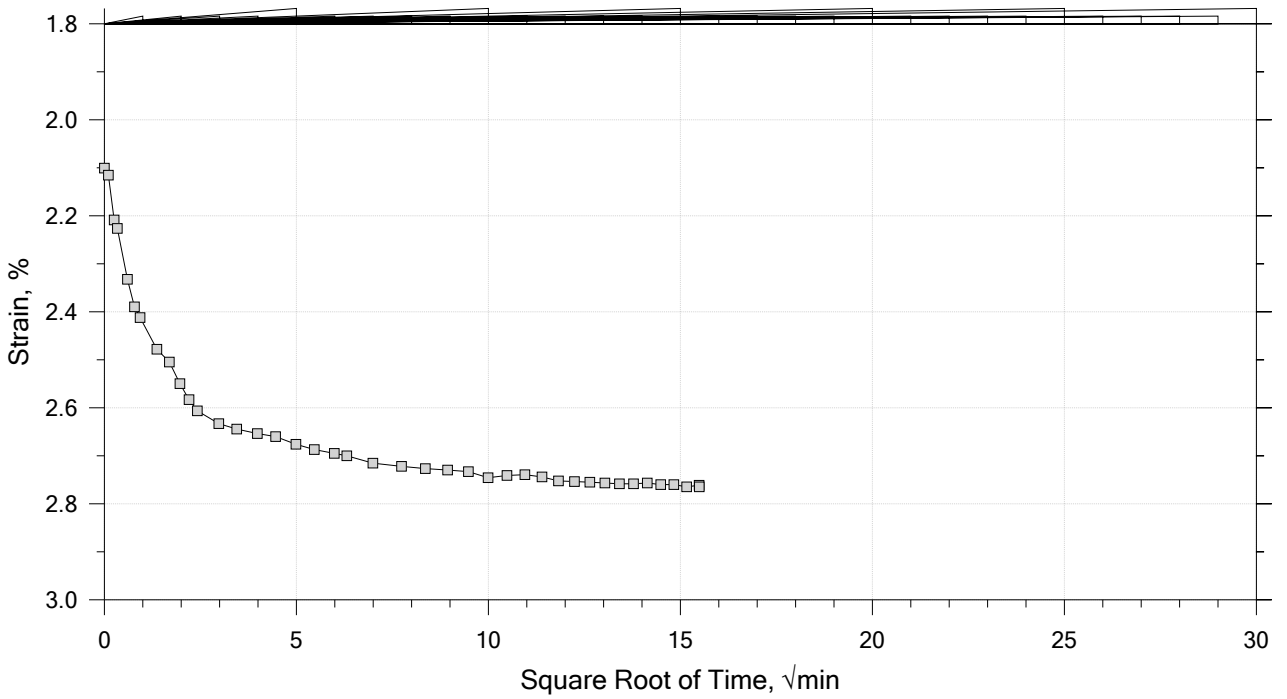


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 5 of 14
Constant Load Step
Stress: 2 tsf

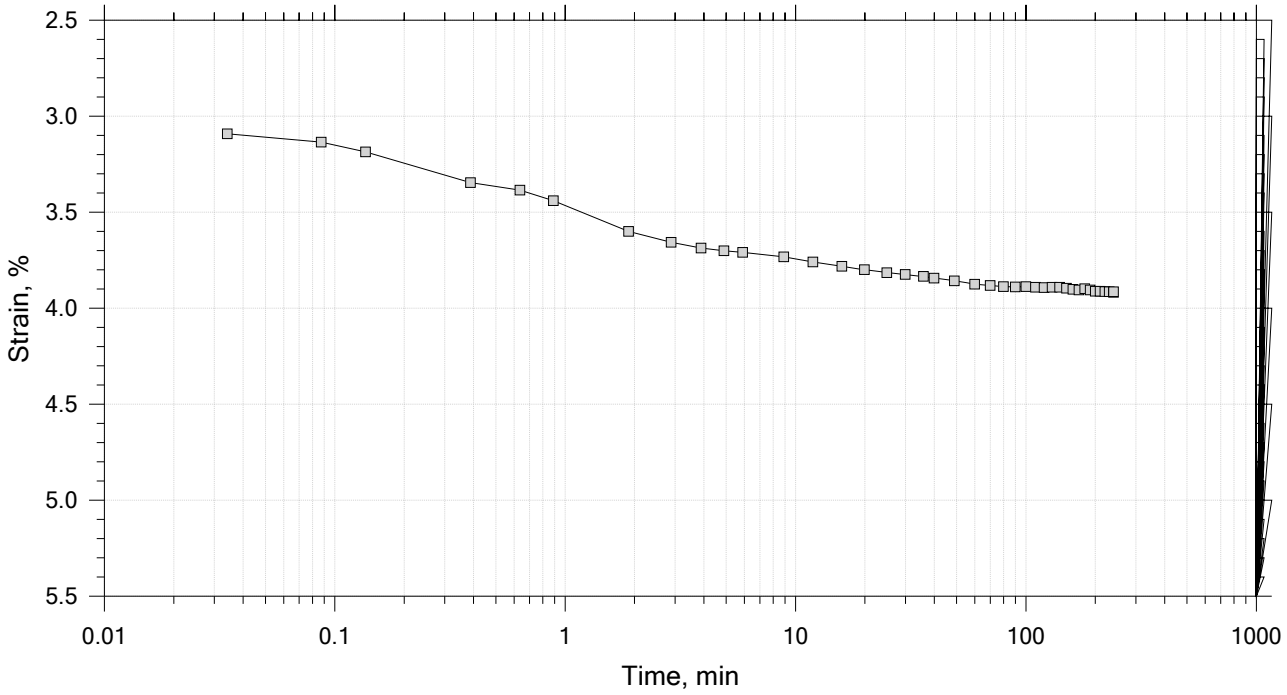


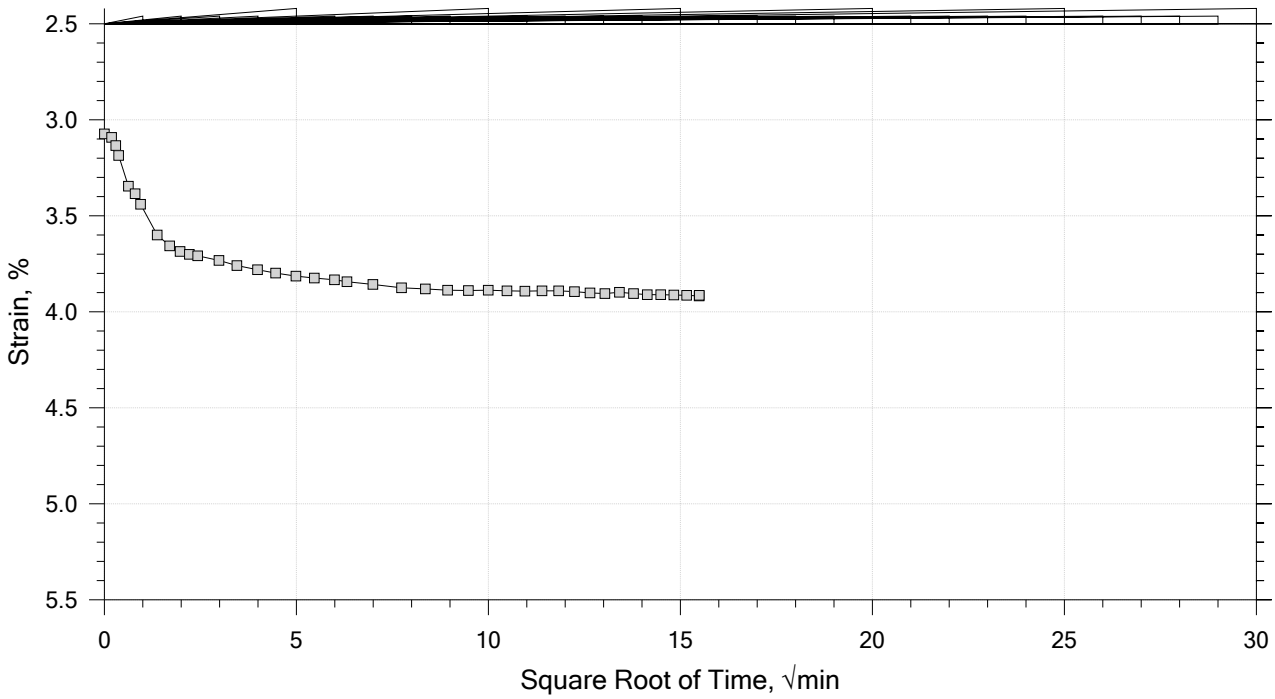


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 6 of 14
Constant Load Step
Stress: 4 tsf

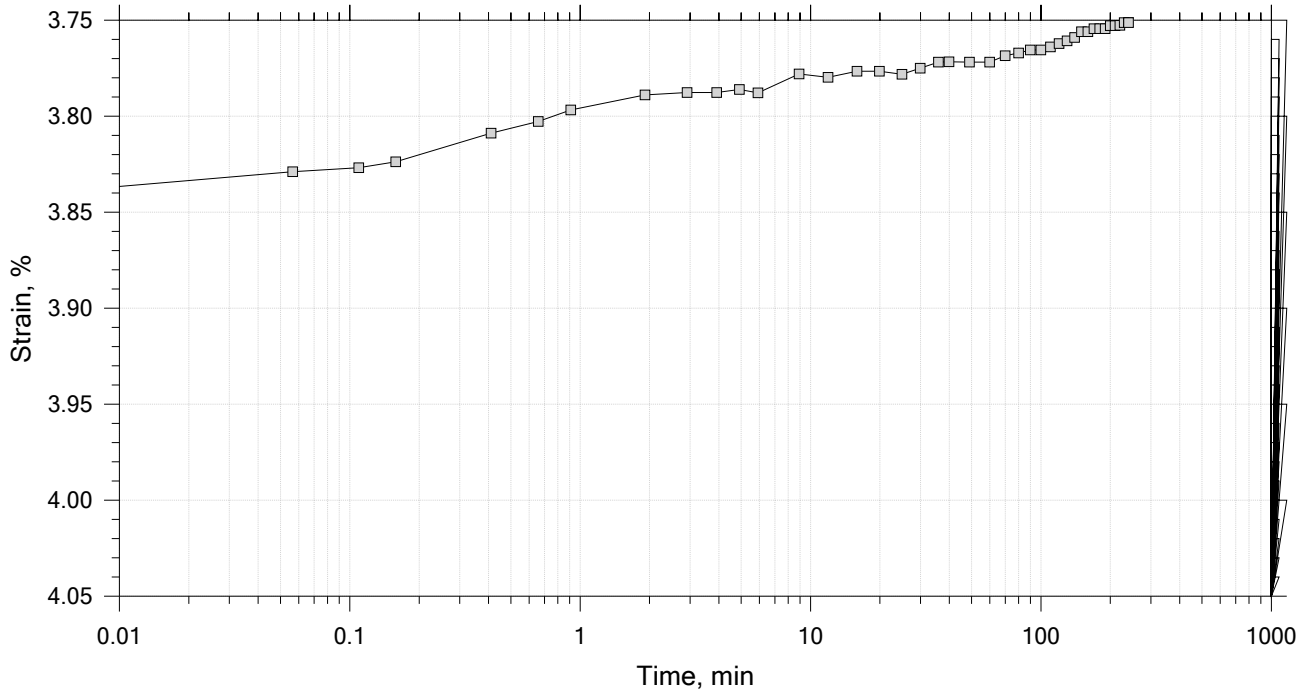


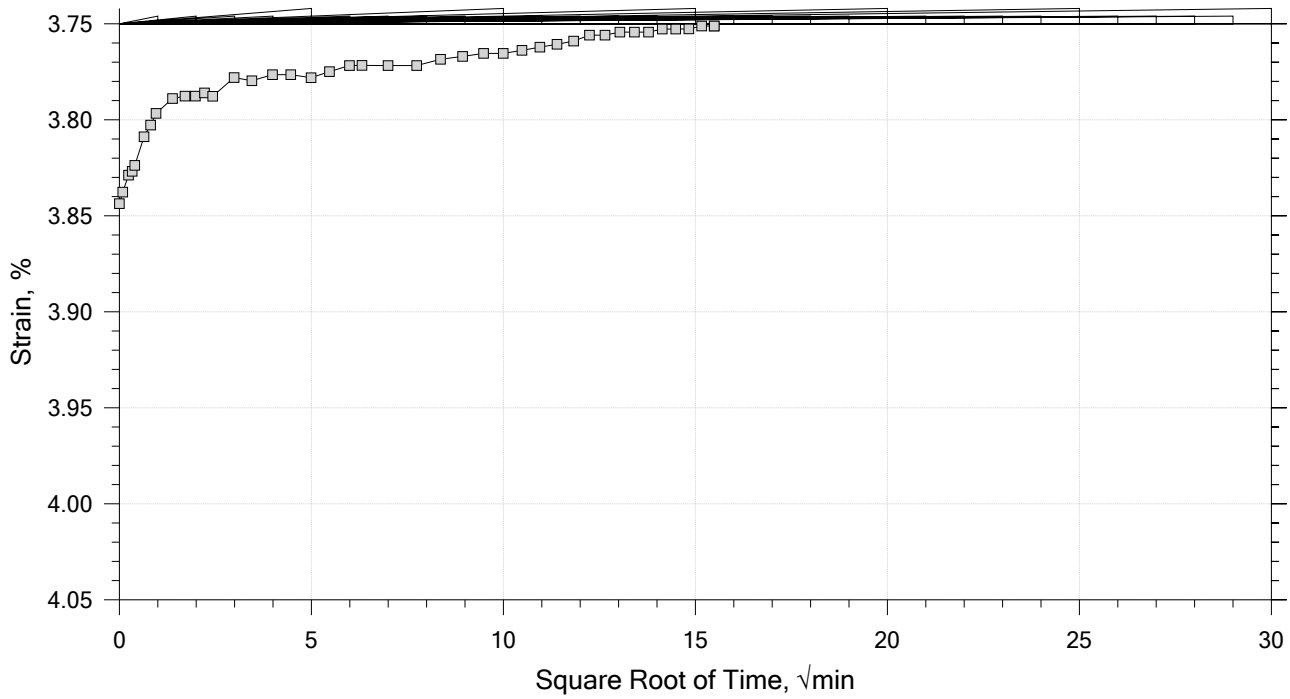


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 7 of 14
Constant Load Step
Stress: 2 tsf





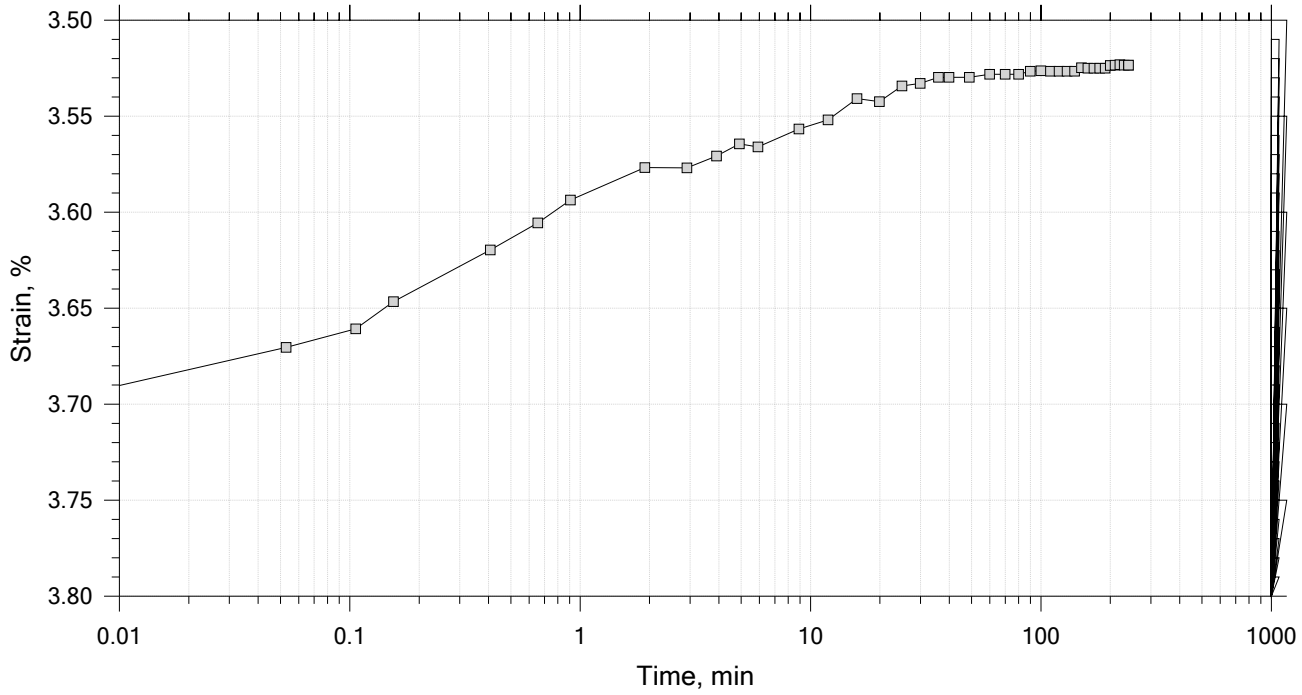
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

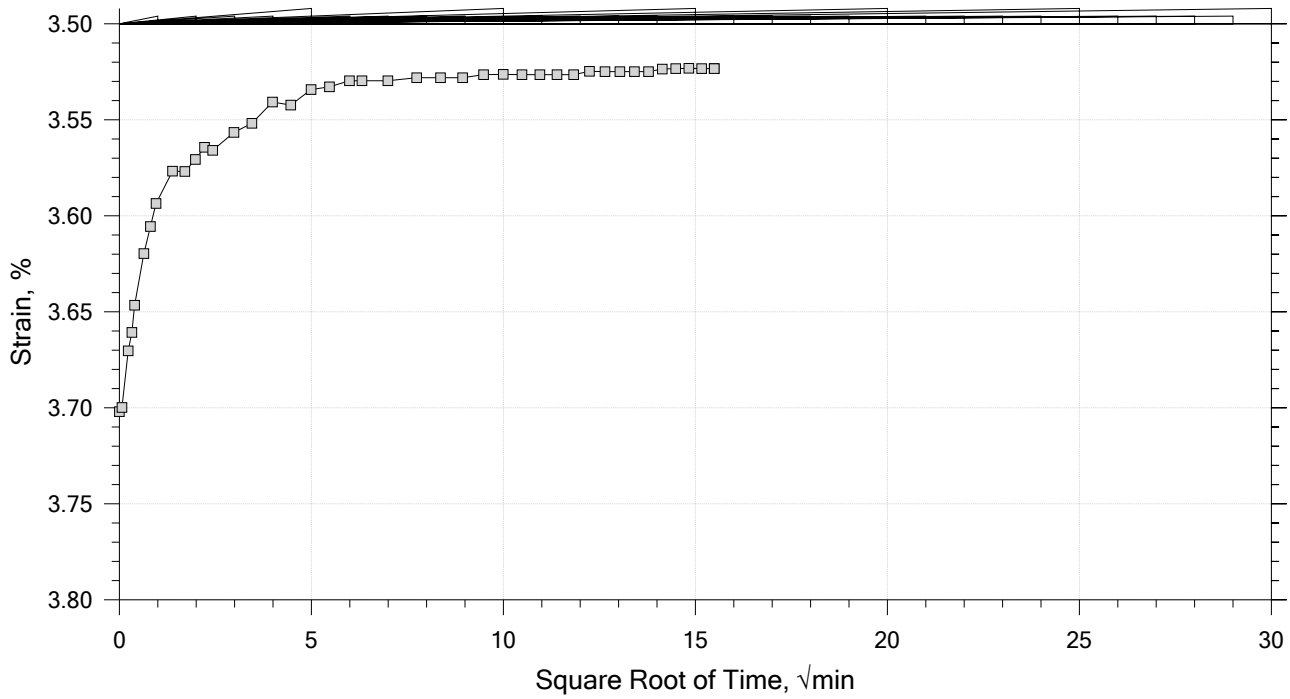
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 8 of 14

Constant Load Step

Stress: 1 tsf

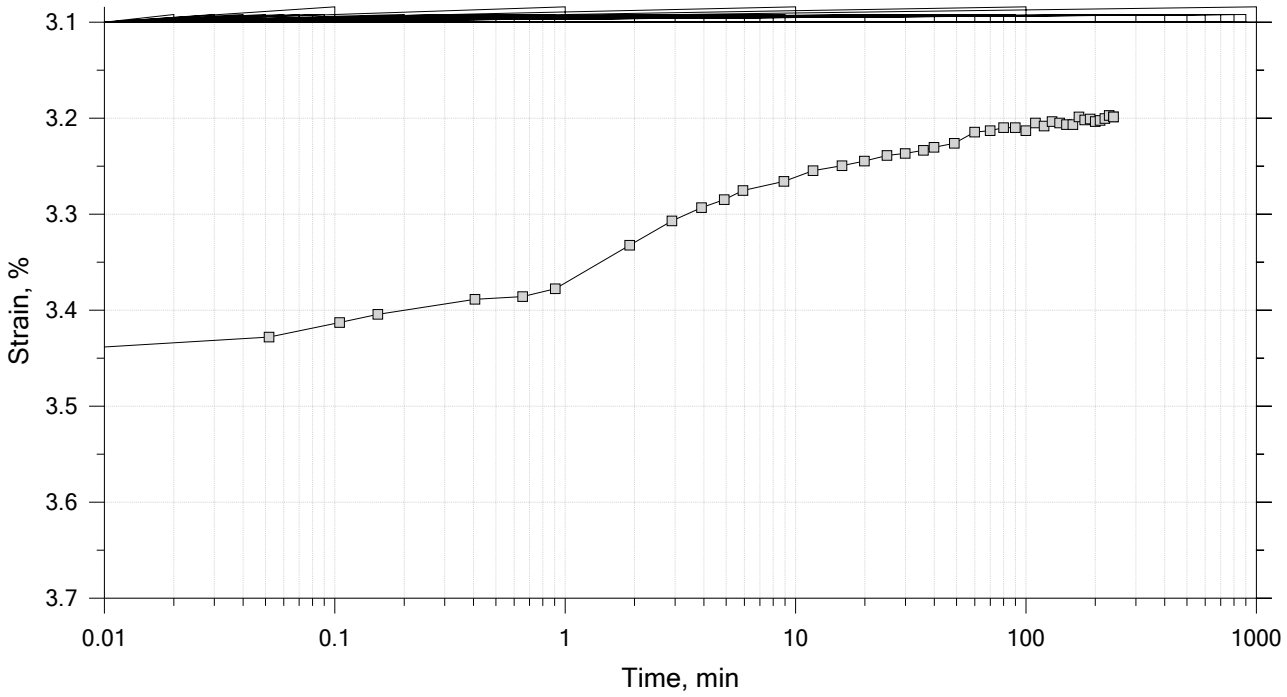


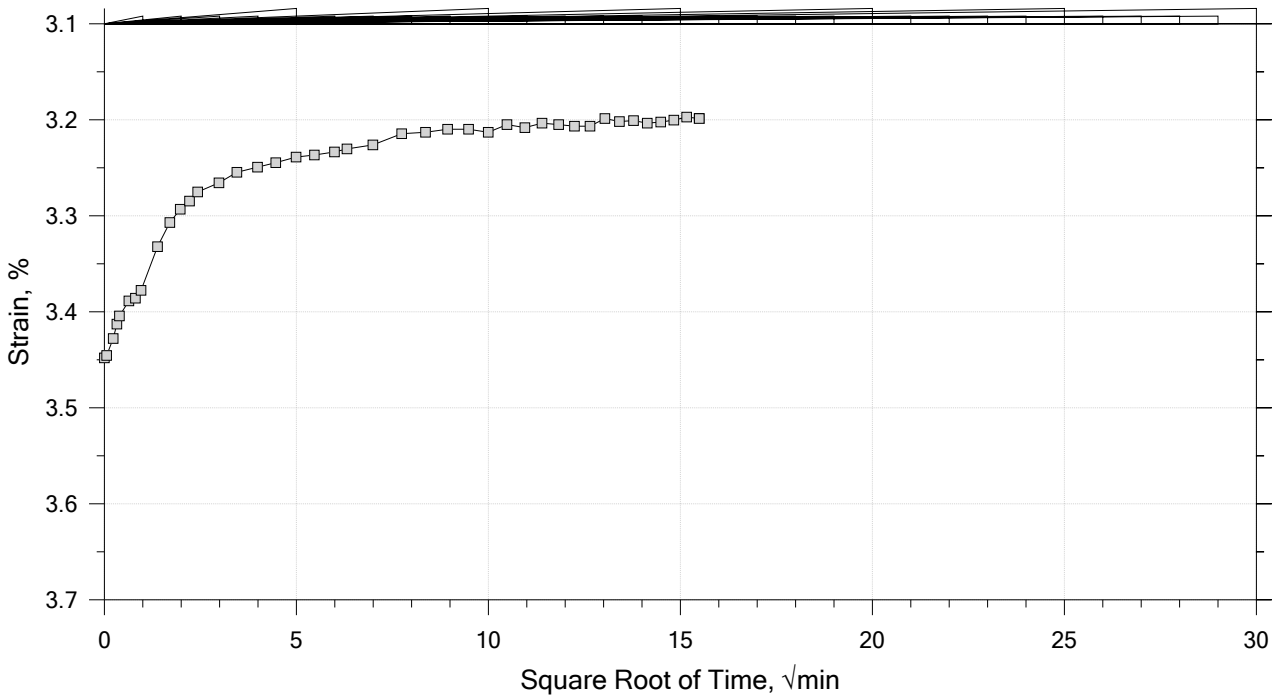


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 9 of 14
Constant Load Step
Stress: 0.5 tsf

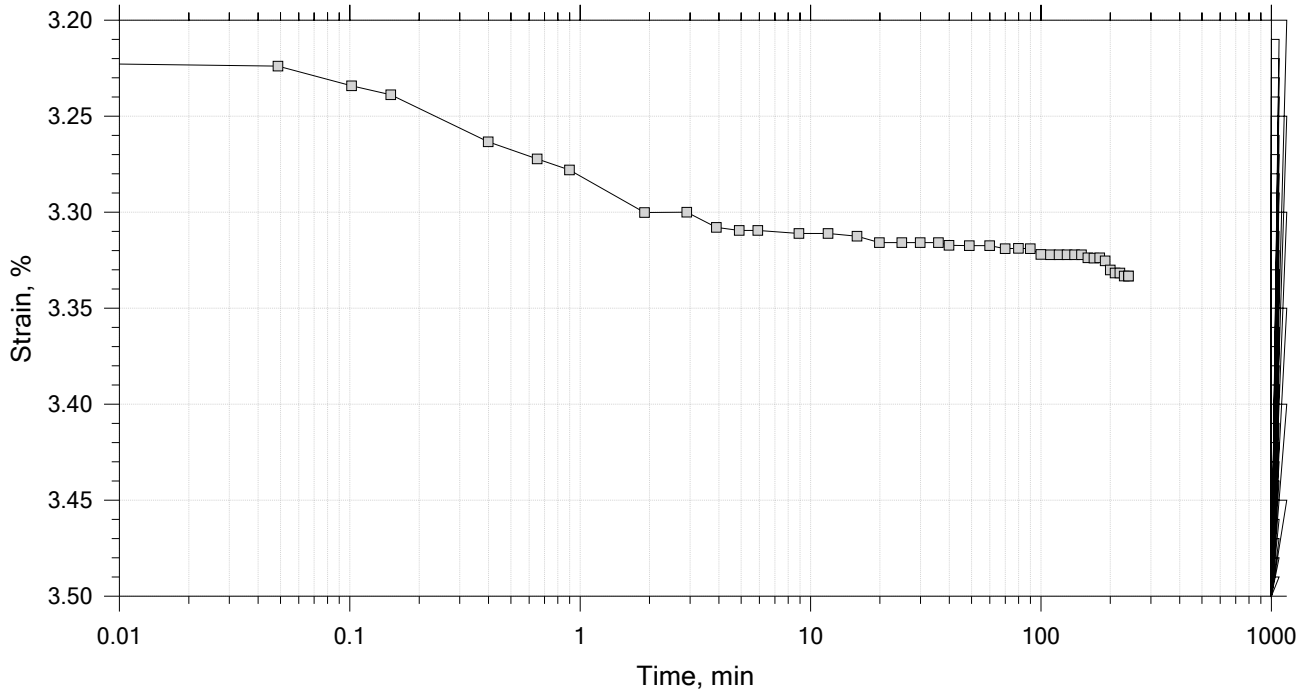


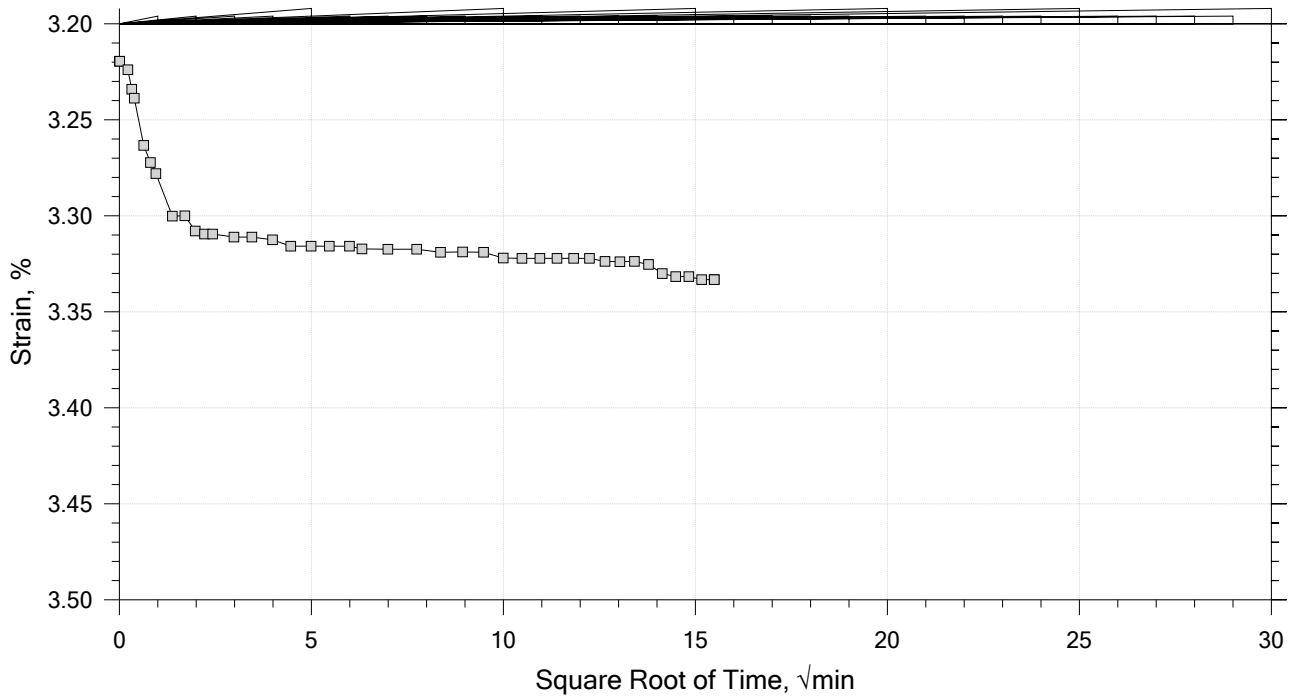


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 10 of 14
Constant Load Step
Stress: 1 tsf

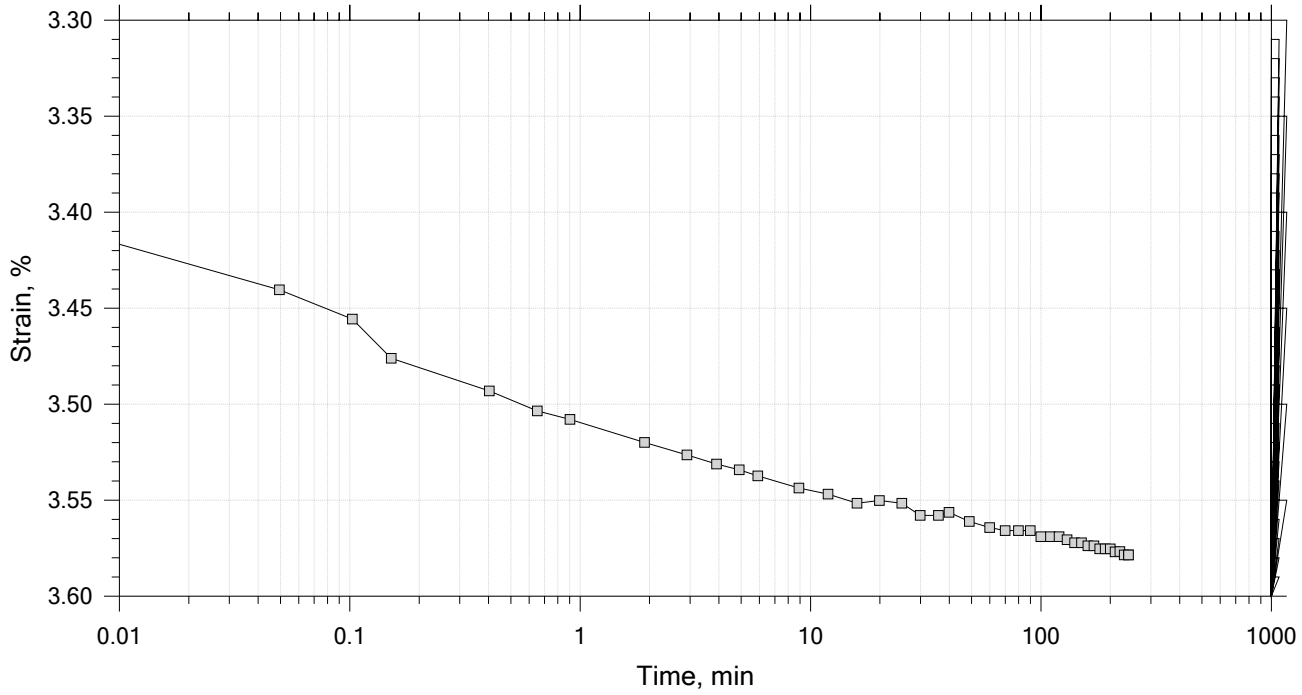


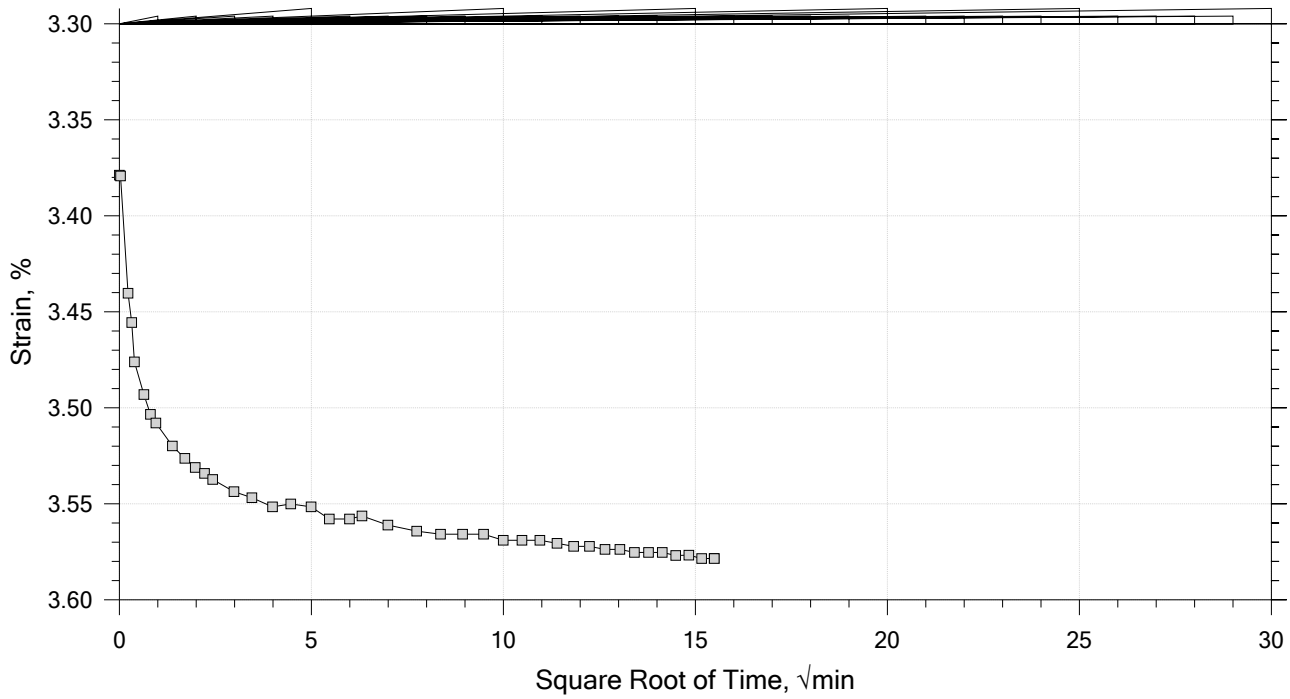


Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
Test No.: IP-1	Sample Type: intact	Elevation: ---
Description: Moist, olive clay		
Remarks: System LTIII-E		

One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 11 of 14
Constant Load Step
Stress: 2 tsf





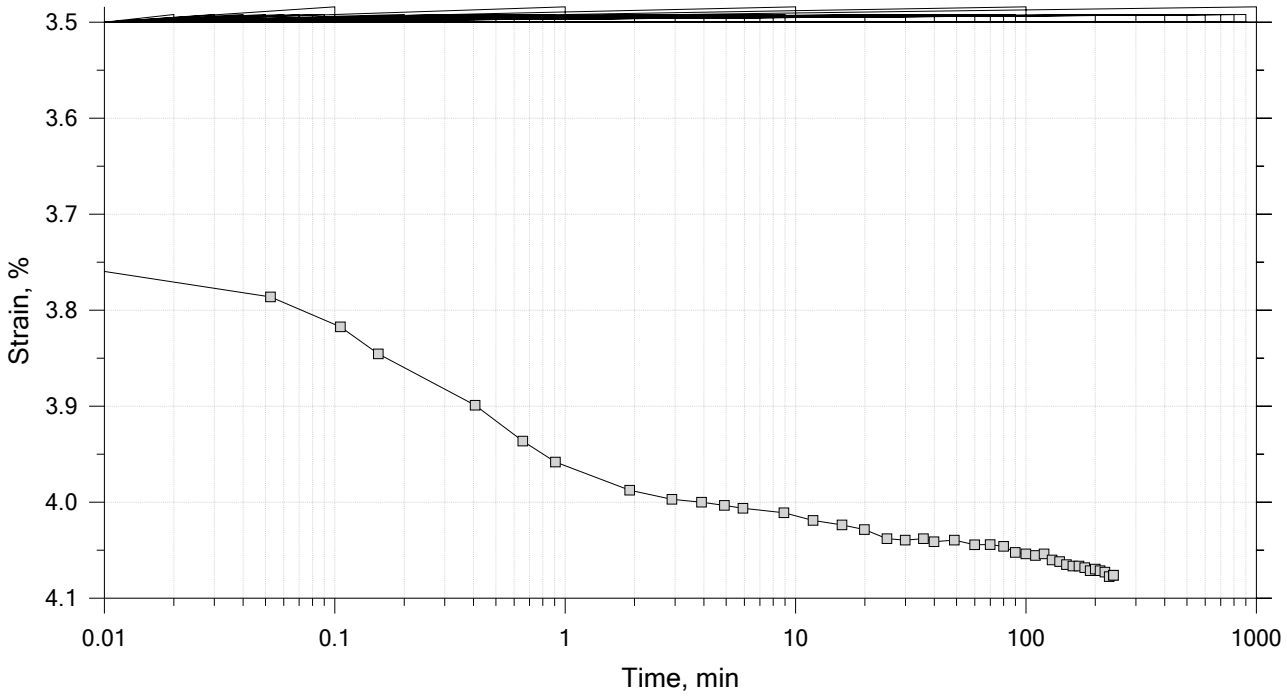
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

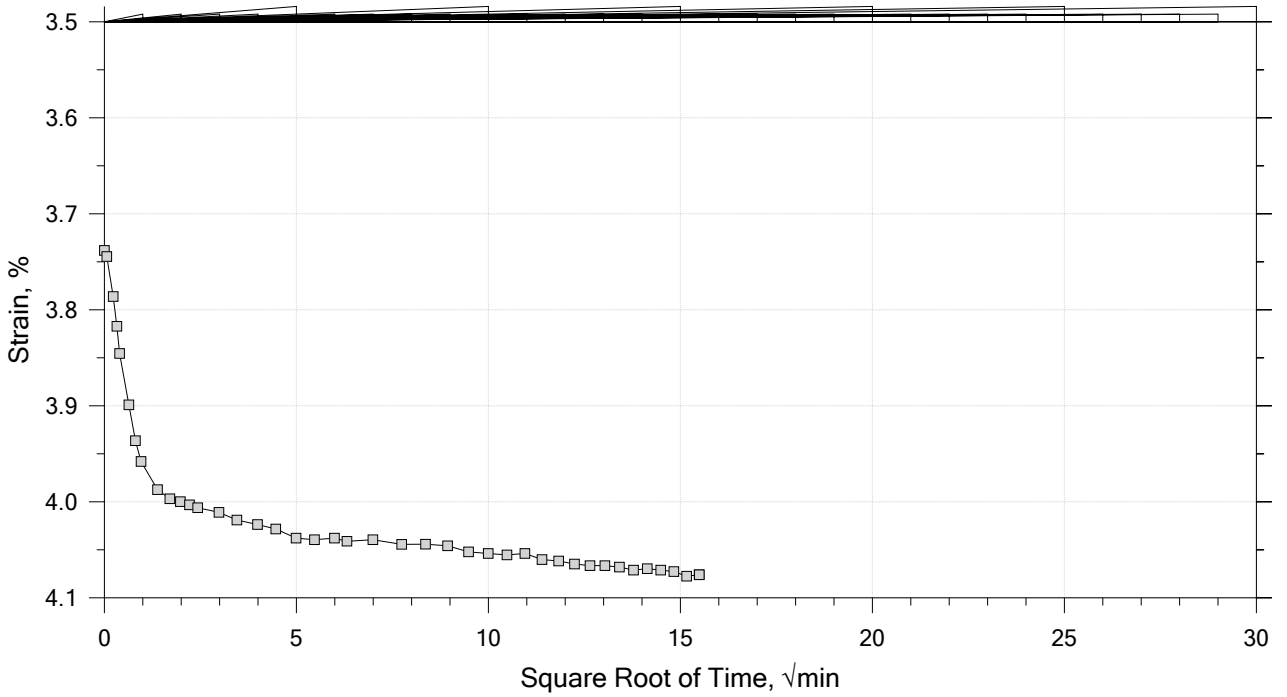
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 12 of 14

Constant Load Step

Stress: 4 tsf





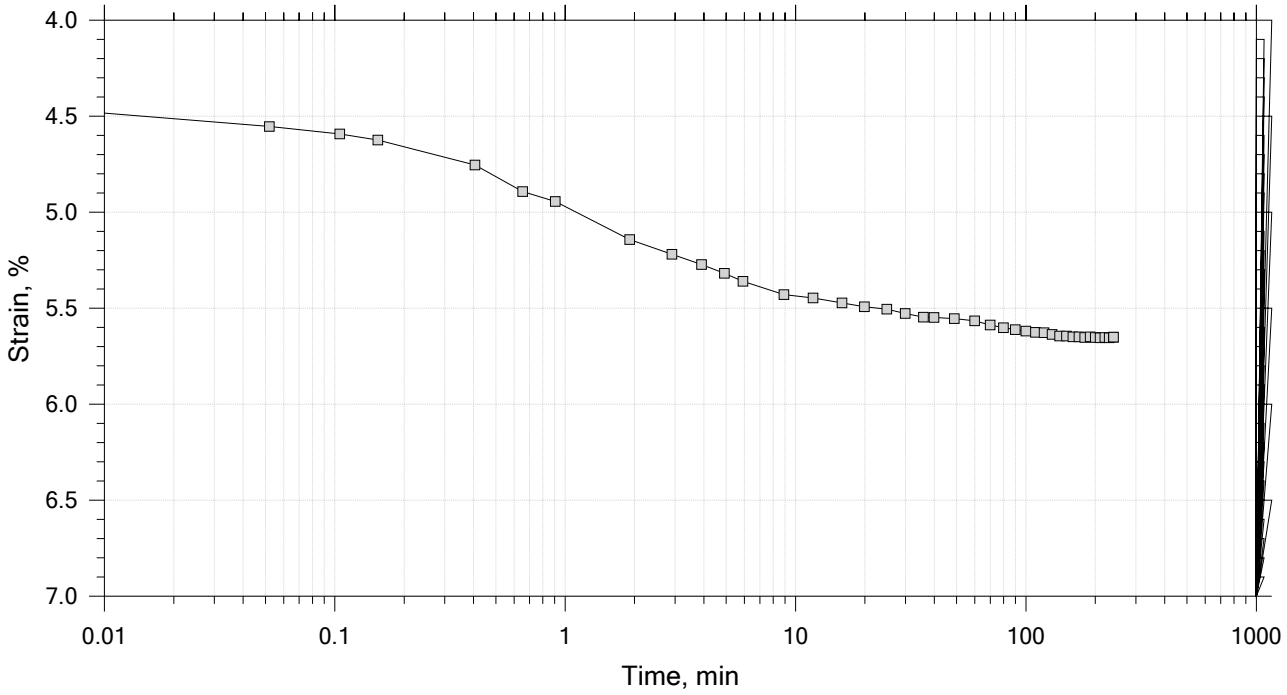
GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

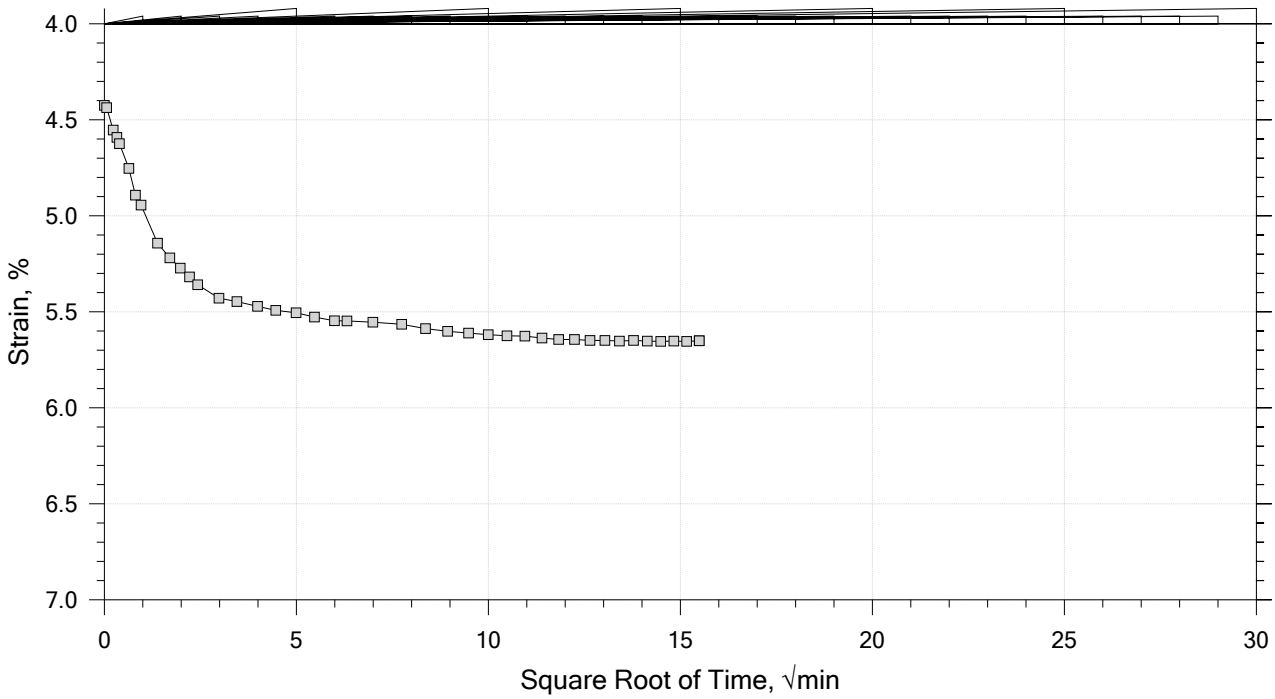
One-Dimensional Consolidation by ASTM D2435 - Method B

Time Curve 13 of 14

Constant Load Step

Stress: 8 tsf

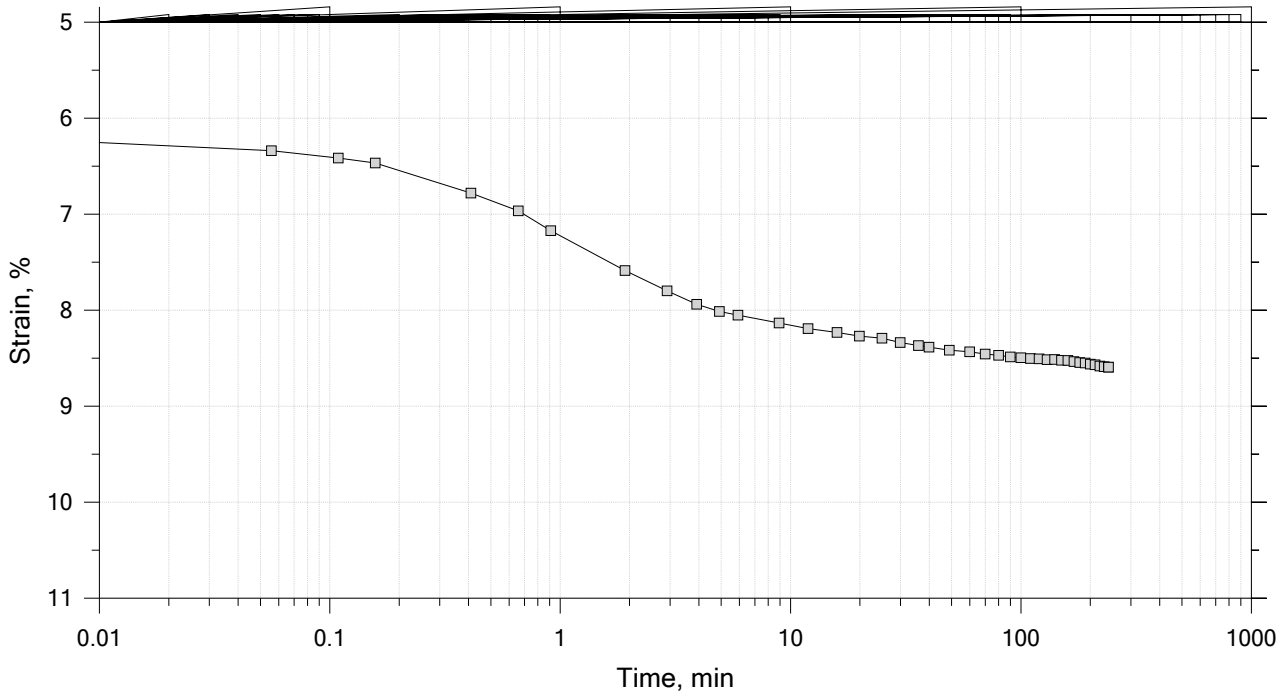


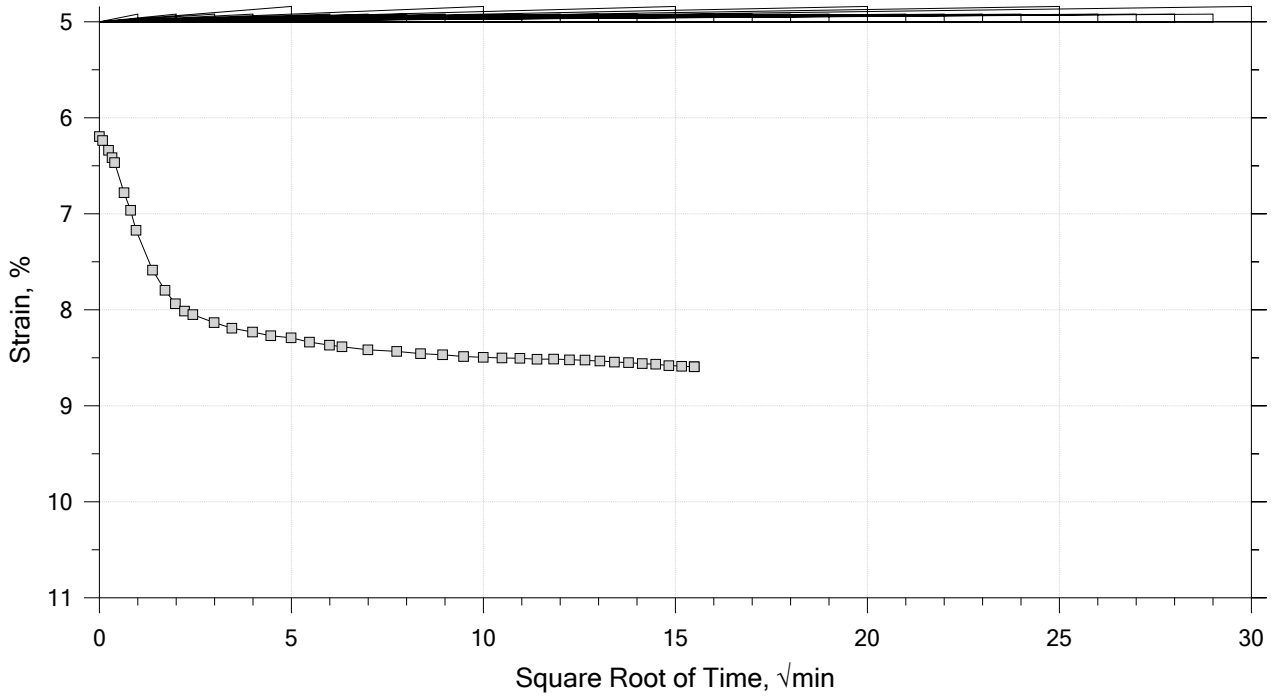


GeoTesting EXPRESS	Project: Hogan Road Highway	Location: Bangor, ME	Project No.: GTX-315483
	Boring No.: HB-BHR-108	Tested By: md	Checked By: anm
	Sample No.: 1U	Test Date: 05/24/22	Depth: 8-10 ft
	Test No.: IP-1	Sample Type: intact	Elevation: ---
	Description: Moist, olive clay		
	Remarks: System LTIII-E		

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

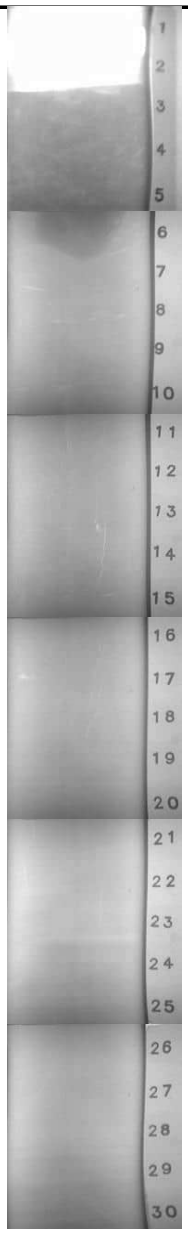
GeoTesting EXPRESS

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		

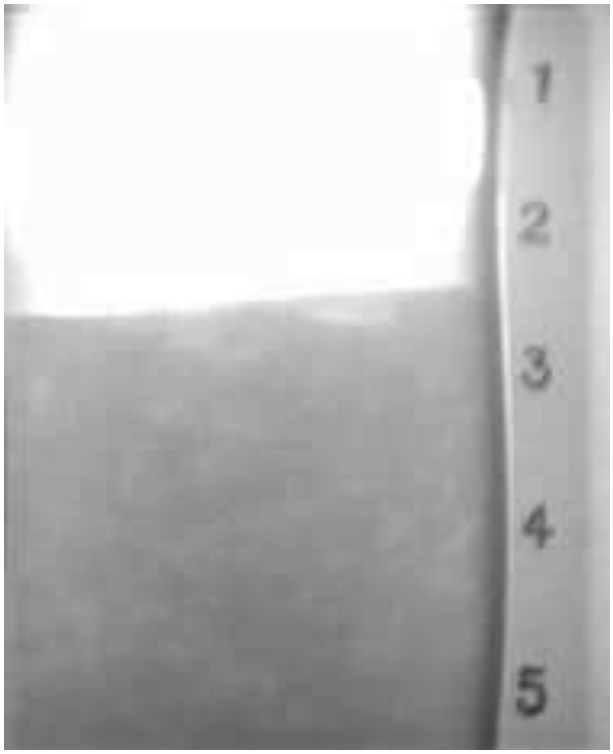
Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



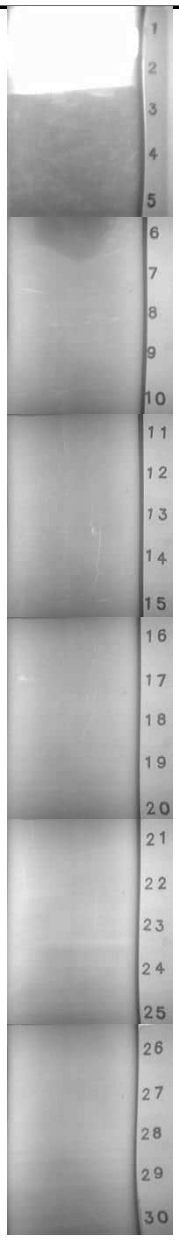
SECTION 1



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-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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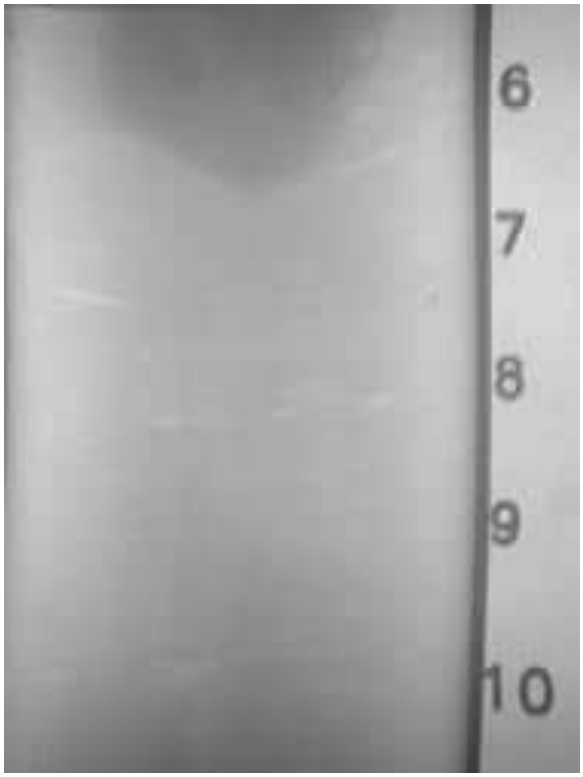
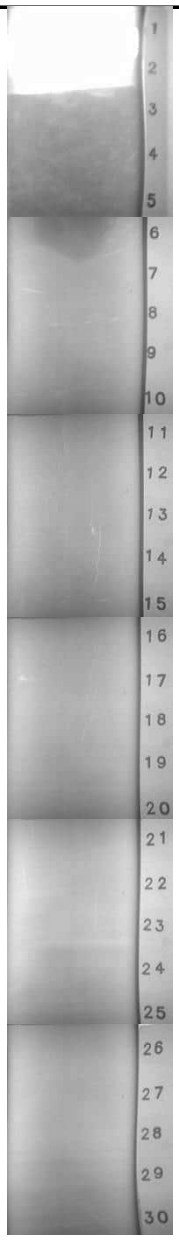


Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube



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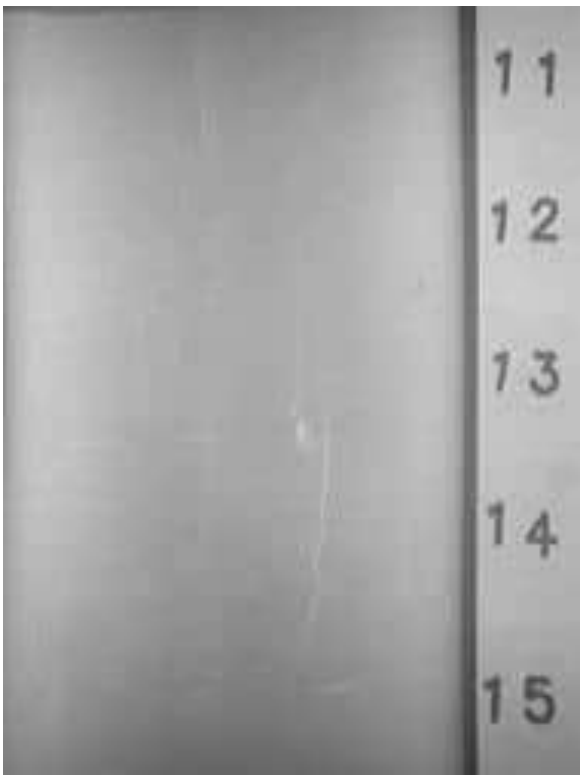
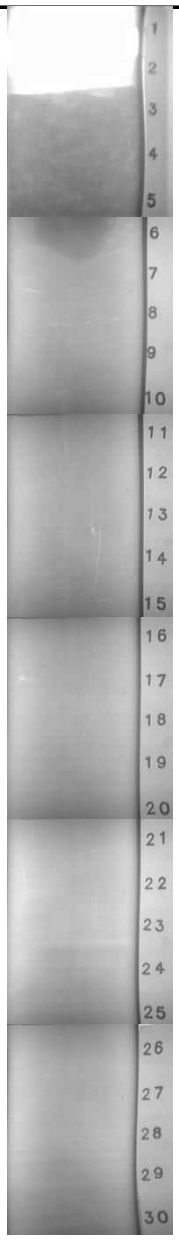


Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
2	
3	
4	
5	
6	
	Bottom of Tube

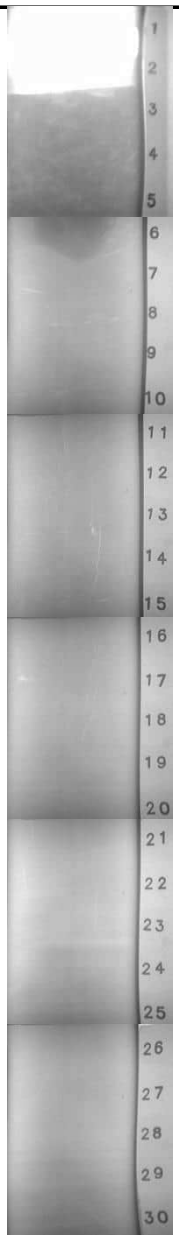


SECTION 4

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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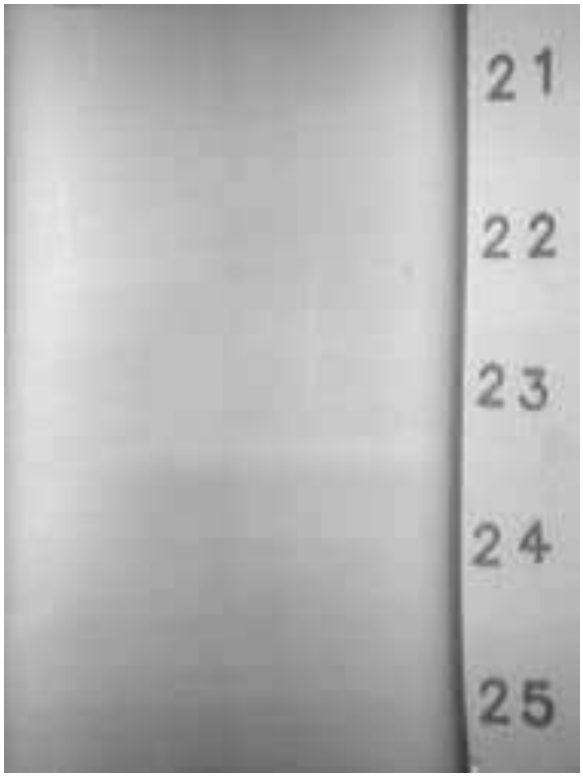

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-101
Sample ID:	1U
Depth:	10-12 ft.

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
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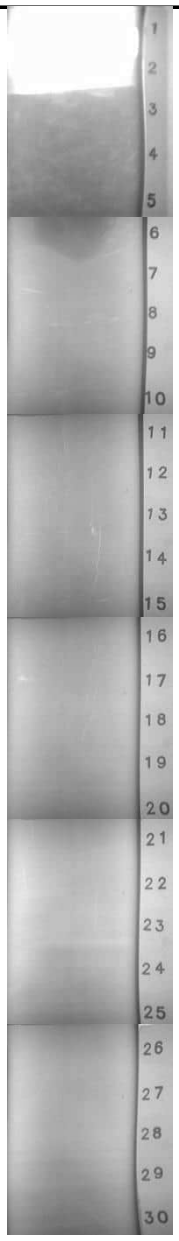
2	
3	
4	
5	
6	
	Bottom of Tube



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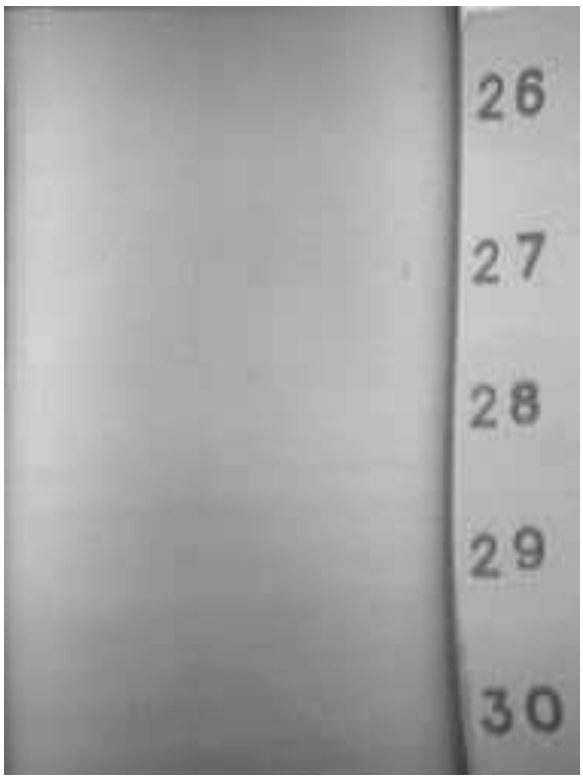

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

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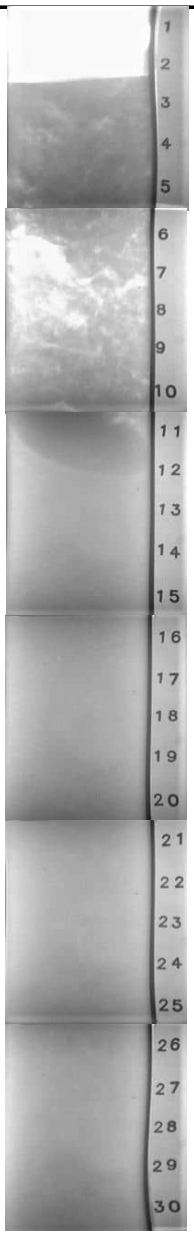
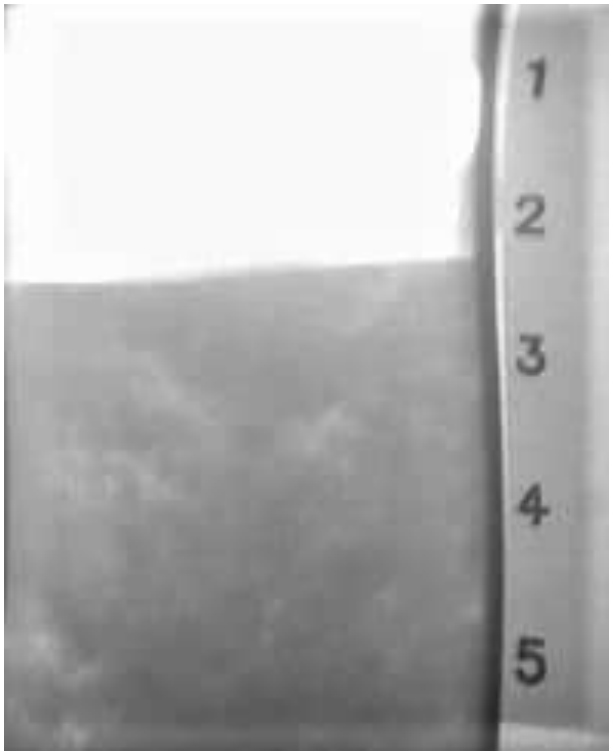
1	
2	
3	
4	
5	
6	
	Bottom of Tube

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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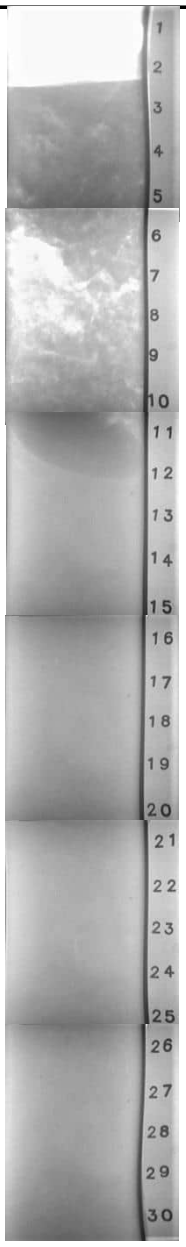
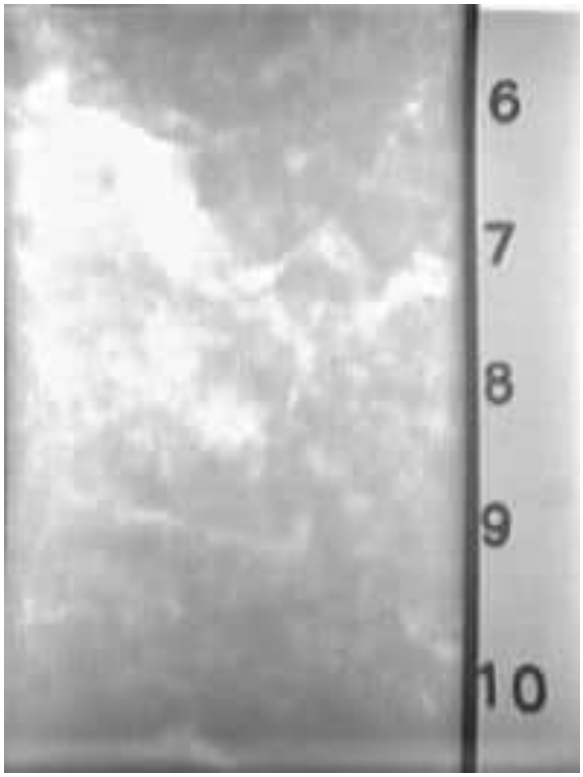
	Bottom of Tube



SECTION 2



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

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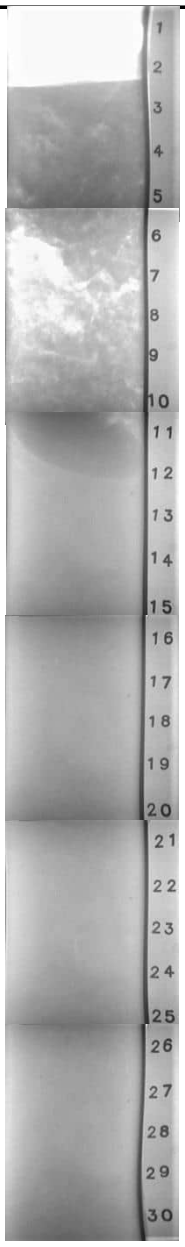
	Bottom of Tube



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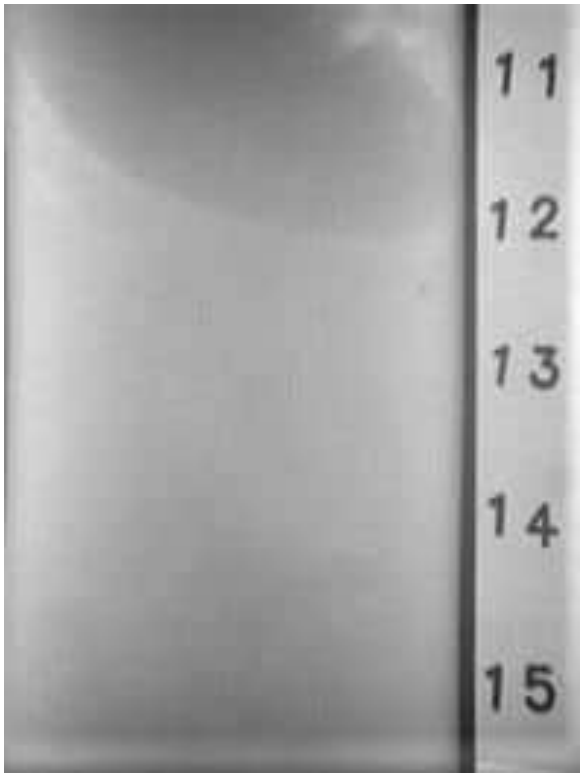

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
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X-Ray of Soil Sample by ASTM D4452

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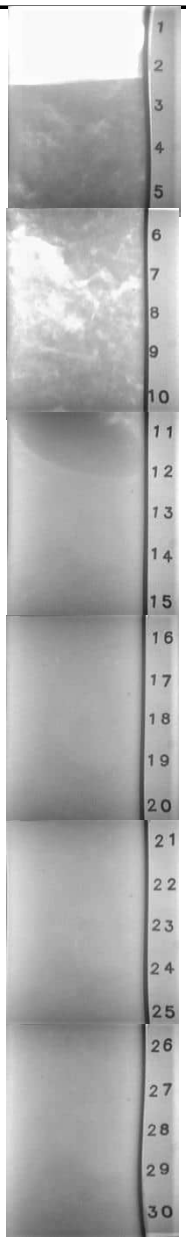

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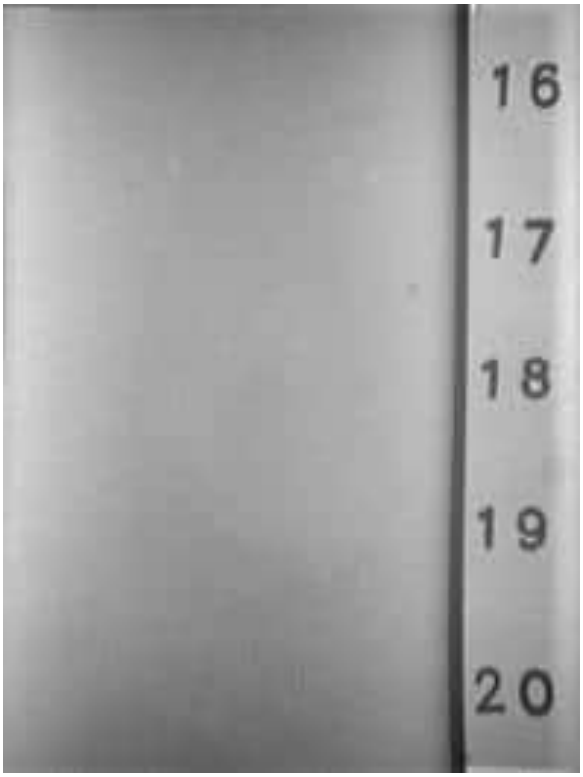

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
Sample ID:	1U
Depth:	10.5-12.5 ft

X-Ray of Soil Sample by ASTM D4452

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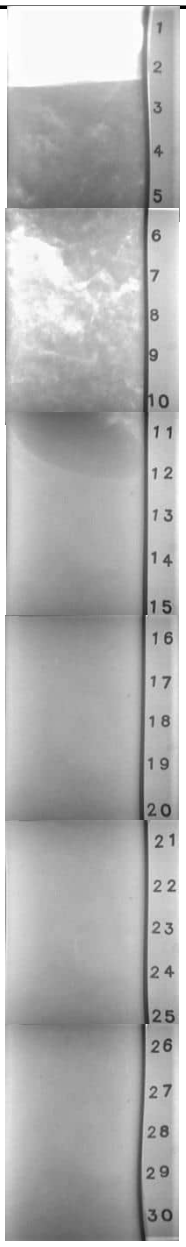

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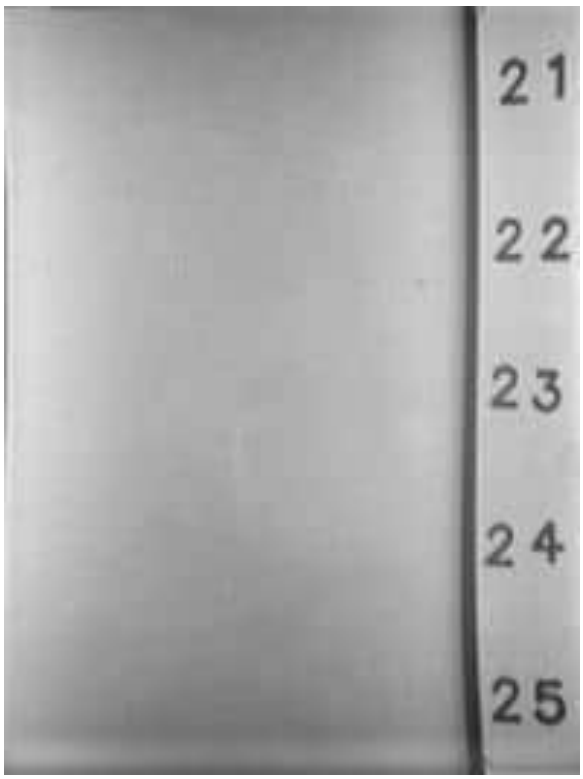

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-102
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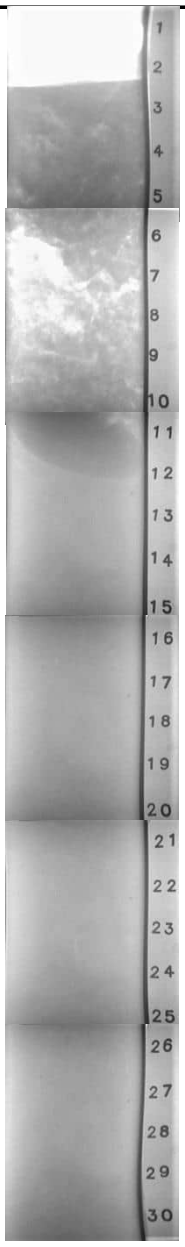
1	
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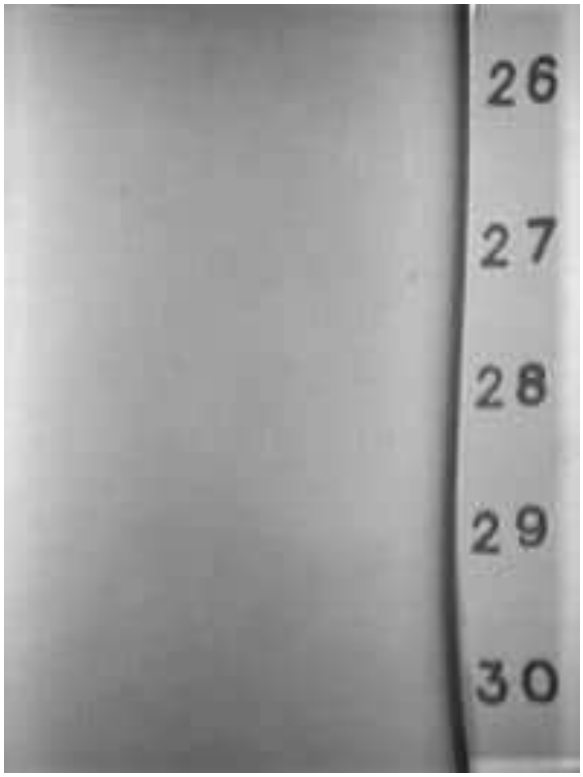
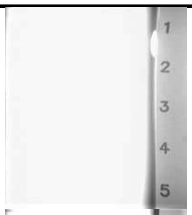
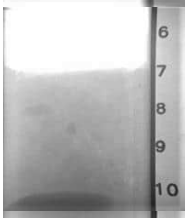
SECTION 6



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
1	
2	
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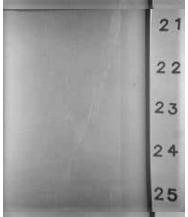
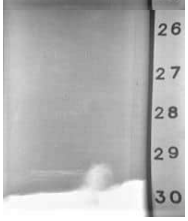
5	
6	
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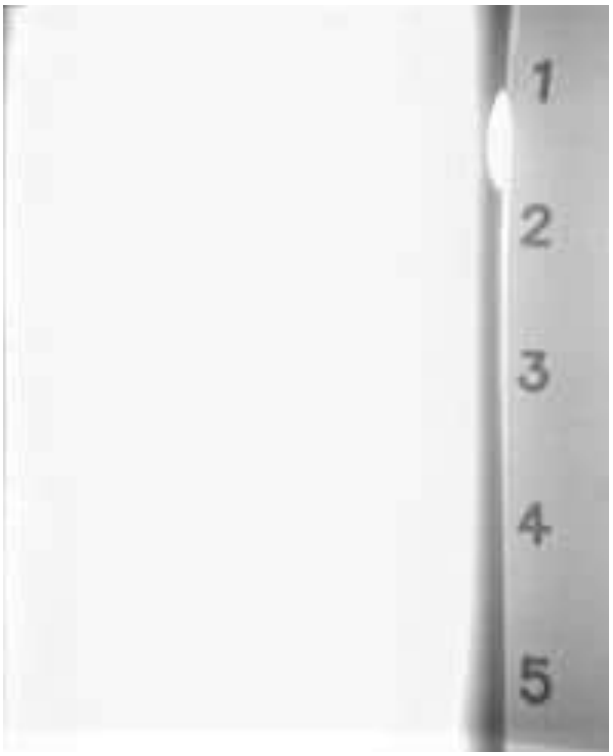
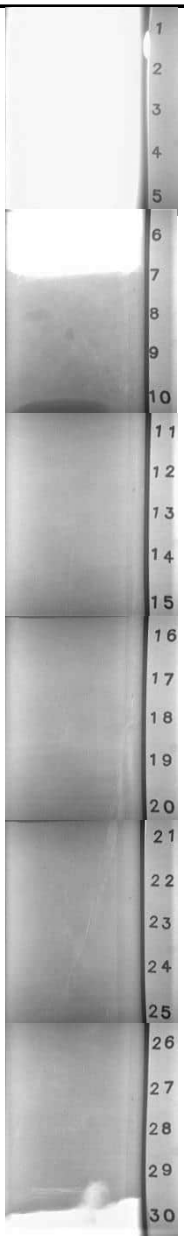
SECTION 1



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-108
Sample ID:	1U
Depth:	80-10 ft

X-Ray of Soil Sample by ASTM D4452

Section	Top of Tube
1	
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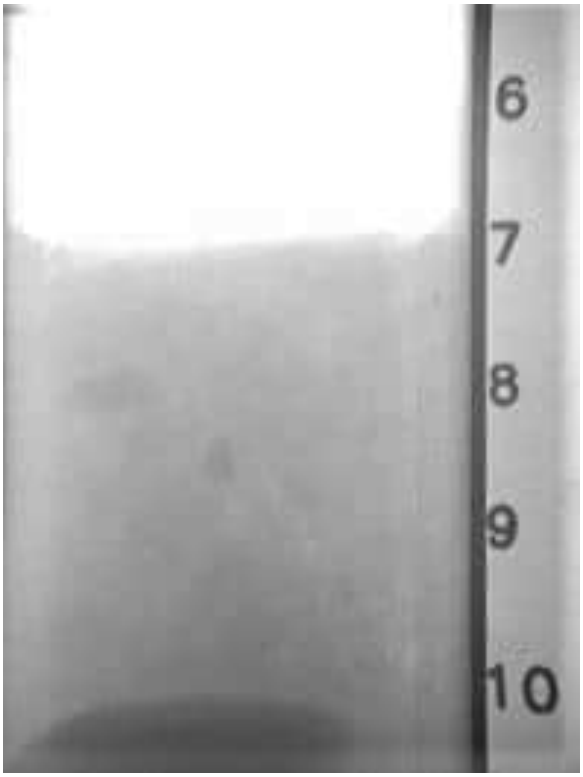

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
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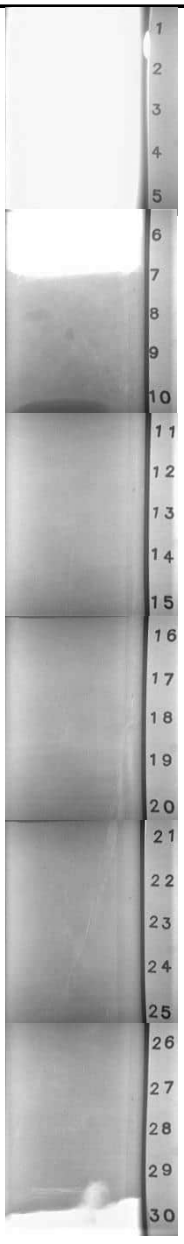
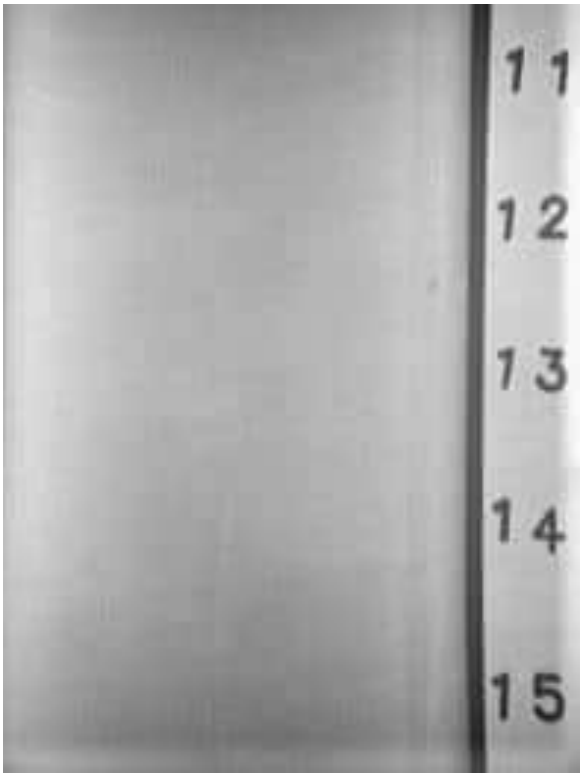


SECTION 3



Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
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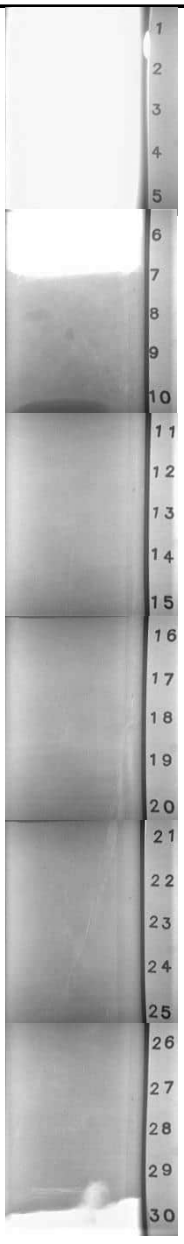

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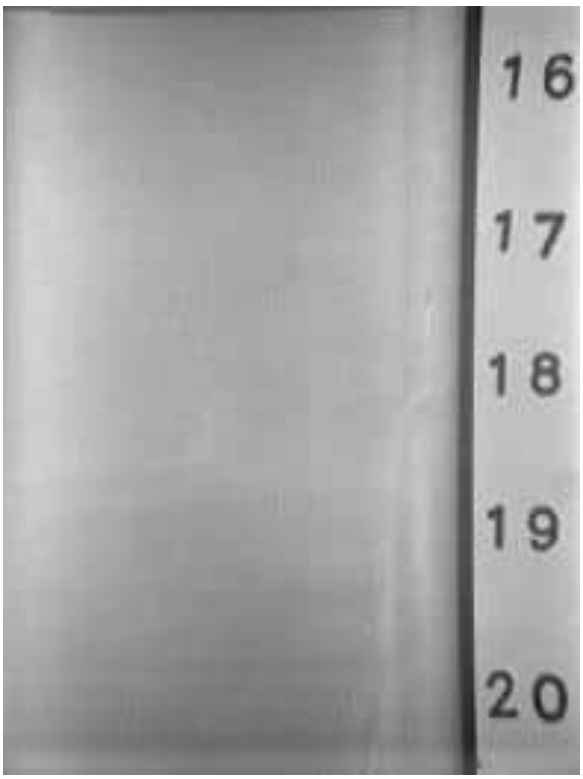

Client:	Golder Associates USA, Inc.
Project Name:	Hogan Road Highway
Project Location:	Bangor, ME
GTX #:	315483
Test Date:	05/16/22
Tested By:	md
Checked By:	jsc
Boring ID:	HB-BHR-108
Sample ID:	1U
Depth:	80-10 ft

X-Ray of Soil Sample by ASTM D4452

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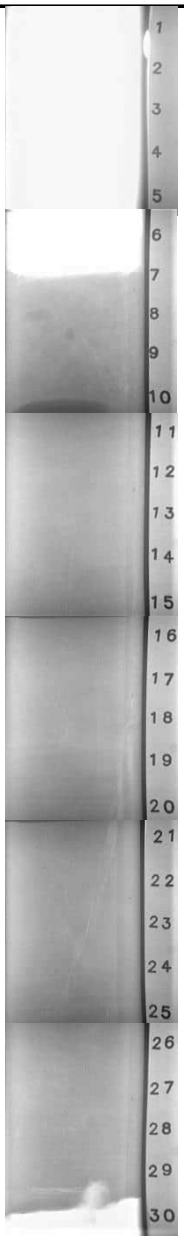
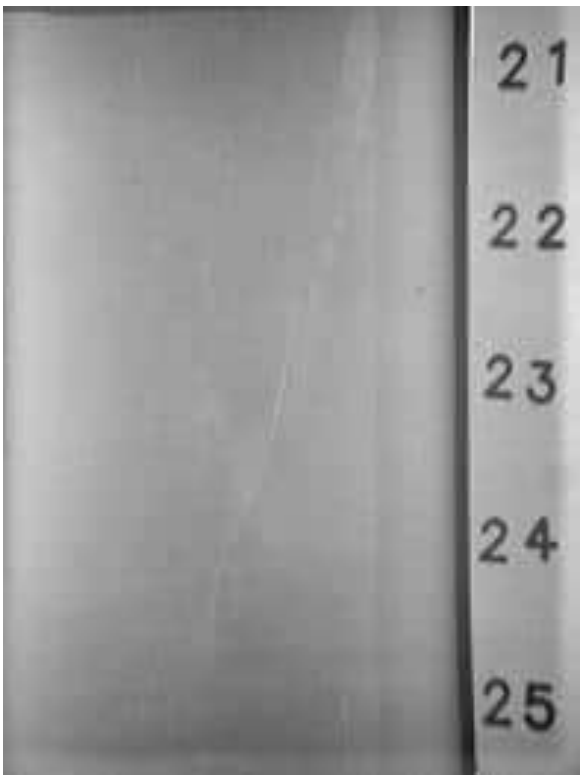


SECTION 5



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

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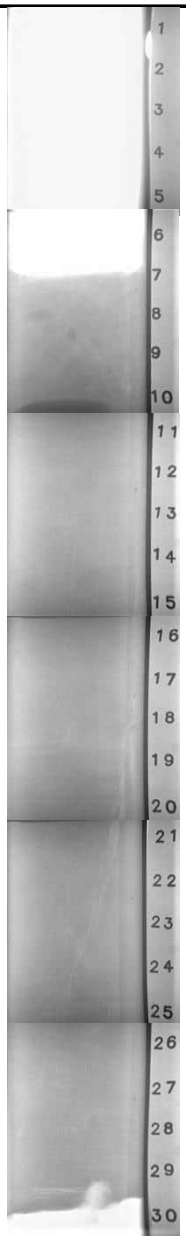
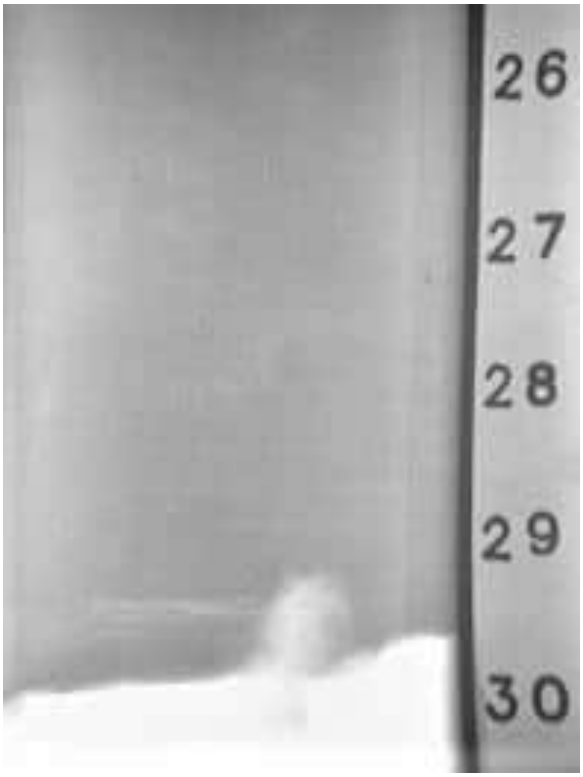


SECTION 6



APPENDIX D

Frost and Seismic Calculations

CALCULATIONS

Date:	3/4/2024	**Made by:**	DEB
Project No.:	314040817.004	**Checked by:**	ATM
Subject:	Frost Penetration Depth - Bridge Structures in Fill	**Reviewed by:**	MEL
Project Short Title:	MaineDOT Bangor Hogan Road Bridge Replacement		

OBJECTIVE

Determine the depth of frost penetration at the existing Bridge #5823 and proposed Bridge #6787 site.

REFERENCES

1. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
2. WSP interpreted subsurface section A-A' and D-D' (Sheet numbers 3 and 6, Geotechnical Design Report, dated September 2025).
3. WSP geotechnical test boring logs (Appendix A, Geotechnical Design Report, dated September 2025).
4. GeoTesting Express laboratory testing results, dated May 2022, June 2022, and November 2023 (Appendix C, Geotechnical Design Report, dated September 2025).

ASSUMPTIONS

1. Based on the available moisture contents in granular fill materials around the existing and proposed bridges, it is assumed the average moisture content in granular fills will be consistent for existing and new fills in this evaluation.

2. Existing bridge pier foundations are assumed to have been undercut and bearing on granular fill and backfilled with granular fill up and away at 1H:1V. Proposed pile caps are assumed to be excavated and backfilled with the same techniques.

3. New fill is assumed to consist of Granular Borrow, Gravel Borrow, or Foamed Glass Aggregate (FGA).

CALCULATION

Follow the procedure in the MaineDOT Bridge Design Guide Section 5.2.1 to determine the depth of frost penetration. The site location is Bangor, Maine. Using the Maine Design Freezing Index Map (Reference 1, Figure 5-1), the design freezing index at the site location is 1726 degree-days.

Based on Reference 2, the existing abutments are founded in fill which is a coarse grained material. The proposed abutments will be founded within new fill, which is a coarse-grained soil (Assumption 3). The existing pier foundations are founded within fill and the proposed pier foundations will be founded in new fill (Assumption 2).

Based on References 3 and 4, the range of water contents are 10% to 12% and the average water content of the samples tested from the existing fill layer is 10.7%.

From Reference 1, Table 5-1, with design freezing index = 1726, coarse-grained soil, and linear interpolation between design freezing index 1700 and 1800 for w = 10% and w = 20%.

Frost Penetration = 87.1 inches = 7.3 feet

CONCLUSIONS

The depth of frost penetration at the proposed Bridge #6787 site is estimated to be 7.3 feet in areas where new fill associated with bridge foundations will be placed. The foundations should be embedded below this depth.

CALCULATIONS

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road / I95 EB Bridge #5823	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine the seismic site class and acceleration coefficient for the existing eastbound bridge #5823 abutment 2 for Hogan Road over I-95 in Bangor, ME

METHOD

Use the procedure outlined in AASHTO LRFD Section 3.10.3.1 Method B (Ref. 1) to determine site class.

REFERENCES

1. AASHTO. (2020). AASHTO LRFD Bridge Design Specifications, 9th Ed. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.
2. WSP boring logs, included as Appendix A of the Preliminary Geotechnical Design Report.
3. WSP Interpretive subsurface cross-section (ISCC), "1.31.24 Existing Abutment ISCC's"
4. WSP N60 correlation from "Lab Field Data Summary Plots V4 3140817.00"
5. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.
6. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exit 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.

ASSUMPTIONS

1. Use average N_{60}-values (corrected only for hammer efficiencies) in the analysis for Method B. Average N_{60} values were taken as averages from the specific associated boring and layer and are based on values presented in Ref. 2 and summarized in Ref. 4.
2. The thicknesses of each soil layer were taken from the ISCC for Abutment 2 at Sta. 113+91.73 of the Existing Bridge.
3. The bottom strata encountered during drilling at each boring was extended to a depth of 100 feet below ground surface to use the method.
4. Average N_{60} values from all the bridge borings for the Glacial Till, and two Silty Clay layers were used for the seismic site class calculation. The N_{60} values for the Proposed Fill layer and the Foam Glass Aggregate (FGA) layer were back calculated from a friction angle of 32 degrees assuming that the backfill material will have the properties of MaineDOT's standard granular borrow (Ref. 6) and 35.5 degrees for the FGA based on WSP's interpretation of Nicks et. al (Ref. 8).

CALCULATIONS

Site Class*	
C	$\bar{N} > 50$; $\bar{s}_u > 2.0$ ksf; 1200 ft/s < $\bar{V}_s < 2500$ ft/s
D	$15 < \bar{N} < 50$; $1.0 < \bar{s}_u < 2.0$ ksf; 600 ft/s < $\bar{V}_s < 1200$ ft/s
E	$\bar{N} < 15$; $\bar{s}_u < 1.0$ ksf; $\bar{V}_s < 600$ ft/s
F	Peats, organic clays, high plasticity clay, thick soft/medium stiff clays

* Site class A & B pertain to rock and are not included in this table.

CALCULATIONS

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road / I95 EB Bridge #5823	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

1. Per AASHTO Table C3.10.3.1-1 (Ref 1), use Method B, $\bar{N}$ method:

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

where:

d_i = Thickness of a layer between 0 and 100 ft.

N_i = SPT blow count, N_{60}, of a layer (not to exceed 100 blows/ft). Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

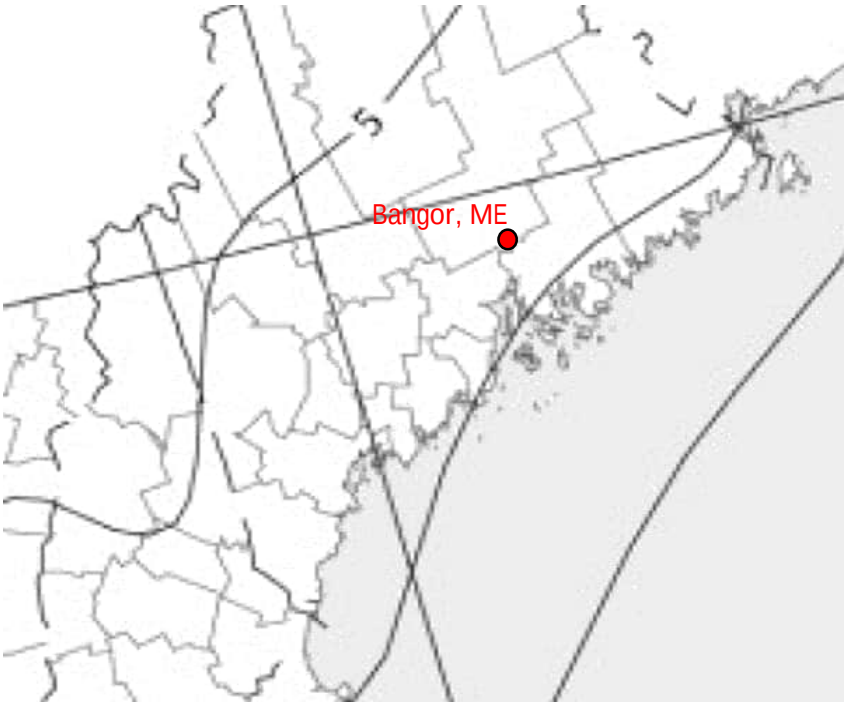
Existing Abutment 2 Soil Profile				
Layer	N_{60}	Deposit	d_i	d_i/N_i
1	24	Fill	22.3	0.93
2	23	Stiff to V. Stiff Clay	12.6	0.55
3	7	Med. Stiff to Soft Clay	1.1	0.16
4	29	Glacial Till	19.6	0.68
5	100	Bedrock	44.4	0.44
		Sum:	100.0	2.75

$\bar{N}$ =	36.3

$\bar{N}$ =	36.3

Site Class:	D

2. Seismic Performance Zone, per AASHTO LRFD 9th Ed. (2020)



S_1 = 0.044 Figure 3.10.2.1-3: 1 second period response acceleration (AASHTO 2020).

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road / I95 EB Bridge #5823	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

AASHTO Equation 3.10.4.2-6 $SD_1 = F_vS_1$
AASHTO Table 3.10.3.2-3 F_v = 2.4 SD_1 = 0.106 Seismic Zone: 1
Seismic Zone 1 per AASHTO 3.10.6 Table 3.10.6-1

3. Calculation of Seismic Parameters using AASHTO LRFD 9th Ed. (2020)
Values for PGA, S_s, and S_1 were obtained comparing the site location to the following geographic charts:

a_1. Using Figure 3.10.2.1-1
*PGA = 0.065
*Horizontal ground coefficient with a 7% probability of exceedance in 75 years

a_2. Using Figure 3.10.2.1-2
*S_s = 0.14
*Horizontal response spectral acceleration at period of 0.2 s with a 7% probability of exceedance in 75 years and 5% critical damping

a_3. Using Figure 3.10.2.1-3
*S_1 = 0.044
*Horizontal response spectral acceleration at a period of 1 s with a 7% probability of exceedance in 75 years and 5% critical damping

b_1. Using Table 3.10.3.2-1
Using a site class value of D, taken from calculation step 1 and a PGA of 0.065, taken from step $3a_1$

b_2. Using Table 3.10.3.2-2
Using a site class value of D, taken from calculation step 1 and a S_s of 0.14, taken from step $3a_2$

b_3. Using Table 3.10.3.2-3
Using a site class value of D, taken from calculation step 1 and a S_1 of 0.044, taken from step $3a_3$

F_{pga} = 1.6 F_a = 1.6 F_v = 2.4

c_1. Using Equation 3.10.4.2-2

$$A_s = F_{pga} * PGA$$

PGA = 0.065
F_{pga} = 1.6
A_s = 0.104

c_2. Using Equation 3.10.4.2-3

$$S_{DS} = F_a * S_S$$

S_S = 0.14
F_a = 1.6
S_{DS} = 0.224

c_3. Using Equation 3.10.4.2-6

$$S_{D1} = F_v * S_1$$

S_1 = 0.044
F_v = 2.4
S_{D1} = 0.106

Reduce the seismic coefficient A_s based on the slope height using the method described in FHWA GEC No. 3 Section 6.2.2 (Ref. 5).

$k_{av} = \alpha * A_s$ Ref. 5, Eq. 6-2
$\alpha = 1 + 0.01 * H * (0.5 * \beta - 1)$ Ref. 5, Eq. 6-3

Where:
H = height of the slope in the direction of sliding
$\beta = F_v * S_1 / A_s$ Ref. 5, Eq. 6-4

H (feet) =	27.8	Ref. 3 Sheet 86
β =	1.02	Ref. 5, Eq. 6-4
α =	0.86	Ref. 5, Eq. 6-3
k_{av} =	0.090	Ref. 5, Eq. 6-2

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road / I95 EB Bridge #5823	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Seismic Parameters						
PGA[1]	A_s[2]	k_{av}[3]	S_{DS}[4]	S_{D1}[5]	Site Class[6]	Zone[7]
-	1/s	1/s	1/s	1/s	-	-
0.065	0.104	0.090	0.224	0.106	D	1

[1]Using AASHTO Figure 3.10.2.1-1
[2]Using AASHTO Figure 3.10.2.1-1, Table 3.10.3.2-1, and Equation 3.10.4.2-2
[3]Using FHWA GEC No.3 Section 6.2.2 Equations 6-2 through 6-4
[4]Using AASHTO Figure 3.10.2.1-2, Table 3.10.3.2-2, and Equation 3.10.4.2-3
[5]Using AASHTO Figure 3.10.2.1-3, Table 3.10.3.2-3, and Equation 3.10.4.2-6
[6]Using result of step 1 of calculation package
[7]Using result of step 2 of calculation package

CONCLUSIONS

Based on the materials encountered and the results of the SPT testing completed during the drilling the subsurface investigations the Seismic Site Class for the site is D.

As per Section 3.10.4.2 of AASHTO (2020), the 1 second period response acceleration at the site is less than 0.15g, therefore the site is in Seismic Performance Zone 1.

CALCULATIONS

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road No. 2 WB Bridge #6787	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine the seismic site class and acceleration coefficient for the proposed westbound bridge #6787 abutment 2 for Hogan Road over I-95 in Bangor, ME

METHOD

Use the procedure outlined in AASHTO LRFD Section 3.10.3.1 Method B (Ref. 1) to determine site class.

REFERENCES

1. AASHTO. (2020). AASHTO LRFD Bridge Design Specifications, 9th Ed. American Association of State Highway and Transportation Officials (AASHTO), Washington, D.C.
2. WSP boring logs, included as Appendix A of the Preliminary Geotechnical Design Report.
3. WSP Interpretive subsurface cross-section (ISCC), "1.31.24 Existing Abutment ISCC's"
4. WSP N60 correlation from "Lab Field Data Summary Plots V4 3140817.00"
5. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.
6.Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
7. Maine Department of Transportation, Preliminary Design Report, Hogan Road/I95 Bridge #5823 over Interstate 95 Exit 187 Diverging Diamond Interchange, Bangor, Maine Federal Project # 1859510, WIN 018595.10, February 3, 2023.
8. 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).

ASSUMPTIONS

1. Use average N_{60}-values (corrected only for hammer efficiencies) in the analysis for Method B. Average N_{60} values were taken as averages from the specific associated boring and layer and are based on values presented in Ref. 2 and summarized in Ref. 4.
2. The thicknesses of each soil layer were taken form the ISCC for Abutment 2 at Sta. 215+00 of the Proposed Bridge.
3. The bottom strata encountered during drilling at each boring was extended to a depth of 100 feet below ground surface to use the method.
4. Average N_{60} values from all the bridge borings for the Glacial Till, and two Silty Clay layers were used for the seismic site class calculation. The N_{60} values for the Proposed Fill layer and the Foam Glass Aggregate (FGA) layer were back calculated from a friction angle of 32 degrees assuming that the backfill material will have the properties of MaineDOT's standard granular borrow (Ref. 6) and 35.5 degrees for the FGA based on WSP's interpretation of Nicks et. al (Ref. 8).

CALCULATIONS

Site Class*	
C	$\bar{N} > 50$; $\bar{s}_u > 2.0$ ksf; 1200 ft/s < $\bar{V}_s < 2500$ ft/s
D	$15 < \bar{N} < 50$; $1.0 < \bar{s}_u < 2.0$ ksf; 600 ft/s < $\bar{V}_s < 1200$ ft/s
E	$\bar{N} < 15$; $\bar{s}_u < 1.0$ ksf; $\bar{V}_s < 600$ ft/s
F	Peats, organic clays, high plasticity clay, thick soft/medium stiff clays

* Site class A & B pertain to rock and are not included in this table.

CALCULATIONS

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road No. 2 WB Bridge #6787	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

1. Per AASHTO Table C3.10.3.1-1 (Ref 1), use Method B, $\bar{N}$ method:

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

where:

d_i = Thickness of a layer between 0 and 100 ft.

N_i = SPT blow count, N_{60}, of a layer (not to exceed 100 blows/ft). Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

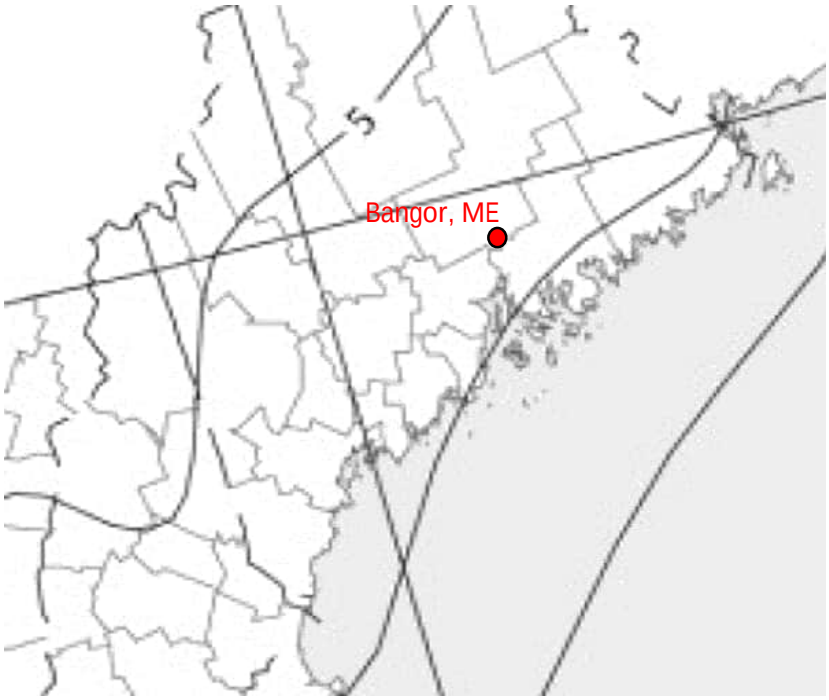
Proposed Abutment 2 Soil Profile				
Layer	N_{60}	Deposit	d_i	d_i/N_i
1	17	Proposed Fill	14.9	0.88
2	30	FGA	18.0	0.60
3	23	Stiff to V. Stiff Clay	2.8	0.12
4	7	Med. Stiff to Soft Clay	6.5	0.93
5	29	Glacial Till	7.1	0.24
6	100	Bedrock	50.7	0.51
		Sum:	100.0	3.28

$\bar{N}$ =	30.5

$\bar{N}$ =	30.5

Site Class:	D

2. Seismic Performance Zone, per AASHTO LRFD 9th Ed. (2020)



S_1 = 0.044 Figure 3.10.2.1-3: 1 second period response acceleration (AASHTO 2020).

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road No. 2 WB Bridge #6787	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

AASHTO Equation 3.10.4.2-6 $SD_1 = F_vS_1$
AASHTO Table 3.10.3.2-3 F_v = 2.4 SD_1 = 0.106 Seismic Zone: 1
Seismic Zone 1 per AASHTO 3.10.6 Table 3.10.6-1

3. Calculation of Seismic Parameters using AASHTO LRFD 9th Ed. (2020)
Values for PGA, S_s, and S_1 were obtained comparing the site location to the following geographic charts:

a_1. Using Figure 3.10.2.1-1
*PGA = 0.065
*Horizontal ground coefficient with a 7% probability of exceedance in 75 years

a_2. Using Figure 3.10.2.1-2
*S_s = 0.14
*Horizontal response spectral acceleration at period of 0.2 s with a 7% probability of exceedance in 75 years and 5% critical damping

a_3. Using Figure 3.10.2.1-3
*S_1 = 0.044
*Horizontal response spectral acceleration at a period of 1 s with a 7% probability of exceedance in 75 years and 5% critical damping

b_1. Using Table 3.10.3.2-1
Using a site class value of D, taken from calculation step 1 and a PGA of 0.065, taken from step $3a_1$

b_2. Using Table 3.10.3.2-2
Using a site class value of D, taken from calculation step 1 and a S_s of 0.14, taken from step $3a_2$

b_3. Using Table 3.10.3.2-3
Using a site class value of D, taken from calculation step 1 and a S_1 of 0.044, taken from step $3a_3$

F_{pga} = 1.6 F_a = 1.6 F_v = 2.4

c_1. Using Equation 3.10.4.2-2

$$A_s = F_{pga} * PGA$$

PGA = 0.065
F_{pga} = 1.6
A_s = 0.104

c_2. Using Equation 3.10.4.2-3

$$S_{DS} = F_a * S_S$$

S_S = 0.14
F_a = 1.6
S_{DS} = 0.224

c_3. Using Equation 3.10.4.2-6

$$S_{D1} = F_v * S_1$$

S_1 = 0.044
F_v = 2.4
S_{D1} = 0.106

Reduce the seismic coefficient A_s based on the slope height using the method described in FHWA GEC No. 3 Section 6.2.2 (Ref. 5).

$k_{av} = \alpha * A_s$ Ref. 5, Eq. 6-2
$\alpha = 1 + 0.01 * H * (0.5 * \beta - 1)$ Ref. 5, Eq. 6-3

Where:
H = height of the slope in the direction of sliding
$\beta = F_v * S_1 / A_s$ Ref. 5, Eq. 6-4

H (feet) =	28.7	Ref. 3 Page 7
β =	1.02	Ref. 5, Eq. 6-4
α =	0.86	Ref. 5, Eq. 6-3
k_{av} =	0.089	Ref. 5, Eq. 6-2

CALCULATIONS

Date:	9/9/2025	**Made by:**	LDN
Project No.:	31404817.004	**Checked by:**	AMR
Subject:	Seismic Site Analysis - Hogan Road No. 2 WB Bridge #6787	**Reviewed by:**	CCB
Project Short Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Seismic Parameters						
PGA[1]	A_s[2]	k_{av}[3]	S_{DS}[4]	S_{D1}[5]	Site Class[6]	Zone[7]
-	1/s	1/s	1/s	1/s	-	-
0.065	0.104	0.089	0.224	0.106	D	1

[1]Using AASHTO Figure 3.10.2.1-1
[2]Using AASHTO Figure 3.10.2.1-1, Table 3.10.3.2-1, and Equation 3.10.4.2-2
[3]Using FHWA GEC No.3 Section 6.2.2 Equations 6-2 through 6-4
[4]Using AASHTO Figure 3.10.2.1-2, Table 3.10.3.2-2, and Equation 3.10.4.2-3
[5]Using AASHTO Figure 3.10.2.1-3, Table 3.10.3.2-3, and Equation 3.10.4.2-6
[6]Using result of step 1 of calculation package
[7]Using result of step 2 of calculation package

CONCLUSIONS

Based on the material encountered and the results of the SPT testing completed during the drilling the subsurface investigations the Seismic Site Class for the site is D.

As per Section 3.10.4.2 of AASHTO (2020), the 1 second period response acceleration at the site is less than 0.15g, therefore the site is in Seismic Performance Zone 1.

APPENDIX E

Soil Corrosivity Evaluation

CALCULATIONS

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

OBJECTIVE

Estimate the corrosion that the existing Hogan Road Bridge HP10x42 piles may experience over the desired remaining service life of 50 years.

METHOD

Use references from AASHTO, FHWA, and NCHRP to determine potential corrosion rates based on the results of pH, electrical resistivity, chloride, and sulfate laboratory testing on soil samples obtained during the subsurface investigation at site.

REFERENCES

1. GeoTesting Express. Laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes. Received by WSP on May 26, 2022, May 31, 2022, January 3, 2024, and March 5, 2024. Summarized in Table 4 and included in Appendix C of the WSP Geotechnical Design Report.

2. American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

3. AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R 27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023

4. Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016.

5. National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011.

6. FHWA. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Publication No. FHWA-NHI-09-087. Dated November 2009.

7. Carroll E. Taylor & Associates for MaineDOT. As-Built Plans: Hogan Road Bridge over Interstate 95 in the city of Bangor, Penobscot County. Dated 1983.

8. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

ASSUMPTIONS

1. Based on field measurements and observations made in the field by WSP and MaineDOT field engineers on July 22nd and July 23rd, 2024, assume no section loss due to corrosion has occurred from pile installation to present.

ATTACHMENTS

1. Summary table of corrosivity laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes

2. Draft boring location plan for bridge and highway 100- and 200-series borings and 300-series hand probes in the vicinity of the existing and proposed bridges

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

CALCULATION

A. Use the corrosivity laboratory test results for the soil at site (Ref. 1, also included as Attachment 1) to determine the corrosion potential at the existing piles.

Per AASHTO LRFD Article 10.7.5 (Ref. 2), the following soil or water conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Resistivity less than 2,000 ohm-cm
- pH less than 5.5
- pH between 5.5 and 8.5 in soils with high organic content
- Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)
- Chloride content greater than 500 ppm

The corrosivity laboratory test results for the soil at site (Ref. 1) are compared to these conditions:

Condition	Range from lab test results	Median from lab test results	Meets condition?
Resistivity less than 2,000 ohm-cm	1,038 ohm-cm to 7,666 ohm-cm	2,212 ohm-cm	Yes*
pH less than 5.5	6.4 to 8.2	7.5	No
pH between 5.5 and 8.5 in soils with high organic content	0.5% to 0.7% organic content	0.6% organic content	No
Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)	<10 ppm to 30 ppm	10 ppm	No
Chloride content greater than 500 ppm	<10 ppm to 120 ppm	48 ppm	No

*While the median value is above the 2,000 ohm-cm threshold, a sufficient number of test results are below 2,000 ohm-cm to warrant further consideration.

Since the lab test results only indicate a marginal potential for pile deterioration due to corrosion, the AASHTO Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications (Ref. 3) is used to further assess corrosion potential.

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

B. Use the flowcharts from the AASHTO Standard Practice (Ref. 3) to determine the potential corrosion situation at the existing piles.

Step 1: use AASHTO flowchart Figure 3 (Ref. 3) and the corrosivity laboratory test results for the soil at site (Ref. 1) to determine the possibility of macrocell and uniform corrosion of the existing piles.

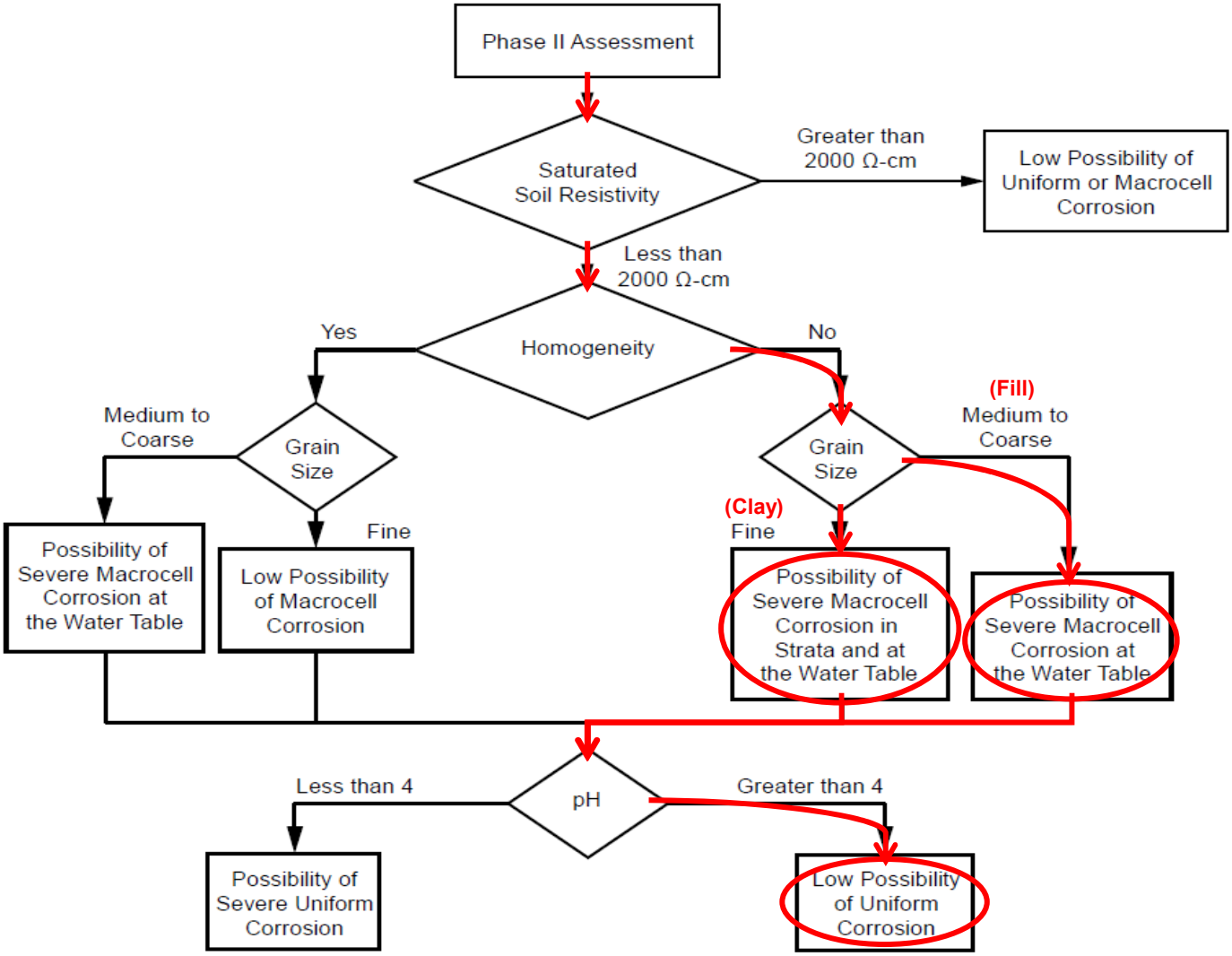


Figure 3—Determination of the Possibility for General and/or Macrocell Corrosion Based on Soil Analysis

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

Step 2: use AASHTO flowchart Figure 4 (Ref. 3) to determine the recommended corrosion assessment procedure for the existing piles.

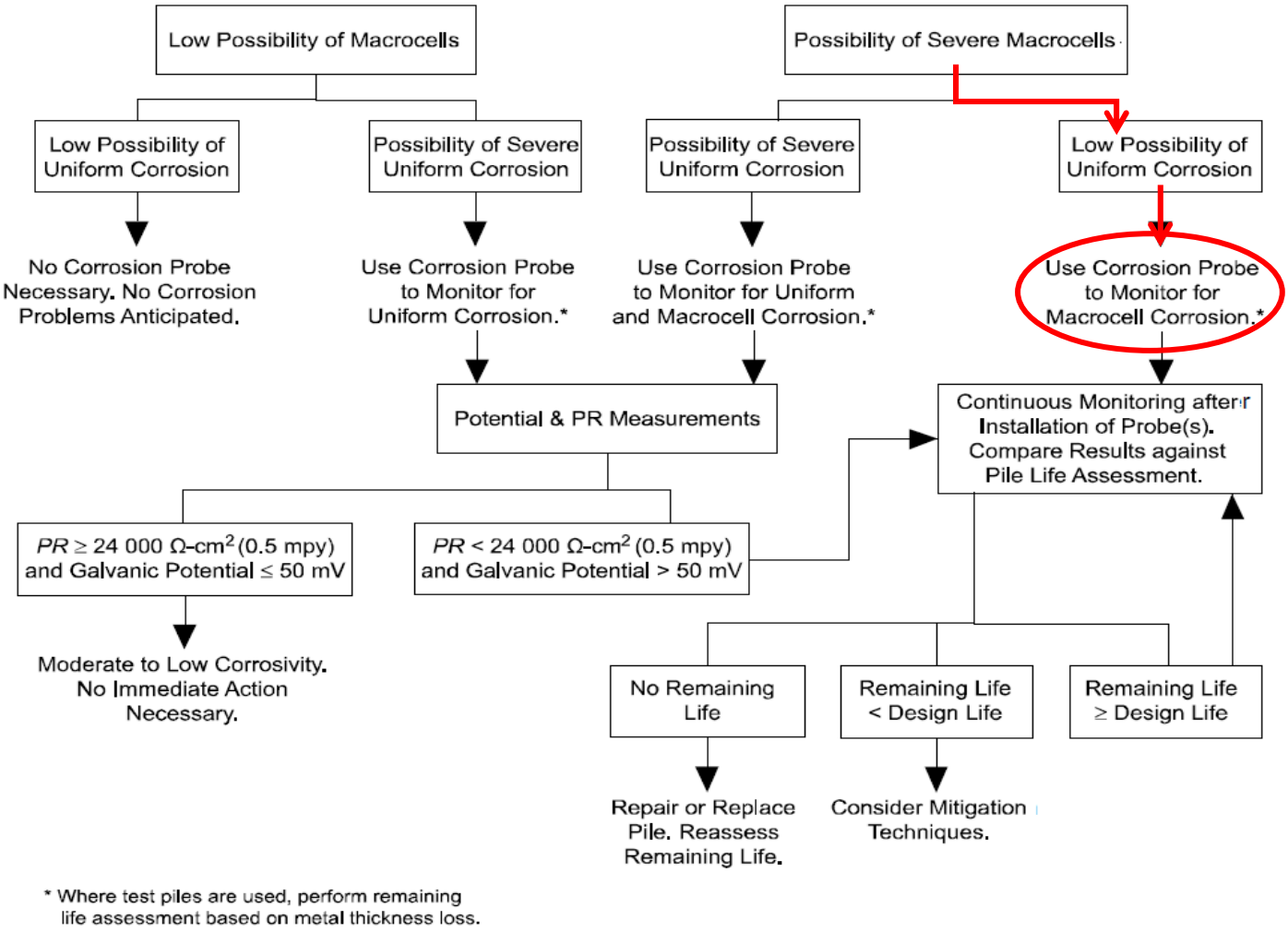


Figure 4—Determination of the Necessity for Electrochemical Testing, Corrosion Monitoring, and Mitigation

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion.

Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP will be used to estimate corrosion loss of the existing piles.

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

C. Estimate corrosion loss of the pile steel and calculate the resulting reduced pile section area.

Linear methods:

Per FHWA GEC-12 Driven Pile Foundations (Ref. 4), Section 6.12.1.1, for steel piles buried in fill or disturbed natural soils, a conservative estimate of the corrosion rate is 0.003 inches per year.

Corrosion loss per year = 0.003 inches

Per NCHRP Report 675 LRFD Metal Loss (Ref 5), Figure 8, for plain steel elements and soil resistivity less than 3,000 ohm-cm, an average corrosion rate is 13.5 µm/yr = 0.0005 in/yr.

Corrosion loss per year = 0.0005 inches

Corrosion loss per year with FS of 2 = 0.0011 inches

Estimate the total corrosion loss that the existing piles may experience over the desired remaining service life of 50 years:

Desired remaining service life = 50 years (2024 to 2074)

	NCHRP	FHWA		
Linear rate of corrosion loss =	0.0011	0.003	inches per year	per steel face or side
Future corrosion loss =	0.05	0.15	inches over 50 years	per steel face or side

Non-linear methods:

Per FHWA Corrosion of Soil Reinforcements (Ref. 6), Section 2.4.a, a general conclusion of studies on underground corrosion is that the rate of corrosion is greatest in the first few years of burial and then levels off to a steady but significantly lower rate. Thus, both Ref. 5 and Ref. 6 suggest a non-linear model of steel corrosion loss:

$$x = kt^n \qquad \text{(Ref. 5, Equation 1)}$$

in which x is the loss of steel thickness per side, k is a constant dependent on soil conditions (i.e., soil corrosivity), t is time in years, and n is a constant dependent on steel conditions (i.e., galvanized or plain).

k =	50	µm (Ref. 5, Table 5, for pH > 5 and resistivity between 700 and 2,000 ohm-cm)
n =	0.8	(Ref. 5 Eq. 3 for plain steel elements; Ref. 6 pg. 2-25 for non-galvanized steel)
t =	50	years
x =	1143	µm over the next 50 years
x =	0.05	inches over the next 50 years (future)

CALCULATIONS

Date:	8/1/2024	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	DEB
Subject:	Soil Corrosivity Evaluation for Bridge Foundations	**Reviewed by:**	CCB
Project Title:	MaineDOT Hogan Rd Bridge Phase II		

Pile section loss:

Using the non-linear total corrosion loss per side = 0.05 inches over the next 50 years

Pile size: HP 10x42 (Ref. 7, Sheet 10)

	Intact Section		Corroded Section	
Depth, d =	9.70	in (Ref. 8)	9.60	in
Width, b_f =	10.1	in (Ref. 8)	10.0	in
Web thickness, t_w =	0.415	in (Ref. 8)	0.315	in
Flange thickness, t_f =	0.420	in (Ref. 8)	0.320	in
Fillet area =	0.2	in^2	0.2	in^2
Section area, A =	12.4	in^2 (Ref. 8)	9.4	in^2

CONCLUSIONS

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion. Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP were used to estimate corrosion loss of the existing piles. Using a non-linear corrosion loss of 0.05 inches per side over the next 50 years (desired future life), the estimated corroded section area of the HP 10x42 piles would be 9.4 in^2, representing 76% of the intact section area. Per the AASHTO Standard Practice, the anticipated corrosion pattern is macrocell rather than uniform, so this estimated corroded section area may vary throughout the pile length.

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Estimate the corrosion that the proposed Hogan Road WB Bridge HP14x89 piles may experience over a design life of 100 years.

METHOD

Use references from AASHTO, FHWA, and NCHRP to determine potential corrosion rates based on the results of pH, electrical resistivity, chloride, and sulfate laboratory testing on soil samples obtained during the subsurface investigation at the site.

REFERENCES

1. GeoTesting Express. Laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes. Received by WSP on May 26, 2022, May 31, 2022, January 3, 2024, and March 5, 2024. Summarized in Table 4 and included in Appendix C of the WSP Geotechnical Design Report.

2. American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

3. AASHTO. Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications. Specification R 27-01 (2023). Technically Revised: 2001, Reviewed but Not Updated: 2023

4. Federal Highway Administration (FHWA). GEC 012: Design and Construction of Driven Pile Foundations – Volume I. Publication No. FHWA-NHI-16-009. Dated July 2016.

5. National Cooperative Highway Research Program (NCHRP). Report 675: LRFD Metal Loss and Service-Life Strength Reduction Factors for Metal-Reinforced Systems. 2011.

6. FHWA. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes. Publication No. FHWA-NHI-09-087. Dated November 2009.

7. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

8. Email from VHB titled Bangor Hogan Road New WB Bridge recieved on 12/02/2024.

ASSUMPTIONS

1. WSP assumes the pile design will consist of HP 14x89 steel piles.

ATTACHMENTS

1. Summary table of corrosivity laboratory test results for bridge and highway 100- and 200-series borings and 300-series hand probes

2. Draft boring location plan for bridge and highway 100- and 200-series borings and 300-series hand probes in the vicinity of the existing and proposed bridges

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Use the corrosivity laboratory test results for the soil at site (Ref. 1, also included as Attachment 1) to determine the corrosion potential of the proposed piles.

Per AASHTO LRFD Article 10.7.5 (Ref. 2), the following soil or water conditions should be considered as indicative of a potential pile deterioration or corrosion situation:

- Resistivity less than 2,000 ohm-cm
- pH less than 5.5
- pH between 5.5 and 8.5 in soils with high organic content
- Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)
- Chloride content greater than 500 ppm

The corrosivity laboratory test results for the soil at site (Ref. 1) are compared to these conditions:

Condition	Range from lab test results	Median from lab test results	Meets condition?
Resistivity less than 2,000 ohm-cm	1,038 ohm-cm to 7,666 ohm-cm	2,212 ohm-cm	Yes*
pH less than 5.5	6.4 to 8.2	7.5	No
pH between 5.5 and 8.5 in soils with high organic content	0.5% to 0.7% organic content	0.6% organic content	No
Sulfate concentration greater than 1,000 ppm (soil) or 500 ppm (water)	<10 ppm to 30 ppm	10 ppm	No
Chloride content greater than 500 ppm	<10 ppm to 120 ppm	48 ppm	No

*While the median value is above the 2,000 ohm-cm threshold, a sufficient number of test results are below 2,000 ohm-cm to warrant further consideration.

Since the lab test results only indicate a marginal potential for pile deterioration due to corrosion, the AASHTO Standard Practice for Assessment of Corrosion of Steel Piling for Non-Marine Applications (Ref. 3) is used to further assess corrosion potential.

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

B. Use the flowcharts from the AASHTO Standard Practice (Ref. 3) to determine the potential corrosion situation of the proposed piles.

Step 1: use AASHTO flowchart Figure 3 (Ref. 3) and the corrosivity laboratory test results for the soil at site (Ref. 1) to determine the possibility of macrocell and uniform corrosion of the proposed piles.

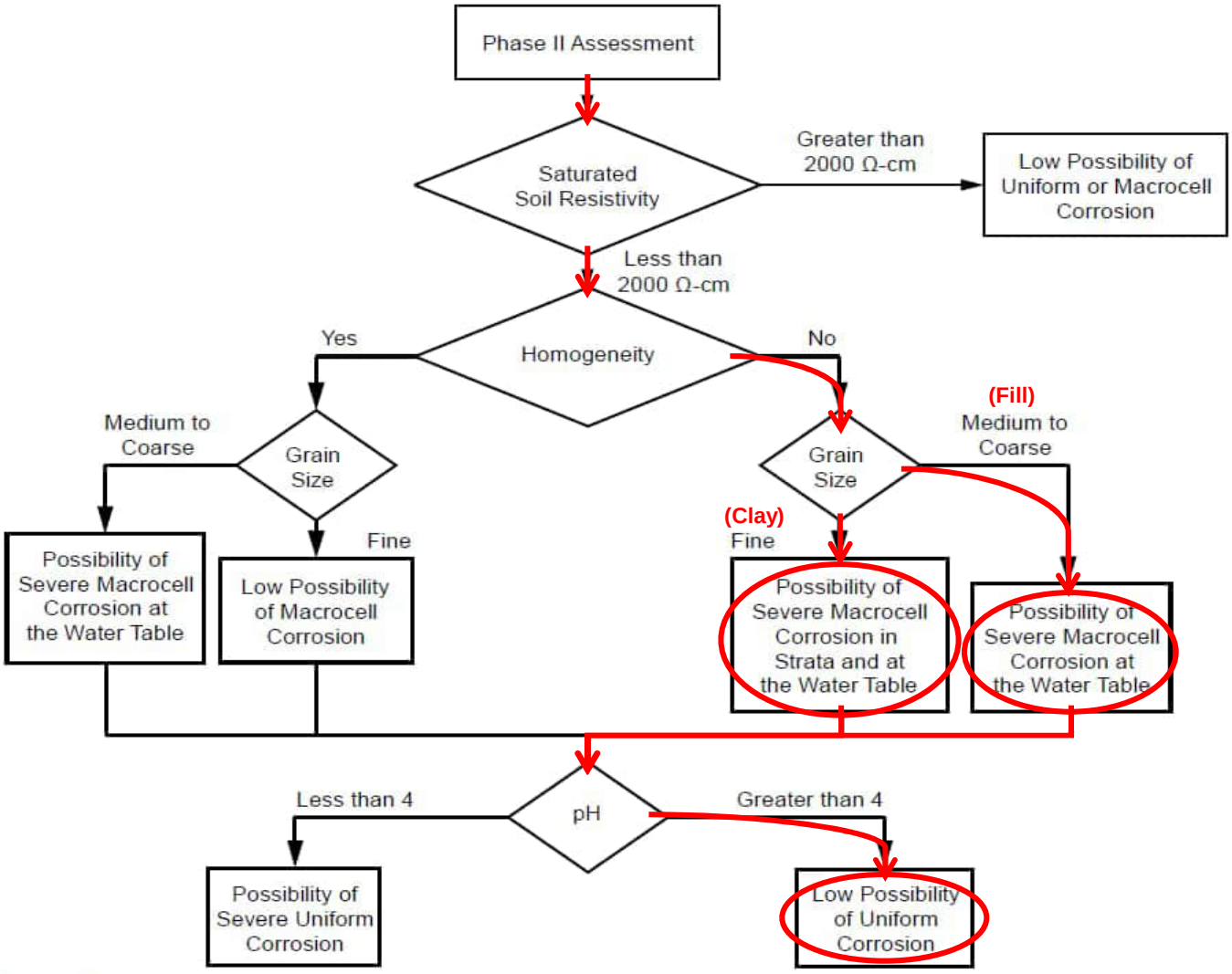


Figure 3—Determination of the Possibility for General and/or Macrocell Corrosion Based on Soil Analysis

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Step 2: use AASHTO flowchart Figure 4 (Ref. 3) to determine the recommended corrosion assessment procedure for the proposed piles.

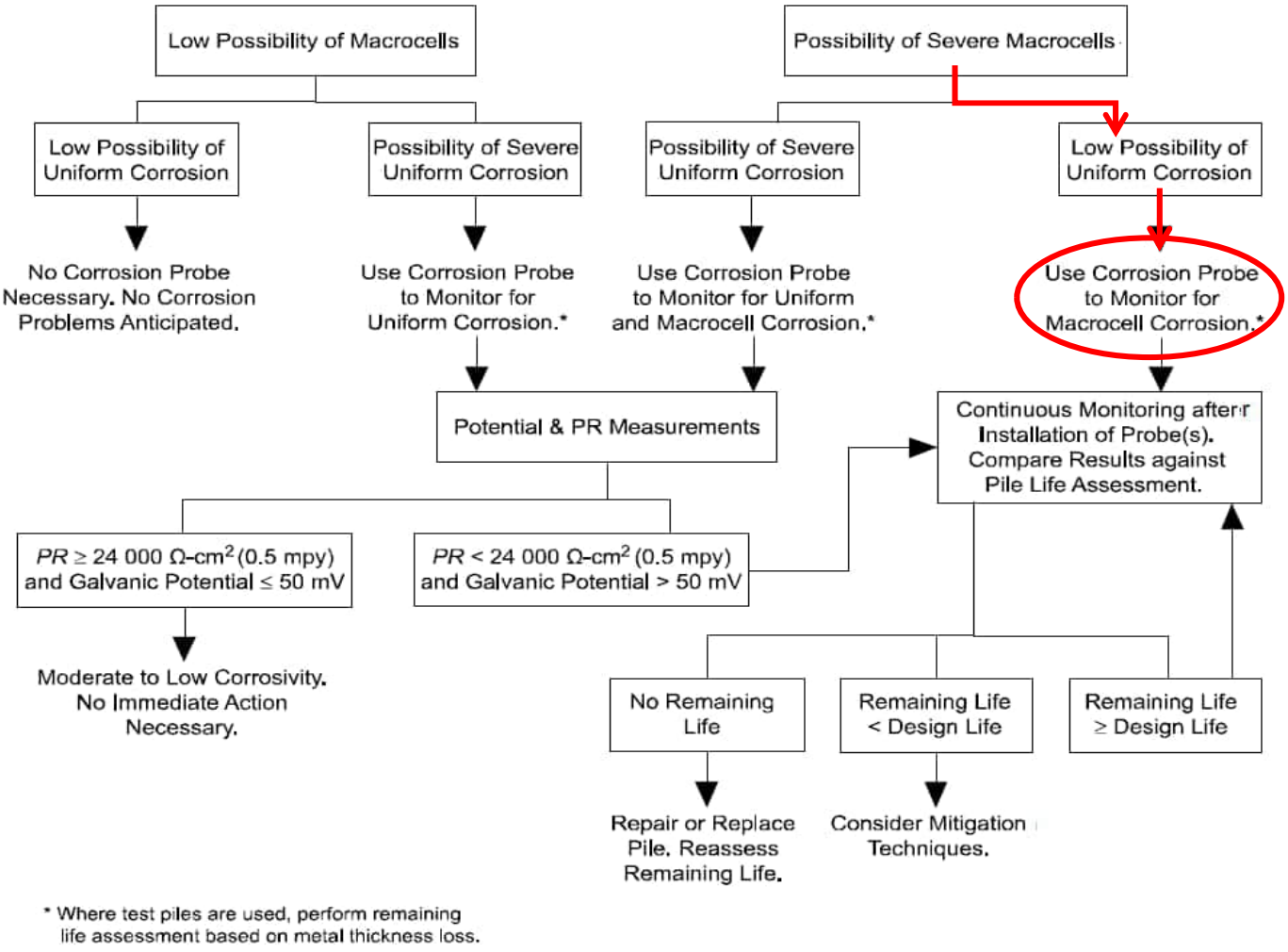


Figure 4—Determination of the Necessity for Electrochemical Testing, Corrosion Monitoring, and Mitigation

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion.

Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP will be used to estimate corrosion loss of the proposed piles.

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

C. Estimate corrosion loss of the pile steel and calculate the resulting reduced pile section area.

Linear methods:

Per FHWA GEC-12 Driven Pile Foundations (Ref. 4), Section 6.12.1.1, for steel piles buried in fill or disturbed natural soils, a conservative estimate of the corrosion rate is 0.003 inches per year.

Corrosion loss per year = 0.003 inches

Per NCHRP Report 675 LRFD Metal Loss (Ref 5), Figure 8, for plain steel elements and soil resistivity less than 3,000 ohm-cm, an average corrosion rate is 13.5 µm/yr = 0.0005 in/yr.

Corrosion loss per year = 0.0005 inches

Corrosion loss per year with FS of 2 = 0.0011 inches

Estimate the total corrosion loss that the proposed piles may experience over a design life of 100 years (Ref. 8):

Desired remaining service life = 100 years (2025 to 2125)

	NCHRP	FHWA		
Linear rate of corrosion loss =	0.0011	0.003	inches per year	per steel face or side
Future corrosion loss =	0.11	0.30	inches over 100 years	

Non-linear methods:

Per FHWA Corrosion of Soil Reinforcements (Ref. 6), Section 2.4.a, a general conclusion of studies on underground corrosion is that the rate of corrosion is greatest in the first few years of burial and then levels off to a steady but significantly lower rate. Thus, both Ref. 5 and Ref. 6 suggest a non-linear model of steel corrosion loss:

$$x = kt^n$$ (Ref. 5, Equation 1)

in which x is the loss of steel thickness per side, k is a constant dependent on soil conditions (i.e., soil corrosivity), t is time in years, and n is a constant dependent on steel conditions (i.e., galvanized or plain).

k =	50	µm (Ref. 5, Table 5, for pH > 5 and resistivity between 700 and 2,000 ohm-cm)
n =	0.8	(Ref. 5 Eq. 3 for plain steel elements; Ref. 6 pg. 2-25 for non-galvanized steel)
t =	100	years
x =	1991	µm over the next 100 years
x =	0.08	inches over the next 100 years (future)

CALCULATIONS

Date:	2/12/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Soil Corrosivity Evaluation for WB Bridge # 6787 Foundations	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Pile section loss:

Using the non-linear total corrosion loss per side = 0.08 inches over the next 100 years

Pile size: HP 14x89 (Assumption 1)

	Intact Section		Corroded Section	
Depth, d =	13.8	in (Ref. 7)	13.64	in
Width, b_f =	14.7	in (Ref. 7)	14.5	in
Web thickness, t_w =	0.615	in (Ref. 7)	0.455	in
Flange thickness, t_f =	0.615	in (Ref. 7)	0.455	in
Fillet area =	0.3	in^2	0.3	in^2
Section area, A =	26.1	in^2 (Ref. 7)	19.3	in^2

CONCLUSIONS

Based on the corrosivity lab test results, the AASHTO Standard Practice recommends using a corrosion probe to monitor for macrocell corrosion. Since a corrosion probe is not currently installed at site, several approximate corrosion rates from FHWA and NCHRP were used to estimate corrosion loss of the proposed piles. Using a non-linear corrosion loss of 0.08 inches per side over the next 100 years (design life), the estimated corroded section area of the HP 14x89 piles would be 19.3 in^2, representing 74% of the intact section area. Per the AASHTO Standard Practice, the anticipated corrosion pattern is macrocell rather than uniform, so this estimated corroded section area may vary throughout the pile length.

APPENDIX F

Global Stability Calculations

CALCULATIONS

Date:	9/15/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road New Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road No. 2 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine the global stability factor of safety for proposed conditions at the proposed Hogan Road No. 2 bridge abutments. The slope stability evaluations were made for shallow and deep failure surfaces. The global factor of safety was calculated at longitudinal profile and abutment transverse sections as follows: Back edge of the approach slab at Abutment 2, STA 215+15, and 50 feet behind the end of the approach slab at Abutment 2 (STA 215+65).

REFERENCES

1. WSP Soil and Rock Exploration Logs, included as Appendix A of the Geotechnical Design Report.

2. WSP Bridge N_{60} Summary of Design Properties titled "Lab Field Data Summary Plots V4 31404817.000.xlsx"

3. WSP Interpretive subsurface profile, included as Figure 3 of the of the Geotechnical Design Report.

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

5. Rocscience Slide2 Software Package Version 9.038 64bit, build date 2/28/2025.

6. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Edition.

7. WSP Seismic Site Class Analysis - Hogan Road Proposed Bridge titled "31404817.004 Hogan Rd Bridge Seismic Site Class.pdf", Dated 9/9/2025.

8. LRFD Seismic Bridge Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual. U.S. Dept. of Transportation, Federal Highway Administration, 2011.

9. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.

10. State of Maine Department of Transportation, US I-95 Exit 187/Hogan Road Interchange, Bangor, Maine (Sheets 1 - 226), Dated 09/2023 (WSP Draft plans)

11. GeoTesting Express laboratory testing results, included as Appendix C and summarized in Table 4, Table 5, and Table 6 of the Geotechnical Design Report.

12. WSP Lightweight Fill Analysis titled "WSP DRAFT Hogan Rd FGA Extents and Thicknesses with Cross-Sections 2024-09.pdf", Dated 09/2024.

13. United States Department of the Interior Geological Survey. 1963. Porosity and Bulk Density of Sedimentary Rocks. Geological Survey Bulletin 1144-E.

14. 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).

15. Maine Department of Transportation, Hogan Road Bridge over I-95, Bangor, Maine, 60% Bridge Plans, dated August 31st, 2024.

16. WSP DRAFT Hogan Road FGA Extents and Thicknesses with Cross-Sections 2025-02

ATTACHMENTS

1. Material Properties table.

2. Slide output figures.

wsp

CALCULATIONS

Date:	9/15/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road New Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road No. 2 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

ASSUMPTIONS

1. Circular surface searches were analyzed using the Spencer method with an auto refine search. Non-circular surfaces searches were analyzed using the Spencer method and cuckoo search with surface altering optimization.

2. As per Maine DOT Bridge Design Guide (Ref. 4) Section 5.9.2 a minimum allowable factor of safety (FS) of 1.5 is used for global stability of embankment slopes supporting abutment footings, and a minimum allowable FS of 1.3 is used for global stability for embankment slopes not supporting a structural element.

3. As per Maine Department of Transportation, the load applied by the road and traffic for final design conditions is modeled as a 2-foot equivalent load of soil (Ref. 4, Table 3-4): 2 x 125 pcf (granular borrow) = 250 psf.

4. For pseudo-static models, a horizontal seismic load of 0.5 x k_{av} was applied in the direction of the slope failure as per FHWA GEC-3 2011 (Ref. 8, Section 6.2.2). The value of k_{av} (height adjusted horizontal peak ground acceleration coefficient scaled by site class factor) was calculated in Ref. 7.

5. As per FHWA Design Guide (Ref. 9) a minimum allowable factor of safety (FS) of 1.1 is recommended for pseudo-static conditions. For evaluations, a FS ≥ 1.1 for pseudo-static loading is recommended for potential failure surfaces on the abutment slopes parallel to the roadway.

6. For the longitudinal slope stability model, deep failure surfaces were considered to be failure surfaces that went underneath the proposed abutments and surficial surfaces were considered to be failure surfaces in the slope in front of the abutments. For the Transverse slope stability models deep failure surfaces were considered to be failure surfaces that passed through in-situ soils and shallow failure surfaces were considered to be failure surfaces that were at least 1 foot deep and traveled only through proposed fill materials.

7. The existing grading, proposed grading and construction design features were obtained from the Proposed Plans (Ref. 10).

8. Subsurface stratigraphy and water table were determined by interpolation between the nearest boring locations to the cross-section and profile (Ref. 1 and 3).

9. Infinite Strength conditions with sliding allowed along the layer boundary was assumed for the bedrock layers.

10. Topsoil at the proposed abutment locations will be stripped and removed prior to any construction. Topsoil originally present in these areas will be replaced with structural fill. Areas between structures still contain topsoil.

11. Granular borrow will be used as the proposed slope fill to cover on the front slope. Lightweight fill extents and thicknesses were based on Ref. 12 and Ref. 16.

12. The approach embankment slopes for Abutment 1 were determined by inspection to be not significant enough to warrant global stability models.

CALCULATION

1. Determine input parameters to build the soil model in Slide2.

The material parameters selected for use in the Slide2 models are shown in Table 1, and in Attachment 1 Figure 1.

a. The cohesion parameters for the stiff to very stiff clay and soft to medium stiff clay were based off the undrained shear strength values from correlation from the plasticity index values obtained from laboratory testing, and the field vane shear testing summarized in Tab "Bridge N60" of Ref. 2.

b. The friction angle parameters for the existing fill, silt, 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. 1) 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, 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. 11).

CALCULATIONS

Date:	9/15/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road New Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road No. 2 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

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. 4. The gravel borrow friction angle was used for aggregate subbase. The friction angles for the Heavy Riprap and Pavement layers were determined from engineering judgement and past experience.

e. The unit weight parameter for the metawacke bedrock encountered at the site was selected as 158 pcf based on Ref. 13, Table 1.

f. The friction angle and unit weight parameters for the Foamed Glass Aggregate were selected based on engineering judgement and Ref.14.

g. 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.

Table 1: Material Properties

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)[1]	Friction Angle (°)[2]
Pavement	140	Mohr-Coulomb	0	45
Topsoil	110	Mohr-Coulomb	300	0
Existing Fill	125	Mohr-Coulomb	0	34
Fill (Granular Borrow)[3]	125	Mohr-Coulomb	0	32
Fill (Aggregate subbase)[3]	135	Mohr-Coulomb	0	36
Heavy Riprap[3]	140	Mohr-Coulomb	0	42
Foamed Glass Aggregate	18	Mohr-Coulomb	0	35.5
Silt	125	Mohr-Coulomb	0	30
Stiff Clay	122	Undrained (Cohesion Type: Constant)	1340	0
Medium Clay	121	Undrained (Cohesion Type: Constant)	888	0
Glacial Till	130	Mohr-Coulomb	0	35
Bedrock	158	Infinite Strength	-	-
Concrete	150	Infinite Strength	-	-

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. [3] Proposed material.

CALCULATIONS

Date:	9/15/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road New Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road No. 2 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

2. Model the proposed geometry and interpreted stratigraphy in Slide2.

a. The proposed ground surface and interpreted stratigraphy for the longitudinal model was derived from the WSP interpretive subsurface model (Reference 3) and the proposed abutments were derived from the 60% Bridge Plans (Reference 15) to evaluate the proposed embankment geometry in the Slide2 software.

b. The proposed embankment geometry evaluated in the Slide2 software was taken from the Sections at the Stations corresponding to the back of the approach slab and 50 feet behind the approach slab included in the Highway Draft Plans (Ref. 10).

3. Use the material properties and proposed embankment geometry to analyze the global stability with Slide2.

The results of the Slide2 global stability analyses completed along the longitudinal sections on the proposed Hogan Road Westbound Abutments between STA. 209+00 and STA. 216+00 for the proposed conditions are summarized in the following table. The analyses were made for deep failure surfaces defined as potential failure surfaces under the abutments, and surficial surfaces on the foreslope in front of the abutments, for static and pseudo-static loading.

			Global Factor of Safety		
Location	**Condition**	**Surface Type**	**Deep Surface**	**Foreslopes**	**Figure # (Non-Circular)**
Abutment 1	Static loading	Non-Circular	1.68	1.39	1A / 1B
		Circular	1.93	1.49	-
	Pseudo-static loading	Non-Circular	1.54	1.26	1C / 1D
		Circular	1.70	1.34	-
Abutment 2	Static loading	Non-Circular	1.67	1.33	2A / 2B
		Circular	1.78	1.39	-
	Pseudo-static loading	Non-Circular	1.47	1.19	2C / 2D
		Circular	1.58	1.25	-

The results of the Slide2 global stability analyses completed along the transverse sections behind the proposed Hogan Road West Bound Abutment 2 at STA. 215+15 and STA. 215+65 for the proposed conditions are summarized in the following table.

Location	**Condition**	**Surface Type**	**Global Factor of Safety**	**Figure #**
Abutment 2 - Back edge of the approach slab	Static loading	Non-Circular	1.30	3A
		Circular	1.32	-
	Pseudo-static loading	Non-Circular	1.18	3B
		Circular	1.19	-
Abutment 2 - 50 feet behind the end of the approach slab	Static loading	Non-Circular	1.32	4A
		Circular	1.35	-
	Pseudo-static loading	Non-Circular	1.19	4B
		Circular	1.22	-

CALCULATIONS

Date:	9/15/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road New Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road No. 2 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

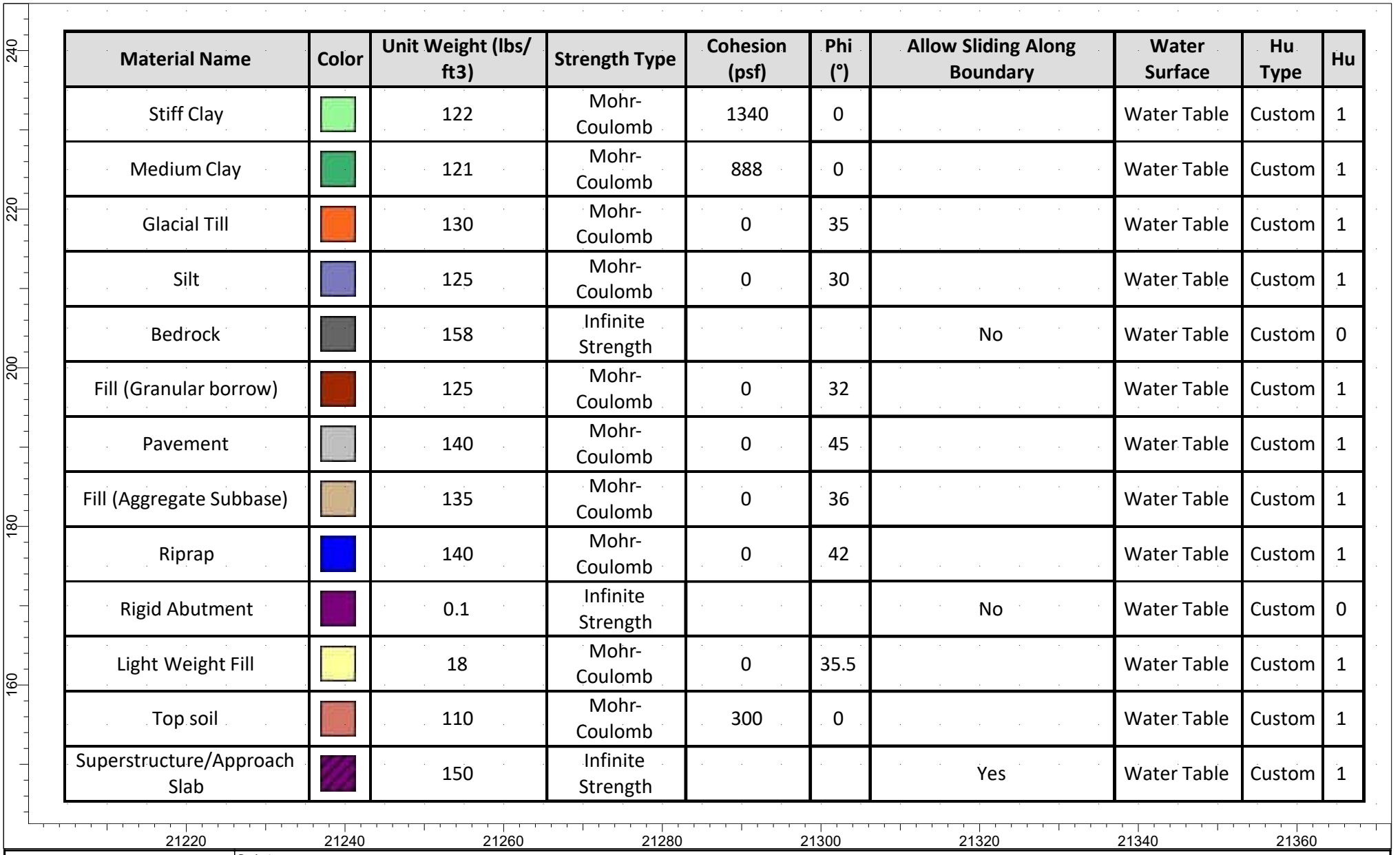
Conclusions

1. The analyzed global stability of the proposed site conditions along the Hogan Road (WB) profile from STA. 209+00 to STA. 216+00 have a computed factor of safety greater than the recommended FS of 1.5 against deep seated failures under static loading conditions. The proposed conditions meet the minimum FS of 1.1 against deep seated failures under pseudo-static loading conditions.

2. The analyzed global stability of the proposed foreslope conditions along the Hogan Road (WB) profile from STA. 209+00 to STA. 216+00 have a computed factor of safety greater than the recommended FS of 1.3 against foreslope failures under static loading conditions. The proposed conditions meet the minimum FS of 1.1 against foreslope failures under pseudo-static loading conditions.

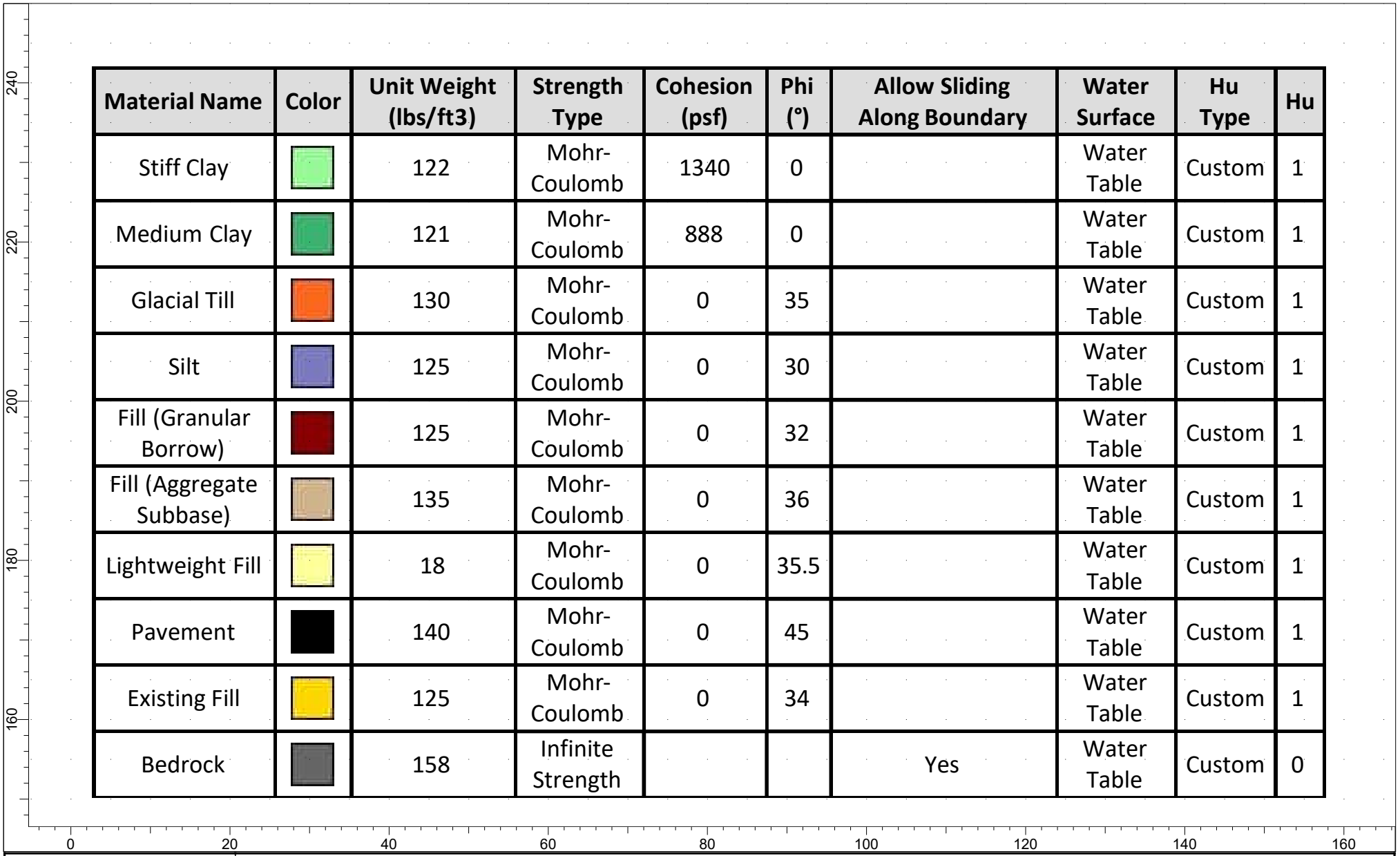
3. The analyzed sections transverse to Hogan Road (WB) at the back of the Abutment 2 approach slab and 50 feet behind the approach slab have a computed factor of safety that meets the recommend FS of 1.3 against embankment slope failures under static loading conditions. The proposed conditions meet the minimum FS of 1.1 against embankment failures under pseudo-static loading conditions.

Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu
Stiff Clay		122	Mohr-Coulomb	1340	0		Water Table	Custom	1
Medium Clay		121	Mohr-Coulomb	888	0		Water Table	Custom	1
Glacial Till		130	Mohr-Coulomb	0	35		Water Table	Custom	1
Silt		125	Mohr-Coulomb	0	30		Water Table	Custom	1
Bedrock		158	Infinite Strength			No	Water Table	Custom	0
Fill (Granular borrow)		125	Mohr-Coulomb	0	32		Water Table	Custom	1
Pavement		140	Mohr-Coulomb	0	45		Water Table	Custom	1
Fill (Aggregate Subbase)		135	Mohr-Coulomb	0	36		Water Table	Custom	1
Riprap		140	Mohr-Coulomb	0	42		Water Table	Custom	1
Rigid Abutment		0.1	Infinite Strength			No	Water Table	Custom	0
Light Weight Fill		18	Mohr-Coulomb	0	35.5		Water Table	Custom	1
Top soil		110	Mohr-Coulomb	300	0		Water Table	Custom	1
Superstructure/Approach Slab		150	Infinite Strength			Yes	Water Table	Custom	1



Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
Analysis Description	Material Properties			
Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 0A**
Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			

SLIDE 9.038

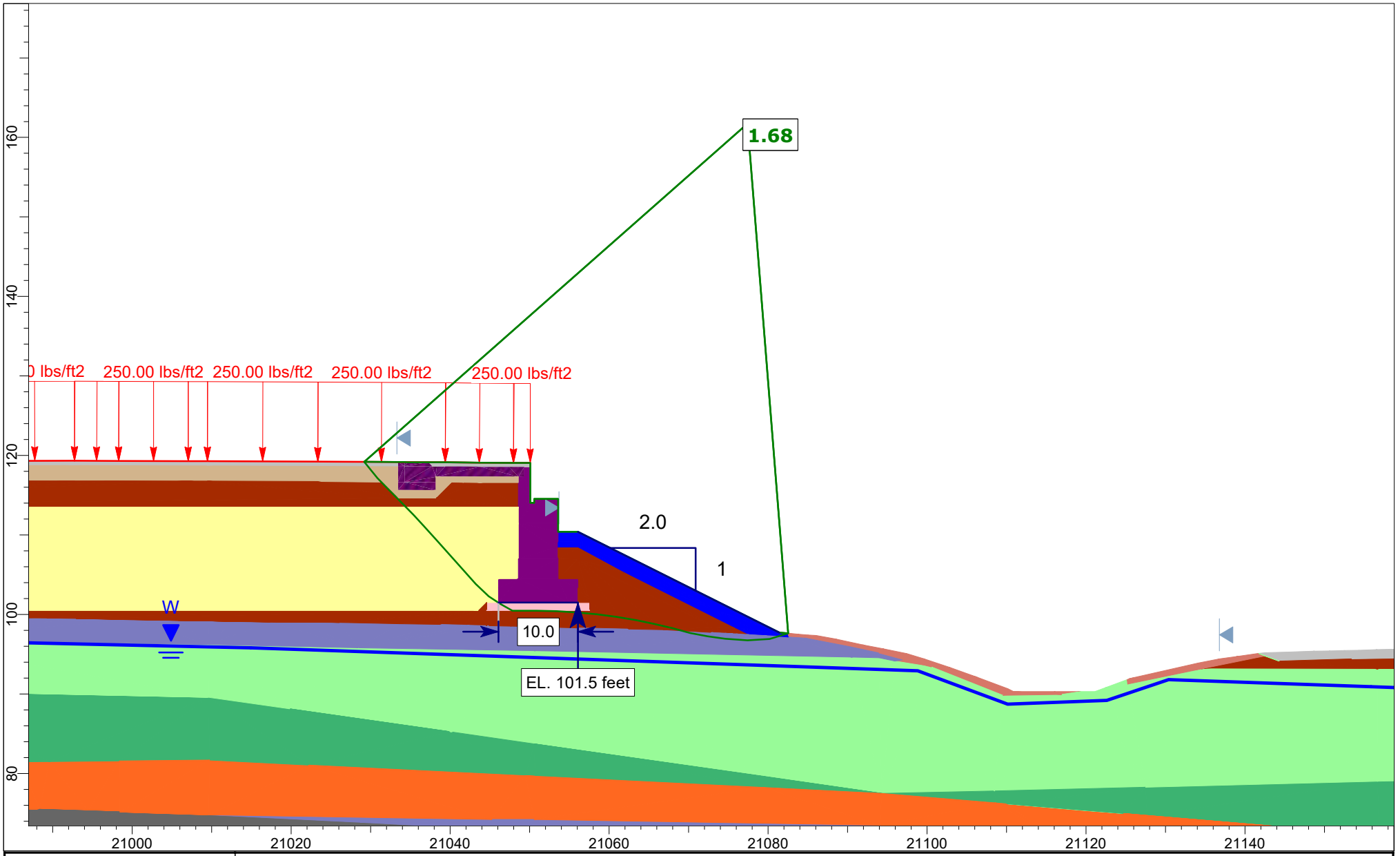


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu
Stiff Clay		122	Mohr-Coulomb	1340	0		Water Table	Custom	1
Medium Clay		121	Mohr-Coulomb	888	0		Water Table	Custom	1
Glacial Till		130	Mohr-Coulomb	0	35		Water Table	Custom	1
Silt		125	Mohr-Coulomb	0	30		Water Table	Custom	1
Fill (Granular Borrow)		125	Mohr-Coulomb	0	32		Water Table	Custom	1
Fill (Aggregate Subbase)		135	Mohr-Coulomb	0	36		Water Table	Custom	1
Lightweight Fill		18	Mohr-Coulomb	0	35.5		Water Table	Custom	1
Pavement		140	Mohr-Coulomb	0	45		Water Table	Custom	1
Existing Fill		125	Mohr-Coulomb	0	34		Water Table	Custom	1
Bedrock		158	Infinite Strength			Yes	Water Table	Custom	0

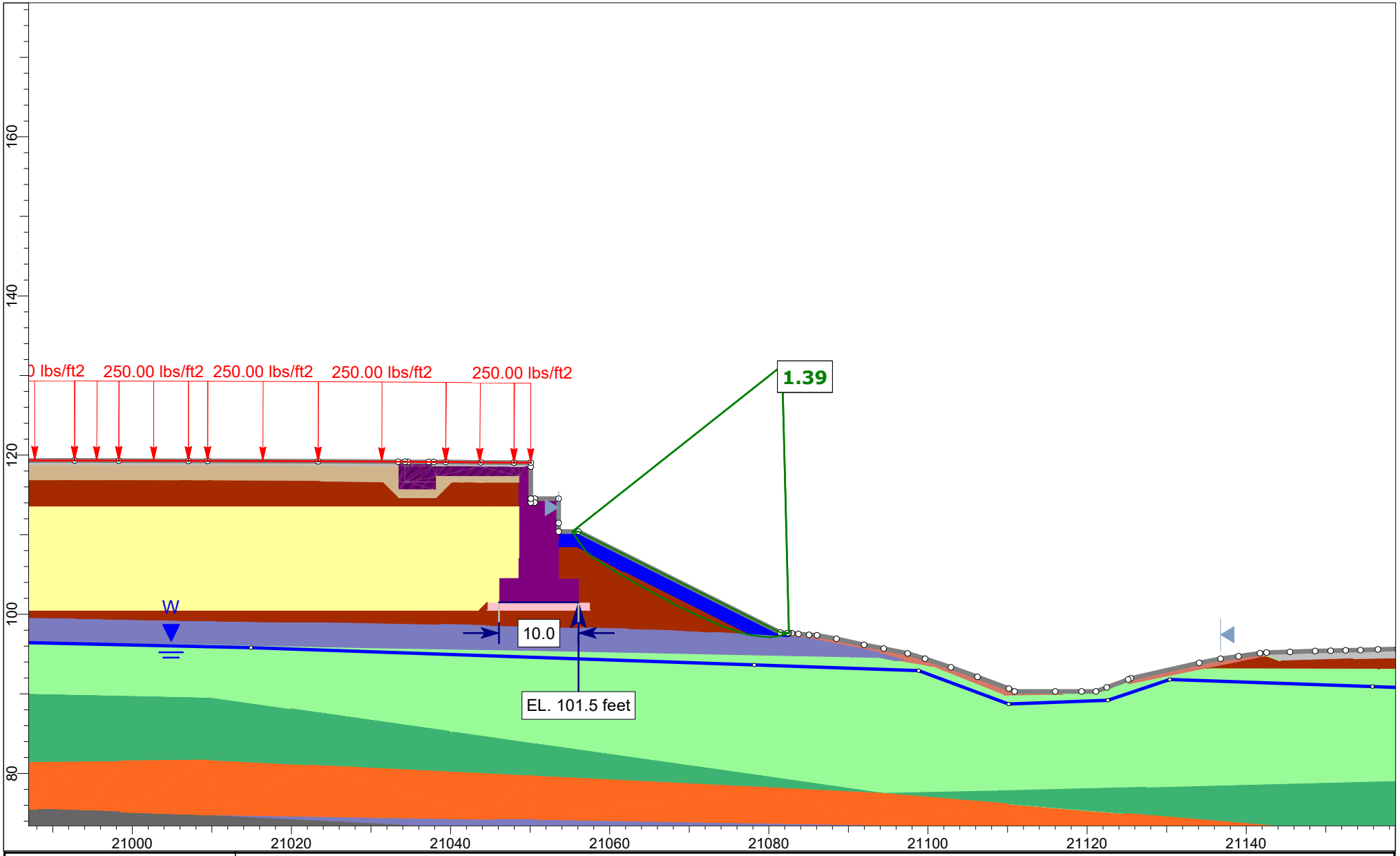
wsp

SLIDE 9.038

Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		
Analysis Description	Abutment 2 Transverse Stability at the Back of the Approach Slab		
Drawn By: ATM	Checked By: LDN	Reviewed By: CCB	Scale: 1:200
Date: 9/18/2025	File Name: Transverse Stability - Abutment 2 Back of Approach Slab Sta. 215+15.slmd		**Figure 0B**



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 1 - Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 1A**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



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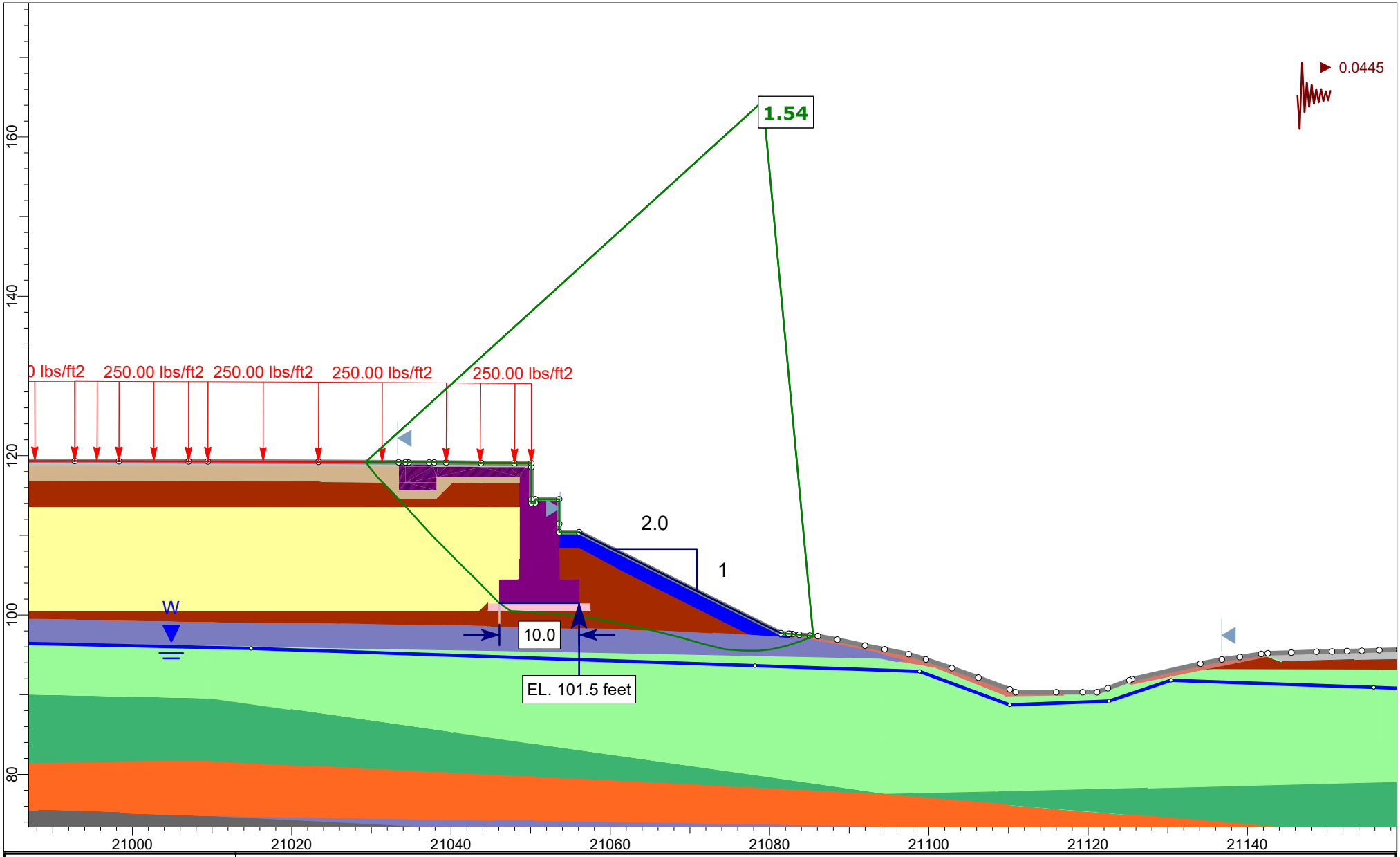
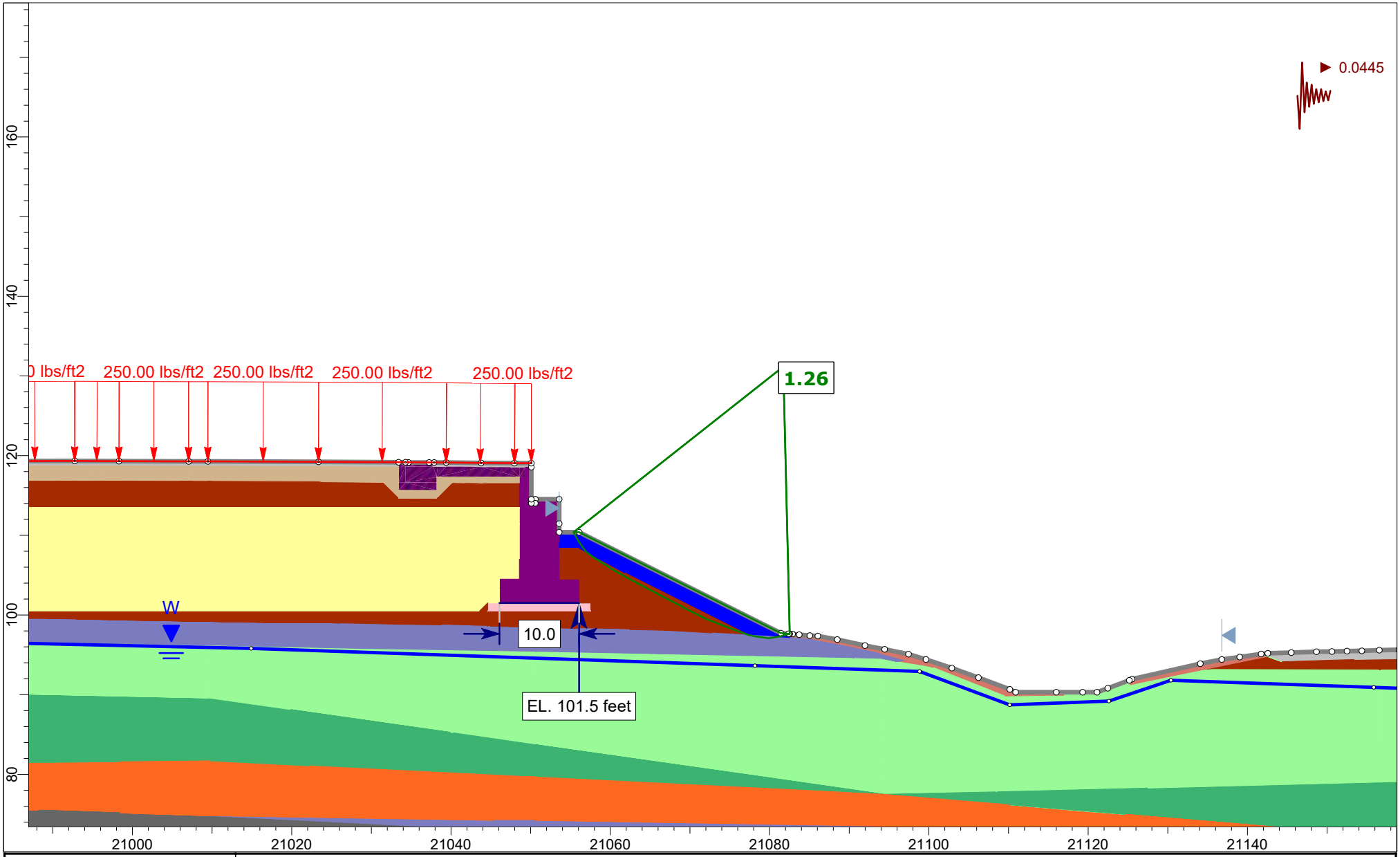
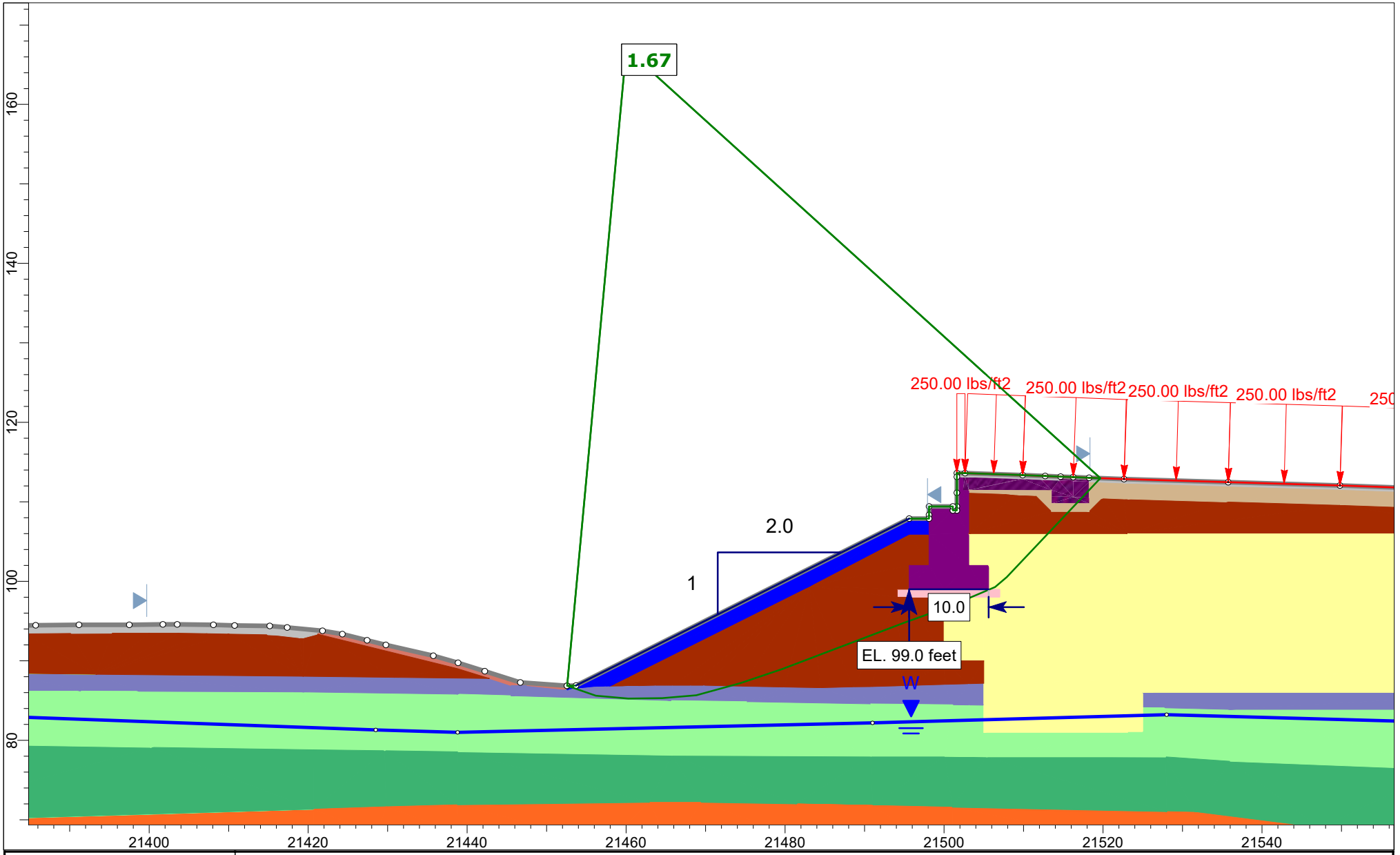
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 1 Foreslope - Static Non-Circular			
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd		**Figure 1B**



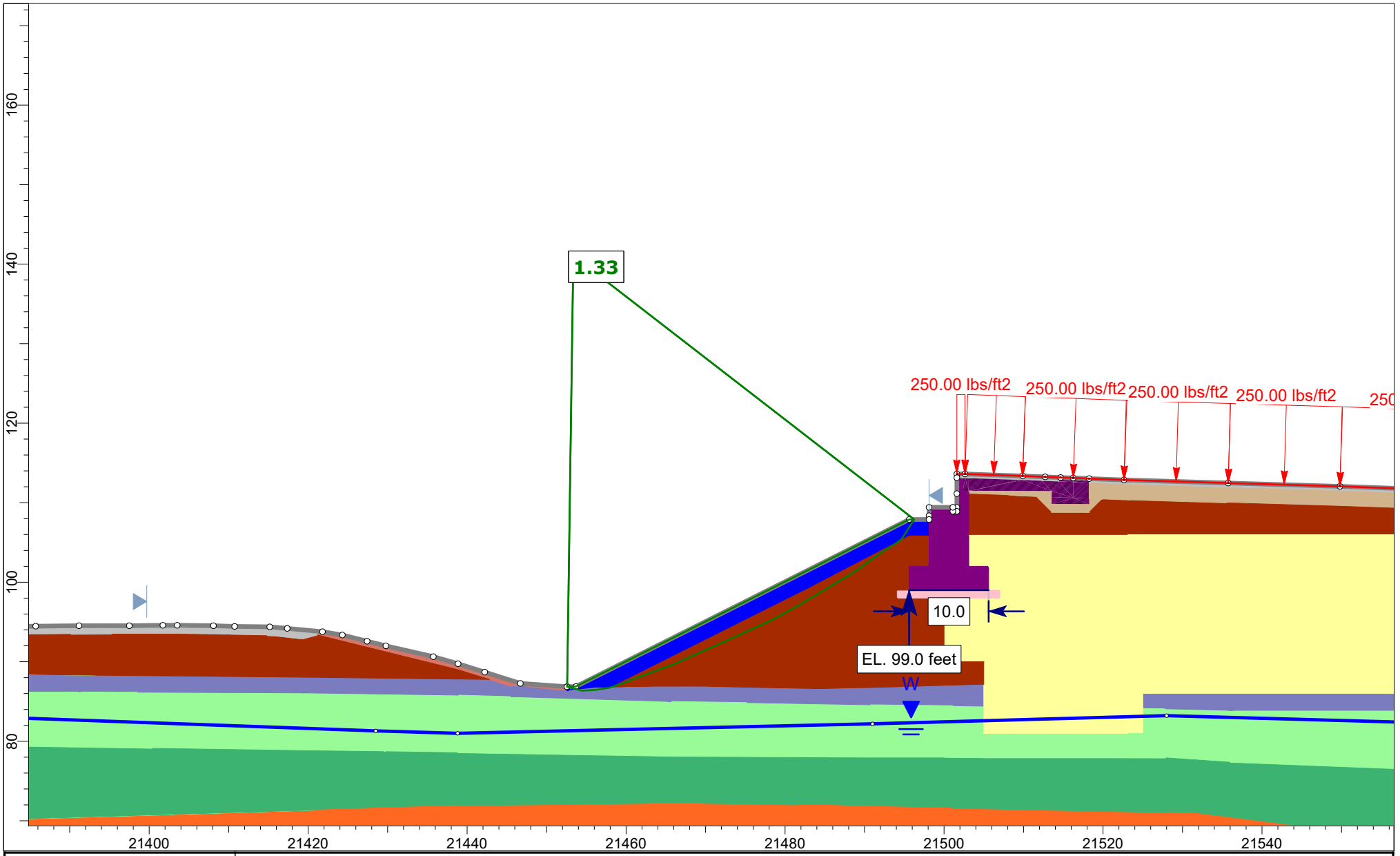
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 1 - Pseudo Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 1C**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



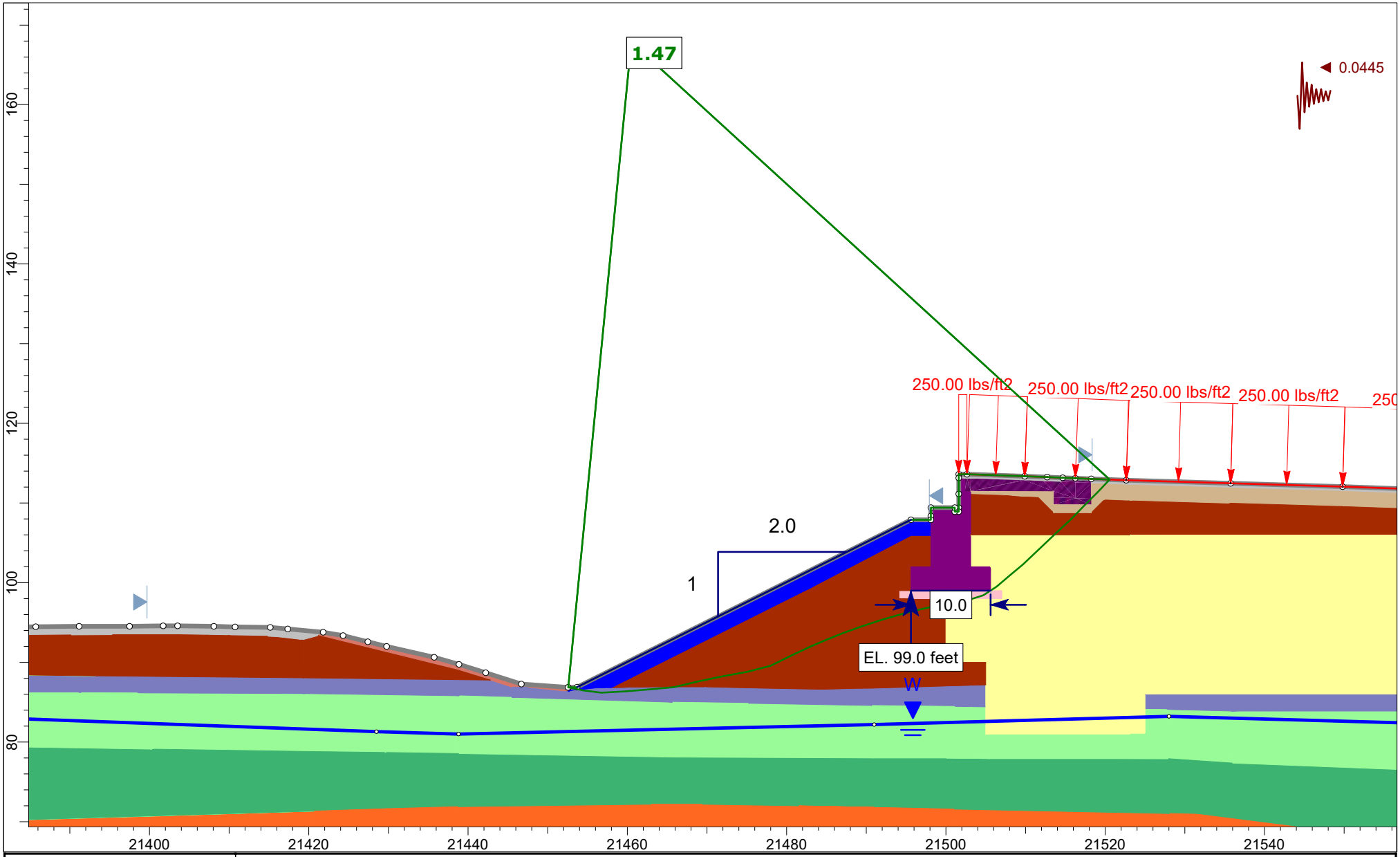
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 1 Foreslope - Pseudo Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 1D**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



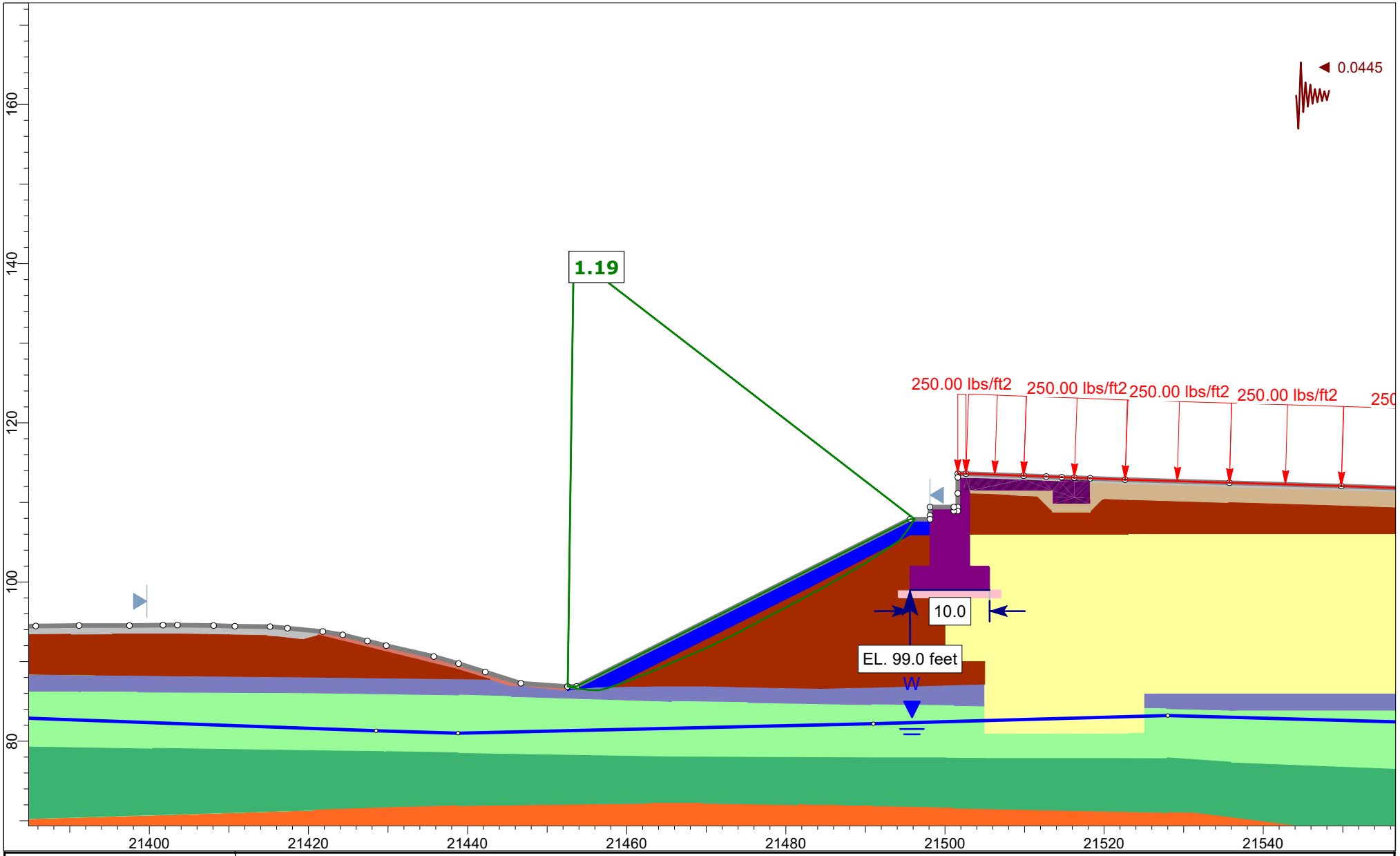
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 2 - Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 2A**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



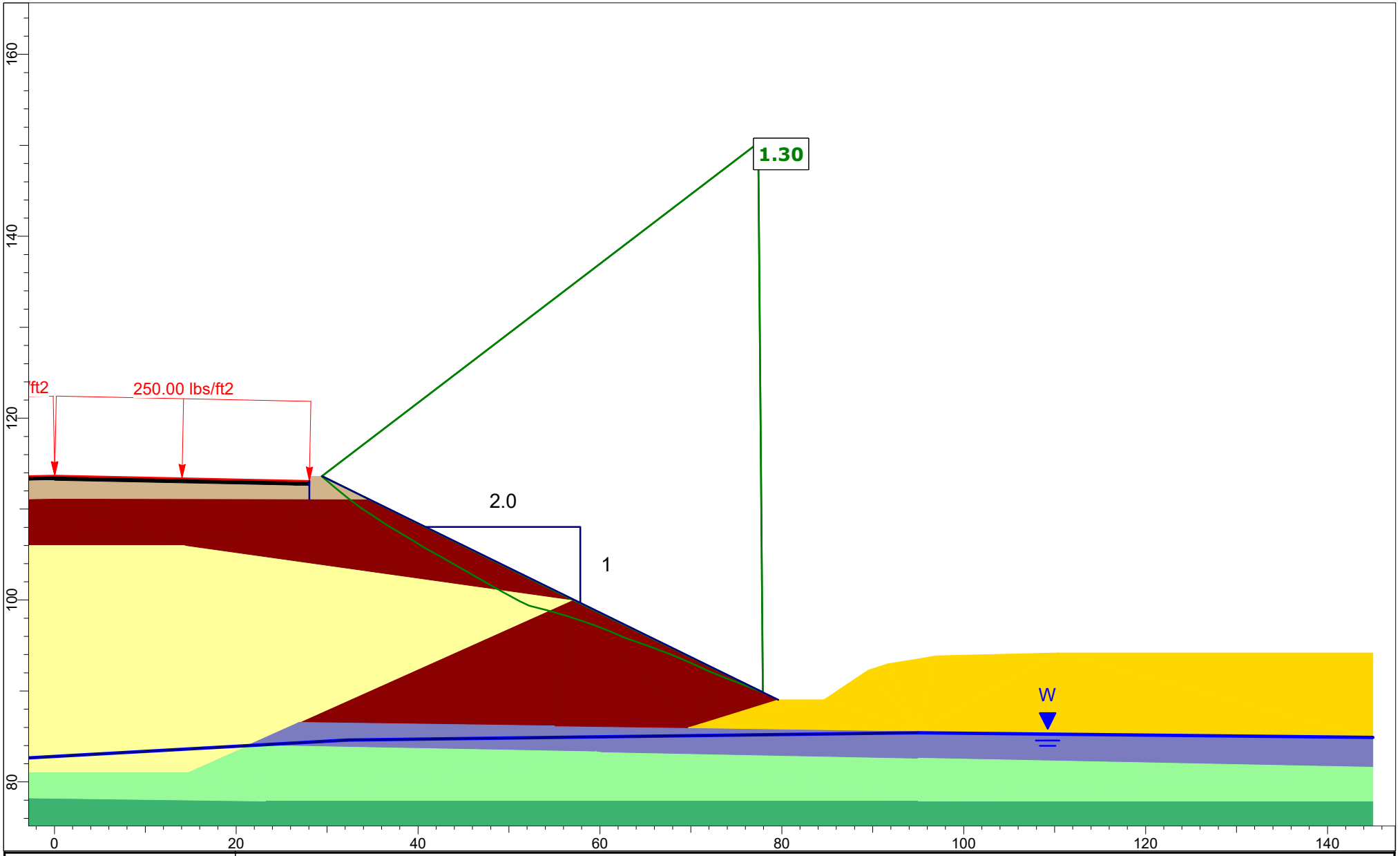
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 2 Foreslope - Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 2B**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



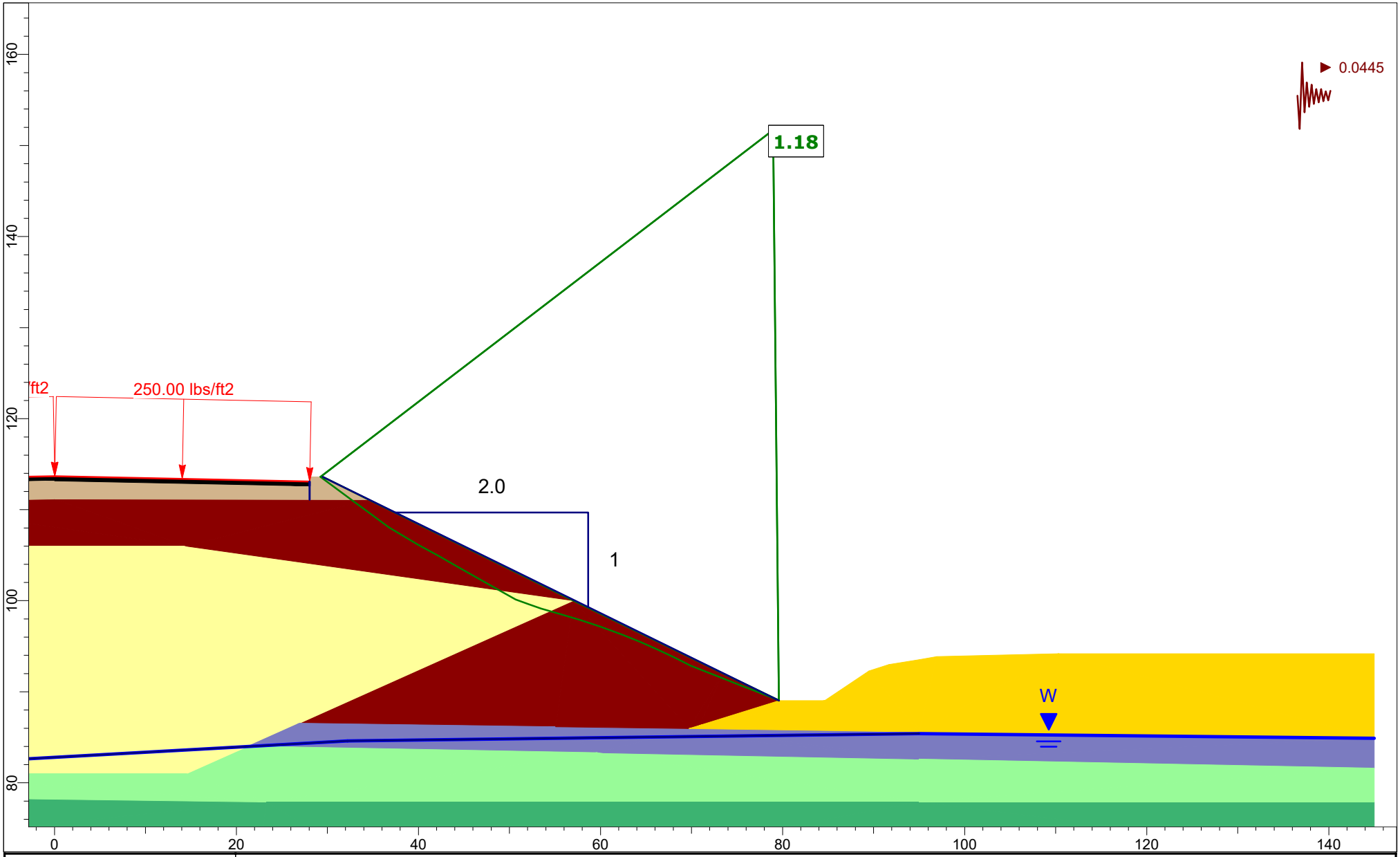
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 2 - Pseudo Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 2C**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



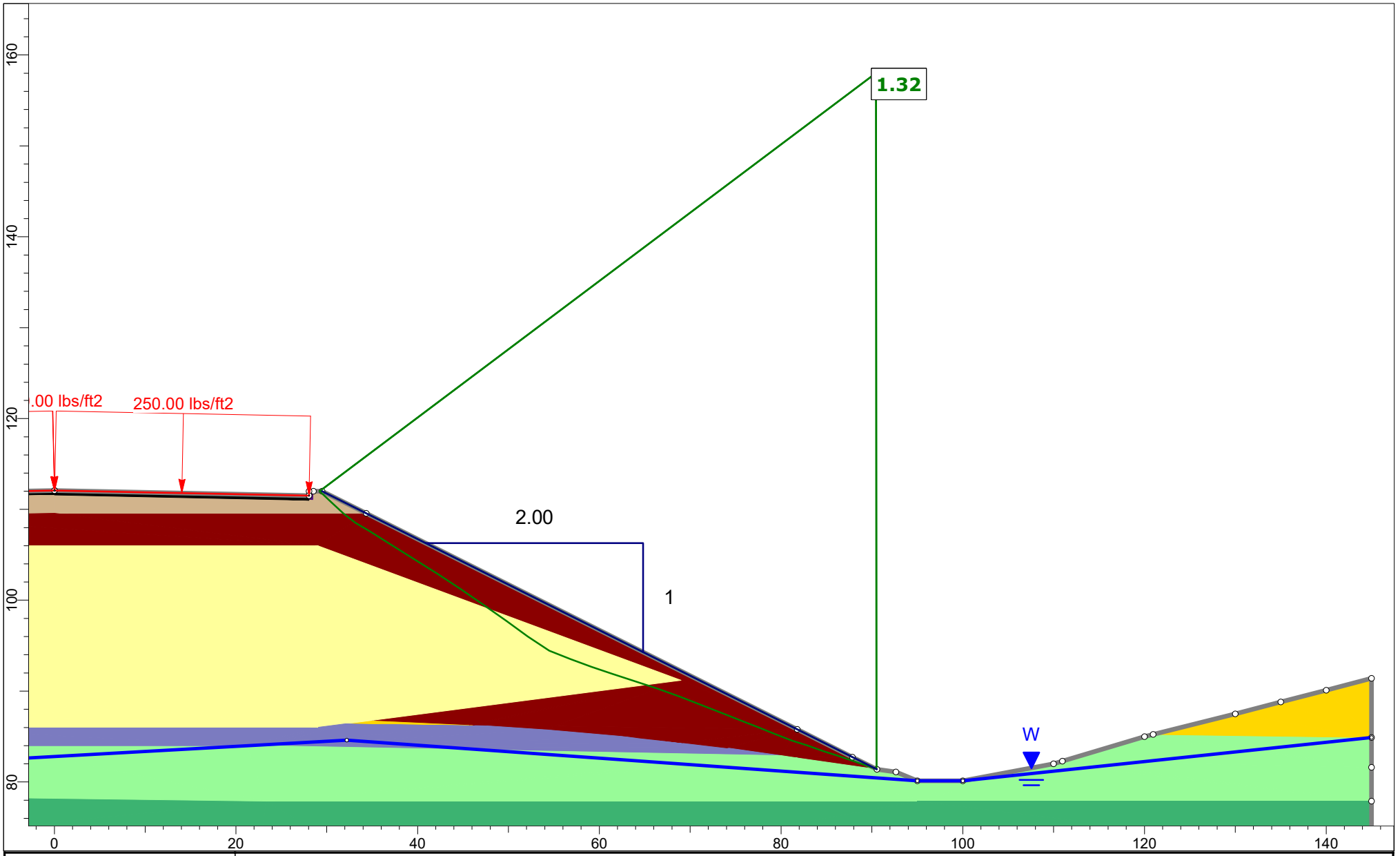
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road New Bridge Longitudinal Global Stability Abutment 2 Foreslope - Pseudo Static Non-Circular				
	Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:200	**Figure 2D**
SLIDEINTERPRET 9.038	Date: 9/18/2025	File Name: Hogan Road Proposed Bridge Longitudinal Model.slmd			



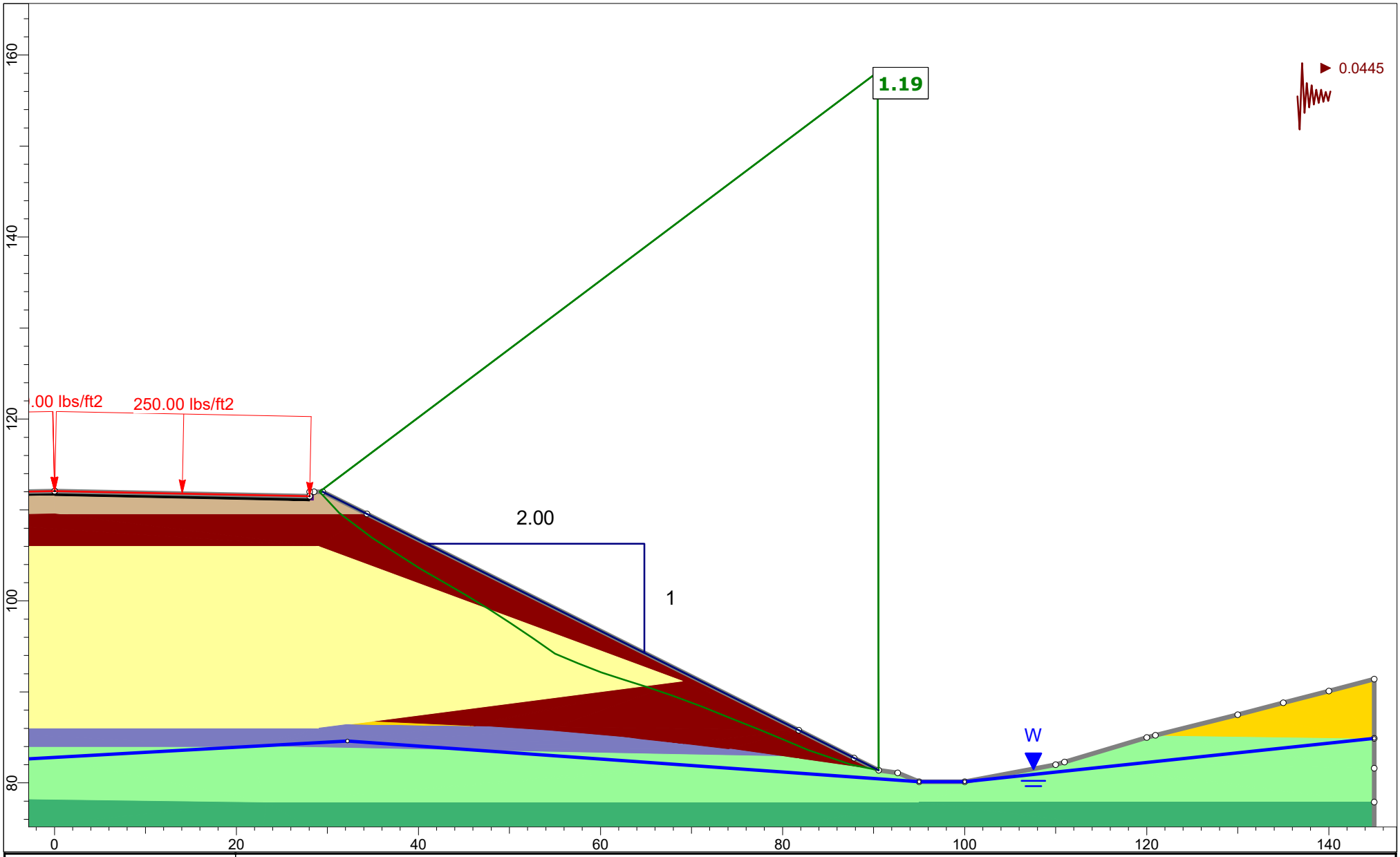
wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
	Analysis Description: Abutment 2 Transverse Stability at the Back of the Approach Slab			
	Drawn By: ATM	Checked By: LDN	Reviewed By: CCB	Scale: 1:175
SLIDEINTERPRET 9.038	Date: 9/19/2025	File Name: Transverse Stability - Abutment 2 Back of Approach Slab Sta. 215+15.slmd		**Figure 3A**



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME				
	Analysis Description: Abutment 2 Transverse Stability at the Back of the Approach Slab				
	Drawn By: ATM	Checked By: LDN	Reviewed By: CCB	Scale: 1:175	**Figure 3B**
SLIDEINTERPRET 9.038	Date: 9/19/2025	File Name: Transverse Stability - Abutment 2 Back of Approach Slab Sta. 215+15.slmd			



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME				
	Analysis Description: Abutment 2 Transverse Stability 50 feet Behind Approach Slab				
	Drawn By: ATM	Checked By: LDN	Reviewed By: CCB	Scale: 1:175	**Figure 4A**
SLIDEINTERPRET 9.038	Date: 9/19/2025	File Name: Transverse Stability - Abutment 2 Back of Approach Slab Sta. 215+15.slmd			



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
	Analysis Description: Abutment 2 Transverse Stability 50 feet Behind Approach Slab			
	Drawn By: ATM	Checked By: LDN	Reviewed By: CCB	Scale: 1:175
	Date: 9/19/2025	File Name: Transverse Stability - Abutment 2 Back of Approach Slab Sta. 215+15.slmd		**Figure 4B**

SLIDEINTERPRET 9.038

CALCULATIONS

Date:	9/16/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road Eastbound Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road I-95 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine the global stability factor of safety for proposed conditions at the Eastbound Hogan Road bridge abutments. The slope stability evaluations were made for shallow and deep failure surfaces. The global factor of safety was calculated at longitudinal profile sections.

REFERENCES

1. WSP Soil and Rock Exploration Logs, included as Appendix A of the Geotechnical Design Report.

2. WSP Bridge N_{60} Summary of Design Properties titled "Lab Field Data Summary Plots V4 31404817.000.xlsx"

3. WSP Interpretive Subsurface Profile D-D, included in the Geotechnical Design Report.

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

5. Rocscience Slide2 Software Package Version 9.038 64bit, build date 2/28/2025.

6. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Edition.

7. WSP Seismic Site Class Analysis - Hogan Road Proposed Bridge titled "31404817.004 Hogan Rd Bridge Seismic Site Class.pdf", Dated 9/9/2025.

8. LRFD Seismic Bridge Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual. U.S. Dept. of Transportation, Federal Highway Administration, 2011.

9. FHWA. August 2011. Geotechnical Engineering Circular No. 3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations: Reference Manual, Rev. 1. Publication No. FHWA-NHI-11-032.

10. State of Maine Department of Transportation, US I-95 Exit 187/Hogan Road Interchange, Bangor, Maine (Sheets 1 - 229), Dated 03/2025

11. GeoTesting Express laboratory testing results, included as Appendix C and summarized in Table 4, Table 5, and Table 6 of the Geotechnical Design Report.

12. United States Department of the Interior Geological Survey. 1963. Porosity and Bulk Density of Sedimentary Rocks. Geological Survey Bulletin 1144-E.

13. Maine Department of Transportation, Hogan Road Bridge over I-95, Bangor, Maine, 60% Bridge Plans, dated August 31st, 2024.

14. VHB, Bangor DDI Hogan Rd EB (1983) Bridge Bearing Loads, provided to WSP in the PDF titled "DDI Hogan EB Abut Brg Loads 2025-4-23" on April 23rd, 2025.

ATTACHMENTS

1. Material Properties table.

2. Slide output figures.

ASSUMPTIONS

1. Circular surface searches were analyzed using the Spencer method with an auto refine search. Non-circular surfaces searches were analyzed using the Spencer method and cuckoo search with surface altering optimization.

2. The Eastbound Bridge abutments are evaluated for longitudinal global stability using a minimum allowable factor of safety (FS) of 1.3. The minimum allowable FS was determined as per AASHTO LRFD (Ref. 6) Section 11.6.3.7 and an understanding that the existing bridge structure with similar geometry and embankments has been in place without slope stability issues subsequent to its construction. This analysis also accounts for the additional lateral resistance that the proposed piles will contribute to the overall stability of the abutments.

wsp

CALCULATIONS

Date:	9/16/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road Eastbound Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road I-95 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

3. As per Maine Department of Transportation, the load applied by the road and traffic for final design conditions is modeled as a 2-foot equivalent load of soil (Ref. 4, Table 3-4): 2 x 125 pcf (granular borrow) = 250 psf.

4. For pseudo-static models, a horizontal seismic load of 0.5 x k_{av} was applied in the direction of the slope failure as per FHWA GEC-3 2011 (Ref. 9, Section 6.2.2). The value of k_{av} (height adjusted horizontal peak ground acceleration coefficient scaled by site class factor) was calculated in Ref. 7.

5. As per FHWA Design Guide (Ref. 9) a minimum allowable factor of safety (FS) of 1.1 is recommended for pseudo-static conditions. For evaluations, a FS ≥ 1.1 for pseudo-static loading is recommended for potential failure surfaces on the abutment slopes parallel to the roadway.

6. For the longitudinal slope stability model, deep failure surfaces were considered to be failure surfaces that went underneath the proposed abutments and surficial surfaces were considered to be failure surfaces in the slope in front of

7. The existing grading, proposed grading and construction design features were obtained from the Proposed Plans (Ref.

8. Subsurface stratigraphy and water table were determined by interpolation between the nearest boring locations to the cross-section and profile (Ref. 1 and 3).

9. Infinite Strength conditions with sliding allowed along the layer boundary was assumed for the bedrock and concrete. The rigid abutment section was modeled with a unit weight of 0.1 pcf as it will be supported on piles. The concrete for the superstructure and approach slab concrete was modeled with a unit weight of 150pcf.

10. Topsoil at the proposed abutment locations will be stripped and removed prior to any construction. Topsoil originally present in these areas will be replaced with structural fill. Areas between structures still contain topsoil.

CALCULATION

1. Determine input parameters to build the soil model in Slide2.

The material parameters selected for use in the Slide2 models are shown in Table 1, and in Attachment 1 Figure 1.

a. The cohesion parameters for the stiff clay were based off the undrained shear strength values from correlation from the plasticity index values obtained from laboratory testing, and the field vane shear testing summarized in Tab "Bridge N60" of Ref. 2.

b. The friction angle parameters for the existing fill, and glacial till layers are based on empirical correlation to the average of the N_{60}-values encountered in nearby borings for each layer (Ref. 1) 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 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. 11).

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. 4. The gravel borrow friction angle was used for aggregate subbase. The friction angles for the Crushed Stone for Slope Protection and Pavement layers were determined from engineering judgement and past experience.

e. The unit weight parameter for the metawacke bedrock encountered at the site was selected as 158 pcf based on Ref. 12, 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.

CALCULATIONS

Date:	9/16/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road Eastbound Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road I-95 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

Table 1: Material Properties

Material Name	Unit Weight (pcf)	Strength Type	Cohesion (psf)[1]	Friction Angle (°)[2]
Pavement	140	Mohr-Coulomb	0	45
Topsoil	110	Mohr-Coulomb	300	0
Existing Fill	125	Mohr-Coulomb	0	34
Fill (Granular Borrow)[3]	125	Mohr-Coulomb	0	32
Fill (Aggregate subbase)[3]	135	Mohr-Coulomb	0	36
Crushed Stone for Slope Protection[3]	140	Mohr-Coulomb	0	42
Crushed Stone below Abutment	130	Mohr-Coulomb	0	36
Stiff Clay	122	Undrained (Cohesion Type: Constant)	1340	0
Glacial Till	130	Mohr-Coulomb	0	35
Bedrock	158	Infinite Strength	-	-
Concrete	150	Infinite Strength	-	-

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. [3] Proposed material.

2. Model the proposed geometry and interpreted stratigraphy in Slide2.

The proposed ground surface and interpreted stratigraphy for the longitudinal model was derived from the WSP interpretive subsurface model (Reference 3) and the proposed abutment geometry was derived from the 60% Bridge Plans (Reference 13) to evaluate the proposed embankment geometry in the Slide2 software. Abutment bottom of footing, top of rigid abutment, and top of superstructure elevations were obtained from EB abutment loads package (Reference 14).

3. Use the material properties and proposed embankment geometry to analyze the global stability with Slide2.

The results of the Slide2 global stability analyses completed along the longitudinal sections on the proposed Hogan Road Eastbound Abutments between STA. 108+00 and STA. 115+00 for the proposed conditions are summarized in the following table. The analyses were made for deep failure surfaces defined as potential failure surfaces under the abutments, and surficial surfaces on the foreslope in front of the abutments, for static and pseudo-static loading.

CALCULATIONS

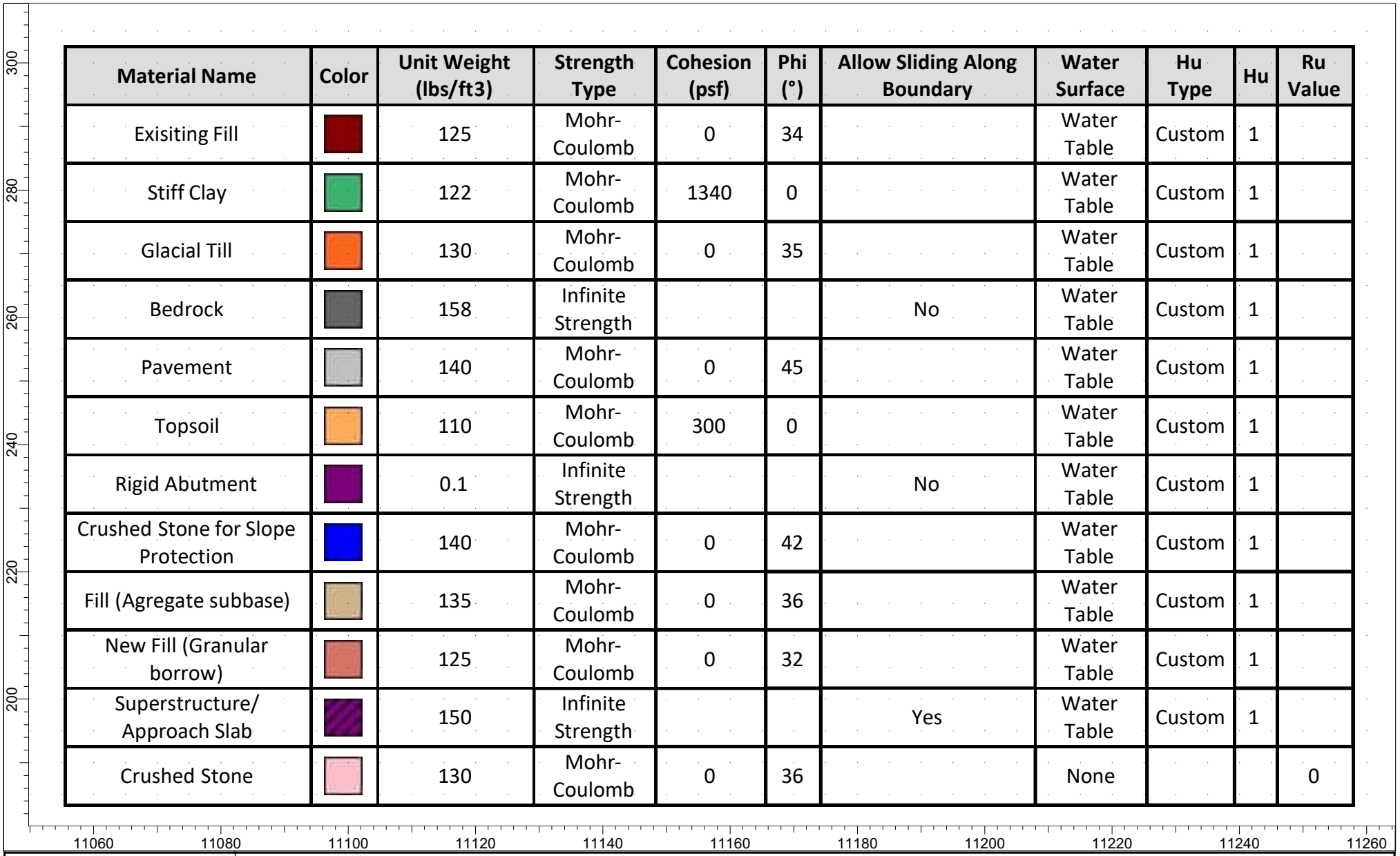
Date:	9/16/2025	**Made by:**	RJN
Project No.:	31404817.004	**Checked by:**	LDN
Subject:	Global Stability Analysis - Hogan Road Eastbound Bridge	**Reviewed by:**	CCB
Project Short	Hogan Road I-95 Bridge (Exit 187) Bangor, ME (WIN 18595.10)		

			Global Factor of Safety		
Location	**Condition**	**Surface Type**	**Deep Surface**	**Foreslopes**	**Figure # (Non-Circular)**
Abutment 1	Static loading	Non-Circular	1.42	1.53	1A / 1B
		Circular	1.53	1.54	-
	Pseudo-static loading	Non-Circular	1.27	1.38	1C / 1D
		Circular	1.40	1.39	-
Abutment 2	Static loading	Non-Circular	1.32	1.45	2A / 2B
		Circular	1.58	1.53	-
	Pseudo-static loading	Non-Circular	1.23	1.30	2C / 2D
		Circular	1.45	1.38	-

Conclusions

The analyzed global stability of the proposed site conditions along the Hogan Road (EB) profile from STA. 108+00 to STA. 115+00 have a computed factor of safety greater than the recommended FS of 1.3 against deep seated failures under staic loading conditions. The proposed conditions meet the minimum FS of 1.1 or greater against deep seated failures under pseudo-static loading conditions.

The analyzed global stability of the proposed foreslope conditions along the Hogan Road (EB) profile from STA. 108+00 to STA. 115+00 have a computed factor of safety greater than the recommended FS of 1.3 against foreslope failures under static loading conditions. The proposed conditions meet the minimum FS of 1.1 against foreslope failures under pseudo-static loading conditions.

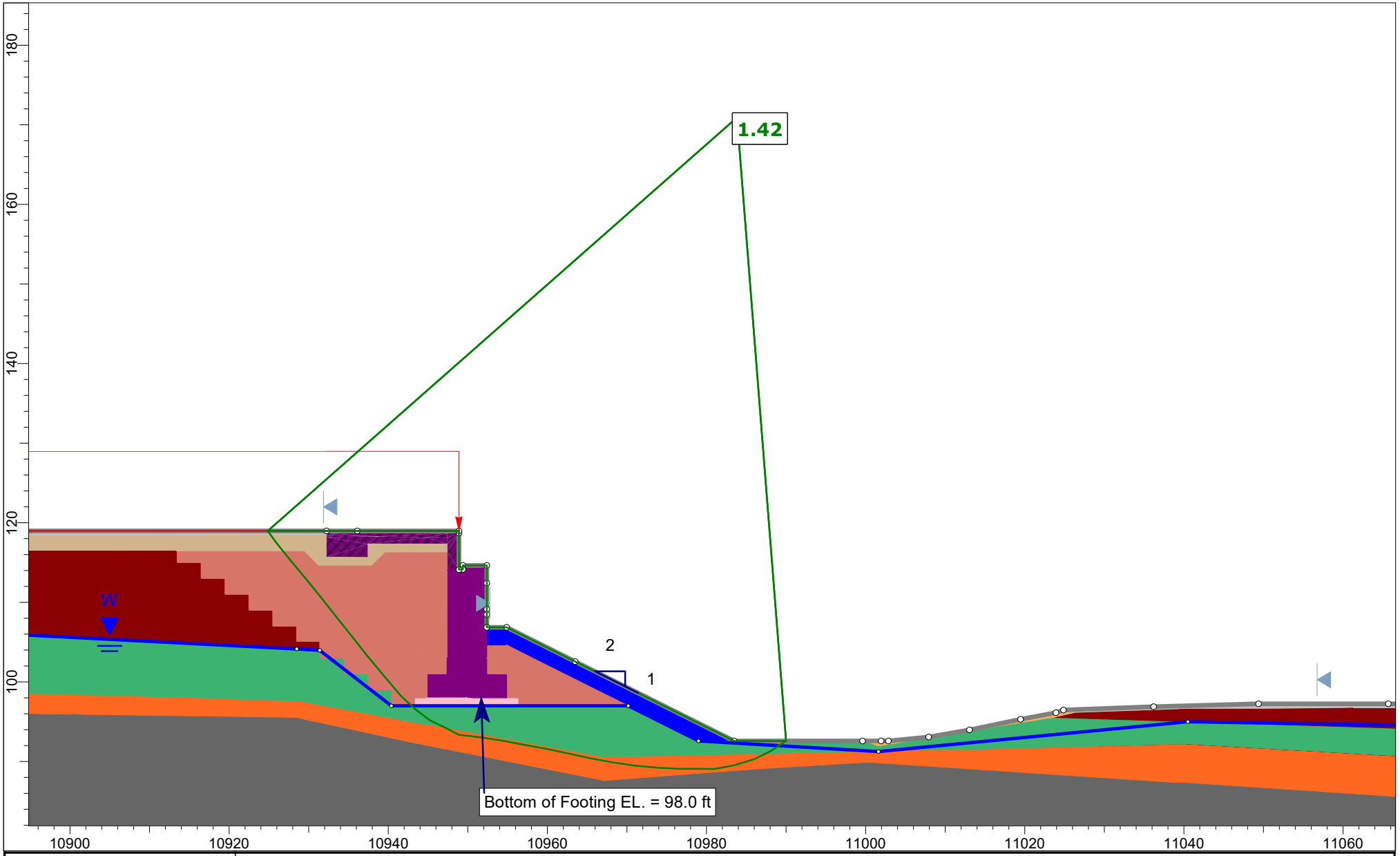


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary	Water Surface	Hu Type	Hu	Ru Value
Exisiting Fill		125	Mohr-Coulomb	0	34		Water Table	Custom	1	
Stiff Clay		122	Mohr-Coulomb	1340	0		Water Table	Custom	1	
Glacial Till		130	Mohr-Coulomb	0	35		Water Table	Custom	1	
Bedrock		158	Infinite Strength			No	Water Table	Custom	1	
Pavement		140	Mohr-Coulomb	0	45		Water Table	Custom	1	
Topsoil		110	Mohr-Coulomb	300	0		Water Table	Custom	1	
Rigid Abutment		0.1	Infinite Strength			No	Water Table	Custom	1	
Crushed Stone for Slope Protection		140	Mohr-Coulomb	0	42		Water Table	Custom	1	
Fill (Agregate subbase)		135	Mohr-Coulomb	0	36		Water Table	Custom	1	
New Fill (Granular borrow)		125	Mohr-Coulomb	0	32		Water Table	Custom	1	
Superstructure/ Approach Slab		150	Infinite Strength			Yes	Water Table	Custom	1	
Crushed Stone		130	Mohr-Coulomb	0	36		None			0

wsp

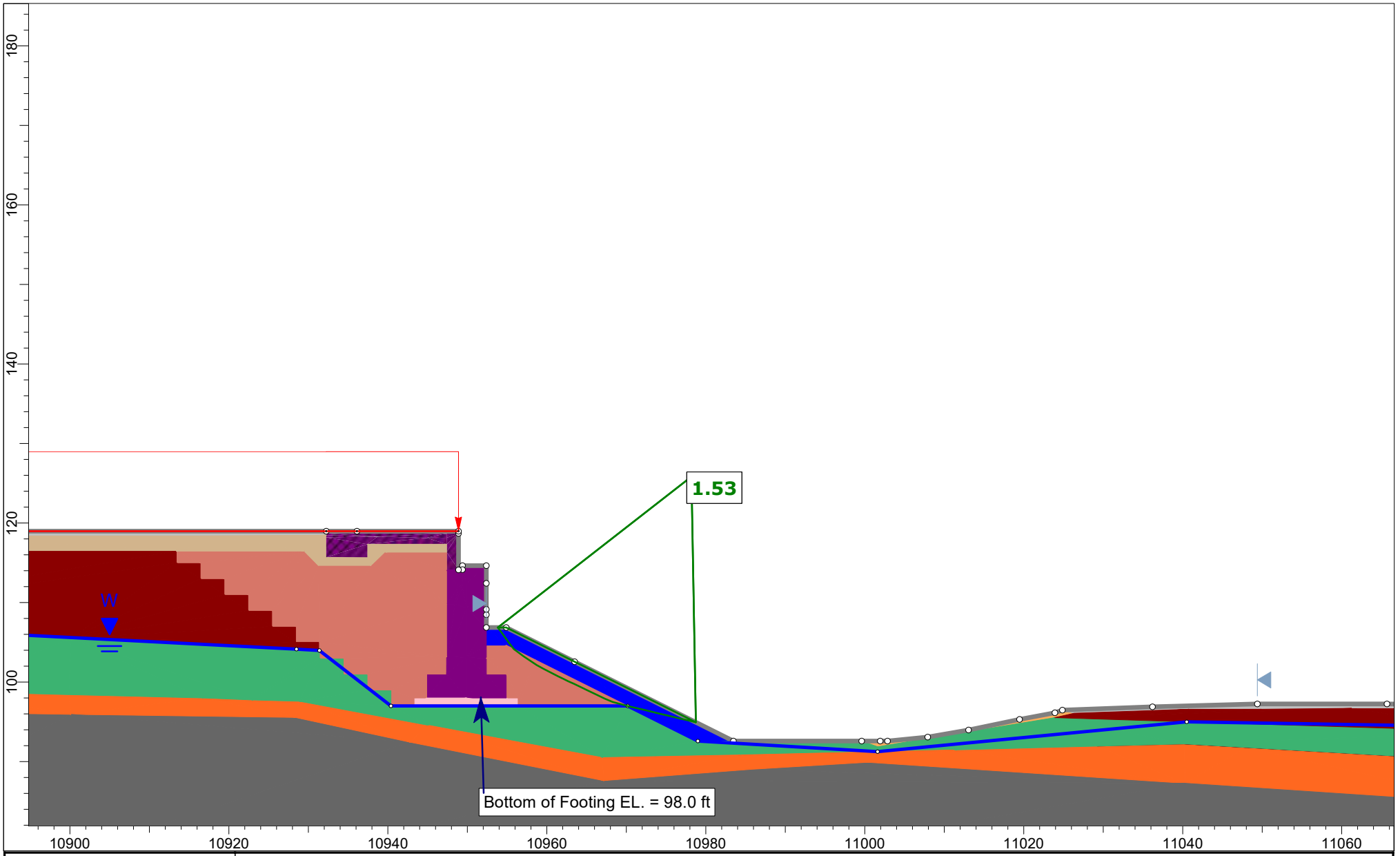
SLIDE 9.038

Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
Analysis Description: Material Properties			
Drawn By: RJN	Checked By: LDN	Reviewed By: CCB	Scale: 1:250
Date: 9/18/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 0**

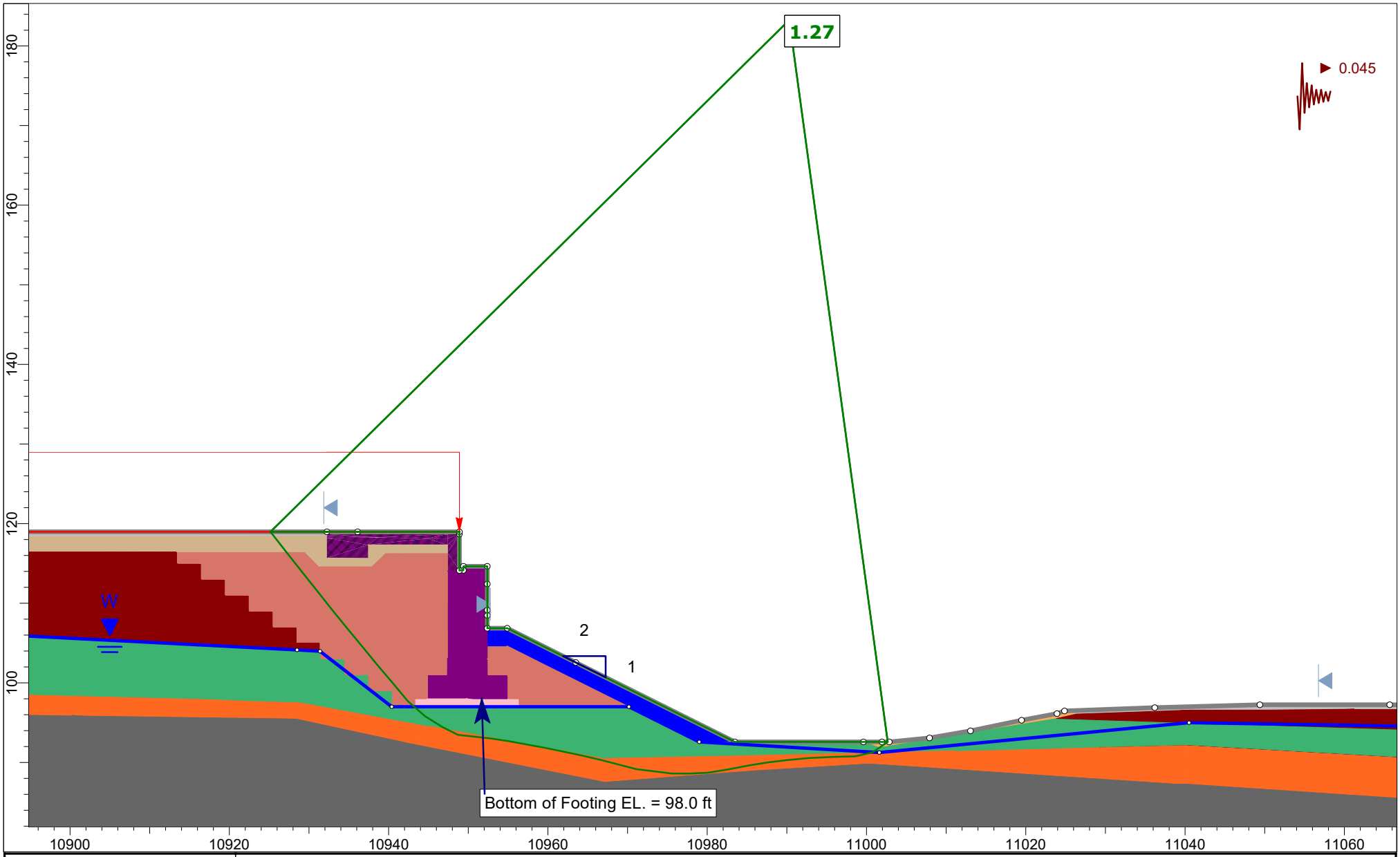


wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 1 - Static Non-Circular			
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200
	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 1A**

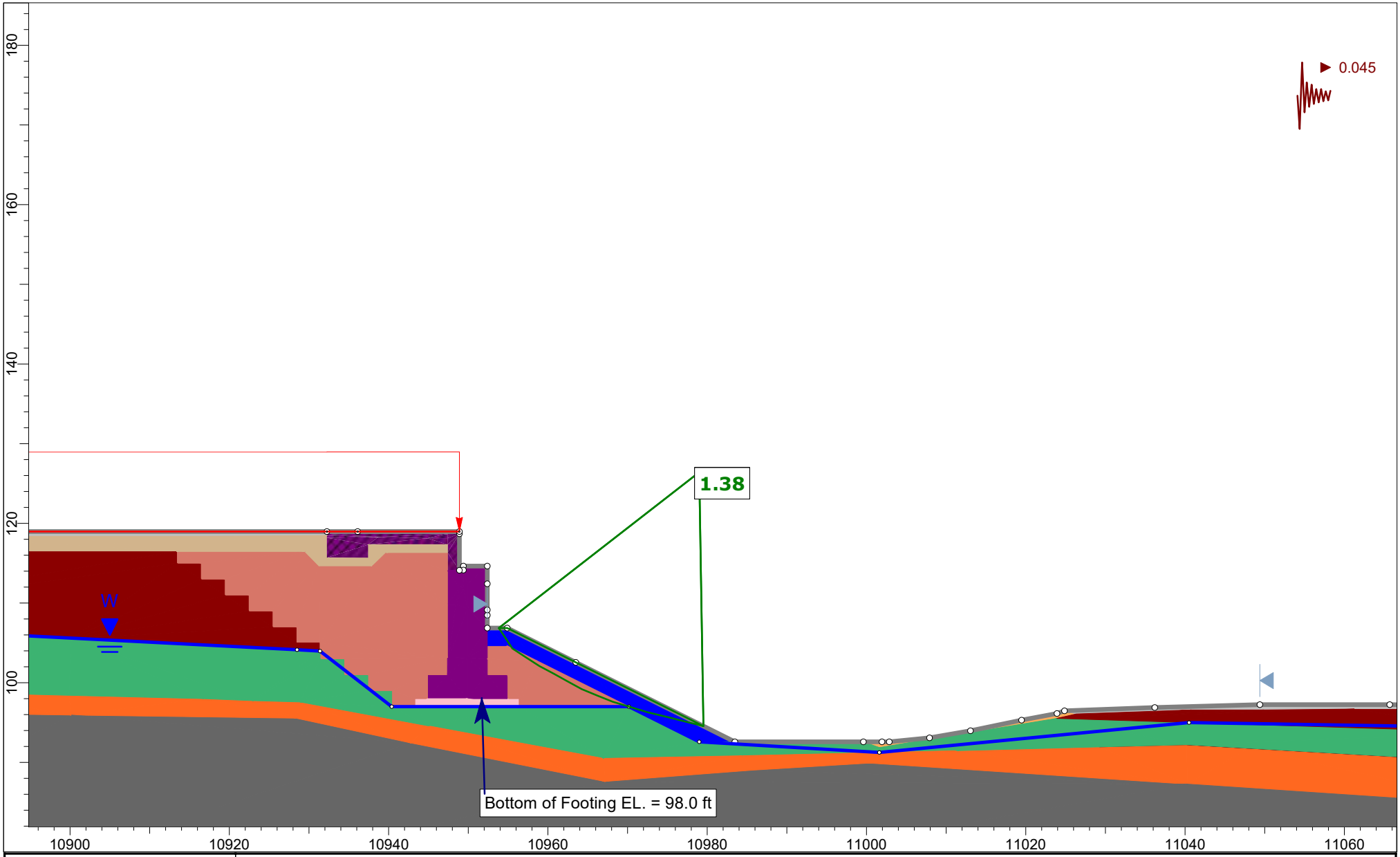
SLIDEINTERPRET 9.038



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 1 Foreslope - Static Non-Circular			
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200
SLIDEINTERPRET 9.038	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 1B**

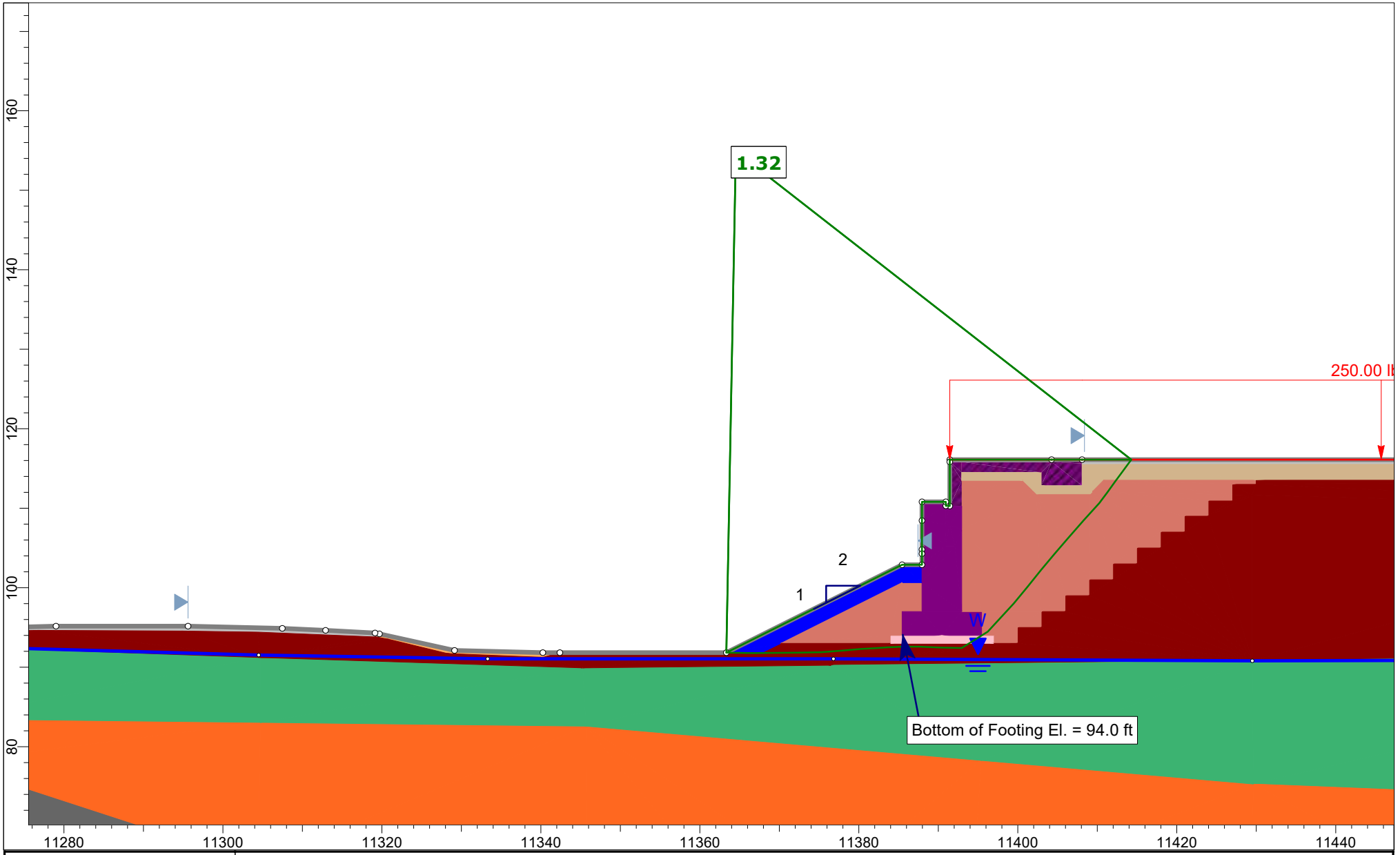


wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 1 - Pseudo Static Non-Circular				
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200	**Figure 1C**
SLIDEINTERPRET 9.038	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd			

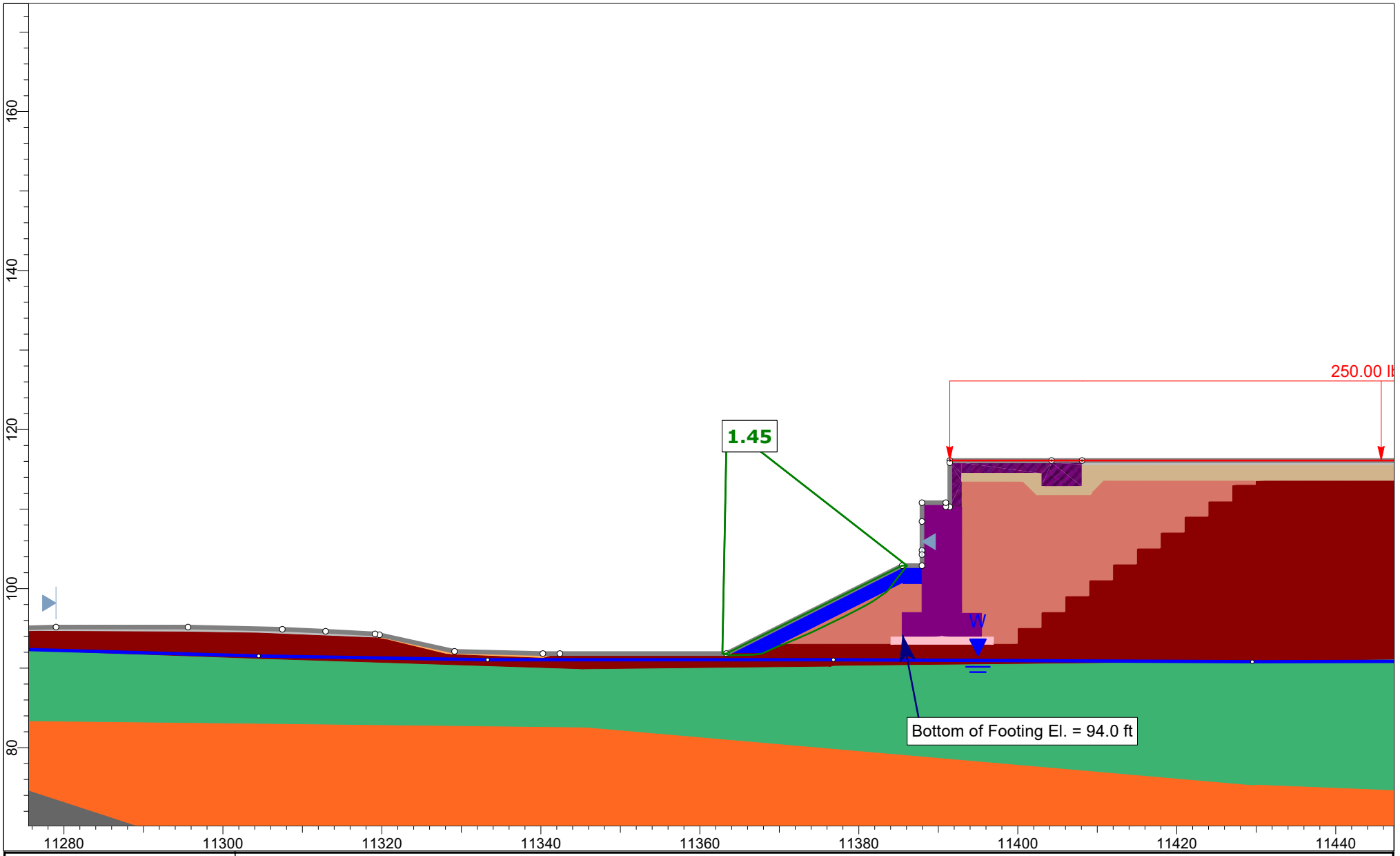


wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 1 Foreslope - Pseudo Static Non-Circular			
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200
	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 1D**

SLIDEINTERPRET 9.038

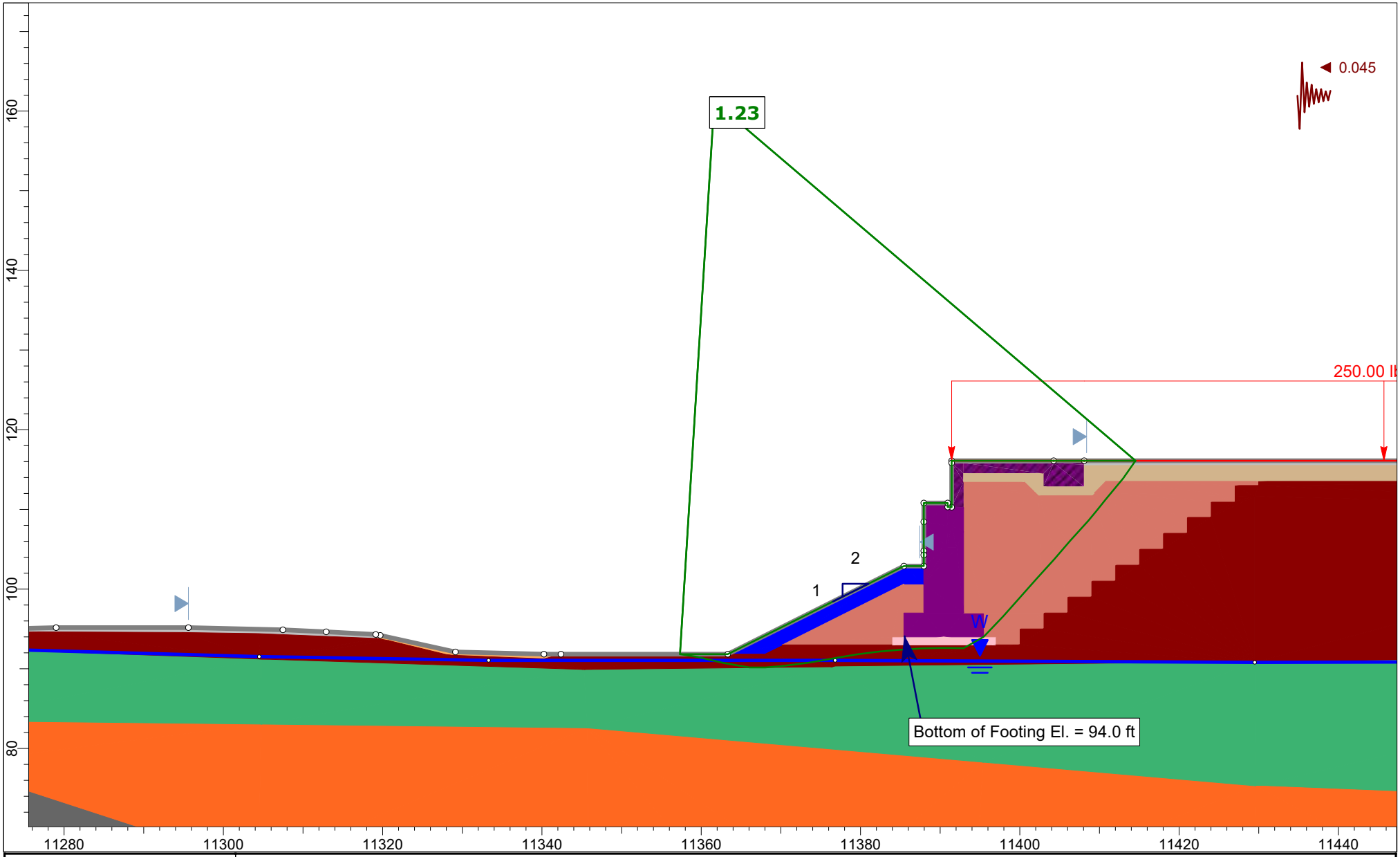


wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)				
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 2 - Static Non-Circular				
	Drawn By: RJN	Checked By: ATM / LDN	Reviewed By: CCB	Scale: 1:200	**Figure 2A**
SLIDEINTERPRET 9.038	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd			

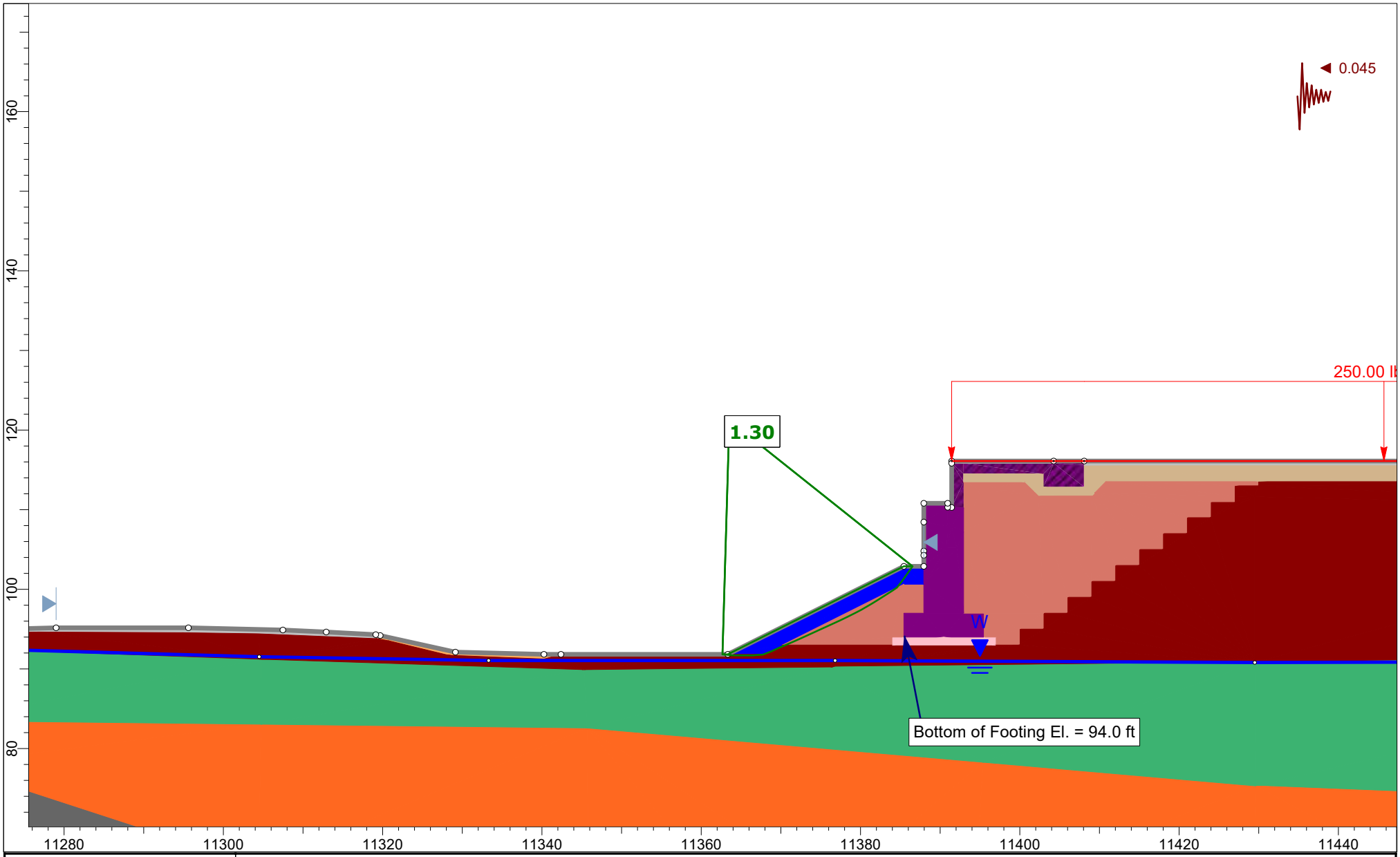


wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 2 Foreslope - Static Non-Circular			
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200
	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 2B**

SLIDEINTERPRET 9.038



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 2 - Pseudo-Static Non-Circular			
	Drawn By: RJN	Checked By: ATM / LDN	Reviewed By: CCB	Scale: 1:200
SLIDEINTERPRET 9.038	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 2C**



wsp	Project: I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)			
	Analysis Description: Hogan Road EB Bridge Longitudinal Global Stability Abutment 2 Foreslope- Pseudo Static Non-Circular			
	Drawn By: RJN	Checked By: ATM/LDN	Reviewed By: CCB	Scale: 1:200
SLIDEINTERPRET 9.038	Date: 9/17/2025	File Name: Eastbound Bridge Longitudinal Model.slmd		**Figure 2D**

APPENDIX G

Settlement Calculations

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Bridge	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Bridge Replacement		

OBJECTIVE

Estimate the anticipated total and differential settlement due to the proposed embankment fills at Hogan Road Eastbound (#5823) and Westbound (#6787) Proposed Bridges 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,

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 Bridge, Bangor, Maine, WIN 018595.10, WSP Project 31404817.000. Dated September 1, 2022.

CALCULATIONS

Date:	6/27/2025	**Made by:**	KAR
Project No.:	31404817.004	**Checked by:**	LMP
Subject:	Settlement Analysis with Lightweight Fill - Bridge	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Bridge Replacement		

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 at Westbound Bridge #6787 Abutment 1 at 75 years after construction
8. Total settlement at Westbound Bridge #6787 Abutment 2 at 75 years after construction
9. Total settlement at Eastbound Bridge #5823 Abutment 1 at 75 years after construction
10. Total settlement at Eastbound Bridge #5823 Abutment 2 at 75 years after construction
11. Graph of anticipated time rate of total settlement over 75 years after construction
12. Graph of anticipated time rate of total settlement over initial two years after construction
13. Graph of anticipated time rate of primary consolidation over 75 years after construction
14. 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
  - 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.

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.

CALCULATIONS

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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.

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.

CALCULATIONS

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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).

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).

$$\frac{E_s}{p_a} = \alpha N_{60}$$ (Ref. 7, Eqn. 2.29)

CALCULATIONS

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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

$E_s = \beta s_u$ (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

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:

$K_0 = 1 - \sin\phi'$ (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:

CALCULATIONS

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$$K_0 = (1 - \sin\phi')OCR^{\sin\phi'}$$ (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.

CALCULATIONS

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RESULTS

The total settlements anticipated at the proposed westbound (#6787) and eastbound (#5823) Abutments 1 and 2 are summarized in Table 1 below. Settlement values are provided at 75 years after construction, which is the anticipated life of the bridge. Model output is provided in Attachment Figures 1 - 14.

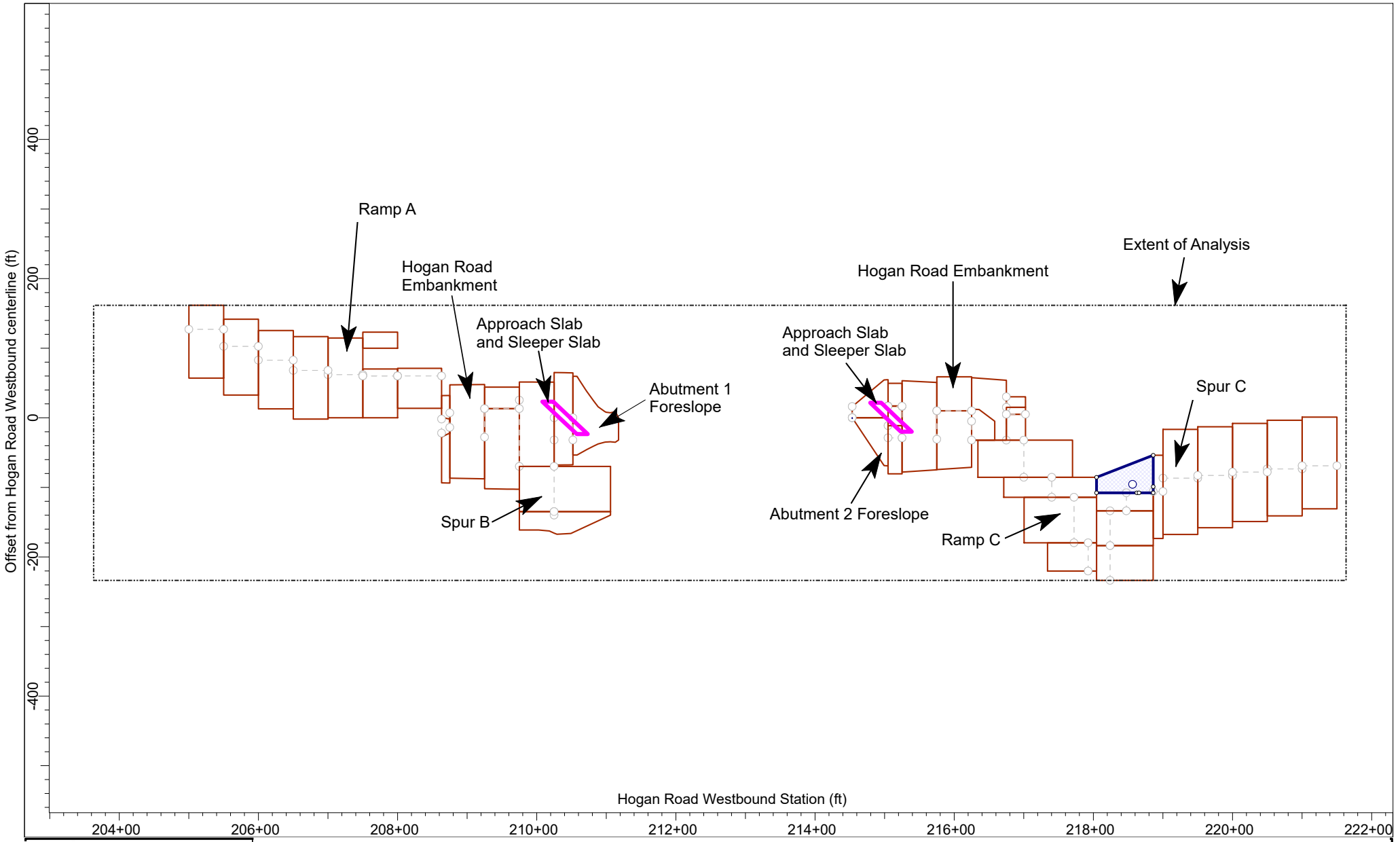
- Images of the model geometry, soil stratigraphy, and loading are shown in Attachment Figures 1 - 6.
- Images of the model results for the abutments are shown in Attachment Figures 7 - 10.
- Plots of the time rate of settlement at point of maximum settlement for each modeled feature are shown in Attachment Figures 11 - 14.

The embankments behind the westbound bridge abutments 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 (Assumption 1).

At the proposed westbound bridge (#6787) (with lightweight fill), the total settlement along the back edge of Abutment 1 ranges from 0.5 to 2.1 inches, and the total settlement along the back edge of Abutment 2 ranges from 0.4 to 1.5 inches. At the proposed eastbound bridge (with conventional fill), the total settlement along the back edge of Abutment 1 ranges from 0.4 to 0.5 inches, and the total settlement along the back edge of Abutment 2 ranges from 0.6 to 0.8 inches.

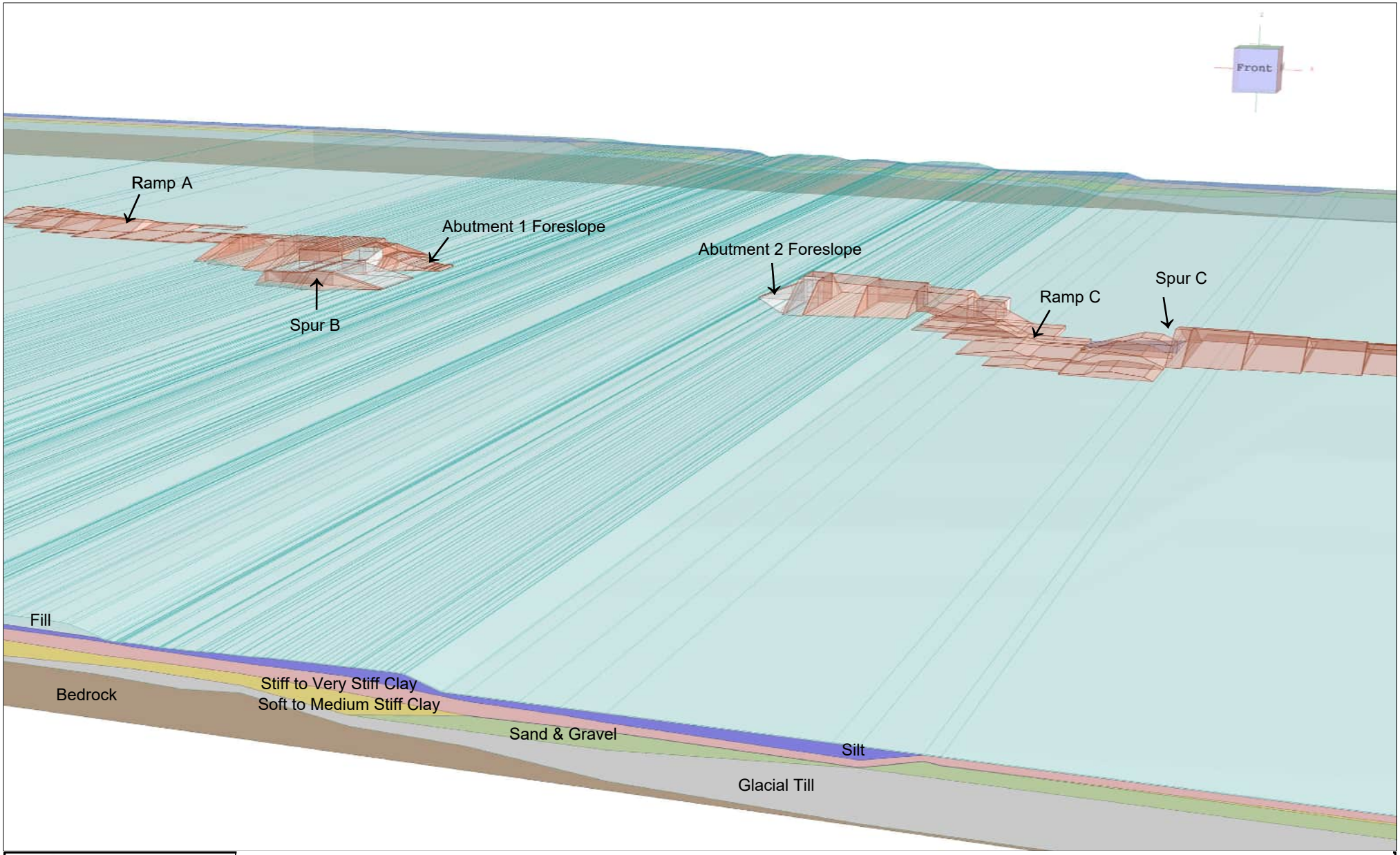
Table 1: Summary of Settlement Analysis Results for Proposed Bridge Abutments

Structure		Westbound Bridge (#6787)		Eastbound Bridge (#5823)	
Abutment		Abut. 1	Abut. 2	Abut. 1	Abut. 2
Elevation (ft)		98.4	87.0	113.1	110.0
Total Settlement (in.)	Maximum point	2.1	1.5	0.5	0.8
	Minimum point	0.5	0.4	0.4	0.6
	Attachment number	#7	#8	#9	#10
Immediate Settlement (in.)	Maximum point	0.7	0.4	0.2	0.5
	Minimum point	0.1	0.1	0.1	0.3
Primary Consolidation Settlement (in.)	Maximum point	1.1	0.8	0.0	0.0
	Minimum point	0.1	0.1	0.0	0.0
Secondary Consolidation Settlement (in.)	Maximum point	0.3	0.3	0.3	0.3
	Minimum point	0.3	0.2	0.3	0.2



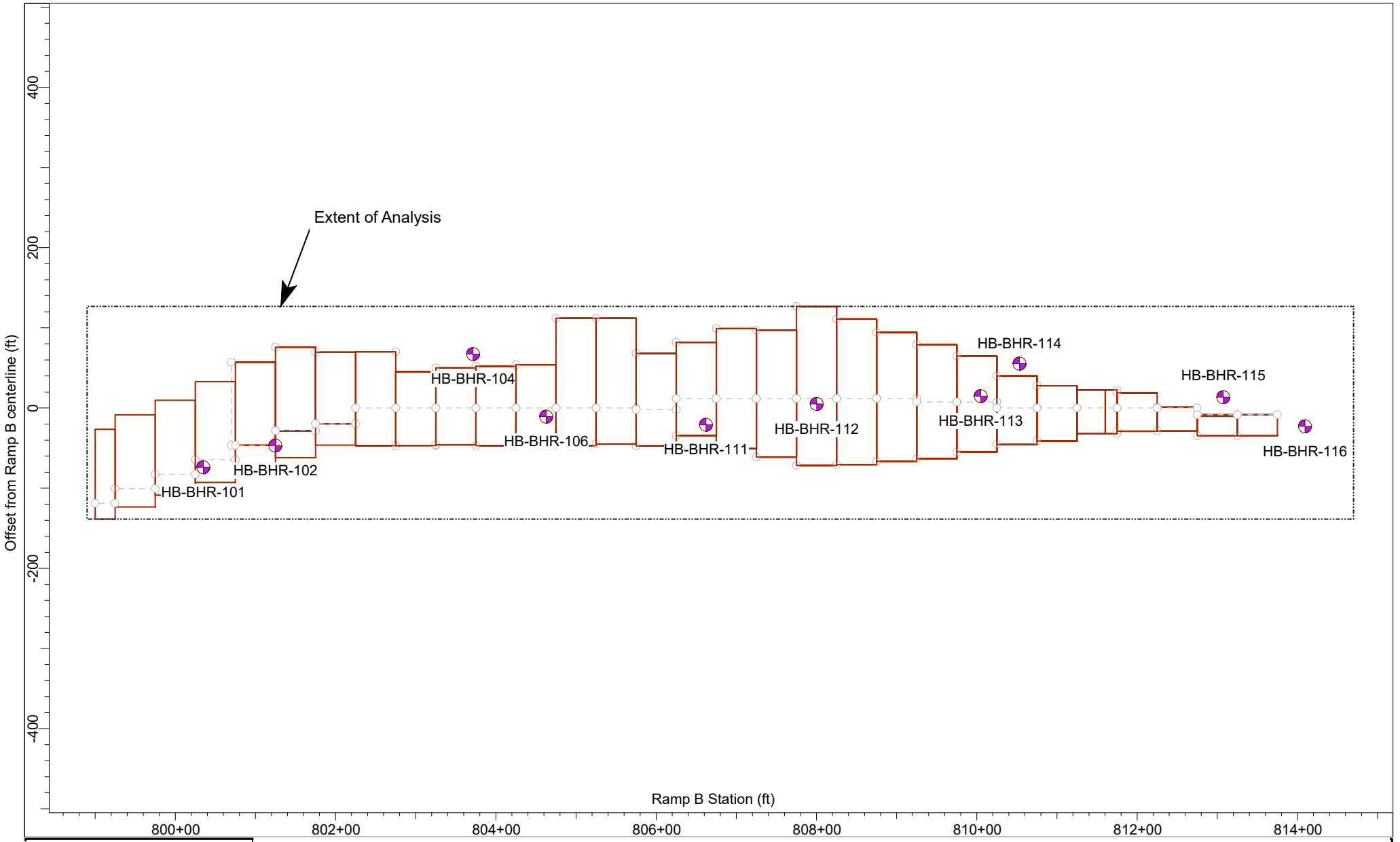
Project	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange, Project #31404817.003			
Analysis Description	Plan View of Loading in Settle3 Model (Highway Interchange)			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 1**
Date: 5/29/2025	File Name: Hogan Rd Highway 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



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**
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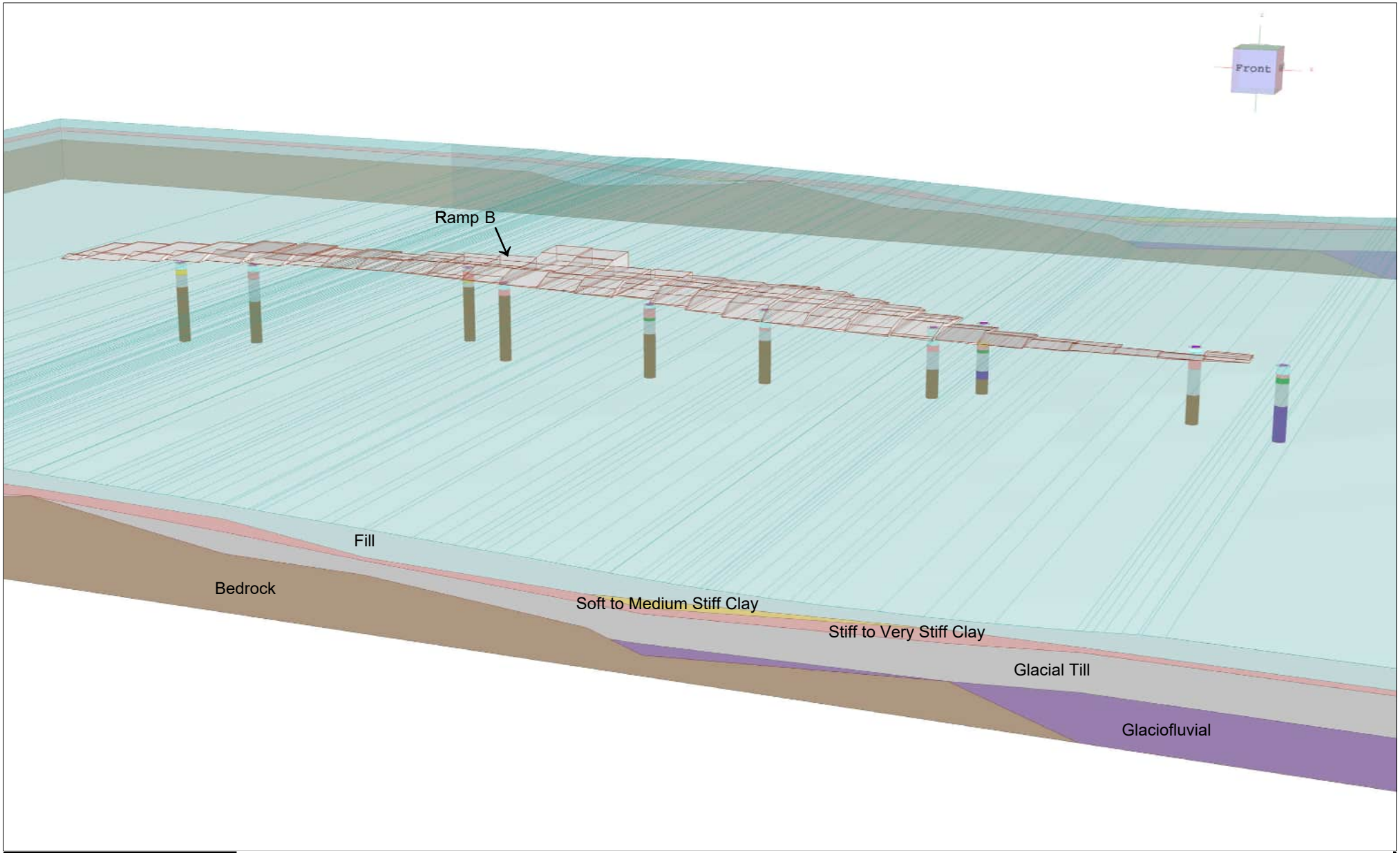
SETTLE3 5.025



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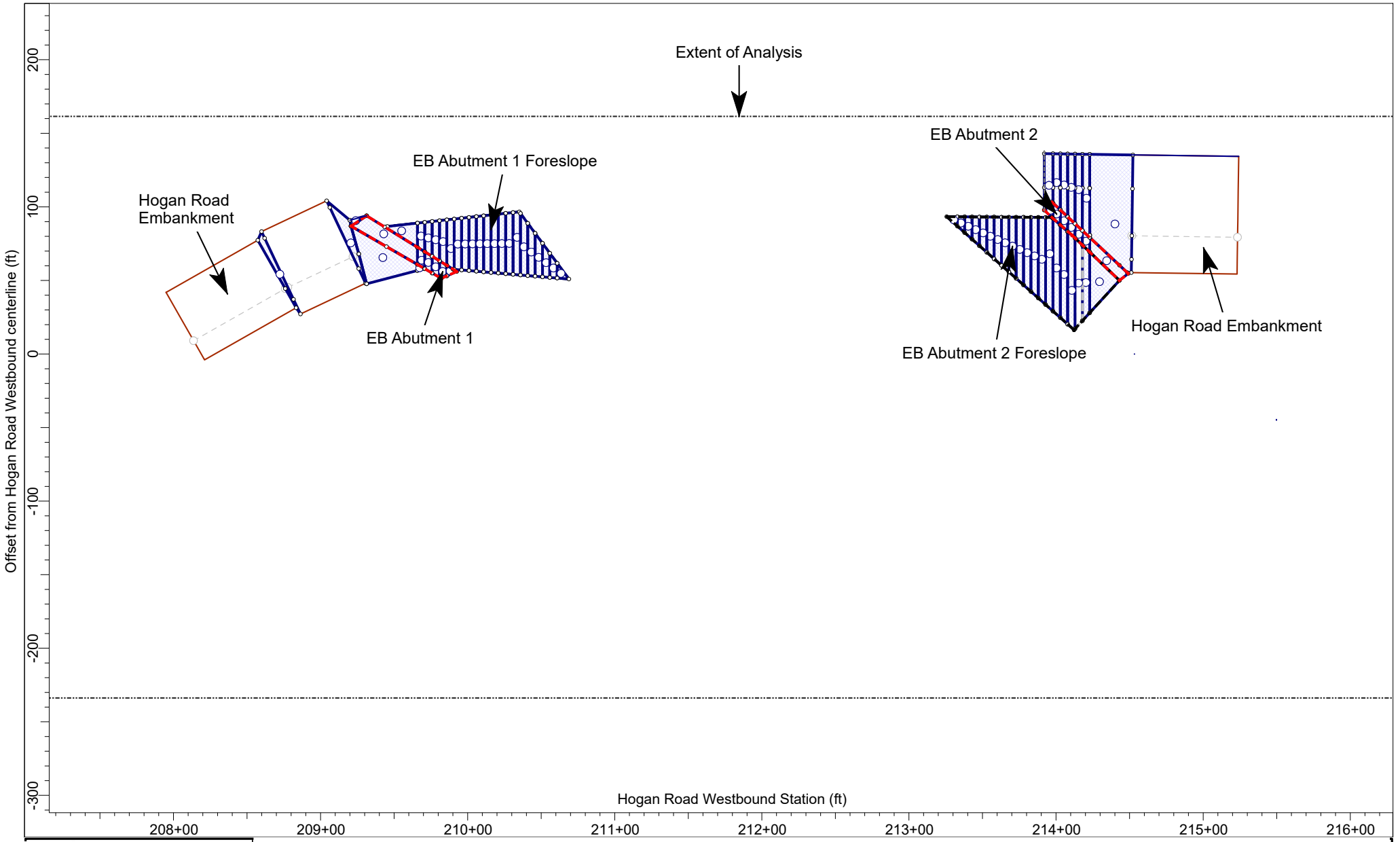
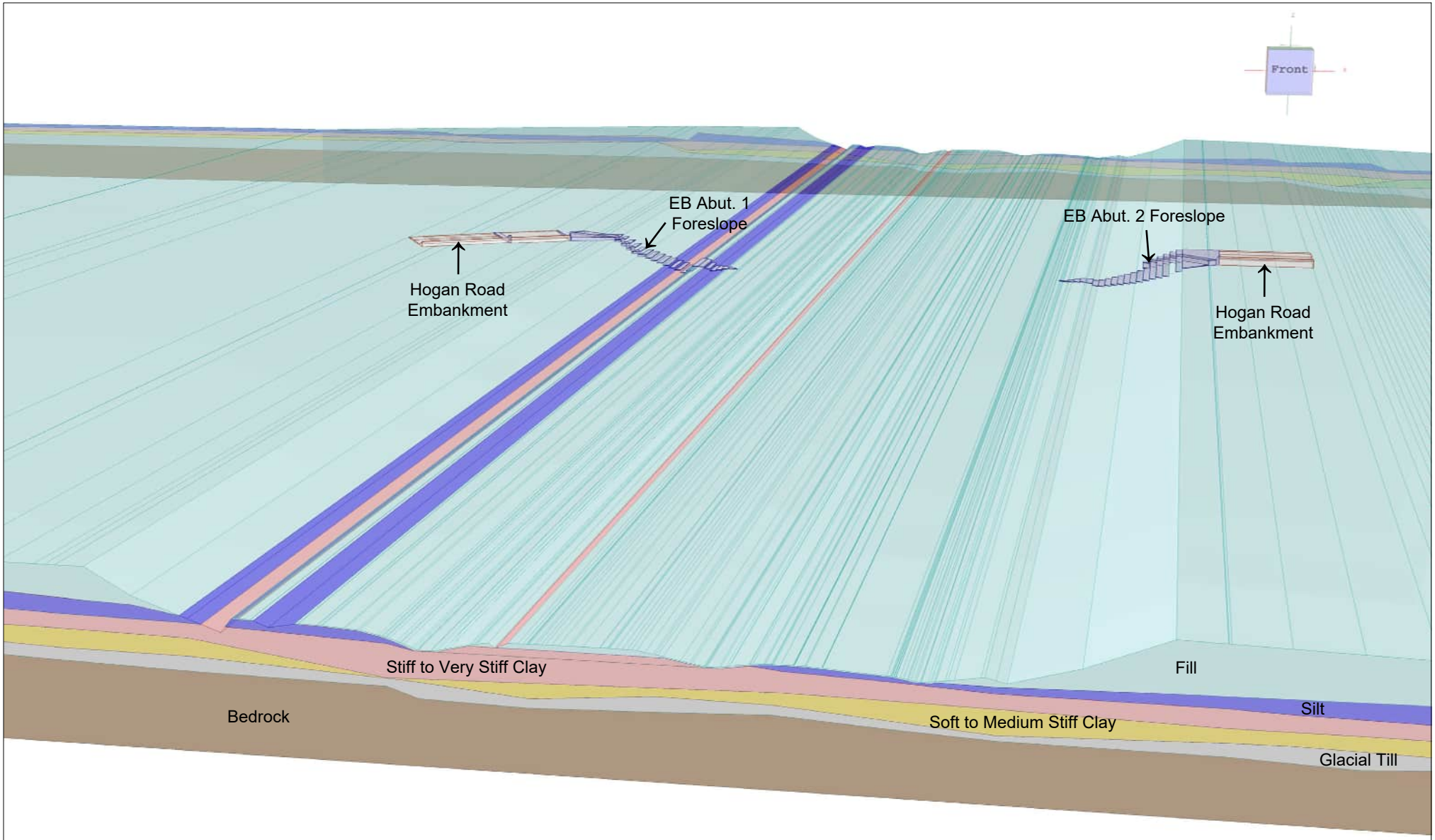
	Analysis Description: 3D View of Soil Stratigraphy and Loading in Settle3 Model (Ramp B)			
	Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale:
SETTLE3 5.025	Date: 3/20/2025	File Name: Hogan Rd Highway Ramp B Phase2 Jan2024.s3z		**Figure 4**



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



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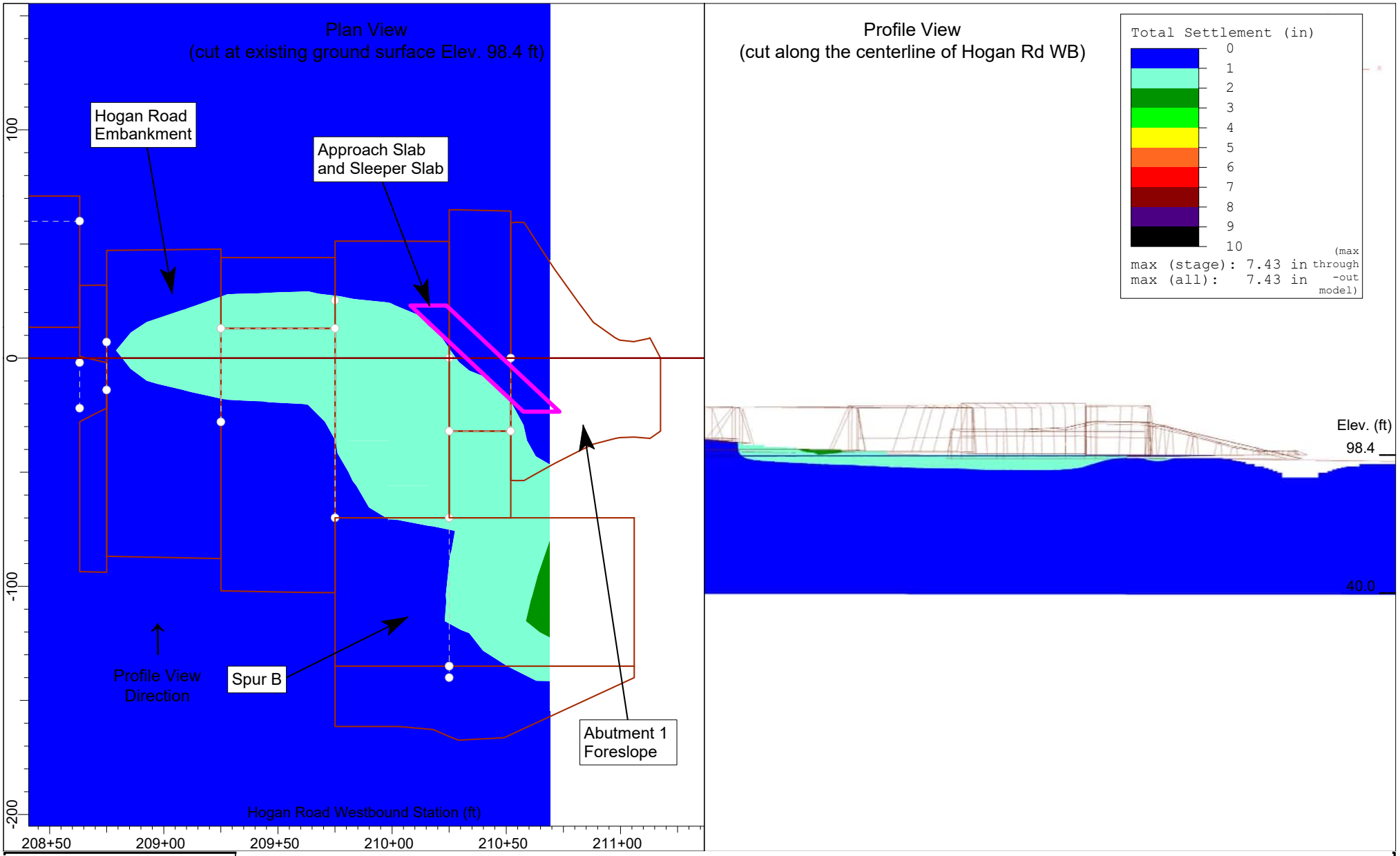
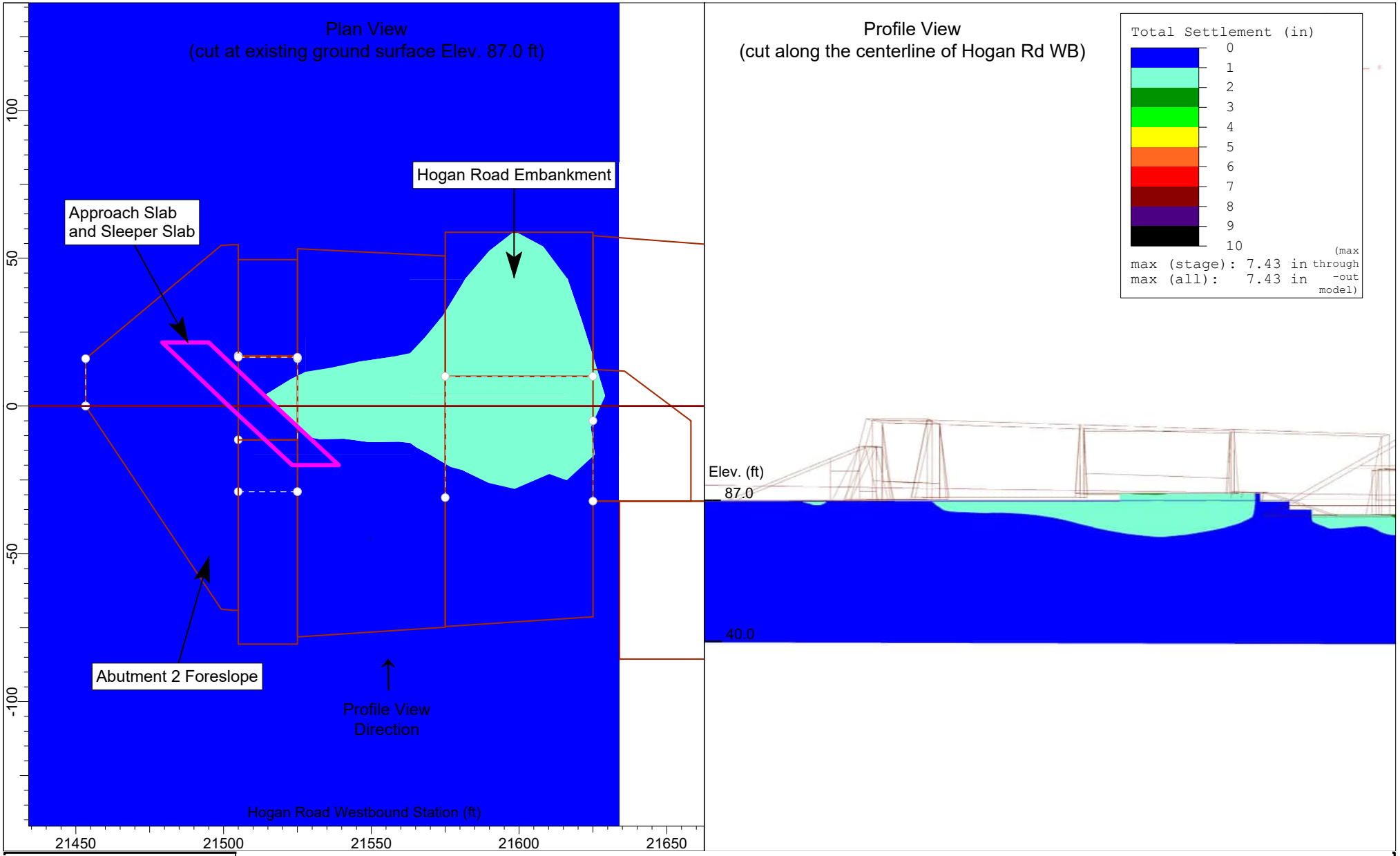
	Analysis Description: 3D View of Soil Stratigraphy and Loading in Settle3 Model (Hogan Road Eastbound)			
	Drawn By: LMP	Checked By: KAR	Reviewed By: CCB	Scale:
SETTLE3 5.025	Date: 3/20/2025	File Name: Hogan Rd Highway Interchange Phase2 EB V1 - Foreslope - with query points.s3z		**Figure 6**



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement at Westbound Bridge #6787 Abutment 1 at 75 Years after Lightweight Fill Placement			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 7**
Date: 6/10/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

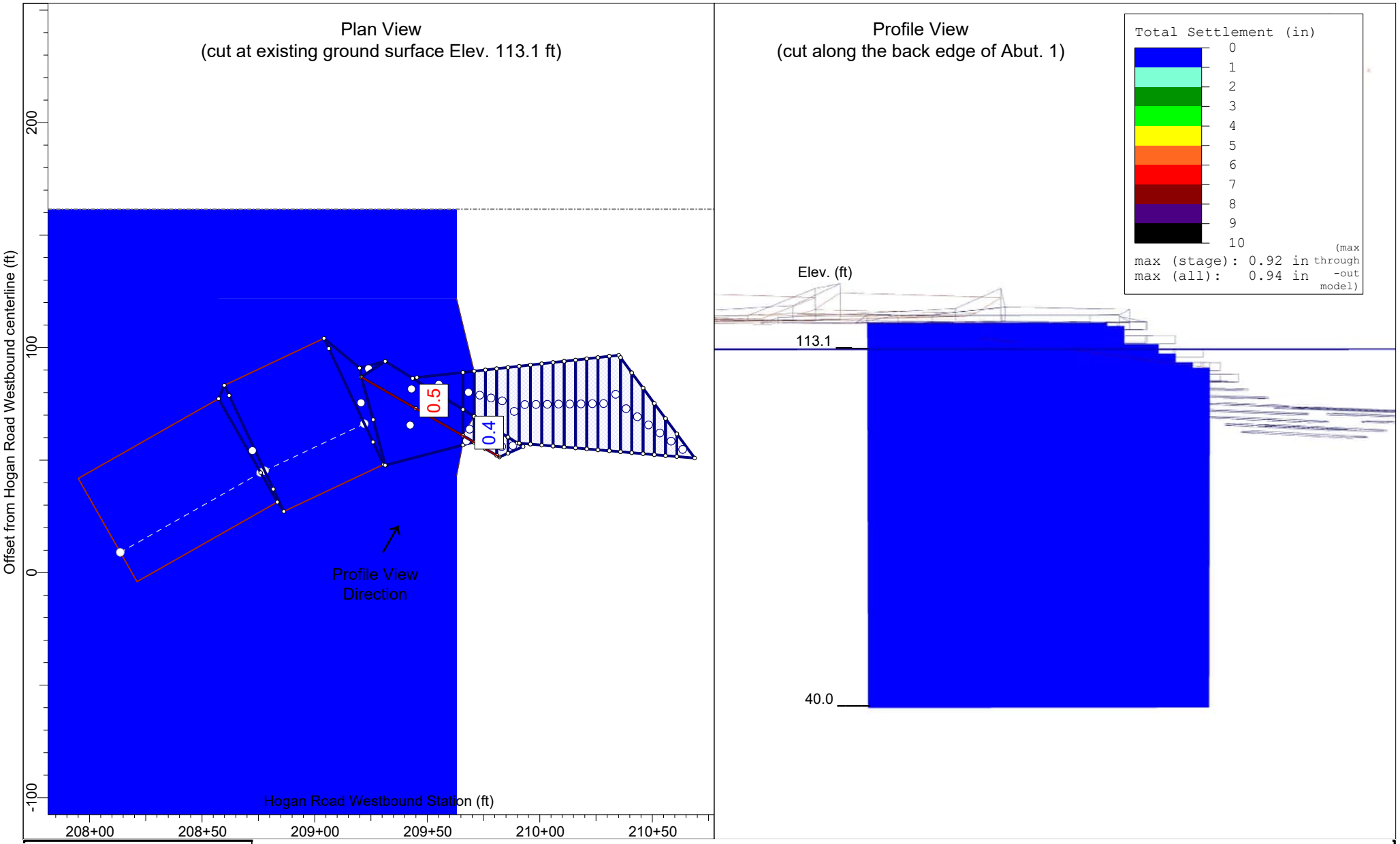
SETTLE3 5.025



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement at Westbound Bridge #6787 Abutment 2 at 75 Years after Lightweight Fill			
Drawn By: KAR	Checked By: LMP	Reviewed By: CCB	Scale: as shown	**Figure 8**
Date: 5/29/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading.s3z			

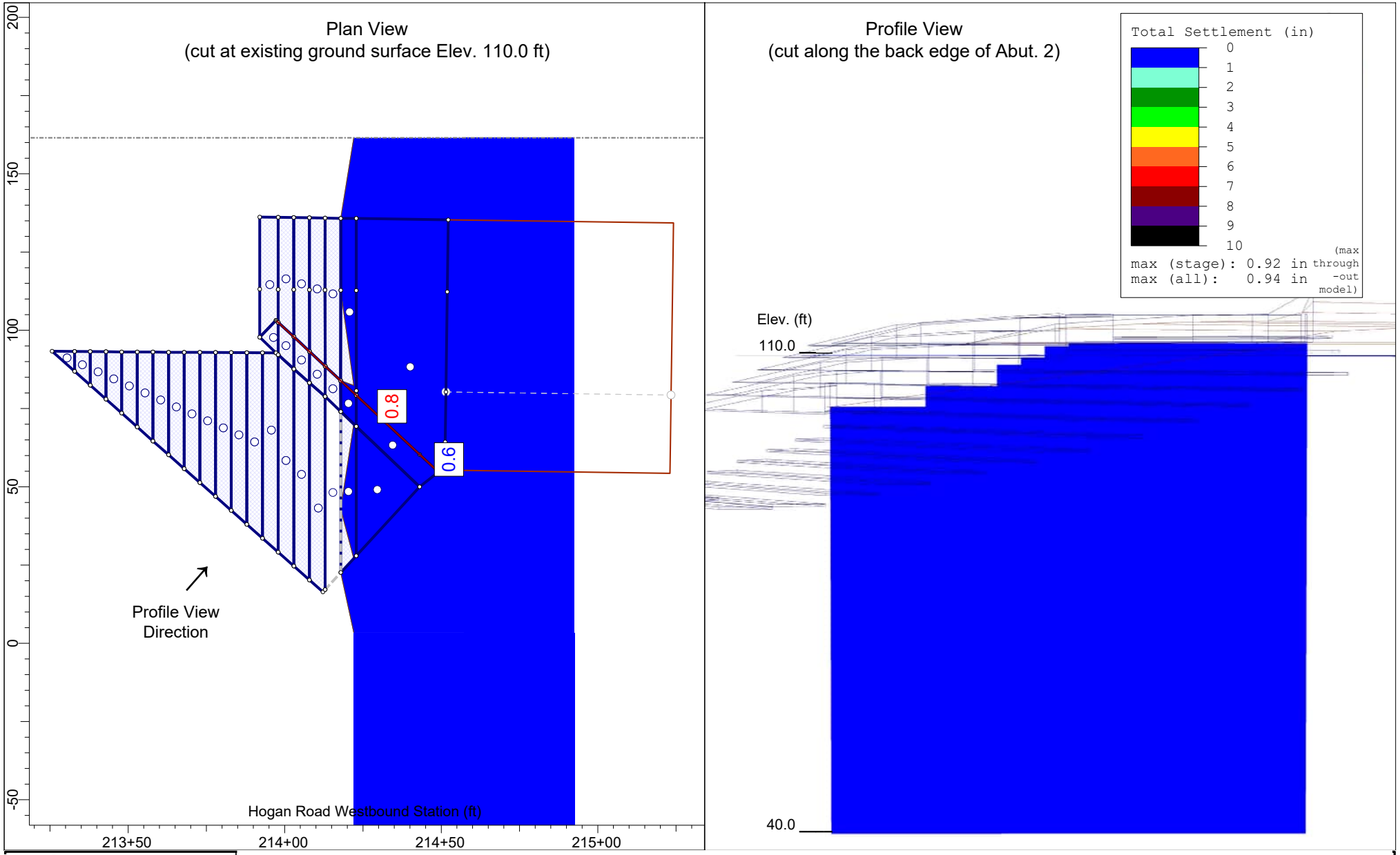
SETTLE3 5.025

Placement



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement at Eastbound Bridge #5823 Abutment 1 at 75 Years after Conventional Fill Placement			
Drawn By: LMP	Checked By: KAR	Reviewed By: CCB	Scale: as shown	**Figure 9**
Date: 3/20/2025	File Name: Hogan Rd Highway Interchange Phase2 EB V1 - Foreslope - with query points.s3z			

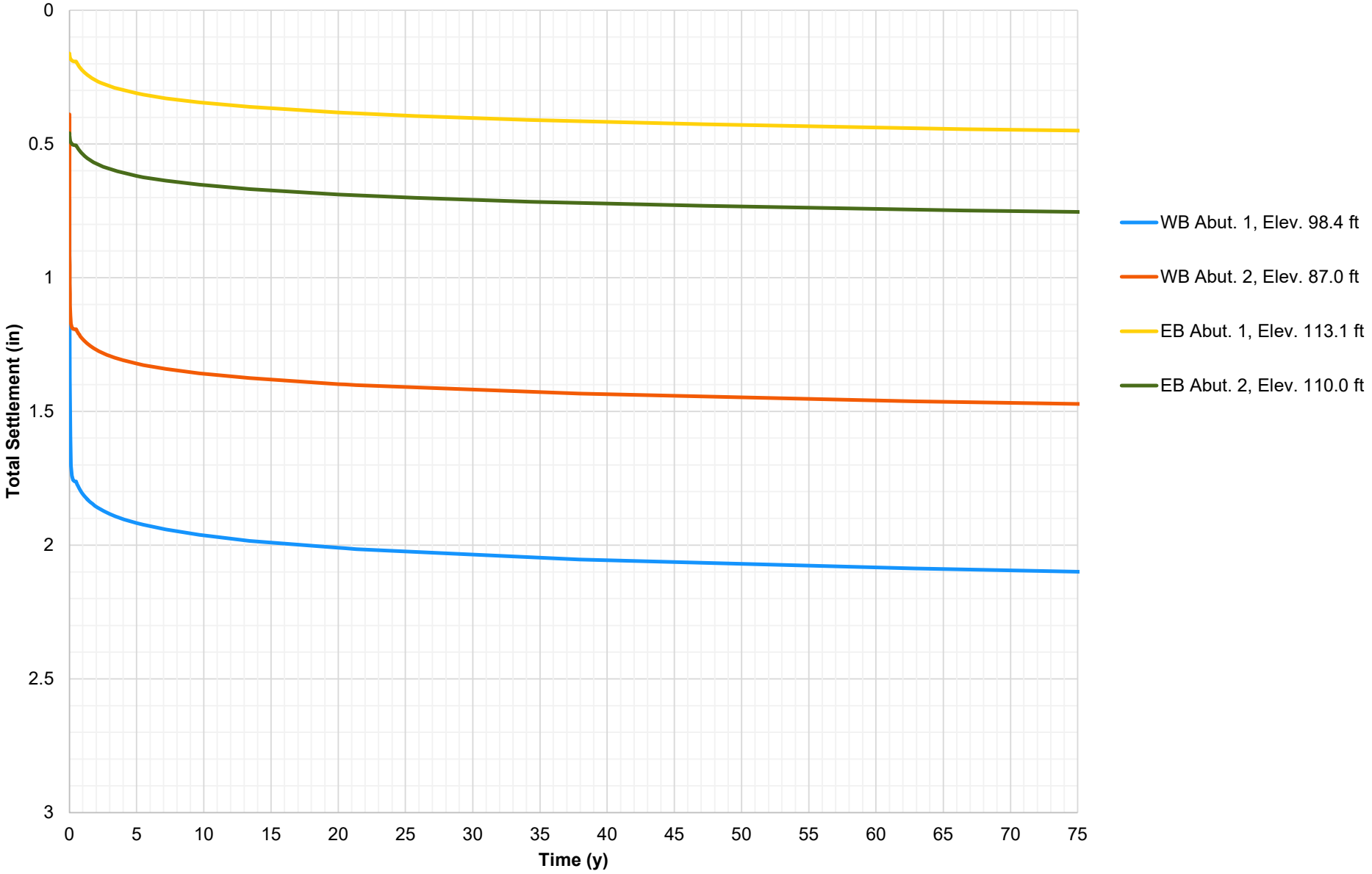
SETTLE3 5.025



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement at Eastbound Bridge #5823 Abutment 2 at 75 Years after Conventional Fill Placement			
Drawn By: LMP	Checked By: KAR	Reviewed By: CCB	Scale: as shown	**Figure 10**
Date: 3/20/2025	File Name: Hogan Rd Highway Interchange Phase2 EB V1 - Foreslope - with query points.s3z			

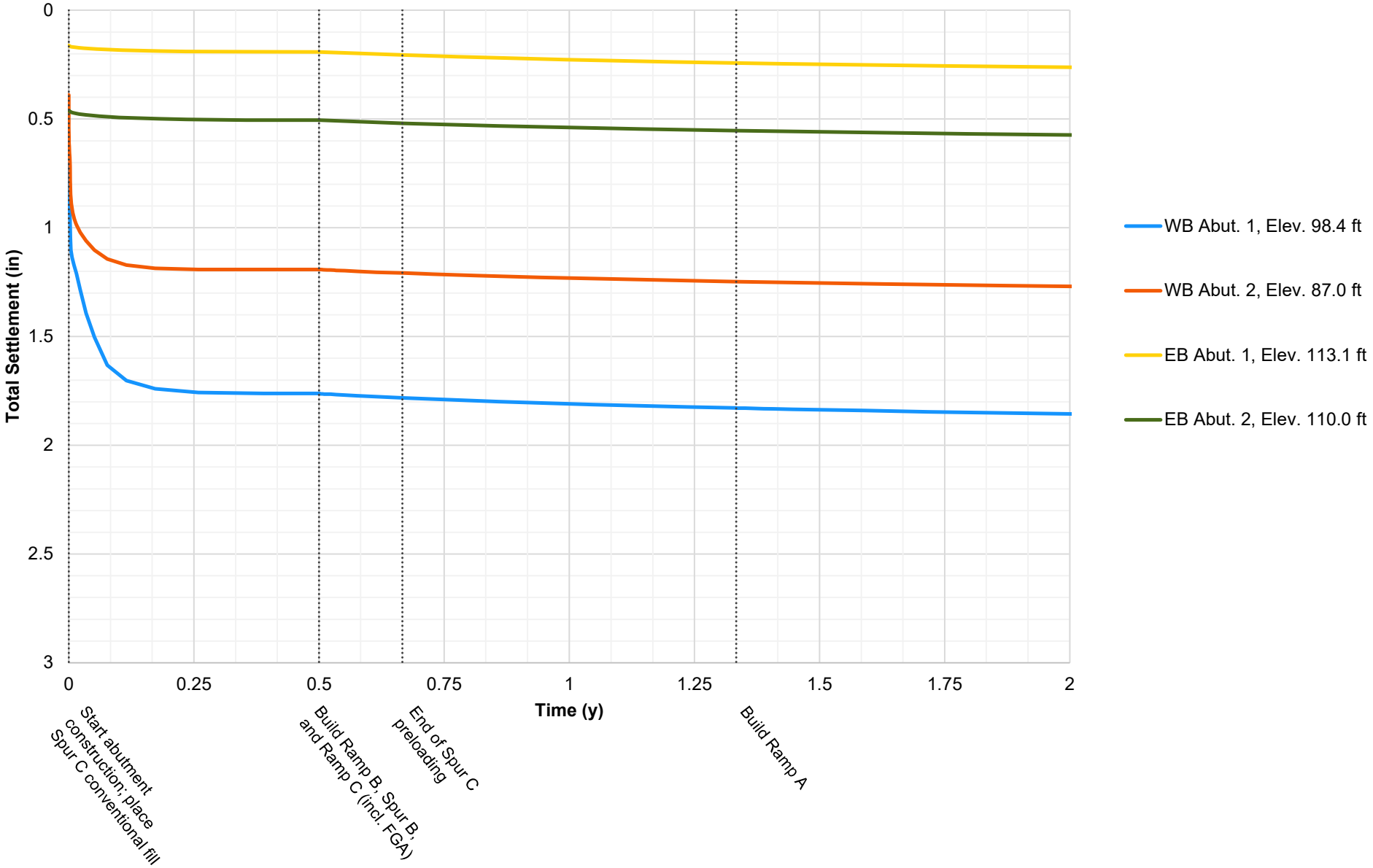
SETTLE3 5.025

Figure 11: Time Rate of Total Settlement over 75 Years



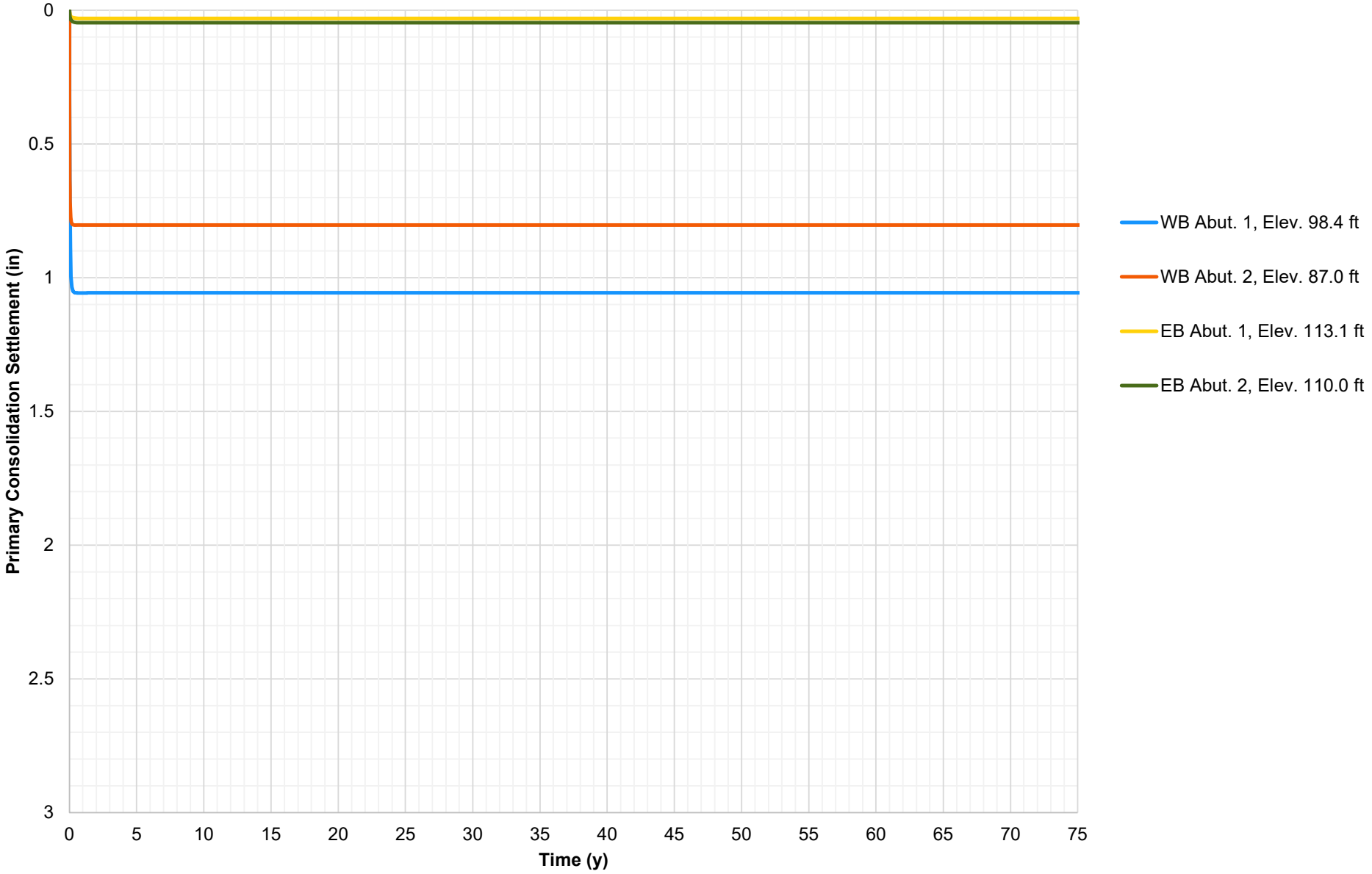
Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 12: Time Rate of Total Settlement over Initial Two Years



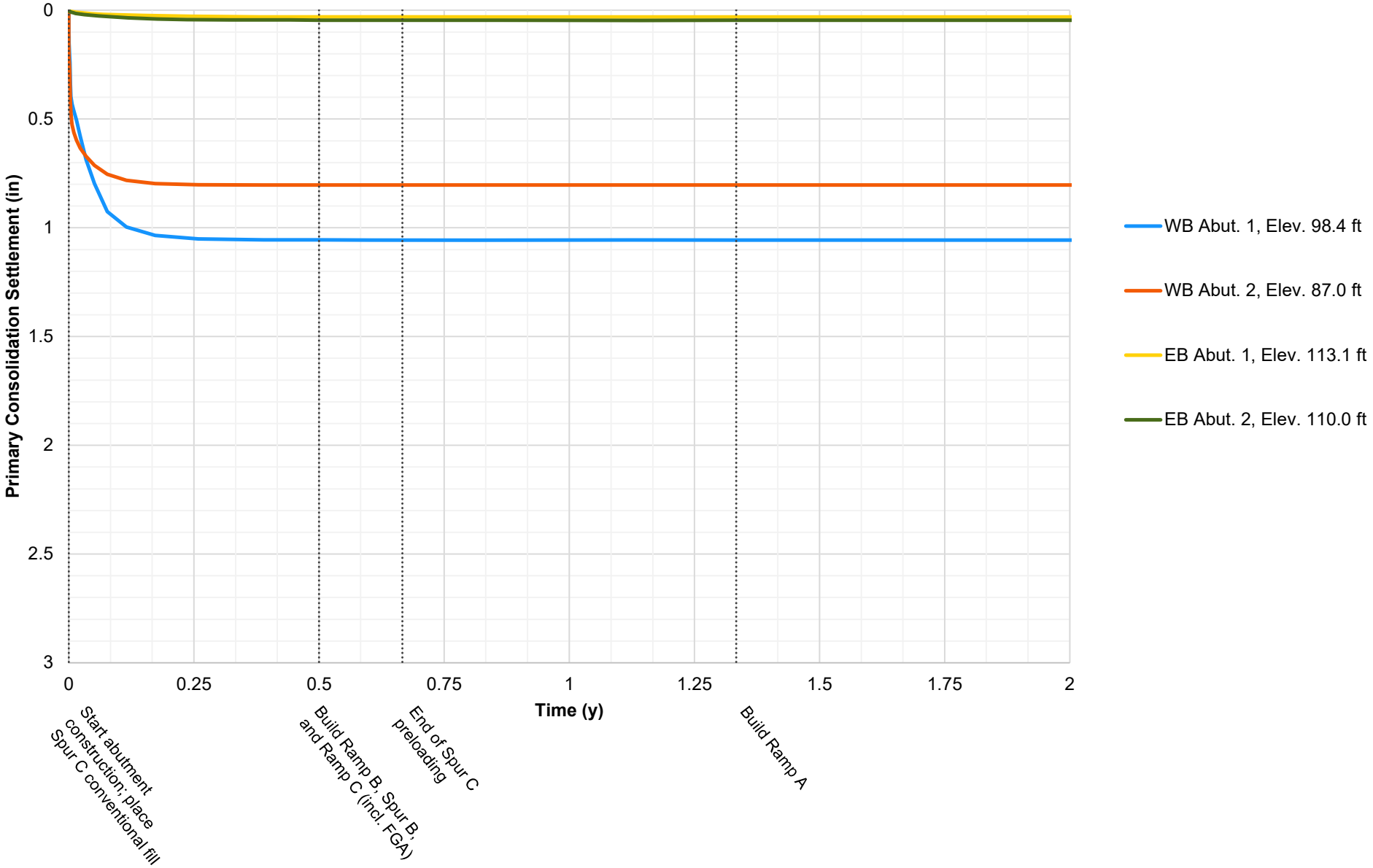
Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 13: Time Rate of Primary Consolidation over 75 Years



Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

Figure 14: Time Rate of Primary Consolidation over Initial Two Years



Date: 6/10/2025
Prepared: KAR
Checked: LMP
Reviewed: CCB

APPENDIX H

Lateral Earth Pressure and Elastic Inclusion Calculations

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

OBJECTIVE

Determine lateral earth pressures acting on the proposed Abutment 1 located at STA 109+43.83 and Abutment 2 located at STA 113+91.73 of Bridge #5823. The lateral earth pressure on the proposed semi-integral abutments is calculated on the superstructure back wall and the rigid abutment.

REFERENCES

1. Das, Braja M. 2011. Principles of Foundation Engineering, 7th ed. Cengage Learning, Stamford, CT.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
4. Hogan Road Bridge Rehabilitation plan mark provided by VHB on 1/22/2025, titled: "5823 Bangor 1983 Hogan Rd Bridge Rehab plan markups (003).pdf"
5. VHB for State of Maine Department of Transportation. Bangor US I-95 Exit 187/Hogan Road Interchange, Draft PIC, dated September 1, 2023.
6. State of Maine Department of Transportation, Bangor, Penobscot County, Hogan Road Bridge over I-95, Hogan Road West Bound (Sheets B1 - B49), Dated 8/30/2024 (60% DDI Plans).
7. State of Maine Department of Transportation, Bangor, Penobscot County, Hogan Road Bridge over I-95, Hogan Road East Bound (Sheets B5 - B8), Dated 4/25.

ASSUMPTIONS

1. The backfill surface behind each abutment is assumed to be horizontal.
2. The backfill is assumed to be free draining (i.e., no water pressure is allowed to build up behind the abutment wall).
3. The geometry of backwall and rigid abutment height were based on the Ref. 7 plan set dated 4/25.

CALCULATION

1. Calculate At-Rest, Active and Passive pressure coefficients.

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

$$K_0 = 1 - \sin\phi$$ (Ref. 1, Eqn. 13.5) For coarse-grained soils

where:

ϕ_{Fill} = 32 degrees - Granular borrow (Ref. 2 Table 3-3)

$K_{0\ Fill}$ = 0.47

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

1.2. Calculate Rankine active earth pressure coefficient using Rankine theory.

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right)$$ (Ref. 2, page 3-7) For a horizontal backfill

for horizontal backfill surface, where:

ϕ_{Fill} = 32 degrees - Granular borrow (Ref. 2 Table 3-3)

$K_{a\,Fill}$ = 0.31

1.4. Calculate the full passive earth pressure.

Per the MaineDOT Bridge Design Guide Section Chapter 3, section 3.6.6 (Ref. 2), Values of the coefficient of passive lateral earth pressure, K_p, may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD (Ref. 3).

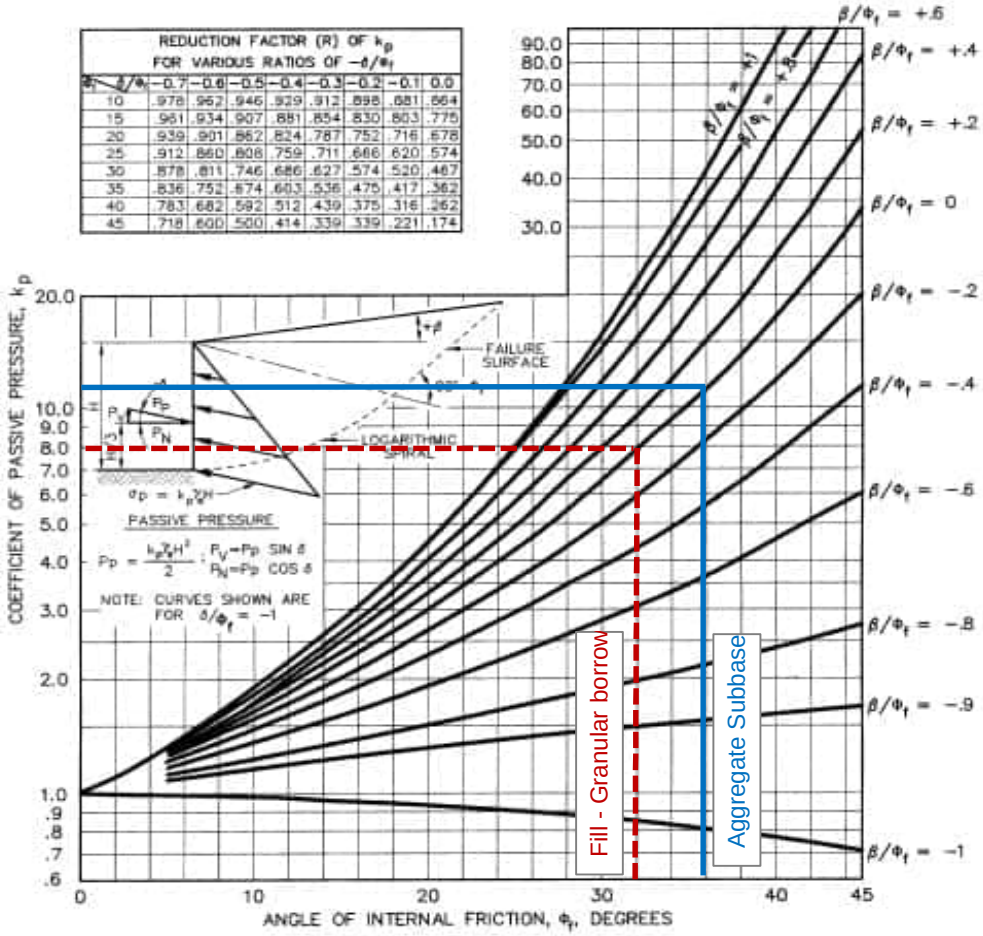


Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

For the Fill layer, and assuming a horizontal backfill surface, where:

ϕ_{Fill} =	32	degrees - Granular borrow (Ref. 2 Table 3-3)
β = Angle of backfill to the horizontal =	0	degrees (horizontal)
β/ϕ_{Fill} =	0	
Uncorrected $K_{p\ Fill}$ =	8.00	Ref. 3, Figure 3.11.5.4-2
δ = Friction angle between fill and wall =	17	Clean sand, silty sand-gravel mixture, Ref. 3, Table C3.11.5.3-1
δ/φ_{Fill}=	0.5	
Reduction Factor =	0.72	Ref. 3, Figure 3.11.5.4-2
$K_{p\ Fill}$ =	5.7	Ref. 3, Figure 3.11.5.4-2

For the Aggregate Subbase, and assuming a horizontal backfill surface, where:

$\phi_{\text{internal friction angle of Aggregate Subbase}}$ =	36	degrees (MaineDOT Gravel Borrow Ref. 2 Table 3-3)
β = Angle of backfill to the horizontal =	0	degrees (horizontal)
$\beta/\phi_{\text{Aggregate Subbase}}$ =	0	
Uncorrected $K_{p\ \text{Aggregate Subbase}}$ =	11	Ref. 3, Figure 3.11.5.4-2
δ = Friction angle between fill and wall =	22	Clean gravel, Ref. 3, Table C3.11.5.3-1
$\delta/\varphi_{\text{Aggregate Subbbase}}$ =	0.6	
Reduction Factor =	0.74	Ref. 3, Figure 3.11.5.4-2
$K_{p\ \text{Aggregate Subase}}$ =	8.1	Ref. 3, Figure 3.11.5.4-2

2. Determine the earth pressure acting on the superstructure backwall and rigid abutments.

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), semi-integral abutments should be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the soil surcharge from the pavement surface to the bottom of the backwall. The superstructure backwall should be designed for full passive pressure.

Per AASHTO LRFD section 3.11.6.4 (Ref. 3) a live load Surcharge shall be applied where vehicular load is expected to act on the surface of the backfill. A live load surchage is included in the lateral earth pressure acting over the superstructure back wall and the rigid abutment.

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

2.1. Determine the unfactored passive pressure on the superstructure backwall.

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), the superstructure backwall should be designed for full passive pressure, and include a live load surcharge where vehicular load is expected to act on the surface of the backfill as indicated in AASHTO LRFD section 3.11.6.4 (Ref. 3). Figure 1 shows an schematic of the earth pressure distribution behind the backwall, the materials show in the schematic are based on the materials required in the settlement evaluation.

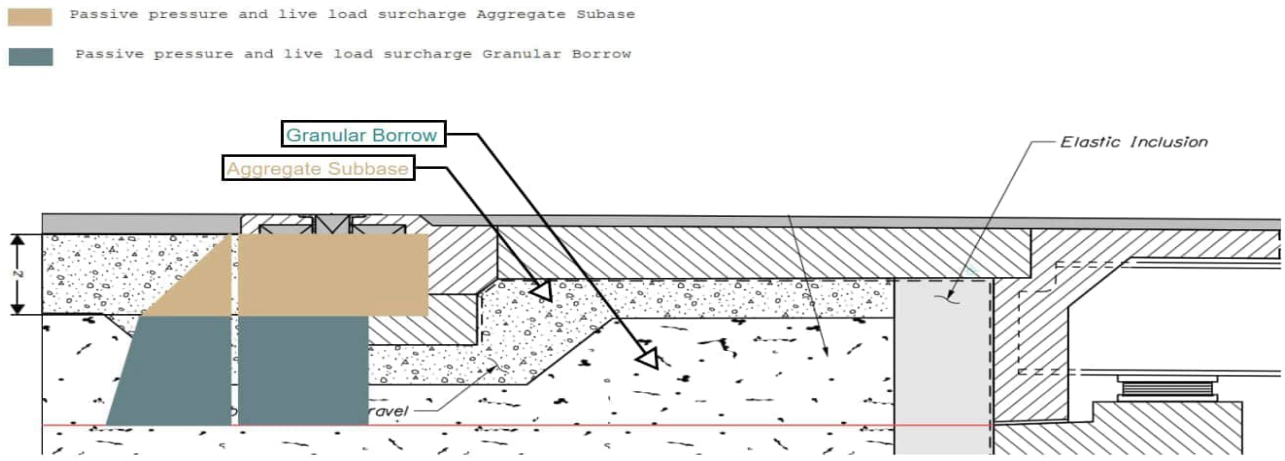


Figure 1. Diagram of earth pressure distribution on the superstructure backwall.

The increase in horizontal pressure due to live load surcharge is estimated based on AASHTO LRFD section 3.11.6.4 (Ref. 3)

$$\Delta_{p_i} = k_{p_i} \cdot \gamma_{s_i} \cdot h_{eq}$$

where:

Δ_{p_i} = constant horizontal earth pressure due to live load surcharge

k_{p_i} = coefficient of lateral earth pressure - Per Ref.1 the passive earth pressure coefficient is used in the backwall

γ_{s_i} = total unit weight of soil

h_{eq} = soil height, calculated up to the equivalent height of soil for vehicular load (Heq).

The force per foot of backwall width due to live load surcharge (LS) is calculated as:

$$P_{\Delta_p} = \Delta_{p_i} \cdot h_i$$

where:

h_i = soil height, calculated to the bottom of the backwall

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

Abutment 1

Average Backwall height (ft) = 4.0 (Ref. 7 Sheet 6)
Rigid abutment height (ft) = 17.0 (Ref. 7 Sheet 6)
Abutment height (ft) = 21.0
Heq (ft) = 2.0 Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment height.

Heq- LS Aggregate Subbase (ft) = 1.2
Heq- Granular Borrow (ft) = 0.8

Layer	Depth below top of the backwall (ft)	Unit weight (pcf)	Kp (-) - per section 1.4	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [2]
LS - Aggregate Subbase [1]	0 - 1.2	135	8.1	2122	1.2	2547	3.44
LS - Granular Borrow	1.2 - 4.0	125	5.7	1493	2.8	4241	1.42

Notes:
1. The height of the Granular Borrow thickness is based on the proposed grade raise on Abutment 1 per Ref. 5 Sheet 53.
2. Moment arm measured from the base of the backwall.

Abutment 2

Average Backwall height (ft) = 5.0 (Ref. 7 Sheet 8)
Rigid abutment height (ft) = 17.48 (Ref. 7 Sheet 8)
Abutment height (ft) = 22.5
Heq (ft) = 2.0 Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment height.

Heq- LS Aggregate Subbase (ft) = 2.0
Heq- Existing Fill (ft) = 0.0

Layer	Depth below top of the backwall (ft)	Unit weight (pcf)	Kp (-) - per section 1.4	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [2]
LS - Aggregate Subbase [1]	0 - 3	135	8.1	2187	3	6561	3.53
LS - Granular Borrow	3 - 5.0	125	5.7	1539	2.0	3116	1.01

Notes:
1. The height of the Granular Borrow thickness is based on the proposed grade raise on Abutment 2 per Ref. 5 Sheet 53.
2. Moment arm measured from the base of the backwall.

Calculate the passive pressure behind the backwall, using the earth pressure diagram shown on Fig.1 .

$$P_p = P_{p\ Aggregate\ Subbase} + P_{p\ Fill-triangular} + P_{p\ Fill-rectangular}$$

$$P_{p\ Aggregate\ Subbase} = \frac{1}{2} \cdot \gamma_{Aggregate\ Subbase} \cdot H_{Aggregate\ Subbase}^{\ 2} \cdot k_{p\ Aggregate\ Subbase}$$

$$P_{p\ Fill-triangular} = \frac{1}{2} \cdot \gamma_{Fill} \cdot H_{Fill}^{\ 2} \cdot k_{p\ Fill}$$

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

$$P_{p\ Fill\ -rectangular} = \gamma_{Aggregate\ Subbase} \cdot H_{Aggregate\ Subbase} \cdot H_{Fill} \cdot k_{p\ Fill}$$

Abutment 1
Average Backwall height (ft) = 4.04 (Ref. 7 Sheet 6)

Pressure	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	h_i (ft)	P (lbs per foot)	d (ft) [1]
P_p Agg. Subbase	0	-	1.2	135	8.1	1.2	787	3.24
P_p Fill - triangular	1.2	-	4.04	125	5.7	2.84	2873	0.95
P_p Fill - rectangular	1.2	-	4.04	135	5.7	2.84	2622	1.42

Notes:
1. Moment arm measured from the base of the backwall.

Abutment 2
Average Backwall height (ft) = 5.03 (Ref. 7 Sheet 8)

Pressure	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	h_i (ft)	P (lbs per foot)	d (ft) [1]
P_p Agg. Subbase	0	-	3	135	8.1	3	4921	3.03
P_p Fill - triangular	3	-	5.0	125	5.7	2.025	1461	0.67
P_p Fill - rectangular	3	-	5.0	135	5.7	2.025	4675	1.01

Notes:
1. Moment arm measured from the base of the backwall.

Calculate the passive pressure acting on the superstructure backwall and the elevation where the equivalent pressure acts.

Abutment 1

Pressure	P (lbs per foot)	d (ft)	Moment (lbs·ft per foot)
LS - Aggregate Subbase	2547	3.44	8760
LS - Granular Borrow	4241	1.42	6023
Pp Agg. Subbase	787	3.24	2551
Pp Fill - triangular	2873	0.95	2720
Pp Fill - rectangular	2622	1.42	3724

Abutment 1 - superstructure backwall P_p = 13,071 lbs per foot of backwall width

CALCULATIONS

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Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

The resultant lateral earth load acts at a height calculated as the resultant moment divided by the resultant lateral force. The resultant height (Hp) is measured from the the base of the backwall:

Abutment 1 - superstructure backwall H_p =	1.82	feet measured from the base of the backwall
Abutment 1 - Average superstructure backwall bottom elevation =	114.95	feet elevation (Ref. 7 Sheet 6)
For Abutment 1 backwall, the load Pp acts at:	116.8	feet elevation

Abutment 2

Pressure	P (lbs per foot)	d (ft)	Moment (lbs·ft per foot)
LS - Granular Borrow	6561	3.525	23128
LS - Granular Borrow	3116	1.0125	3155
Pp Agg. Subbase	4921	3.03	14885
Pp Fill - triangular	1461	0.67	986
Pp Fill - rectangular	4675	1.01	4733

Abutment 2 - superstructure backwall P_p = 20,734 lbs per foot of backwall width

The resultant lateral earth load acts at a height calculated as the resultant moment divided by the resultant lateral force. The resultant height (Hp) is measured from the the base of the backwall:

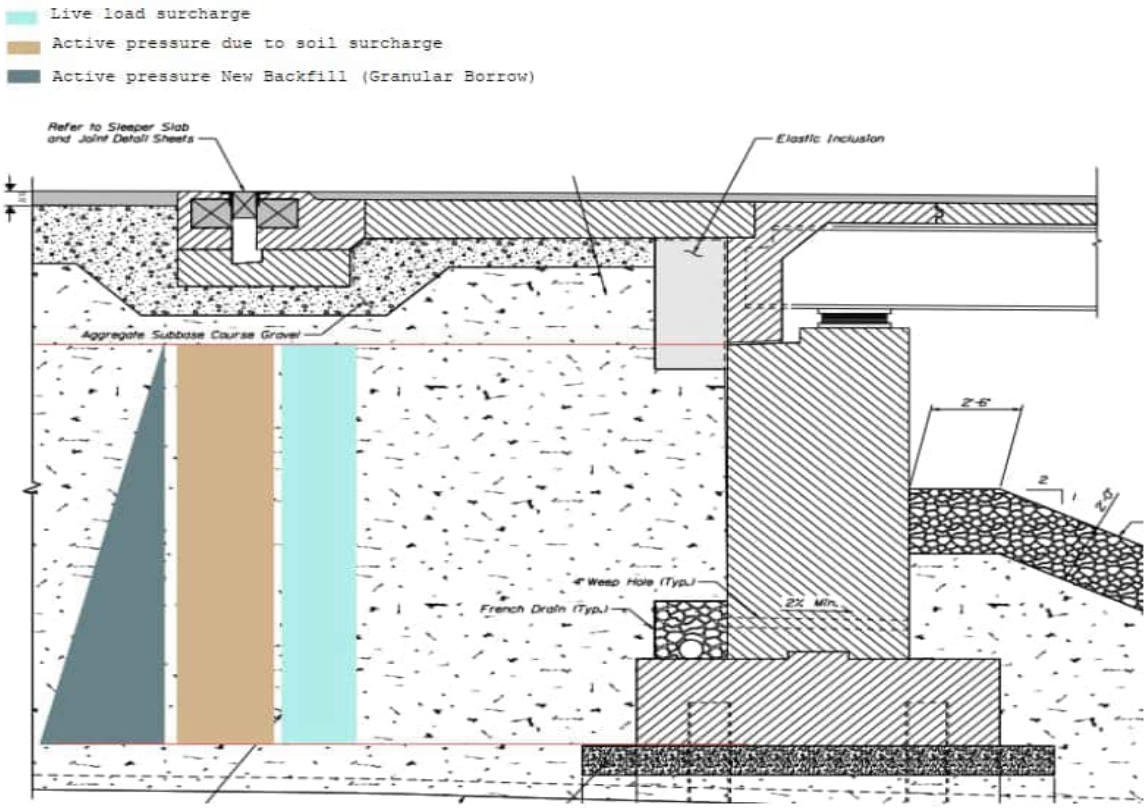
Abutment 2 - superstructure backwall H_p =	2.26	feet measured from the base of the backwall
Abutment 2 - superstructure backwall bottom elevation =	111.48	feet elevation (Ref. 7 Sheet 8)
For Abutment 2 backwall, the load Pp acts at:	113.7	feet elevation

2.2. Determine the unfactored active pressure in the rigid abutment .

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), for semi-integral abutments the rigid abutments should be designed for full active earth pressure over the rigid abutment height and a uniform pressure distribution due to the soil surcharge from the pavement surface to the bottom of the backwall. A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill as indicated in AASHTO LRFD section 3.11.6.4 (Ref. 3). Figure 2 shows an scheme of the earth pressure distribution behind the rigid abutment, the materials show in the schematic are based on the materials required in the settlement evaluation.

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		



The increase in horizontal pressure due to live load surcharge is estimated based on AASHTO LRFD section 3.11.6.4 (Ref. 3)

$$\Delta_{a_i} = k_{a_i} \cdot \gamma_{s_i} \cdot h_{eq}$$

where:

Δ_{a_i} = constant horizontal earth pressure due to live load surcharge
k_{a_i} = active leteral earth pressure coefficient of Existing Fill
γ_{s_i} = total unit calculated up to Heq (measured from the top of the abutment)
h_{eq} = soil height, calculated up to the equivalent height of soil for vehicular load (Heq).

The force per foot of backwall width due to live load surcharge (LS) is calculated as:

$$P_{\Delta_a} = \Delta_{a_i} \cdot h_i$$

where:

h_i = soil height, calculated to the bottom of the rigid abutment

Calculate the pressure due to soil surcharge:

$$P_{a\ surcharge} = k_{a-Fill} \cdot H_{abut} \cdot \sigma_{v\ -top\ of\ rigid\ abutment}$$

where:

CALCULATIONS

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Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

$\sigma_{v\,-top\ of\ rigid\ abutment}$ = soil surcharge above the rigid abutment

Calculate the Fill active pressure:

$$P_a = \frac{1}{2} \cdot \gamma_{Fill} \cdot H_{abut}{}^2 \cdot k_{a-Fill}$$

Abutment 1

Abutment height (ft) =	21.0	
Rigid abutment height (ft) =	17.0	(Ref. 7 Sheet 6)
Fill unit weight (pcf) =	125	(Ref. 2)
k_aFill =	0.31	
Heq (ft) =	2.0	Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment height.

Soil surcharge above the rigid abutmet:

Layer	Depth below top of the backwall (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Aggregate Subbase	0.5	-	1.2	135	0.7	170
Granular Borrow	1.2	-	4.0	125	2.84	525

$\sigma_{v\,-top\ of\ rigid\ abutment}$ = 525 psf

Pressure	P (lbs per foot)	d (ft) [1]	Moment (lbs·ft per foot)
Live Load Surcharge	1365	8.5	11564
P_a Soil Surcharge	2732	8.5	23150
P_a Fill	5517	5.7	31173

Notes:
1. Moment arm measured from the base of the abutment.

Abutment 1 - rigid abutment P_a = 9,613 lbs per foot of abutment width

The resultant lateral earth load acts at a height calculated as the resultant moment divided from the resultant lateral force. The resultat height (H_a) is measured from the the base of the rigid abutment:

Abutment 1 - rigid abutment H_a =	6.85	feet measured from the base of the rigid abutment
Abutment 1 - pile cap elevation =	98.00	feet elevation (Ref. 7 Sheet 6)
For Abutment 1 rigid abutment, the load P_a acts at:	104.9	feet elevation

Abutment 2

Abutment height (ft) =	22.5	
Rigid abutment height (ft) =	17.5	(Ref. 7 Sheet 8)
Fill unit weight (pcf) =	125	(Ref. 2)
k_aFill =	0.31	
Heq (ft) =	2.0	Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment height.

CALCULATIONS

Date:	4/25/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #5823 Abutments	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

Soil surcharge above the rigid abutmet:

Layer	Depth below top of the backwall (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Gravel Borrow	0.5	-	3	135	2.5	413
Existing Fill	3	-	5.025	125	2.025	666

$\sigma_{v\,-top\ of\ rigid\ abutment}$ = 666 psf

Pressure	P (lbs per foot)	d (ft) [1]	Moment (lbs·ft per foot)
Live Load Surcharge	1450	8.7	12674
P_a Soil Surcharge	3575	8.7	31245
P_a Fill	5868	5.8	34189

Notes:
1. Moment arm measured from the base of the abutment.

Abutment 2 - rigid abutment P_a = 10,893 lbs per foot of abutment width

The resultant lateral earth load acts at a height calculated as the resultant moment divided from the resultant lateral force. The resultat height (Hp) is measured from the the base of the rigid abutment:

Abutment 2 - rigid abutment Ha = 7.17 feet measured from the base of the rigid abutment
Abutment 2 - pile cap elevation = 94.00 feet elevation (Ref. 7 Sheet 8)
For Abutment 2 rigid abutment, the load P_a acts at: 101.2 feet elevation

CONCLUSIONS

The semi-integral abutments should be designed for passive pressure on the superstructure backwall, and active pressure for the rigid abutments. A live load surcharge was applied to account for vehicular loading in both the superstructure backwall and rigid abutments. For the recommended soil parameters and given abutment geometry, the passive earth force acting on the superstructure backwall is P_p = 13,071 pounds per linear foot for Abutment 1 acting at elevation 116.8 feet and P_p = 20,734 pounds per linear foot for Abutment 2 acting at elevation 113.7 feet. The active earth force acting on the rigid abutment is P_a = 9,613 pounds per linear foot for Abutment 1 acting at elevation 104.9 feet and P_a = 10,893 pounds per linear foot for Abutment 2 acting at elevation 101.2 feet.

CALCULATIONS

Date:	2/27/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #6787	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

OBJECTIVE

Determine lateral earth pressures acting on the proposed Abutment 1 located at STA 210+52 and Abutment 2 located at STA 215+05 of the proposed Bridge #6787. The lateral earth pressure on the proposed semi-integral abutments is calculated on the superstructure back wall and the rigid abutment.

REFERENCES

1. Das, Braja M. 2011. Principles of Foundation Engineering, 7th ed. Cengage Learning, Stamford, CT.
2. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
3. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
4. State of Maine Department of Transportation, Bangor, Penobscot County, Hogan Road Bridge over I-95, Hogan Road West Bound (Sheets B1 - B49), Dated 8/30/2024 (60% DDI Plans).
5. 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 47, no. 1 (January/February 2024): 351–370. https://doi.org/10.1520/GTJ20220258
6. WSP Settlement Analysis titled: WSP DRAFT Hogan Rd FGA Extents and Thicknesses with Cross-Sections 2025-02.pdf

ASSUMPTIONS

1. The backfill surface behind each abutment is assumed to be horizontal.
2. The backfill is assumed to be free draining (i.e., no water pressure is allowed to build up behind the abutment wall).
3. The interface friction angle of concrete to Foam Glass Aggregate (UL-FGA) is assumed to equal the value of clean gravel on concrete.
4. The unit weight of the Foam Glass Aggregate (UL-FGA) was assumed as 20 pcf, this value is based on engineering judgement and project-specific laboratory data for othe projects. The friction angle of the FGA was assumed to be 35.5 degrees based on the average laboratory results reported on Ref. 5.
5. The thickness of the FGA on each abutment was determined based on the WSP settlement analysis reported in Ref. 6, using the geometry on STA210+00 for Abutment 1 and STA215+00 for Abutment 2.

CALCULATION

1. Calculate At-Rest, Active and Passive pressure coefficients.

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

$$K_0 = 1 - \sin\phi$$ (Ref. 1, Eqn. 13.5) For coarse-grained soils

where:

$\phi_{\text{internal friction UL-FGA}}$ = 35.5 degrees - Ultra-Lightweight Foam Glass Aggregate (UL-FGA)

$K_{0\ \text{UL-FGA}}$ = 0.42 (Assumption 4)

CALCULATIONS

Date:	2/27/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Lateral Earth Pressure at the proposed Bridge #6787	**Reviewed by:**	CCB
Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

1.2. Calculate Rankine active earth pressure coefficient using Rankine theory.

$$K_a = \tan^2\left(45° - \frac{\Phi}{2}\right)$$ (Ref. 2, page 3-7) For a horizontal backfill

for horizontal backfill surface, where:

$\Phi_{\text{internal friction Granular Borrow}}$ = 32 degrees (MaineDOT Gravel Borrow Ref. 2 Table 3-3)

$K_{a\ \text{Granular Borrow}}$ = 0.31

$\Phi_{\text{internal friction UL-FGA}}$ = 35.5 degrees (Assumption 4)

$K_{a\ \text{UL-FGA}}$ = 0.27

1.4. Calculate the full passive earth pressure.

Per the MaineDOT Bridge Design Guide Section Chapter 3, section 3.6.6 (Ref. 2), Values of the coefficient of passive lateral earth pressure, K_p, may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD (Ref. 3).

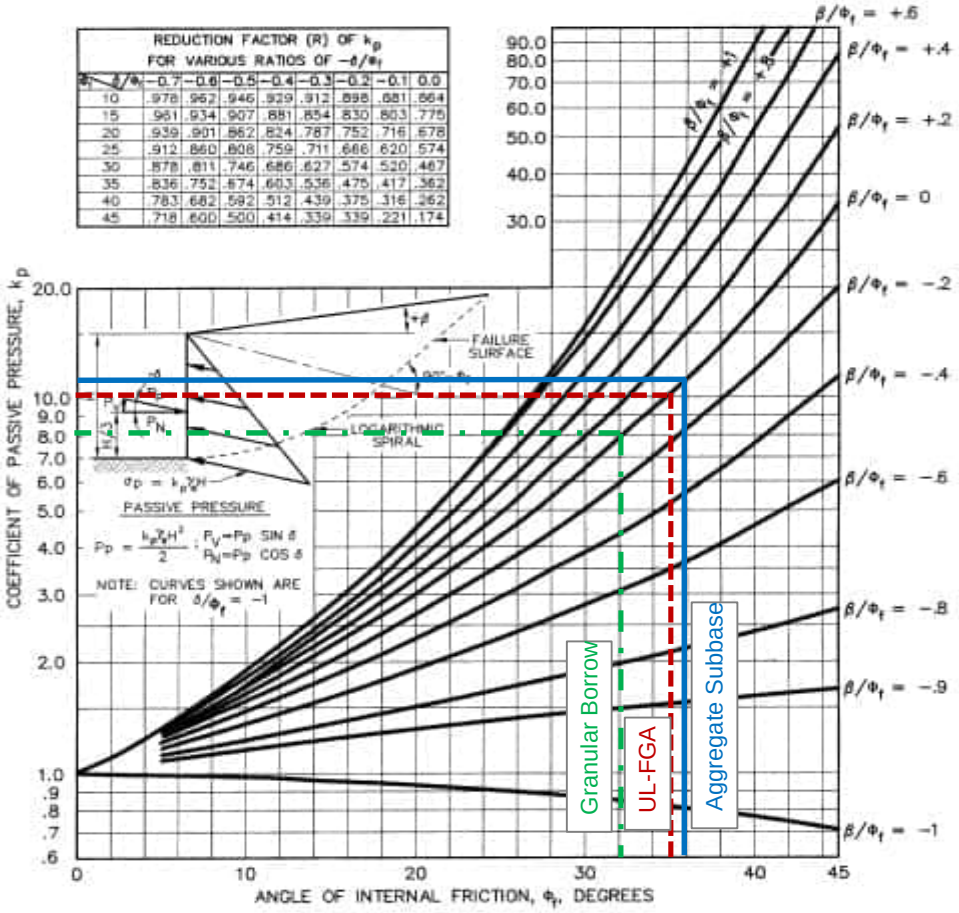

Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

CALCULATIONS

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For the Ultra-Lightweight Foam Glass Aggregate (UL-FGA), and assuming a horizontal backfill surface, where:

$\phi_{\text{internal friction UL-FGA}}$ =	35.5	degrees (foamed glass aggregate backfill)
β = Angle of backfill to the horizontal =	0	degrees (horizontal)
$\beta/\phi_{\text{UL-FGA}}$ =	0	
Uncorrected $K_{p\ \text{UL-FGA}}$ =	10	Ref. 3, Figure 3.11.5.4-2
δ = Friction angle between fill and wall =	22	Clean gravel to concrete, Ref. 3, Table C3.11.5.3-1
$\delta/\varphi_{\text{UL-FGA}}$ =	0.6	
Reduction Factor =	0.75	Ref. 3, Figure 3.11.5.4-2
$Kp_{\text{UL-FGA}}$ =	7.5	Ref. 3, Figure 3.11.5.4-2

For the Aggregate Subbase, and assuming a horizontal backfill surface, where:

$\phi_{\text{internal friction angle of Aggregate Subase}}$ =	36	degrees (MaineDOT Gravel Borrow Ref. 2 Table 3-3)
β = Angle of backfill to the horizontal =	0	degrees (horizontal)
$\beta/\phi_{\text{Aggregate Subase}}$ =	0	
Uncorrected $K_{p\ \text{Aggregate Subase}}$ =	11	Ref. 3, Figure 3.11.5.4-2
δ = Friction angle between fill and wall =	22	Clean gravel, Ref. 3, Table C3.11.5.3-1
$\delta/\varphi_{\text{Aggregate Subase}}$ =	0.6	
Reduction Factor =	0.74	Ref. 3, Figure 3.11.5.4-2
$K_{p\ \text{Aggregate Subase}}$ =	8.1	Ref. 3, Figure 3.11.5.4-2

For the Granular Borrow, and assuming a horizontal backfill surface, where:

$\phi_{\text{Granular borrow}}$ =	32	degrees (MaineDOT Gravel Borrow Ref. 2 Table 3-3)
β = Angle of backfill to the horizontal =	0	degrees (horizontal)
$\beta/\phi_{\text{Granular borrow}}$ =	0	
Uncorrected $K_{p\ \text{Granular Borrow}}$ =	8	Ref. 3, Figure 3.11.5.4-2
δ = Friction angle between fill and wall =	17	Clean sand, silty sand-gravel, Ref. 3, Table C3.11.5.3-1
$\delta/\varphi_{\text{Granular Borrow}}$ =	0.5	
Reduction Factor =	0.72	Ref. 3, Figure 3.11.5.4-2
$K_{p\ \text{Granular Borrow}}$ =	5.7	Ref. 3, Figure 3.11.5.4-2

2. Determine the earth pressure acting on the superstructure backwall and rigid abutments.

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), semi-integral abutments should be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the soil surcharge from the pavement surface to the bottom of the backwall. The superstructure backwall should be designed for full passive pressure.

Per AASHTO LRFD section 3.11.6.4 (Ref. 3) a live load Surcharge shall be applied where vehicular load is expected to act on the surface of the backfill. A live load surchage is included in the lateral earth pressure acting over the superstructure back wall and the rigid abutment.

CALCULATIONS

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2.1. Determine the unfactored passive pressure on the superstructure backwall.

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), the superstructure backwall should be designed for full passive pressure, and include a live load surcharge where vehicular load is expected to act on the surface of the backfill as indicated in AASHTO LRFD section 3.11.6.4 (Ref. 3). Figure 1 shows an schematic of the earth pressure distribution behind the backwall, the materials show in the schematic are based on the materials required in the settlement evaluation.

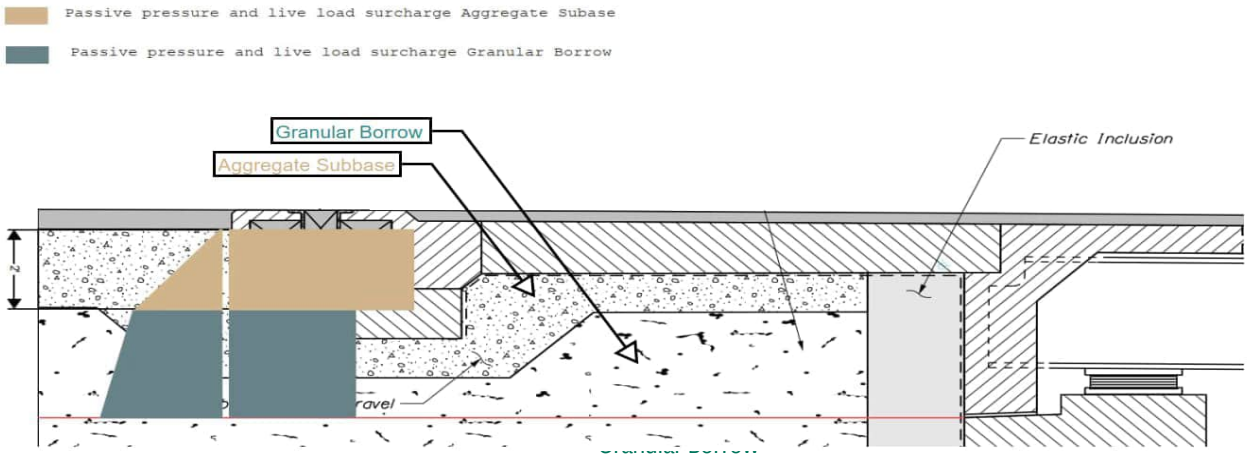

Figure 1. Diagram of earth pressure distribution on the superstructure backwall.

The increase in horizontal pressure due to live load surcharge is estimated based on AASHTO LRFD section 3.11.6.4 (Ref. 3)

$$\Delta_{p_i} = k_{p_i} \cdot \gamma_{s_i} \cdot h_{eq}$$

where:

Δ_{p_i} = constant horizontal earth pressure due to live load surcharge

k_{p_i} = coefficient of lateral earth pressure - Per Ref.1 the passive earth pressure coefficient is used in the backwall

γ_{s_i} = total unit weight of soil

h_{eq} = soil height, calculated up to the equivalent height of soil for vehicular load (Heq).

The force per foot of backwall width due to live load surcharge (LS) is calculated as:

$$P_{\Delta_p} = \Delta_{p_i} \cdot h_i$$

where:

h_i = soil height, calculated to the bottom of the backwall

Abutment 1

Average Backwall height (ft) =	5.3	(Ref. 4 Sheet B20)
Rigid abutment height (ft) =	13.0	(Ref. 4 Sheet B20)
Abutment height (ft) =	18.3	
Heq (ft) =	2.17	Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment

CALCULATIONS

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Heq- LS Aggregate Subbase (ft) = 2.00

Heq- Granular Borrowl (ft) = 0.17

Layer	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [1]
LS - Aggregate Subbase	0	-	2	135	8.1	2360	2	4720	4.26
LS - Granular Borrow	2	-	5.3	125	5.7	1661	3.3	5414	1.63

Notes:
1. Moment arm measured from the base of the backwall.

Abutment 2

Average Backwall height (ft) = 5.2 (Ref. 4 Sheet B23)

Rigid abutment height (ft) = 10.4 (Ref. 4 Sheet B23)

Abutment height (ft) = 15.6

Heq (ft) = 2.44 Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment

Heq- LS Aggregate Subbase (ft) = 2.00

Heq- Existing Fill (ft) = 0.44

Layer	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [1]
LS - Aggregate Subbase	0	-	2	135	8.1	2633	2	5265	4.23
LS - Granular Borrow	2	-	5.23	125	5.7	1853	3.23	5984	1.62

Notes:
1. Moment arm measured from the base of the backwall.

Calculate the passive pressure behind the backwall, using the earth pressure diagram shown on Fig.1 .

$$P_p = P_{p\ Aggregagate\ Subbase} + P_{p\ Granular\ Borrow\ -triangular} + P_{p\ Granular\ Borrow\ -rectangular}$$

$$P_{p\ Aggregagate\ Subbase} = \frac{1}{2} \cdot \gamma_{Aggregate\ Subbase} \cdot H_{Aggregate\ Subbase}^{\ 2} \cdot k_{p\ Aggregate\ Subbase}$$

$$P_{p\ Granular\ Borrow\ -triangular} = \frac{1}{2} \cdot \gamma_{Granular\ Borrow} \cdot H_{Granular\ Borrow}^{\ 2} \cdot k_{p\ Granular\ Borr}$$

$$P_{p\ Granular\ Borrow\ -rectangular} = \gamma_{Aggregate\ Subbase} \cdot H_{Aggregate\ Subbase} \cdot H_{Granular\ Borrow} \cdot k_{p\ Granular\ Borrow}$$

CALCULATIONS

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Abutment 1
Average Backwall height (ft) = 5.26 (Ref. 4 Sheet B20)

Pressure	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	h_i (ft)	P (lbs per foot)	d (ft) [1]
Fp aggregate Subbase	0	-	2	135	8.1	2	2187	3.93
Fp Granular Borrow- triangular	2	-	5.26	125	5.7	3.26	3786	1.09
Fp Granular Borrow - rectangular	2	-	5.26	135	5.7	3.26	5017	1.63

Notes:
1. Moment arm measured from the base of the backwall.

Abutment 2
Average Backwall height (ft) = 5.2 (Ref. 4 Sheet B23)

Pressure	Depth below top of the backwall (ft)			Unit weight (pcf)	Kp (-) - per section 1.4	h_i (ft)	P (lbs per foot)	d (ft) [1]
Fp aggregate Subbase	0	-	2	135	8.1	2	2187	3.90
Fp Granular Borrow- triangular	2	-	5.23	125	5.7	3.23	3717	1.08
Fp Granular Borrow - rectangular	2	-	5.23	135	5.7	3.23	4971	1.62

Notes:
1. Moment arm measured from the base of the backwall.

Calculate the passive pressure acting on the superstructure backwall and the elevation where the equivalent pressure acts.

Abutment 1

Pressure	P (lbs per foot)	d (ft)	Moment (lbs·ft per foot)
LS - Aggregate Subbase	4720	4.26	20108
LS - Granular Borrow	5414	1.63	8825
Fp aggregate Subbase	2187	3.93	8588
Fp Granular Borrow- triangular	3786	1.09	4114
Fp Granular Borrow - rectangular	5017	1.63	8178

Abutment 1 - superstructure backwall P_p = 21,125 lbs per foot of backwall width

CALCULATIONS

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The resultant lateral earth load acts at a height calculated as the resultant moment divided by the resultant lateral force. The resultant height (Hp) is measured from the the base of the backwall:

Abutment 1 - superstructure backwall H_p =	2.4	feet measured from the base of the backwall
Abutment 1 - superstructure backwall bottom elevation =	114.5	feet elevation (Ref. 4 Sheet B20)
For Abutment 1 backwall, the load Pp acts at:	116.9	feet elevation

Abutment 2

Pressure	P (lbs per foot)	d (ft)	Moment (lbs·ft per foot)
LS - Aggregate Subbase	5265	4.23	22271
LS - Granular Borrow	5984	1.62	9663
Fp aggregate Subbase	2187	3.90	8522
Fp Granular Borrow- triangular	3717	1.08	4002
Fp Granular Borrow - rectangular	4971	1.62	8028

Abutment 2 - superstructure backwall P_p = 22,123 lbs per foot of backwall width

The resultant lateral earth load acts at a height calculated as the resultant moment divided by the resultant lateral force. The resultant height (Hp) is measured from the the base of the backwall:

Abutment 2 - superstructure backwall H_p =	2.4	feet measured from the base of the backwall
Abutment 2 - superstructure backwall bottom elevation =	109.4	feet elevation (Ref. 4 Sheet B23)
For Abutment 2 backwall, the load Pp acts at:	111.7	feet elevation

2.2. Determine the unfactored active pressure in the rigid abutment .

Per the MaineDOT Bridge Design Guide Section 5.4.3 (Ref. 2), for semi-integral abutments the rigid abutments should be designed for full active earth pressure over the rigid abutment height and a uniform pressure distribution due to the soil surcharge from the pavement surface to the bottom of the backwall. A live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill as indicated in AASHTO LRFD section 3.11.6.4 (Ref. 3). Figure 2 shows an scheme of the earth pressure distribution behind the rigid abutment, materials show are required based on the materials required in the settlement evaluation.

CALCULATIONS

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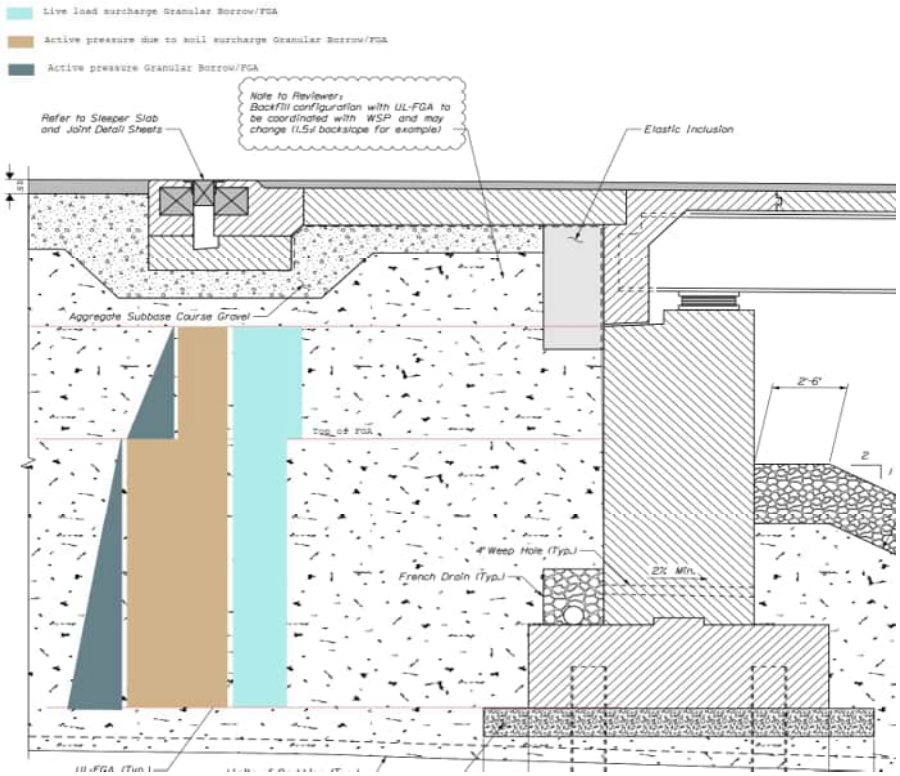

Figure 2. Diagram of earth pressure distribution on the rigid abutment.

The increase in horizontal pressure due to live load surcharge is estimated based on AASHTO LRFD section 3.11.6.4 (Ref. 3)

$$\Delta_{a_i} = k_{a_i} \cdot \gamma_{s_i} \cdot h_{eq}$$

where:

Δ_{p_i} = constant horizontal earth pressure due to live load surcharge
k_{a_i} = active leteral earth pressure coefficient of Granular Borrow or FGA
γ_{s_i} = total unit calculated up to Heq (measured from the top of the abutment)
h_{eq} = soil height, calculated up to the equivalent height of soil for vehicular load (Heq).

The force per foot of backwall width due to live load surcharge (LS) is calculated as:

$$P_{\Delta_a} = \Delta_{a_i} \cdot h_i$$

where:

h_i = soil height, calculated to the bottom of the rigid abutment

Calculate the pressure due to soil surcharge:

$$P_{a\ surcharge} = k_{a_i} \cdot h_i \cdot \sigma_{v\ -top\ of\ point\ i}$$

where:

$\sigma_{v\ -top\ of\ point\ i}$ = soil surcharge above the layer

CALCULATIONS

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Calculate the active pressure:

$$P_a = \frac{1}{2} \cdot \gamma_{si} \cdot h_i^2 \cdot k_{a_i}$$

Abutment 1

Abutment height (ft) =	18.3	
Rigid abutment height (ft) =	13.0	(Ref. 4 Sheet B20)
Heq (ft) =	2.17	Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment
Heq- LS Aggregate Subbase (ft) =	2.00	
Heq- Granular Borrow (ft) =	0.17	

Calculate pressure due to live load surcharge:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	2.03	125	0.31	90	2.03	182	12.02
UL-FGA	2.03	-	13.03	20	0.27	77	11	850	5.50

Notes:
1. Moment arm measured from the base of the rigid abutment.

Soil surcharge above the rigid abutmet:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Aggregate Subbase	0.5	-	2	135	1.5	278
Granular Borrow	2	-	5.26	125	3.26	685

$\sigma_{v\ -top\ of\ rigid\ abutment}$ = 685 psf

Soil surcharge above the top of FGA:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Aggregate Subbase	0.5	-	2	135	1.5	278
Granular Borrow	2	-	7.29	125	5.29	939

$\sigma_{v\ -top\ of\ FGA}$ = 939 psf

CALCULATIONS

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Project Title:	MaineDOT Bangor I-95 Exit 187 DDI, ME		

Calculate pressure due to soil surcharge:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	h_i (ft)	$P_{a\text{- surcharge}}$ (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	2.03	125	0.31	2.03	427	12.0
UL-FGA	2.03	-	13.03	20	0.27	11.00	2739	5.5

Notes:
1. Moment arm measured from the base of the rigid abutment.

Calculate the soil active pressure:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	h_i (ft)	P_a (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	2.03	125	0.31	2.03	79	11.7
UL-FGA	2.03	-	13.03	20	0.27	11.00	321	3.7

Notes:
1. Moment arm measured from the base of the rigid abutment.

Pressure	P (lbs per foot)	d (ft) [1]	Moment (lbs·ft per foot)
LS - Granular Borrow	182	12.0	2184
LS - UL-FGA	850	5.5	4676
Surcharge - Granular Borrow	427	12.0	5134
Surcharge - UL-FGA	2739	5.5	15065
P_a - Granular Borrow	79	11.7	924
P_a - UL-FGA	321	3.7	1177

Notes:
1. Moment arm measured from the base of the abutment.

Abutment 1 - rigid abutment P_a = 4,598 lbs per foot of abutment width

The resultant lateral earth load acts at a height calculated as the resultant moment divided from the resultant lateral force. The resultat height (H_a) is measured from the the base of the rigid abutment:

Abutment 1 - rigid abutment H_a =	6.34	feet measured from the base of the rigid abutment
Abutment 1 - pile cap elevation =	101.5	feet elevation (Ref. 4 Sheet B20)
For Abutment 1 rigid abutment, the load P_a acts at:	107.8	feet elevation

Abutment 2

Abutment height (ft) =	15.6	
Rigid abutment height (ft) =	10.4	(Ref. 4 Sheet B23)
Heq (ft) =	2.44	Interpolating linearly from Ref.3. Table 3.11.6.4-1, using the Abutment
Heq- LS Aggregate Subbase (ft) =	2.00	
Heq- Existing Fill (ft) =	0.44	

CALCULATIONS

Date:	2/27/2025	**Made by:**	LMP
Project No.:	31404817.004	**Checked by:**	ATM
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Calculate pressure due to live load surcharge:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	Δ_p (psf)	h_i (ft)	$P_{\Delta p}$ (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	3.37	125	0.31	100	3.37	337	8.7
UL-FGA	3.37	-	10.37	20	0.27	86	7	603	3.5

Notes:
1. Moment arm measured from the base of the rigid abutment.

Soil surcharge above the rigid abutmet:

Layer	Depth below top of the backwall (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Aggregate Subbase	0.5	-	2	135	1.5	278
Granular Borrow	2	-	5.23	125	3.23	681

$\sigma_{v\ -top\ of\ rigid\ abutment}$ = 681 psf

Soil surcharge above the top of FGA:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	h_i (ft)	σ_v (psf)
Pavement	0	-	0.5	150	0.5	75
Aggregate Subbase	0.5	-	2	135	1.5	278
Granular Borrow	2	-	8.6	125	6.6	1103

$\sigma_{v\ -top\ of\ FGA}$ = 1103 psf

Calculate pressure due to soil surcharge:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	h_i (ft)	$P_{a\text{- surcharge}}$ (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	3.37	125	0.31	3.37	705	8.7
UL-FGA	3.37	-	10.37	20	0.27	7.00	2047	3.5

Notes:
1. Moment arm measured from the base of the rigid abutment.

Calculate the soil active pressure:

Layer	Depth below top of the rigid abutment (ft)			Unit weight (pcf)	Ka (-) - per section 1.2	h_i (ft)	P_a (lbs per foot)	d (ft) [1]
Granular Borrow	0	-	3.37	125	0.31	3.37	218	8.1
UL-FGA	3.37	-	10.37	20	0.27	7.00	130	2.3

Notes:
1. Moment arm measured from the base of the rigid abutment.

CALCULATIONS

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Project No.:	31404817.004	**Checked by:**	ATM
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Pressure	P (lbs per foot)	d (ft) [1]	Moment (lbs·ft per foot)
LS - Granular Borrow	337	8.7	2923
LS - UL-FGA	603	3.5	2112
Surcharge - Granular Borrow	705	8.7	6126
Surcharge - UL-FGA	2047	3.5	7165
P_a - Granular Borrow	218	8.1	1772
P_a - UL-FGA	130	2.3	303

Notes:
1. Moment arm measured from the base of the abutment.

Abutment 2 - rigid abutment P_a = 4,041 lbs per foot of abutment width

The resultant lateral earth load acts at a height calculated as the resultant moment divided from the resultant lateral force. The resultat height (H_a) is measured from the the base of the rigid abutment:

Abutment 2 - rigid abutment H_a = 5.05 feet measured from the base of the rigid abutment

Abutment 2 - pile cap elevation = 99.0 feet elevation (Ref. 4 Sheet B23)

For Abutment 2 rigid abutment, the load P_a acts at: 104.0 feet elevation

CONCLUSIONS

The semi-integral abutments should be designed for passive pressure on the superstructure backwall, and active pressure for the rigid abutments. A live load surcharge was applied to account for vehicular loading in both the superstructure backwall and rigid abutments. For the recommended soil parameters and given wall geometry, the passive passive earth force acting on the superstructure backwall is P_p = 21,125 pounds per linear foot for Abutment 1 acting at elevation 116.9 feet and P_p = 22,123 pounds per linear foot for Abutment 2 acting at elevation 111.7 feet. The active earth force acting on the rigid abutment is P_a = 4,598 pounds per linear foot for Abutment 1 acting at elevation 107.8 feet and P_a = 4,041 pounds per linear foot for Abutment 2 acting at elevation 104.0 feet.

CALCULATIONS

Date:	9/16/2025	**Made by:**	MEL
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Bridge Abutment Elastic Inclusion	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Evaluate the type and thickness of an elastic inclusion behind bridge abutments for the renovated bridge and new bridge.

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO). LRFD Bridge Design Specifications. 9th Edition, dated 2020.

2. Hoppe, E.J, and Eichenthal, S.L. (2012). Thermal Response of a Highly Skewed Integral Abutment Bridge. FHWA / VCTIR 12-R10. https://vtrc.virginia.gov/media/vtrc/vtrc-pdf/vtrc-pdf/12-r10.pdf

3. Email form VHB to WSP, Subject "Abutment Displacements for Hogan Road - Elastic Inclusion", received on 5/15/25.

4. State of Maine Department of Transportation, Bangor, Penobscot County, Hogan Road over I-95, Hogan Road Eastbound (Sheets B5-B8), included in "DDI Hogan EB Abut Brg Loads 2025-04-23.pdf" provided by VHB

5. State of Maine Department of Transportation, Bangor, Penobscot County, Hogan Road over I-95, Hogan Road Westbound (Sheets B1-B49), dated 8/30/2024.

6. Beaver Plastic, G-TEC Elasticized Expanded Polystyrene, Product Data Sheet

REFERENCE SUMMARY

Hoppe & Eichenthal (2012)

Integral Abutment, 45-deg skew, 110 length, no approach slabs, field performance study

Terraflex Elasticized Geofoam Inclusion

$EPS_t = 10\,[0.01h + 0.5\Delta L]\cos(\theta)$, Ref. 2 Eq. 4

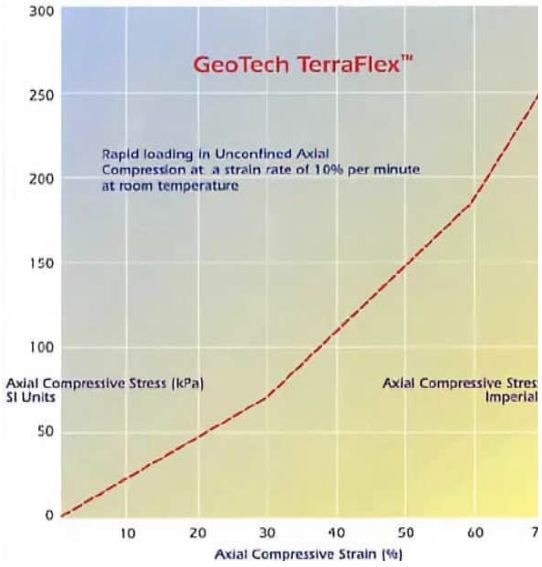
EPSt = required EPS thickness (in)

h = height of integral backwall/abutment (in)

ΔL = total thermally induced longitudinal movement at abutment (in)

θ = bridge skew angle

TerraFlex Elasticized Polystyrene:
https://universalconstructionfoam.com/products/elasticized-polystyrene/terraflex-elasticized/



Horvath, J.S. Geofoam Geosynthetics. Horvath Engineering, P.C., Scarsdale, N.Y., 1995

Elasticized expanded polystyrene (EPS) exhibits linear elastic stress strain behavior up to approximately 10 percent strain, while regular EPS is elastic up to approximately 1 percent strain. EPS is elasticized when its temporarily loaded to 60% to 70% strain.

CALCULATIONS

Date:	9/16/2025	**Made by:**	MEL
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Bridge Abutment Elastic Inclusion	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

CALCULATION

1. Calculate the required thickness of EPS following the method outlined in Hoppe and Eichenthal (Ref 2).

Eastbound Bridge

2.8	ΔL, assumed maximum longitudinal movement (in)	Ref. 3
48.49	θ; bridge skew angle, from VHB plan set (48º 29' 6")	Ref. 4
4.04	h; Height over which passive earth pressure acts - Abutment 1 (ft)	Ref. 4
48.5	h; Height over which active earth pressure acts - Abutment 1 (in)	
12	EPS_t (in)	
5.03	h; Height over which passive earth pressure acts - Abutment 2 (ft)	Ref. 4
60.4	h; Height over which passive earth pressure acts - Abutment 2 (in)	
13	EPS_t (in)	

Westbound Bridge

2.8	ΔL, assumed maximum longitudinal movement (in)	Ref. 3
46.50	θ; bridge skew angle, from VHB plan set (46º 30' 0")	Ref. 5
5.3	h; Height over which passive earth pressure acts - Abutment 1 (ft)	Ref. 5
63.6	h; Height over which passive earth pressure acts - Abutment 1 (in)	
14.0	EPS_t (in)	
5.2	h; Height over which passive earth pressure acts - Abutment 2 (ft)	Ref. 5
62.4	h; Height over which passive earth pressure acts - Abutment 2 (in)	
14.0	EPS_t (in)	

2. Calculate the required thickness of EPS based on the requirements detailed in the G-TEC Elasticized Expanded Polystyrene (EEPS) product data sheet from Universal Construction Foam.

From reference 6, "G-TEC should be specified in a thickness that will accommodate the required movement when compressed to 10 to 20% of its original thickness. 10% maximum compression should be specified when rapid (near daily) earth movement is anticipated. Up to 20% compression may satisfactorily serve less frequent cycling forces."

With the compression value taken as the movement normal to the backwall, 2 inches, the required thickness of the GTEC EEPS is calculated as:

2.0	Maximum longitudinal movement normal to the backwall - provided by VHB (in)
20.0	Minimum thickness of EEPS for 10% compression (in)
1.5	Assumed factor of safety, intended to provide additional compressional thickness
30.0	Recommended design thickness of EEPS (equivalent to 6.7% compression)

CONCLUSIONS

For the maximum abutment longitudinal movements provided by VHB and the proposed abutment geometry, a minimum thickness of EEPS of 30.0 inches is recommended. G-TEC is available in 1220-mm x 1220-mm panels (4' x 4'), in the required thickness.

APPENDIX I

Bridge 5823 Replacement
Abutment Foundation Calculations

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for EB Abutment 1 based on the design load case 2 (controlling) (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 1 (Pile 8), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road Eastbound Bridge Sheets 5 through 8 dated April 2025.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday April 23rd, 2025 with subject line "Bangor DDI WIN #1859510 Hogan EB Abutment Bearing Loads".
15. WSP calculation package titled Lateral Earth Pressure at the proposed East Bound Bridge Abutments, dated 2/27/2025.
16. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.

wsp

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. EB Abutment 1 Minimum and Maximum Rock Socket Design Typical Sections
5. Pier Loads

ASSUMPTIONS

1. The proposed EB Abutment 1 is assumed to be supported by twenty (20) HP 14x89 Grade 50 piles with a minimum center to center spacing of 6 feet in the x-direction and 7 feet in the y-direction shown in Attachment 1.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) were applied at the bearings on the abutment stem wall.
4. All piles are plumb.
5. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
6. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
7. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
8. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
9. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
10. A vertical earth load EV was added to the toe of the EB Abutment 1 pile cap to account for the granular borrow and crushed stone slope on the toe (Ref. 5), and a vertical earth load EV was added to the heal of the EB Abutment 1 pile cap to account for the backfill and traffic live load surcharge (Ref. 15). The EV loads on the toe and heal are added as point loads on the piles, as pile spacing is not uniform. The point loads are based on the pile tributary areas.
11. A horizontal earth load EH of 9,613 plf (Ref. 15) was added to the pile cap on the back side of the abutment within the area of the stem wall.
12. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assumed to begin 1-foot below the top of bedrock surface.
13. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	13.64	in	
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	14.5	in	
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Fillet area =	0.3	in^2		0.3	in^2	
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	19.3	in^2	
Section area =	0.18	ft^2		0.13	ft^2	
Moment of inertia about the x-axis, I_x =				653	in^4	
Moment of inertia about the y-axis, I_y =				233	in^4	
Radius of gyration about the x-axis, r_x =				5.82	in	
Radius of gyration about the y-axis, r_y =				3.47	in	
Elastic section modulus about the x-axis, S_x =				95.8	in^3	
Elastic section modulus about the y-axis, S_y =				32.1	in^3	
Plastic section modulus about the x-axis, Z_x =				105.7	in^3	
Plastic section modulus about the y-axis, Z_y =				48.8	in^3	

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	101.0	ft	Sheet B23 (Ref. 5)
Pile cap thickness =	3	ft	Sheet B23 (Ref. 5)
Pile cap midplane elevation =	99.5	ft	Sheet B23 (Ref. 5)
As-modeled pile cap midplane elevation =	99.5	ft	
Base of pile cap elevation =	98.0	ft	Sheet B23 (Ref. 5)
Pile length =	17.0	ft	
Socket length =	5	ft, includes upper 1-foot of bedrock (Assumption 10).	
Bottom of socket El. =	82.0	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
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Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	98	97	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	97.0	90.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	90.8	87.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	87.0	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 8)					
Maximum axial force in the piles, P_u =	258.8	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	188.6	kip-ft	=	2263.4	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	7.1	kip-ft	=	84.6	in-kips	
Maximum shear force in the piles, V_u =	38.2	kips				
Depth below pile head to fixity =	16.4	ft	=	Elev.	83.10	ft
Depth below pile cap to fixity =	14.9	ft	=	Elev.	83.10	ft

wsp

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.063	0.717	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.024	0.357	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		
	Top	Bottom	
Undrained shear strength, s_u =	1.340	1.340	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	(Ref. 7 Fig.
f_s =	1.179	1.179	ksf

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Nominal shaft resistance:

$$R_s = f_s A_s$$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = (d × 2) + (b_f × 2) = 56.4 in = 4.7 ft (from Part A)

	Granular Borrow	Glacial Till		
Thickness =	1	3.8	ft	(from stratigraphy in Part A)
A_s =	4.7	17.8	ft^2	
R_s per layer =	1.7	0.4	kips	

Total R_s = 2.1 kips

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length: 5 ft Grout below pile toe: 0.5 ft (Assumption 7)

Socket diameter: 30 in (Assumption 4)

Socket fill material: grout with compressive strength of 6,000 psi (Assumption 7)

Additional Rock Properties

Unit weight of rock =	158	pcf	(based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c).

$$q_p = A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u \left[m_b \frac{(\sigma'_{vb})}{q_u} + s \right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
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where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf	(from Step D)
Unit weight of rock =	158	pcf	(from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.5	ksf	(from stratigraphy in Step 3)
Geological Strength Index, GSI =	45		(from Step D)
Hoek-Brown material index constant, m_i =	19		(from Step D)
Disturbance factor, D =	0		(Ref. 16 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022		
a =	0.508		
m_b =	2.665		
A =	70	ksf	
Unit tip resistance q_p =	553	ksf	
Base area of pile shoe plate =	1.6	ft^2	(from Assumption 7)
Nominal tip resistance at base of rock socket =	864	kips	
Resistance factor =	0.50		(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	432	kips	

Compare tip resistance to axial load on pile:

Factored tip resistance		Strength I factored axial	
432	>	259	OK

Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf	(Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips				
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)				
Factored grout resistance beneath shoe plate =	945	kips				

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	259	OK

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	178.84	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 33 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 13)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$$\frac{b}{t} = 15.98 > 13.49 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
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where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	178.84	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	4,940	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.20 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 890 kips

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

P_r = 445 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r,\ axial/\ flexural}$ = 623 kips

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 259 kips (from Part B)

$P_{r,\,axial}$ = 445 kips > P_u = 259 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

wsp

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 2263 in-kips (from Part B)
M_{uy} = 85 in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	2263	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	85	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.4 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.86 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$V_n = CV_p = V_{cr}$

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y = 50 ksi (from Part A)
Web depth, d_w = 12.730 in $d_w = d - (2 \times t_f)$
Web thickness, t_w = 0.455 in (from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k = 5.0 (Ref. 7, Article 6.10.9.2)
D = 12.730 in D= d - (2 × tf)
t_w = 0.455 in (from Part A)

$1.12\sqrt{\frac{Ek}{F_y}}$ = 60.3 ksi (Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

V_p = 168 kips

V_n = 168 kips

$V_u \leq \varphi_v V_n$ φ_v = 1.00 (Ref. 7, Article 6.5.4.2)

V_r = 168 kips

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

V_u = 38.2 kips (from Part B)

V_r = 168 kips > V_u = 38.2 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 17.0 feet, a maximum axial force of 258.8 kips , the corresponding strong axis (x-axis) moment of 2,263 in-kips (188.6 ft-kips) occurs in the proposed HP14x89 piles at the Abutment 1 in Pile 8 of the proposed EB Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the service limit state is less than 0.3 inches in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 4.5-foot pile embedment into a 5.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone, see the minimum length and maxiumum length rock socket details in Attachment 4. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

EB Abutment 1 Summary Table		
Pile Length (ft)		17.0
Depth below Pile Head to Fixity (ft)		16.4
Geotechnical Axial Resistance (kips)	Nominal	864
	Factored	432
Structural Axial Resistance (kips)	Nominal	890
	Factored	445
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.86
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

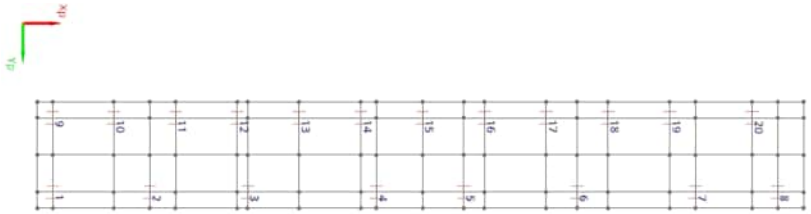


Figure 1b: Profile View of Pile Cap (Transverse)

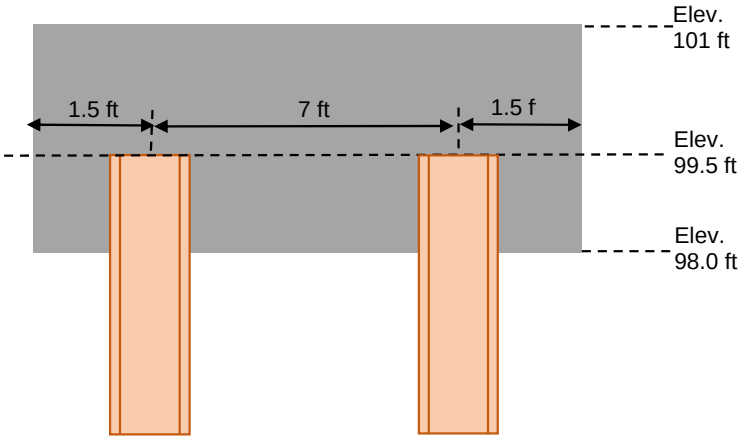


Figure 1c: 3D View of Pile Model

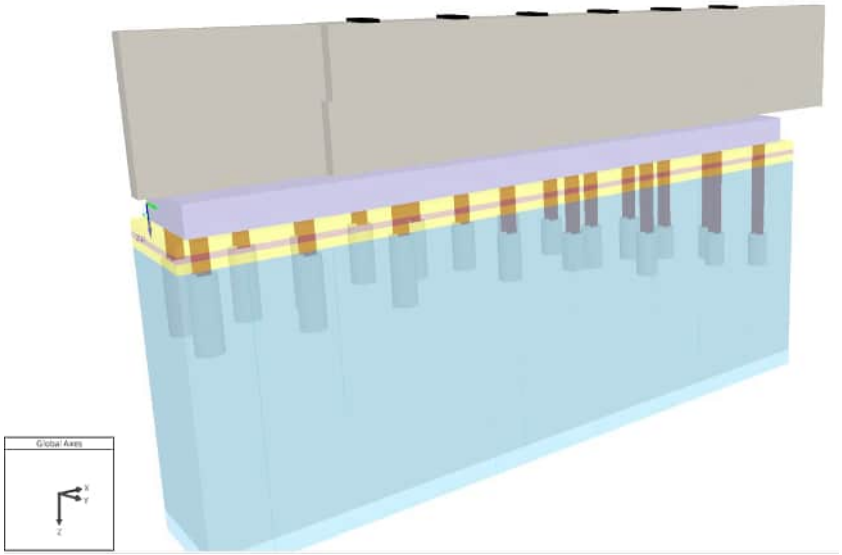
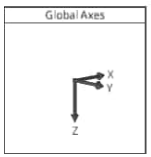


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	98.0	97.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	24		-	-	-
Stiff Clay	2	Cohesive	97.0	90.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	3	Cohesionless	90.8	87.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	357		-	-	-
Bedrock	4	Rock	87.0	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		4764.8	864.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on the EB Abutment 1 profile E-E'.

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

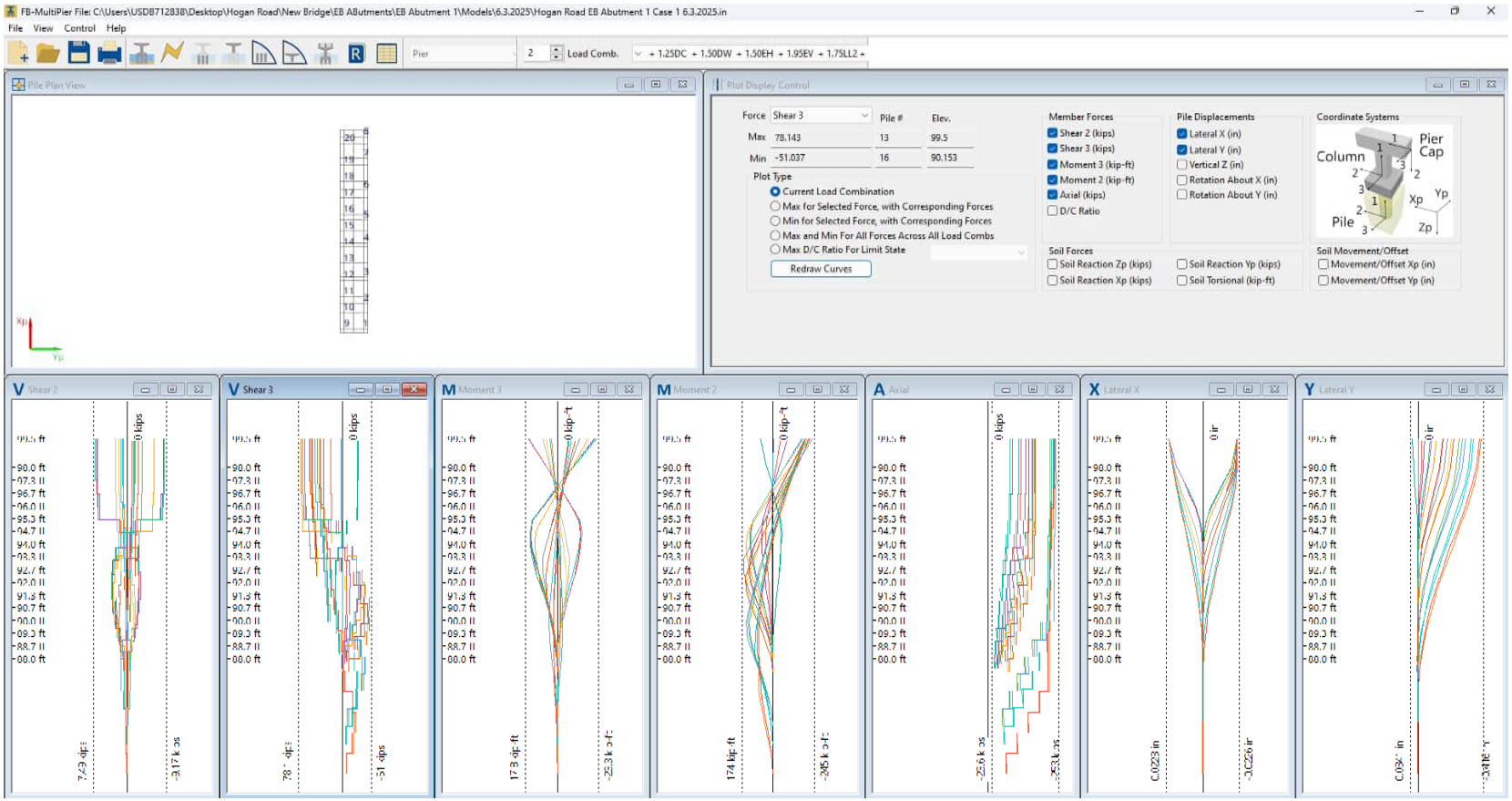
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

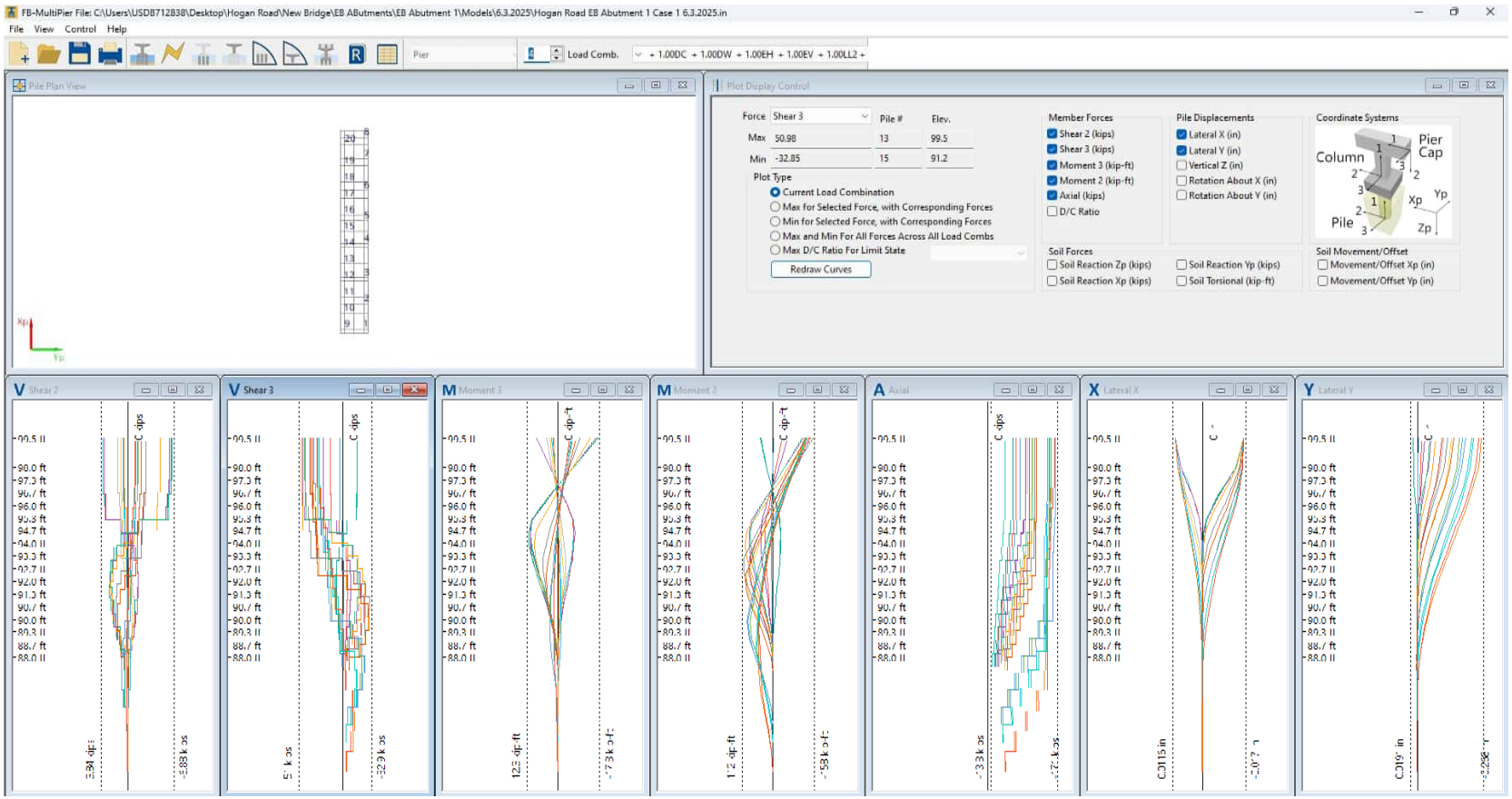
8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed:

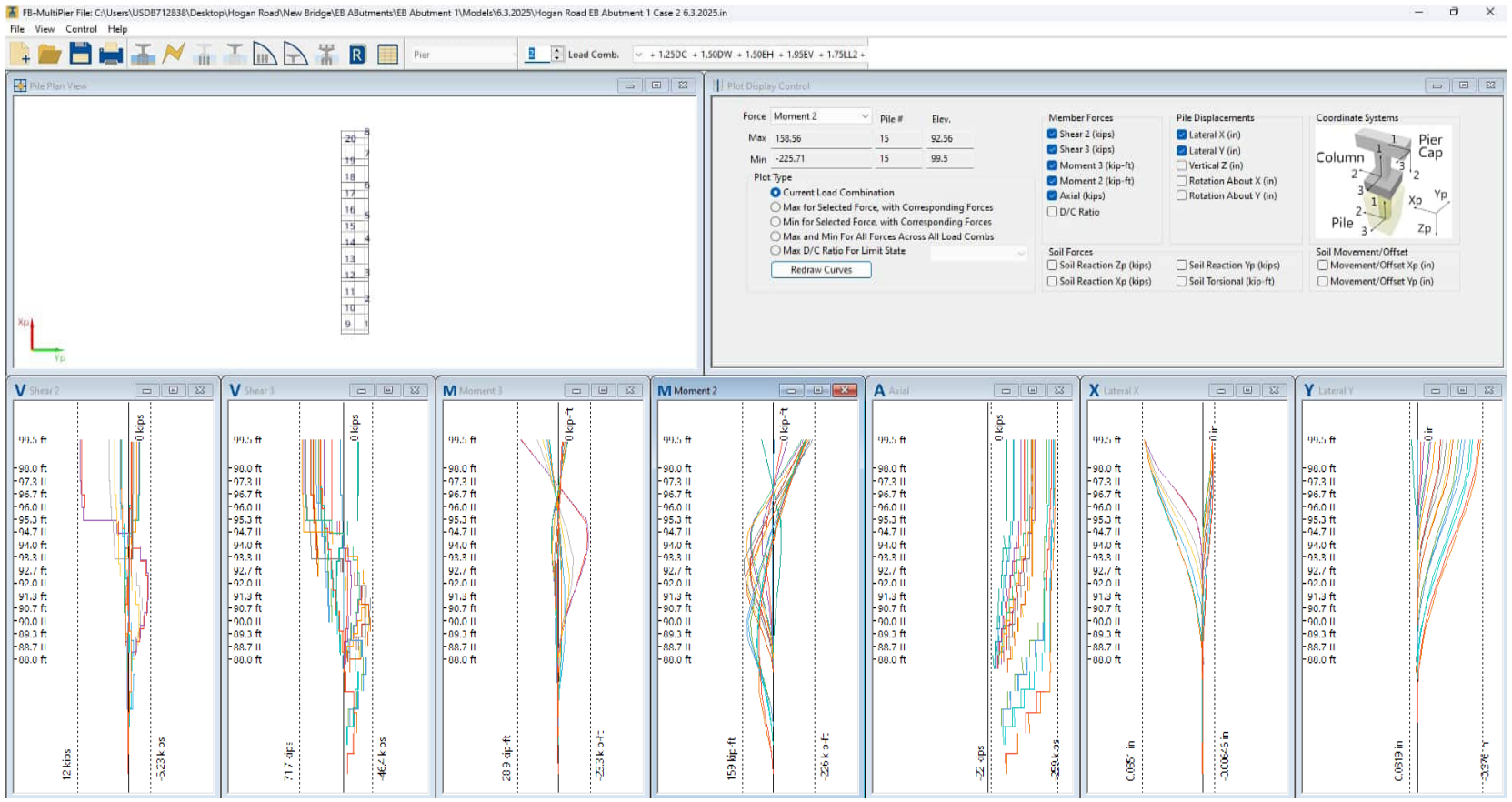
Case 1 Strength I



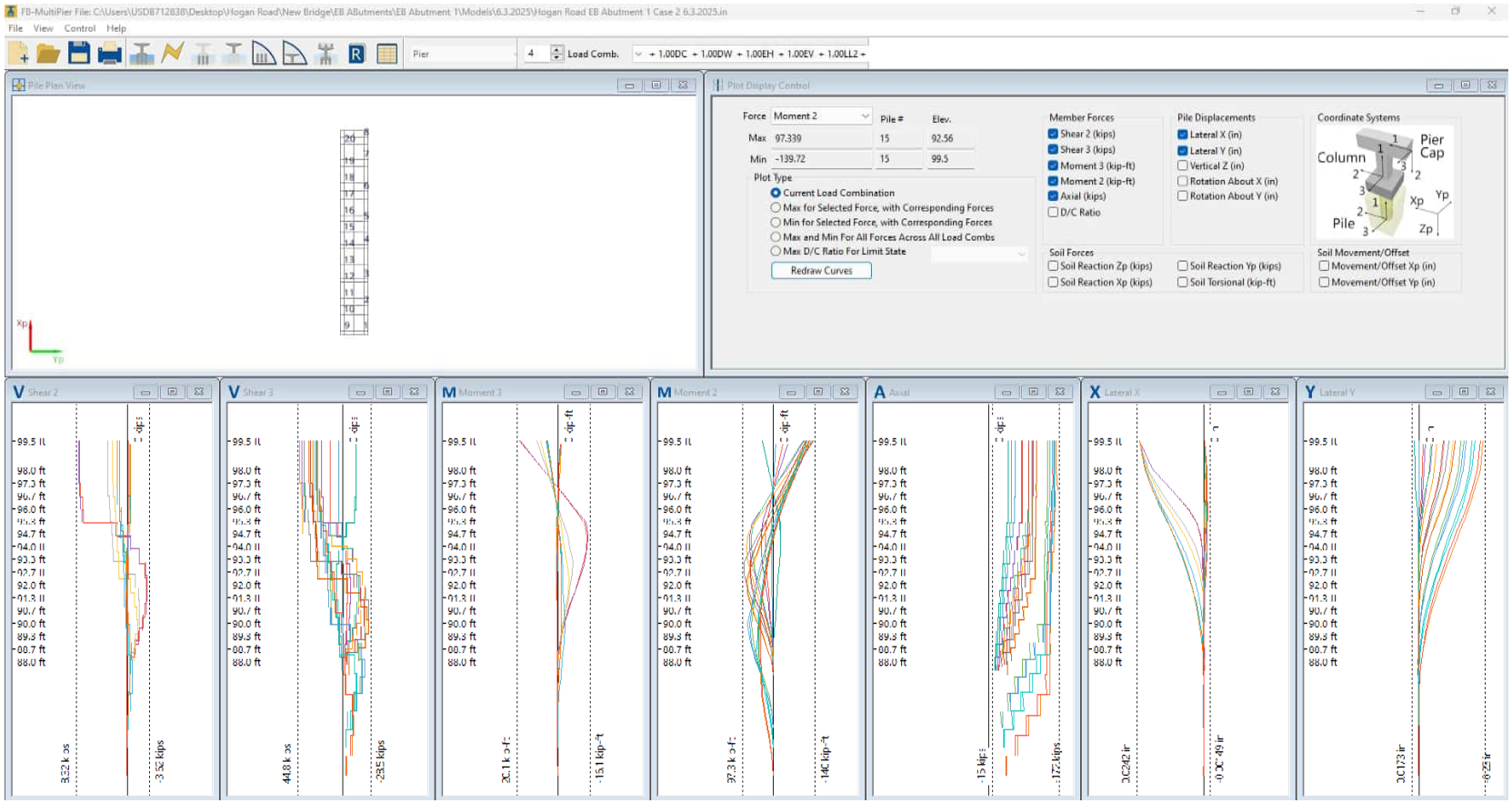
Case 1 Service I

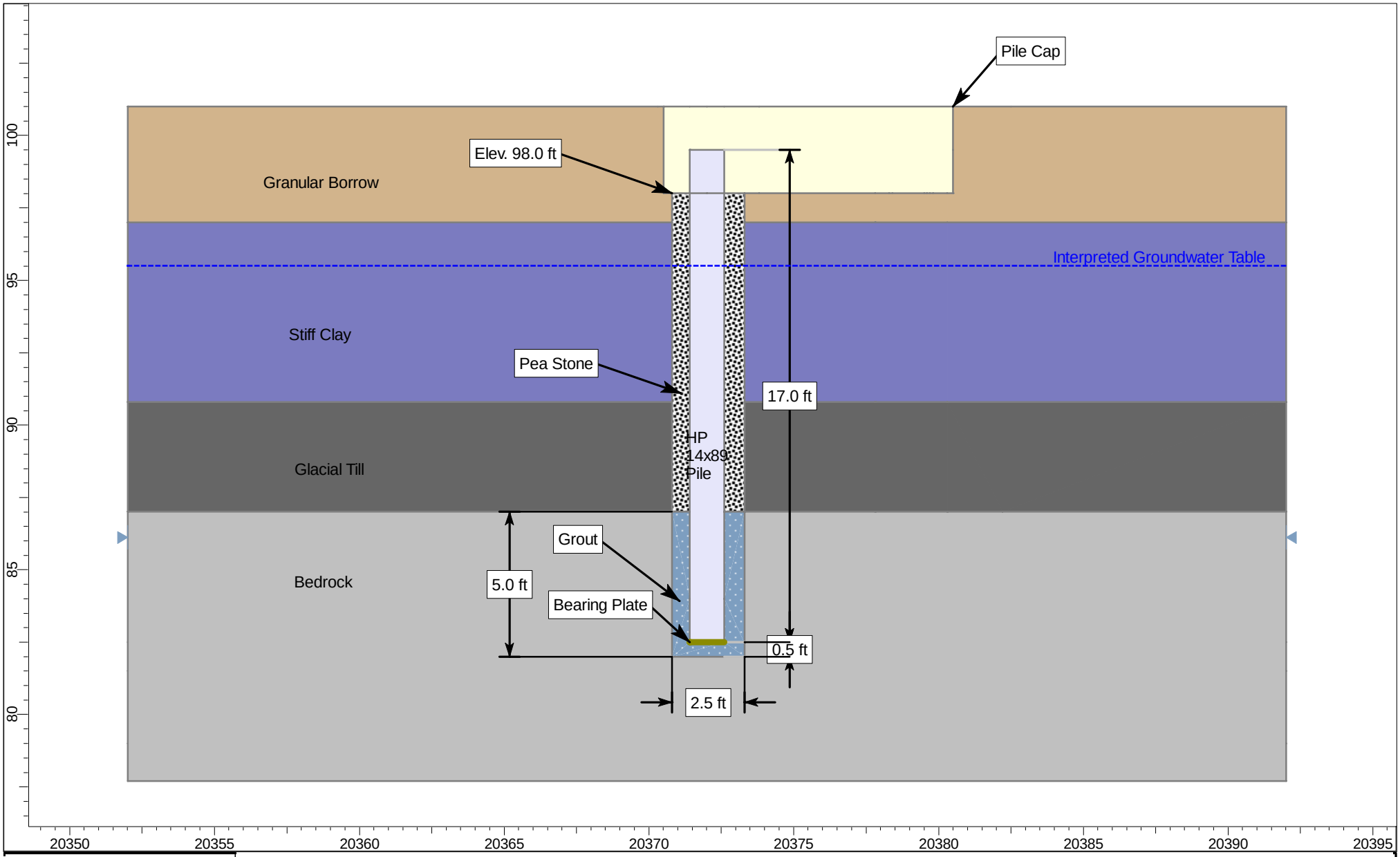


Case 2 Strength I



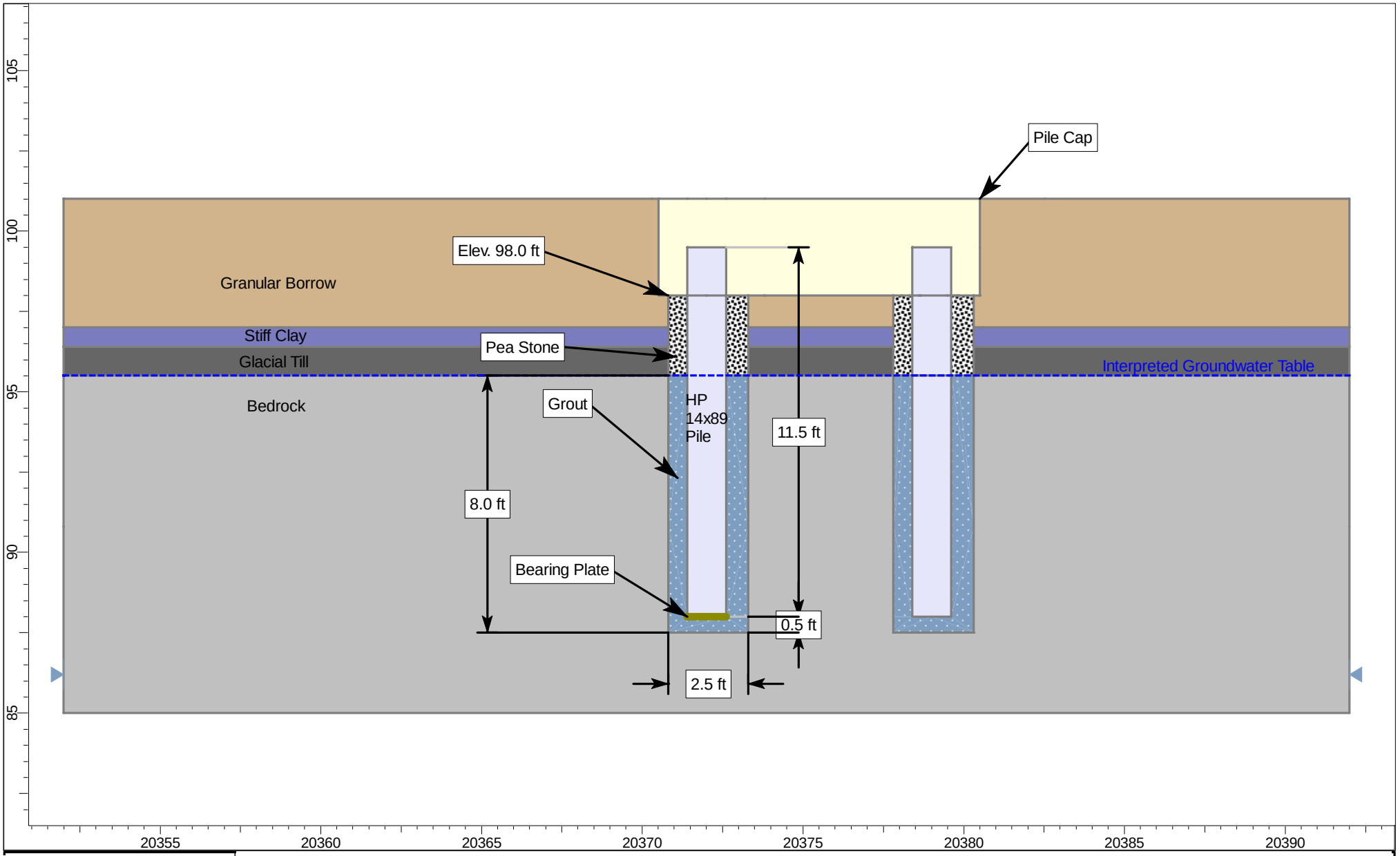
Case 2 Service I





	Project I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
	Analysis Description HP 14x89 Grouted Pile at EB Abutment 1 - Minimum Socket Depth Typical Section			
	Drawn By DEB	Checked By ATM	Reviewed By CCB	Scale 1:55
	Date 6/5/2025	File Name EB Abut 1 sketch.slmd		Attachment 4

SLIDE 9.037



Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at EB Abutment 1 - Maximum Socket Depth Typical Section			
Drawn By: DEB	Checked By: ATM	Reviewed By: CCB	Scale: 1:50	Attachment 4
Date: 6/5/2025	File Name: EB Abut 1 sketch max socket.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	4/18/2025
Checked By	SMH	Date	4/23/2025
Title	Bangor DDI Hogan Rd EB (1983)Bridge Bearing Loads		

Computations

Unfactored Bearing Loads, Abutment

Abutment Bearing Reactions

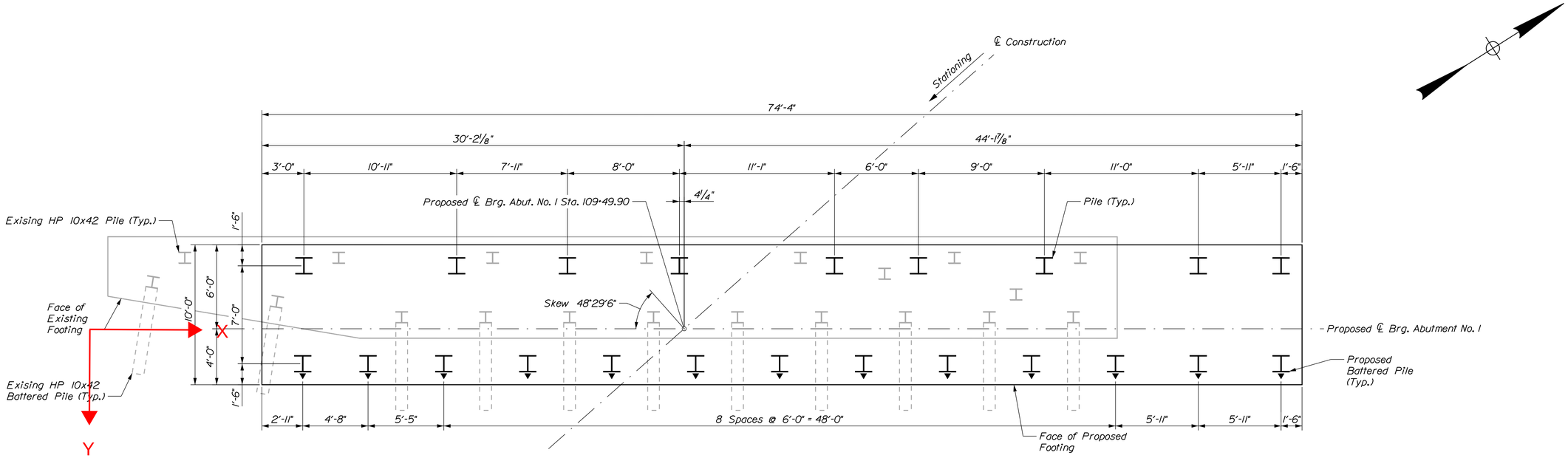
	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	
DC	58	58	58	58	58	58	
DW	12	12	12	12	12	12	
LL 1A	55	85	83	0	0	0	
LL 1B	0	0	0	83	85	55	
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6		G1	G2	G3	G4	G5	G6	
DC														
DW														
LL 1A														
LL 1B														
FR	5	5	5	5	5	5		5	5	5	5	5	5	

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	
DC	58	58	58	58	58	58	
DW	12	12	12	12	12	12	
LL 1A	55	85	83	0	0	0	
LL 1B	0	0	0	83	85	55	
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6		G1	G2	G3	G4	G5	G6	
DC														
DW														
LL 1A														
LL 1B														
FR	-5	-5	-5	-5	-5	-5		-5	-5	-5	-5	-5	-5	



ABUTMENT NO. 1
FOOTING PLAN & PILE LAYOUT

Note to Reviewer:
Foundation design by WSP is in progress. Conceptual pile layout is shown.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

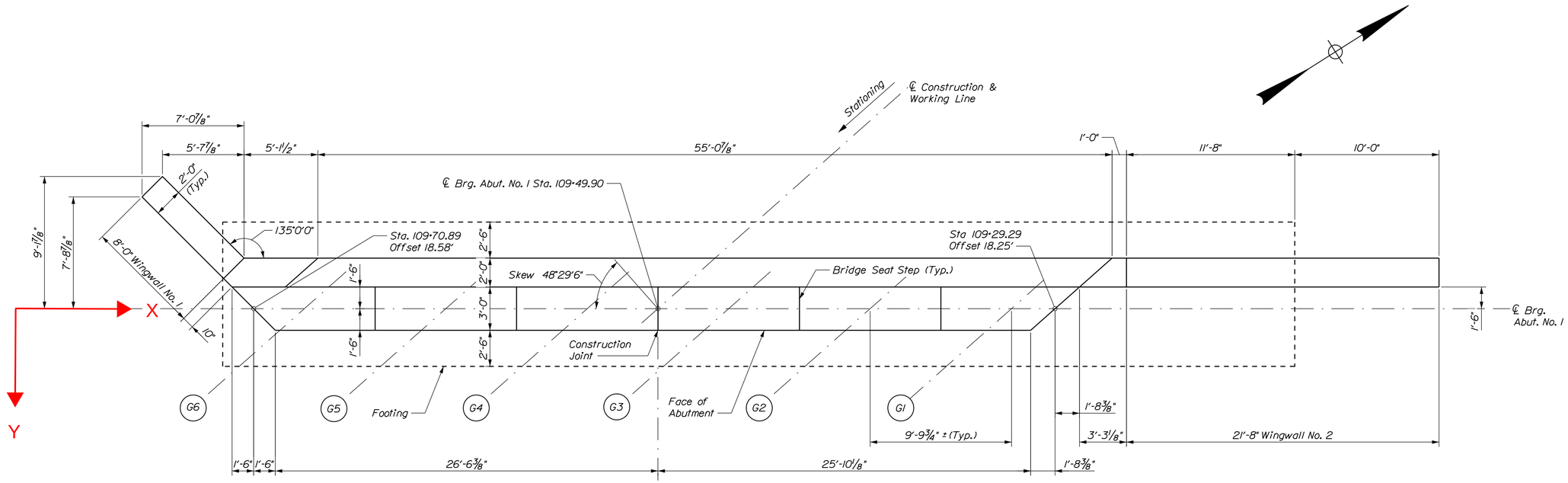
PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	TSR	BY	4/25
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

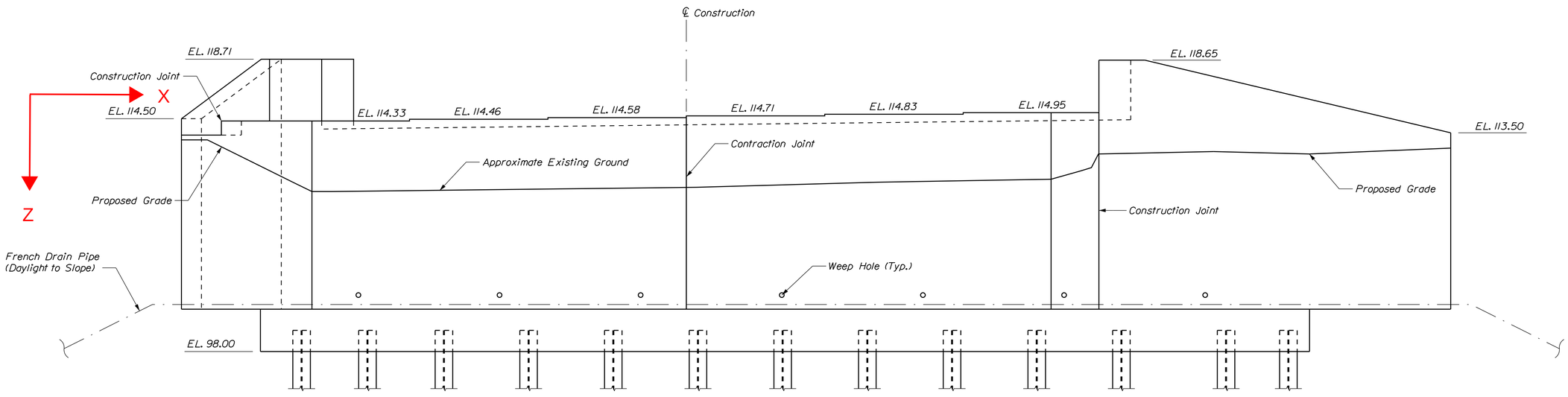
ABUTMENT NO. 1
FOOTING PLAN

SHEET NUMBER
5
OF 20

Filename: ...\XXX_Abut_1_01.dgn Division: HIGHWAY Username: jharris Date:08/29/2025



ABUTMENT NO. 1 PLAN



ABUTMENT NO. 1 ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

	R. MOULTON	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	TSR	TSR	4/25
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

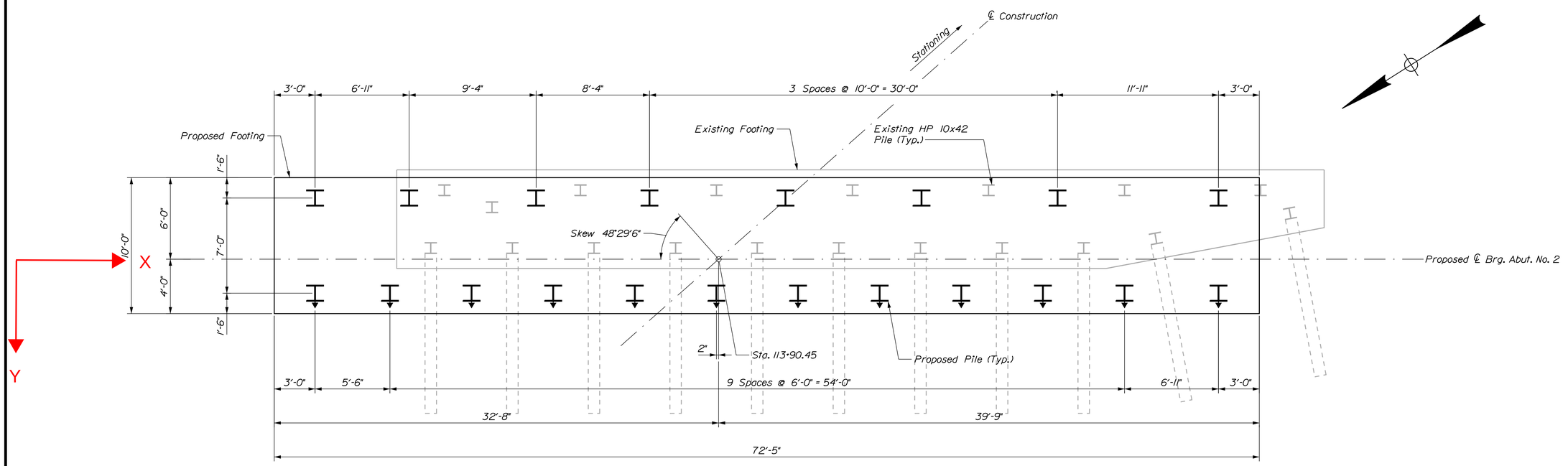
ABUTMENT NO. 1
PLAN & ELEVATION

SHEET NUMBER

6

OF 20

Username: jharris
Division: HIGHWAY
Filename: ...\XXX_Abut_1_02.dgn
Date:08/29/2025



ABUTMENT NO. 2
FOOTING PLAN PILE LAYOUT

Note to Reviewer:
Foundation design by WSP is in progress. Conceptual pile layout is shown.

Filename: ...\XXX_Abut_2_01.dgn Division: HIGHWAY Username: jharris Date:08/29/2025

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	BY		DATE
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			4/25
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 2
FOOTING PLAN

SHEET NUMBER
7
OF 20

ABUTMENT NO. 2 PLAN

ABUTMENT NO. 2
ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	KDW	BY	DATE
CHECKED-REVIEWED	YP	KDW	4/25
DESIGN2-DETAILED2		SMH	4/25
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 2
PLAN & ELEVATION

SHEET NUMBER

8

OF 20

Username: jharris
Date:08/29/2025
Division: HIGHWAY
Filename: ...\XXX_Abut_2_02.dgn

CALCULATIONS

Date:	6/4/3035	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	EB Bridge # 5823 Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles provide adequate support for the replacement EB Abutment 2 based on the design load case 2 (controlling) (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at EB Abutment 2 (Pile 20), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
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9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday April 23rd, 2025 with subject line "Bangor DDI WIN #1859510 Hogan EB Abutment Bearing Loads".
15. WSP calculation package titled Lateral Earth Pressure at the proposed East Bound Bridge Abutments, dated 4/25/2025.
16. WSP calculation package titled Pile Drivability at EB Abutment 2 - HP14x89, dated 6/4/2025.

CALCULATIONS

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Subject:	EB Bridge # 5823 Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Case 1 output
4. FB-Multipier Case 2 output
5. Abutment loads

ASSUMPTIONS

1. The proposed EB Abutment 2 is assumed to be supported by twenty (20) HP 14x89 Grade 50 piles with a least center to center spacing of 5.5 feet in the x-direction and 7 feet in the y-direction shown in Attachment 1.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) were applied at the bearings on the abutment stem wall.
4. All piles are assumed to have reinforced pile tips (Ref. 8).
5. The I-95 facing piles are battered at 3 on 12 (0.25) away from the abutment (Ref. 5).
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. Downdrag loading does not develop along the length of the EB Abutement 2 piles.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added to the toe of the EB Abutment 2 pile cap to account for the granular borrow and crushed stone slope on the toe (Ref. 5), and a vertical earth load EV was added to the heal of the EB Abutment 2 pile cap to account for the backfill and traffic live load surcharge (Ref. 15). The EV loads on the toe and heal are added as point loads on the piles, as pile spacing is not uniform. The point loads are based on the pile tributary areas.
10. A horizontal earth load EH of 10,893 plf (Ref. 15) was added to the pile cap on the back side of the abutment within the area of the stem wall.
11. A φdyn of 0.65 is used to determine the factored pile resistance, based on the MaineDOT standard Specification, PDA testing will be performed on 2% of the production piles.
12. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

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CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	97.0	ft	Sheet 8 (Ref. 5)
Pile cap thickness =	3	ft	Sheet 8 (Ref. 5)
Pile cap midplane elevation =	95.5	ft	Sheet 8 (Ref. 5)
A-modeled pile cap midplane elevation =	95.5	ft	
Base of pile cap elevation =	94.0	ft	Sheet 8 (Ref. 5)
Pile length =	38.3	ft	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	94.0	93.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	93.0	79.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	79.8	77.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	4	Cohesionless	77.1	58.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	58.2	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+91 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section F-F').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 20)

Maximum axial force in the piles, P_u =	229.5	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	122.6	kip-ft	=	1471.7	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	10.2	kip-ft	=	122.9	in-kips	
Maximum shear force in the piles, V_u =	21.4	kips				
Depth below pile head to fixity =	17.6	ft	=	Elev.	77.88	ft
Depth below pile cap to fixity =	16.1	ft	=	Elev.	77.88	ft

wsp

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.063	2.351	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.024	1.170	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.340	1.340	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	0.95	0.95	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.179	1.179	0.844	0.844	ksf

wsp

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 56.4 in = 4.7 ft (from Part A)

	Granular Borrow	Stiff Clay	Medium Clay	Glacial Till		
Thickness =	1.0	13.2	2.7	18.9	ft	(from stratigraphy in Part A)
A_s =	4.7	62.0	12.7	88.8	ft^2	
R_s per layer =	0.1	73.1	10.7	103.9	kips	

Total R_s = 187.8 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD $\leq$ 70%
$q_p = 0.80 q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 1,279 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 422.0 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.13 ft^2 (from Part A)

Total R_p = 56.6 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 244 kips (bedrock)

CALCULATIONS

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$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:
- φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method
- (Ref. 7 Tbl. 10.5.5.2.3-1) φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method
- φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Granular Borrow	Stiff Clay	Medium Clay	Glacial Till	Pile Tip (Bedrock)	
R_s or R_p =	0.1	73.1	10.7	103.9	56.6	kips
φ_{stat} =	0.45	0.35	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	25.6	3.7	46.8	25.5	

Total R_r = 102 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 71% of the steel yield strength (Ref. 14):

R_n = 667 kips (Ref. 14)

φ_{dyn} = 0.65 (Assumption 11)

R_f = 434 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 229 kips (from Part B)

R_r = 434 kips > P_u = 229 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:
- Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
- Unbraced length, l = 193.45 in (from Part B)
- Minimum radius of gyration, r_s = 3.47 in (from Part A)

Check: Kl/r_s = 36 < 120 OK

CALCULATIONS

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Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 12)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	193.45	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	4,221	kips	

CALCULATIONS

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Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D = 12.730 in D= d - (2 × tf)

Web thickness, t_w = 0.455 in (from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.23 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 878 kips

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.50 (Ref. 7, Article 6.5.4.2))

P_r = 439 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c = 0.70 (Ref. 7, Article 6.5.4.2))

$P_{r, axial/ flexural}$ = 615 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 229 kips (from Part B)

$P_{r, axial}$ = 439 kips > P_u = 229 kips OK

CALCULATIONS

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E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	1472	in-kips (from Part B)
M_{uy} =	123	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	1472	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	123	in-kips	OK

wsp

CALCULATIONS

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F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.4 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.7 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$V_n = CV_p = V_{cr}$

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_p = 0.58F_y d_w t_w$ (Ref. 7, 6.10.9.2-2))

$V_n = CV_p = V_{cr}$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

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Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

V_u = 21.4 kips (from Part B)

V_r = 168 kips > V_u = 21.4 kips OK

H. Determine nominal pile driving resistance.

P_u = 229 kips (from Part B)

φ_{dyn} = 0.65 (Ref. 3, Table 7-2)

for driving criteria established by dynamic testing of at least two piles per site condition and no less than 2% of production piles

Nominal pile driving resistance, R_{ndr} = 353 kips

Confirm R_{ndr} does not exceed P_n: R_{ndr} P_n (from Part D)

353 kips < 878 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 38 feet, a maximum axial force of 229.5 kips , the corresponding strong axis (x-axis) moment of 1471.7 in-kips (122.6 ft-kips) occurs in the HP14x89 piles at Eastbound Abutment 2 in Pile 20. The estimated pile length is sufficient for the piles to achieve fixity. The analysis indicates that the maximum top of pile deflection at the service limit state is less than 0.2 inches in the X and Y directions. The EB Abutment 2 piles should be driven to a nominal resistance of 353 kips.This evaluation considers section loss for a 100 year design life, downdrag is not estimated to develop along the length of the EB Abutment 2 piles. A summary table is presented below.

Replacement EB Abutment 2 Summary Table		
Pile Length (ft)		38
Depth below Pile Head to Fixity (ft)		18
Geotechnical Axial Resistance (kips)	Nominal	667
	Factored	434
Structural Axial Resistance (kips)	Nominal	878
	Factored	439
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.7
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

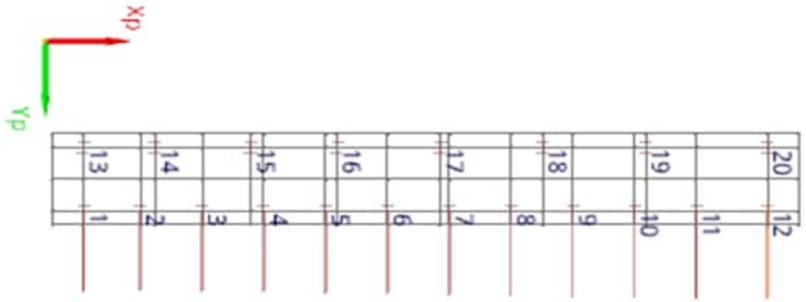


Figure 1b: Profile View of Pile Cap (Transverse)

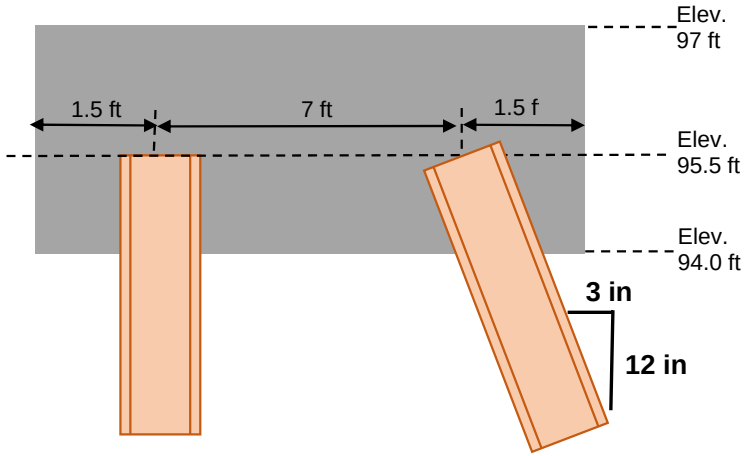


Figure 1c: 3D View of Pile Model

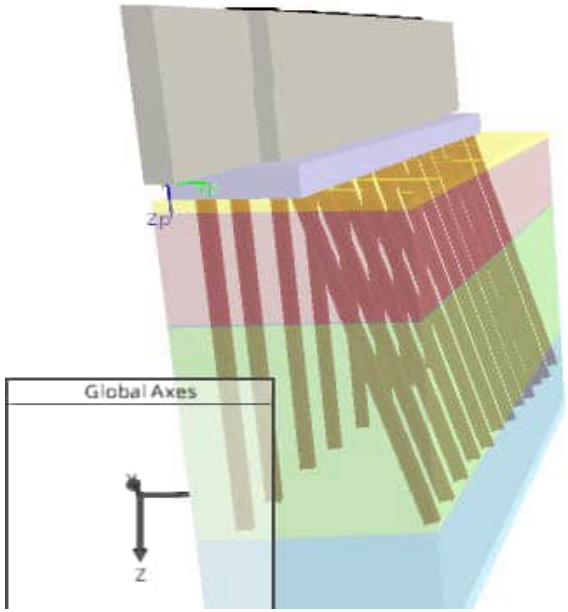
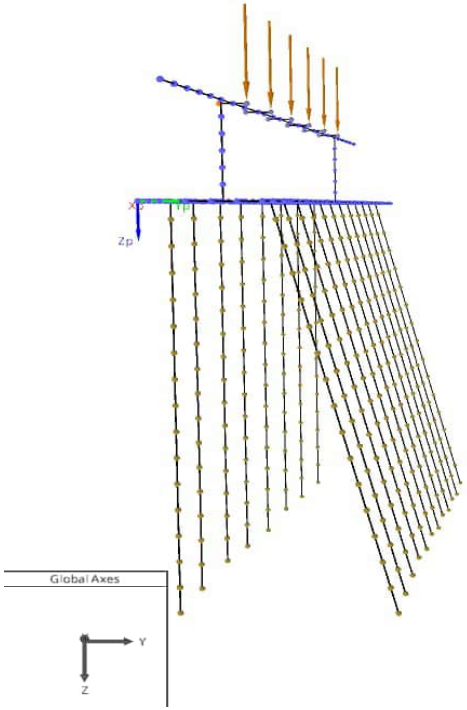


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	94.0	93.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	24		-	-	-
Stiff Clay	2	Cohesive	93.0	79.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medim Clay	3	Cohesive	79.8	77.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	4	Cohesionless	77.1	58.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	1170		-	-	-
Bedrock	5	Rock	58.2	50.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3678.4	667.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on the EB Abutment 2 profile F-F' (Sheet X).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

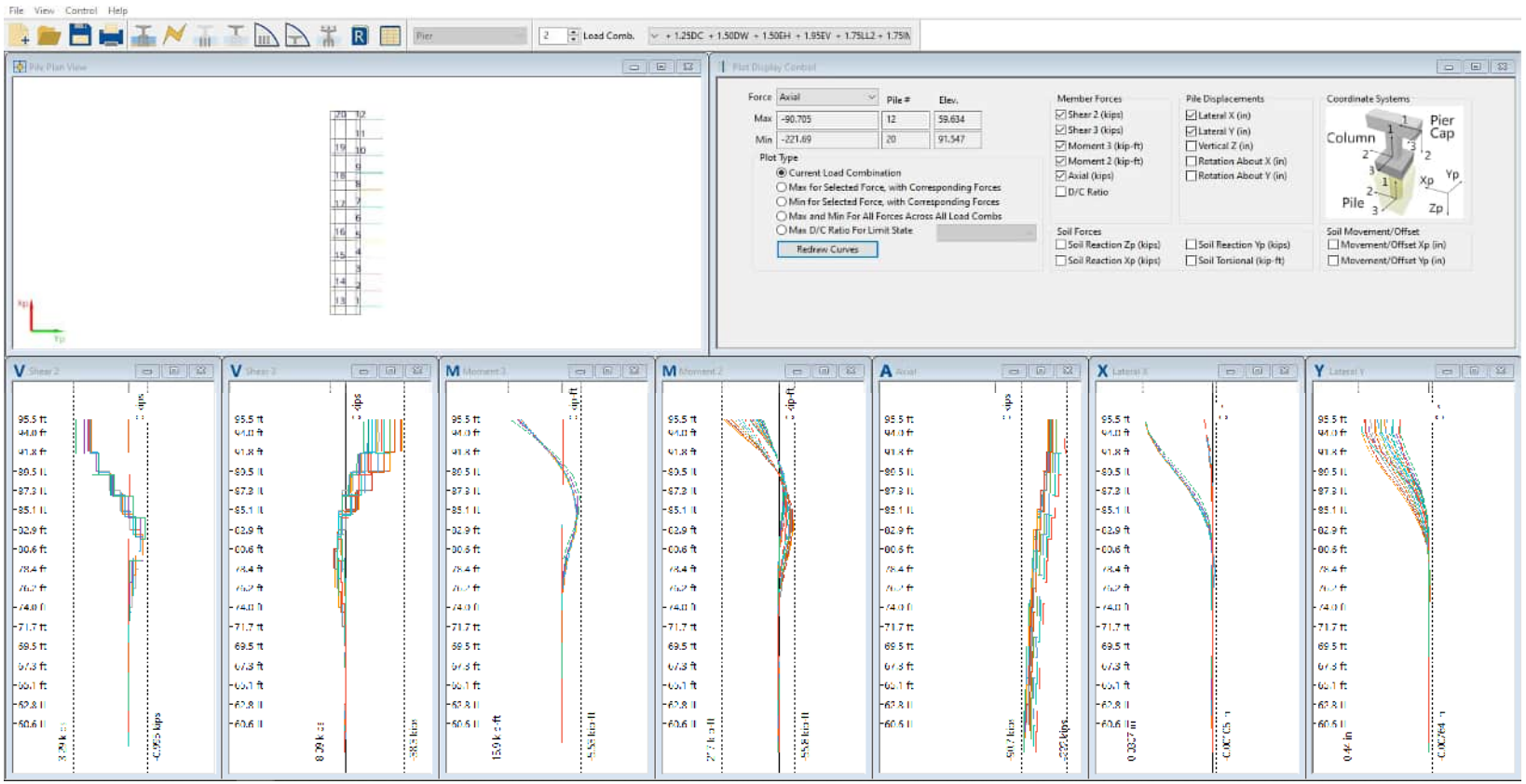
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

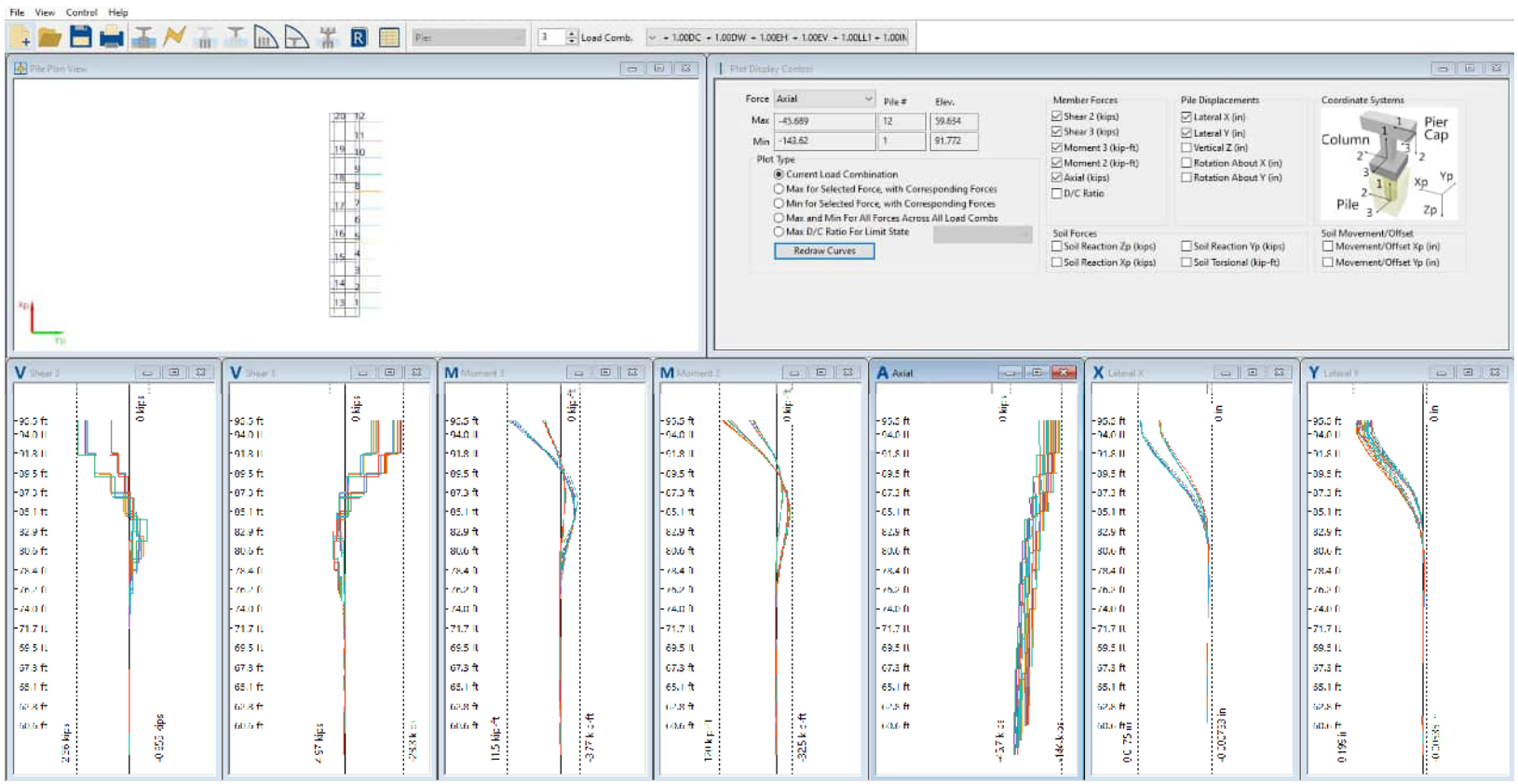
8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed:

Case 1 Strength I



Case 1 Service I



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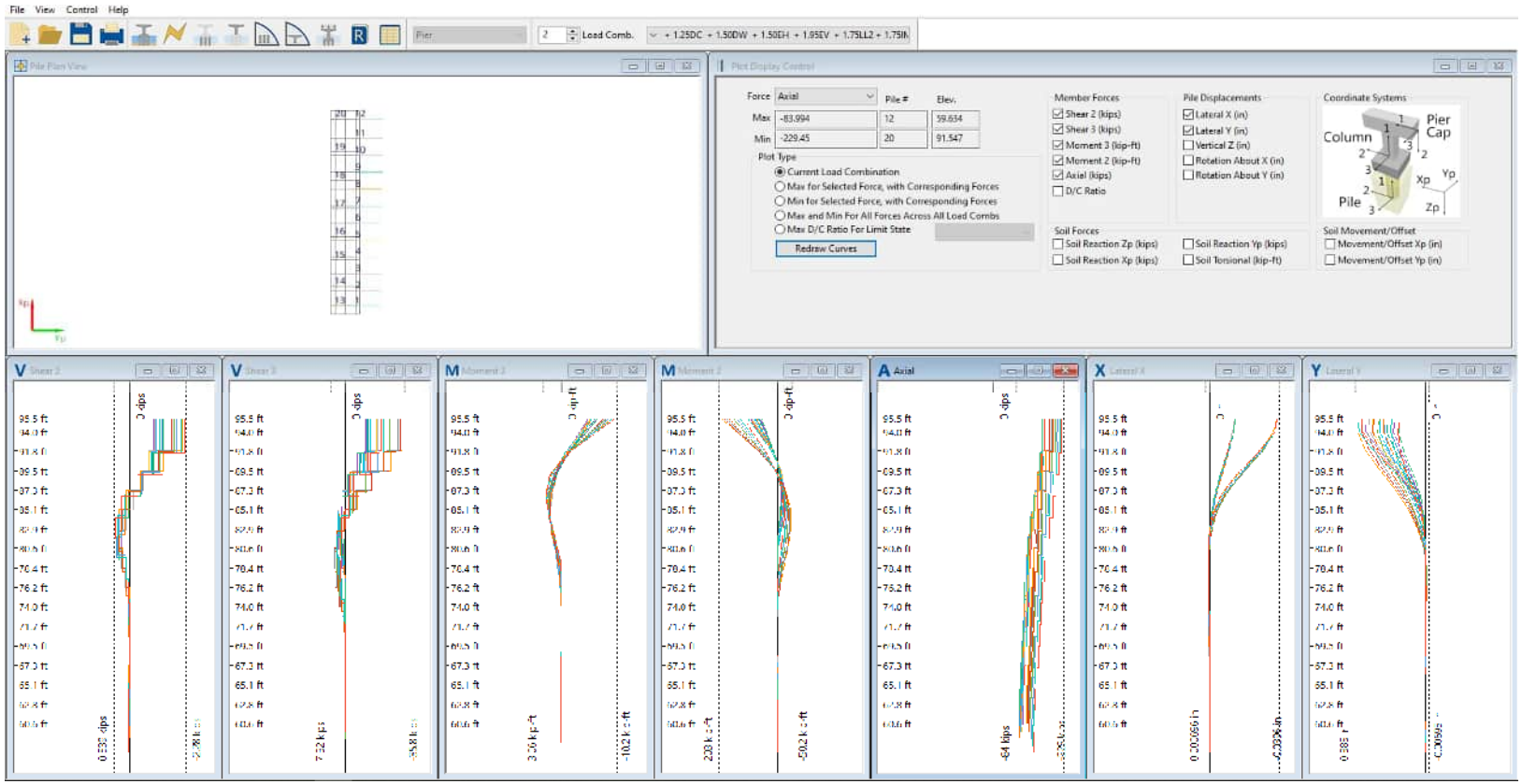
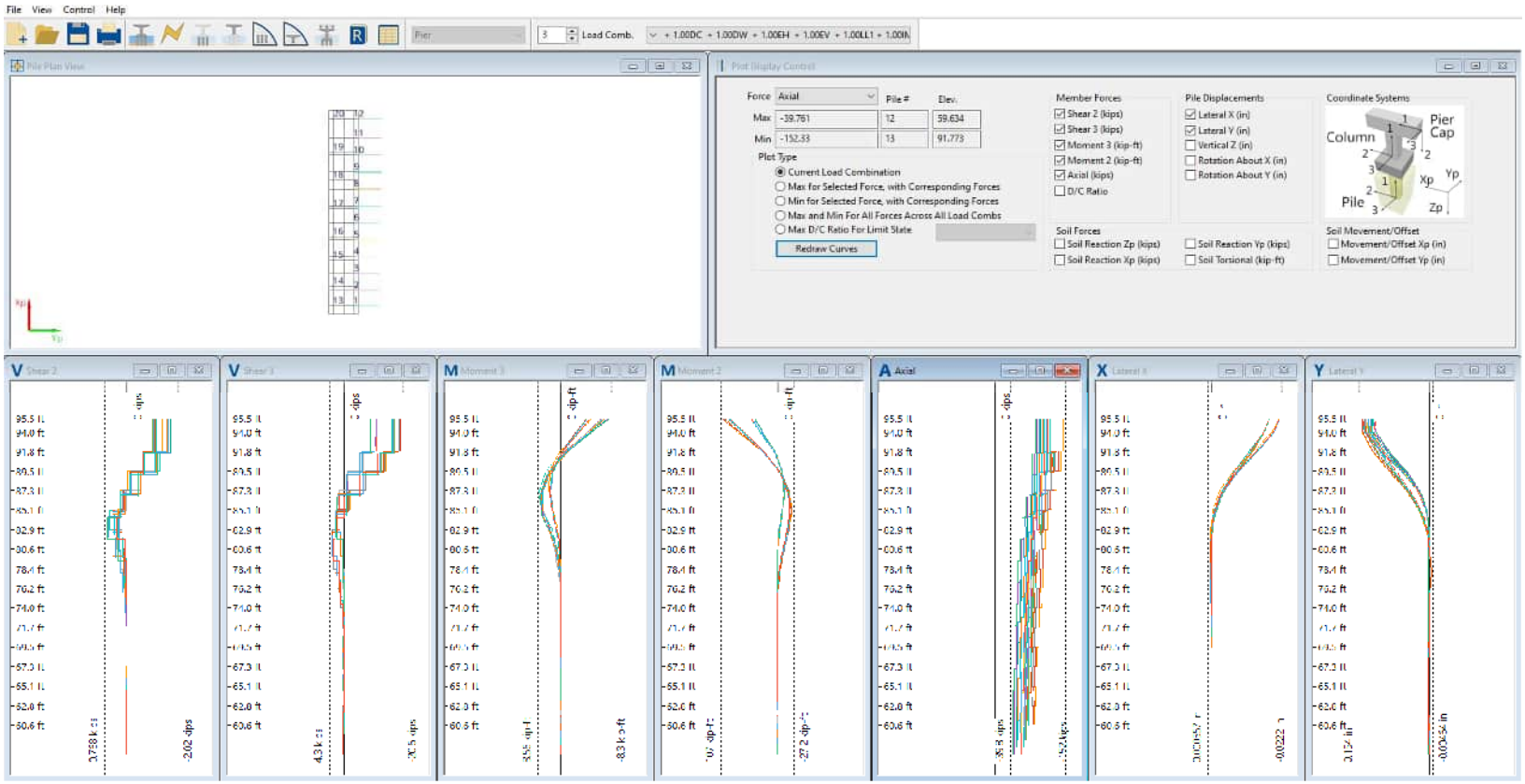


Case 2 Service I



Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	4/18/2025
Checked By	SMH	Date	4/23/2025
Title	Bangor DDI Hogan Rd EB (1983)Bridge Bearing Loads		

Computations

Unfactored Bearing Loads, Abutment

Abutment Bearing Reactions

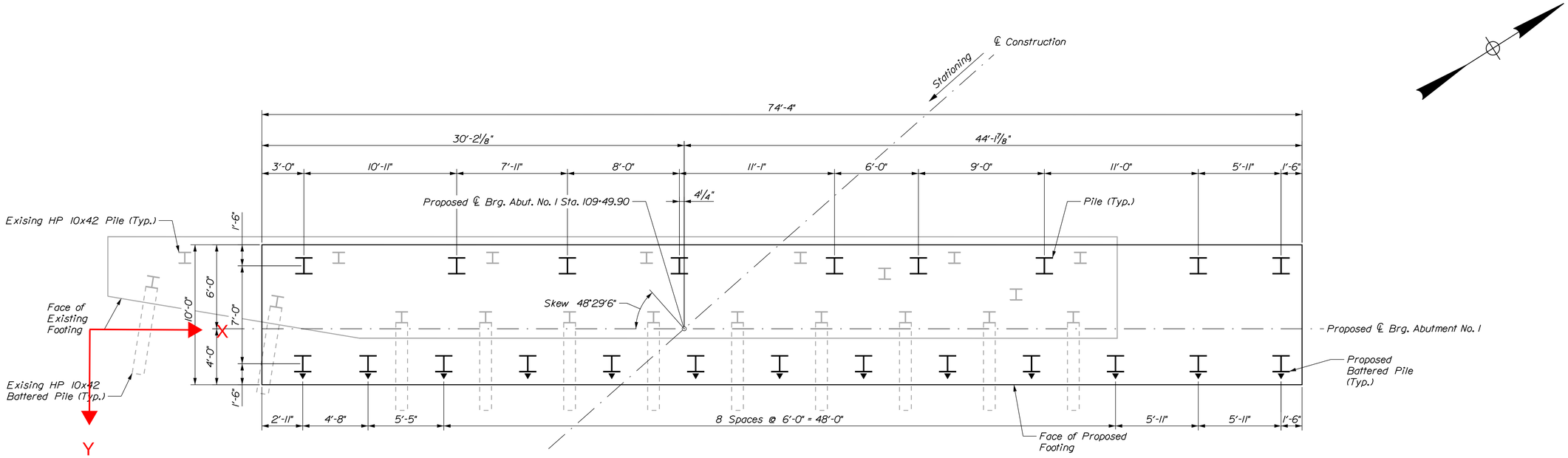
	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	
DC	58	58	58	58	58	58	
DW	12	12	12	12	12	12	
LL 1A	55	85	83	0	0	0	
LL 1B	0	0	0	83	85	55	
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6		G1	G2	G3	G4	G5	G6	
DC														
DW														
LL 1A														
LL 1B														
FR	5	5	5	5	5	5		5	5	5	5	5	5	

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	
DC	58	58	58	58	58	58	
DW	12	12	12	12	12	12	
LL 1A	55	85	83	0	0	0	
LL 1B	0	0	0	83	85	55	
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6		G1	G2	G3	G4	G5	G6	
DC														
DW														
LL 1A														
LL 1B														
FR	-5	-5	-5	-5	-5	-5		-5	-5	-5	-5	-5	-5	



ABUTMENT NO. 1
FOOTING PLAN & PILE LAYOUT

Note to Reviewer:
Foundation design by WSP is in progress. Conceptual pile layout is shown.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

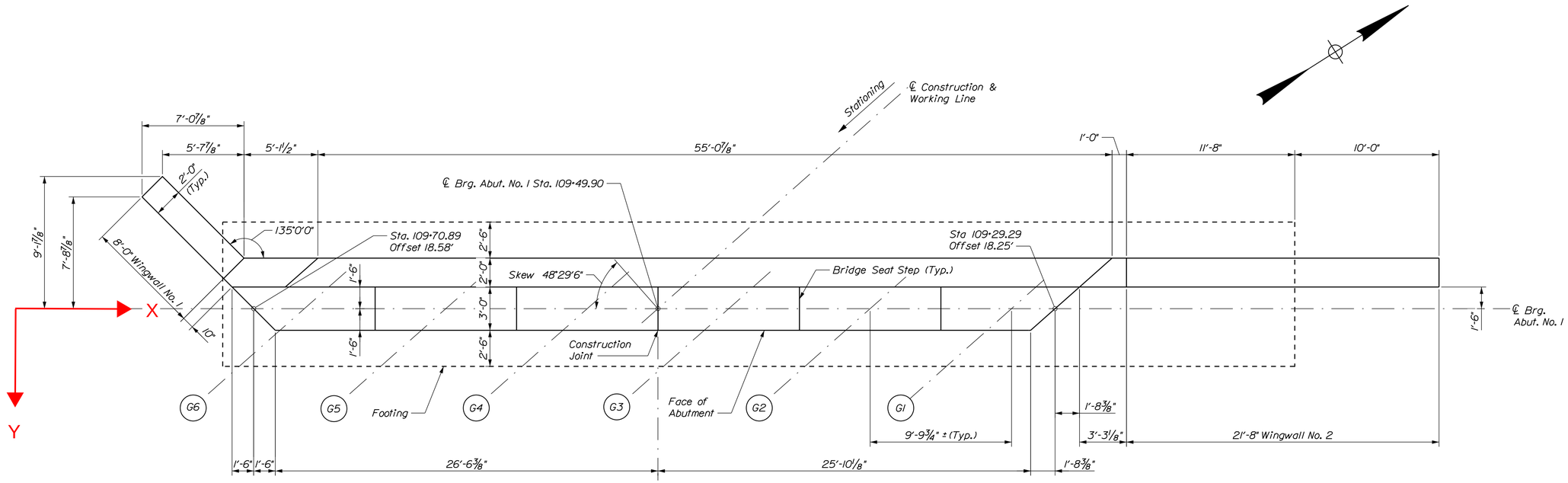
PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	TSR	BY	4/25
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

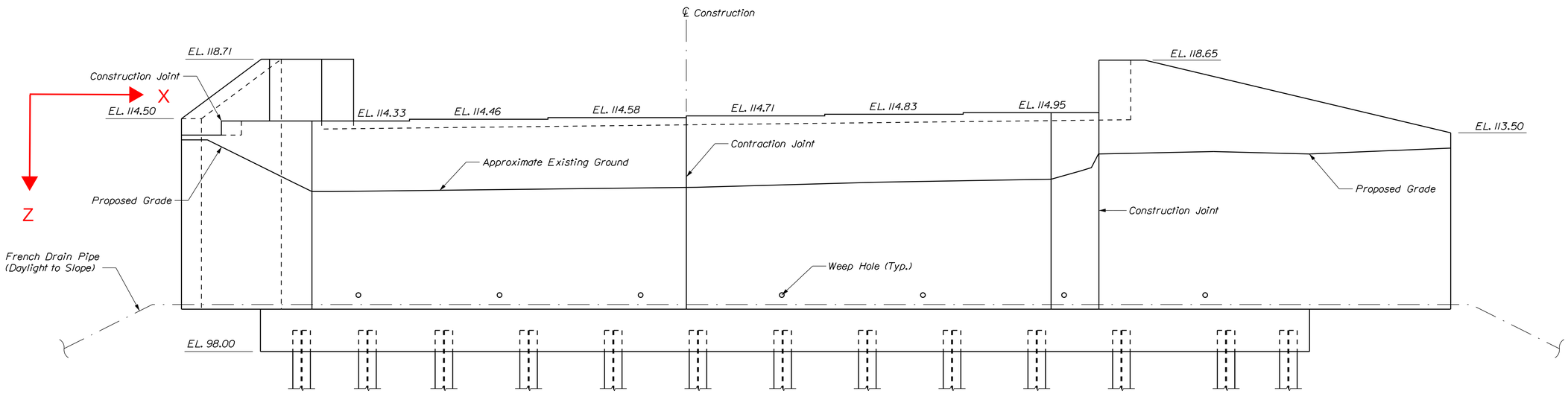
ABUTMENT NO. 1
FOOTING PLAN

SHEET NUMBER
5
OF 20

Filename: ...\XXX_Abut_1_01.dgn Division: HIGHWAY Username: jharris Date:08/29/2025



ABUTMENT NO. 1 PLAN



ABUTMENT NO. 1 ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

	R. MOULTON	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	TSR	TSR	4/25
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

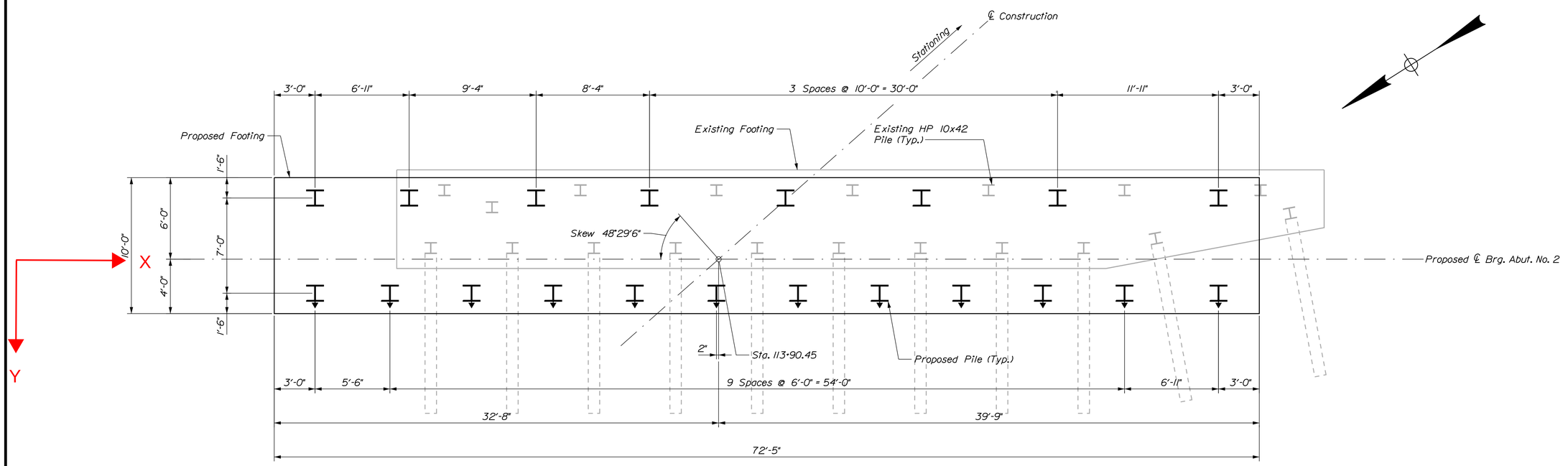
ABUTMENT NO. 1
PLAN & ELEVATION

SHEET NUMBER

6

OF 20

Username: jharris
Division: HIGHWAY
Filename: ...\XXX_Abut_1_02.dgn
Date:08/29/2025



ABUTMENT NO. 2
FOOTING PLAN PILE LAYOUT

Note to Reviewer:
Foundation design by WSP is in progress. Conceptual pile layout is shown.

Filename: ...\XXX_Abut_2_01.dgn Division: HIGHWAY Username: jharris Date:08/29/2025

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	BY		DATE
CHECKED-REVIEWED	YP	SMH	4/25
DESIGN2-DETAILED2			4/25
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 2
FOOTING PLAN

SHEET NUMBER
7
OF 20

ABUTMENT NO. 2 PLAN

ABUTMENT NO. 2
ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
4/15/2025

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	KDW	BY	DATE
CHECKED-REVIEWED	YP	KDW	4/25
DESIGN2-DETAILED2		SMH	4/25
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD EASTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 2
PLAN & ELEVATION

SHEET NUMBER

8

OF 20

Username: jharris
Date:08/29/2025
Division: HIGHWAY
Filename: ...\XXX_Abut_2_02.dgn

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge 5823 Abutment 2 - HP14x89 - Resistan	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine the factored pile resistance of the proposed HP 14x89 piles at EB Abutment 2 based on the driving criteria outline in FHWA GEC 12 and the MaineDOT Standard Specification March 2020.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road Eastbound Bridge Sheet 7, Abutment No. 2 Footing Plan dated 4/2025.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Abutment 1 H-Pile Analysis, dated 6/4/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
9. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-10. WSP boring logs.

ASSUMPTIONS

1. Pile tip elevations are based on an interpretation of the as-drilled top of bedrock elevations (Ref. 10).
2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at EB Abutment 2 (soil set 1).
3. As per FHWA GEC 12 Vol 2. Section 14.2.06C-8 (Ref. 9), practical refusal when using a power hammer is considered to be when penetration does not exceed 1/2 inch in 10 successive blows, and driving stresses shall not exceed 90% of the pile yield strength.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 8).
5. This analysis looks at resistance upon reaching the tip elevation in bedrock and does not consider the resistance during driving. A resistance distribution of 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.
6. A φdyn of 0.65 using (Ref. 9 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge 5823 Abutment 2 - HP14x89 - Resistan	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2	= 0.18 ft^2 (Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = $(d \times 2) + (b_f \times 2)$ =	57.0	in	= 4.8 ft
Pile head elevation =	95.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	60.6	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	34.9	ft	(Ref. 3, Sheet 13 of 53)

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.09	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation =	95.5	ft	
Groundwater elevation =	82.7	ft	= 12.8 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Granular Borrow	94.0	93.0	1.5	2.5	125	32	-	-	-	Sand
Stiff Clay	93.0	82.5	2.5	13.0	122	-	1340	-	-	Clay
Glacial Till	82.5	61.6	13.0	33.9	130	35	-	-	-	Sand
Competent Bedrock	61.6	-	33.9	-	158	-	-	20.0	422.0	Rock Other

CALCULATIONS

Date:	6/4/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge 5823 Abutment 2 - HP14x89 - Resistan	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outline in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
Reaching tip elevation	10%	90%	APE D25-42	45,892	Fuel Setting 2 (74%)	667	19.9	35.67	71%

3. Determine nominal and factored toe resistance.

Nominal toe resistance = 667 kips (Intact section from GRLWEAP)

φ_{dyn} = 0.65 (Ref. 9, Table 10.5.5.2.3-1)

Factored toe resistance = 434 kips

CONCLUSIONS

This analysis indicates that an HP14x89 pile driven with an APE D25-42 hammer operated at 71% of the combustion pressure is expected to develop a nominal bearing resistance of 667 kips at EB Abutment 2 while limiting driving stresses to a maximum of 45 ksi. Blow counts were limited to 10 blows per 1/2" of penetration, in accordance with FHWA GEC 12 (Ref. 9) and Section 501 of the Maine Department of Transportation 2020 Standard Specifications for Construction (Ref. 4). The resulting factored toe resistance from this WEAP analysis is 434 kips.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

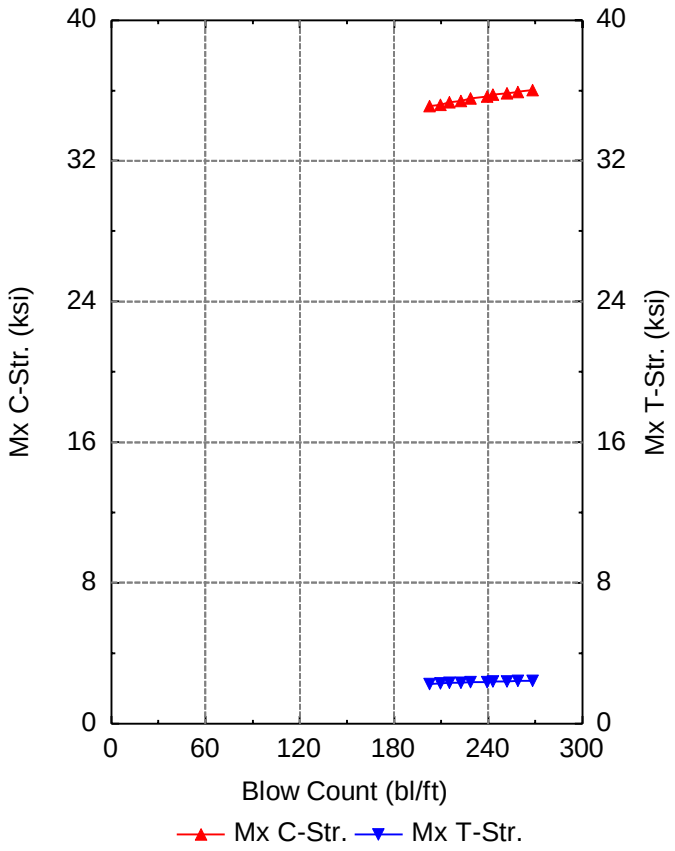
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

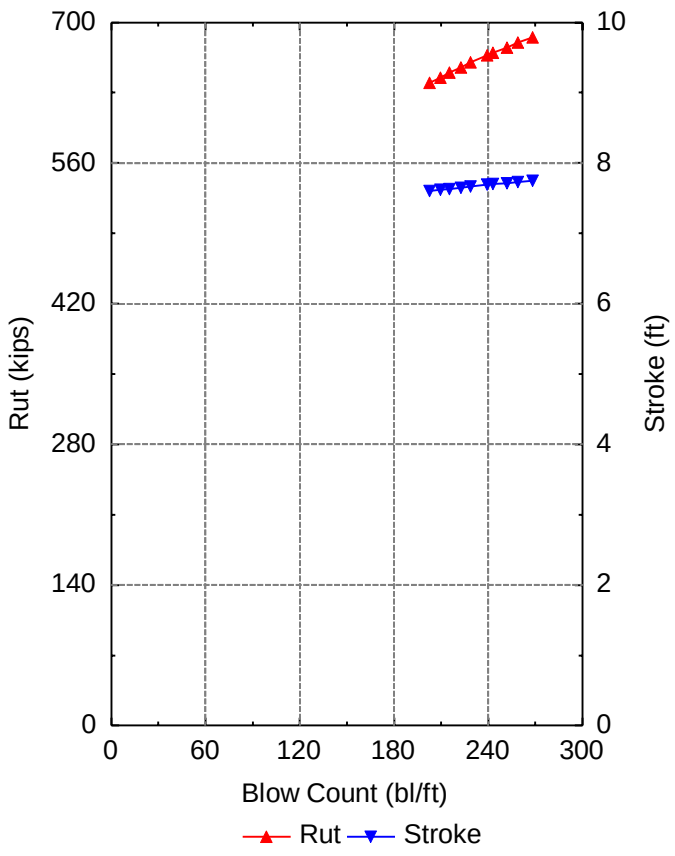
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

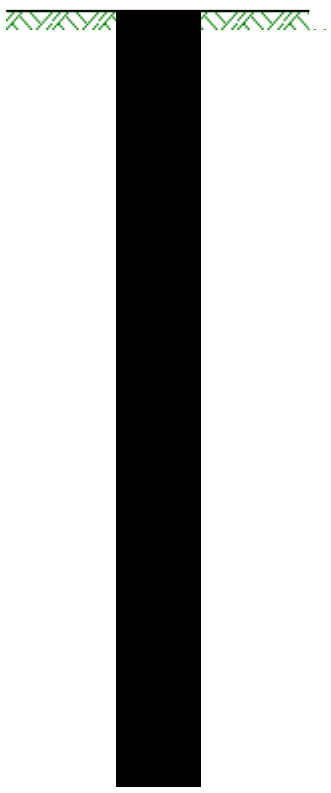
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



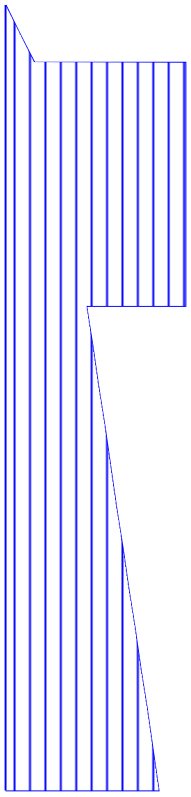
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	33.90	ft
Pile Penetration	33.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
640.0	35.11	30.25	2.25	202.7	7.60	14.80	D 25-42	23.9
645.0	35.17	30.37	2.28	209.5	7.62	14.83	D 25-42	23.9
650.0	35.34	30.56	2.30	215.2	7.63	14.90	D 25-42	24.0
655.0	35.40	30.68	2.32	222.7	7.65	14.92	D 25-42	24.1
660.0	35.54	30.85	2.34	228.6	7.67	14.98	D 25-42	24.2
667.0	35.67	31.05	2.37	239.1	7.69	15.03	D 25-42	24.2
670.0	35.75	31.14	2.38	243.0	7.70	15.08	D 25-42	24.3
675.0	35.83	31.27	2.39	251.8	7.71	15.09	D 25-42	24.3
680.0	35.93	31.42	2.41	258.7	7.73	15.16	D 25-42	24.5
685.0	36.02	31.55	2.42	268.2	7.74	15.18	D 25-42	24.5

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed plumb H-piles at EB Abutment 2 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road Eastbound Bridge Sheet 7, Abutment No. 2 Footing Plan dated 4/2025.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road EB Bridge Abutment 2 H-Pile Analysis, dated 6/4/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on twenty (20) HP14x89 Grade 50 piles with a center-to-center pile spacing of 5.5 feet in the x-direction and 7 feet in the y-direction (Ref. 3). The piles are to be driven to bedrock.
2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at EB Abutment 2 (soil set 1).
3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2	= 0.18 ft^2 (Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = $(d \times 2) + (b_f \times 2)$ =	57.0	in	= 4.8 ft
Pile head elevation =	95.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	60.6	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	34.9	ft	(Ref. 3, Sheet 13 of 53)

Ultimate Capacities:

Axial load on pile, P_u =	230	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	353	kips	

For soil above bedrock while driving pile to bedrock:

Nominal shaft resistance, R_s =	188	kips	(Ref. 6)
Maximum nominal toe resistance of soil, $R_{p\text{-}soil}$ =	57	kips	(Ref. 6)
Total soil resistance =	244	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.09	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Shaft and Toe Resistance (during driving):

Nominal shaft resistance, R_s = 188 kips = 77 % of R_n (Ref. 6)

Nominal toe resistance, $R_{p\text{-}rock}$ = 57 kips = 23 % of R_n (Ref. 6)

Nominal resistance, R_n = 244 kips

Assume that once the pile has encountered bedrock, the resistance distribution will switch to 23% shaft resistance and 77% toe resistance. A second analysis will be performed to model these conditions.

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation = 95.5 ft

Groundwater elevation = 82.7 ft = 12.8 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Granular Borrow	94.0	93.0	1.5	2.5	125	32	-	-	-	Sand
Stiff Clay	93.0	82.5	2.5	13.0	122	-	1340	-	-	Clay
Glacial Till	82.5	61.6	13.0	33.9	130	35	-	-	-	Sand
Competent Bedrock	61.6	-	33.9	-	158	-	-	20.0	422.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	77%	23%	APE D25-42	45,892	Fuel Setting 2 (74%)	353	5.2	23.06	46%
Reaching tip elevation	23%	77%	APE D25-42	45,892	Fuel Setting 2 (74%)	353	5.0	23.62	47%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 2 (74% of the combustion pressure) while driving through soil and upon reaching bedrock should be able to overcome a soil resistance of 244 kips in order to drive the plumb piles at EB Abutment 2 to bedrock and a nominal resistance of 353 kips while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

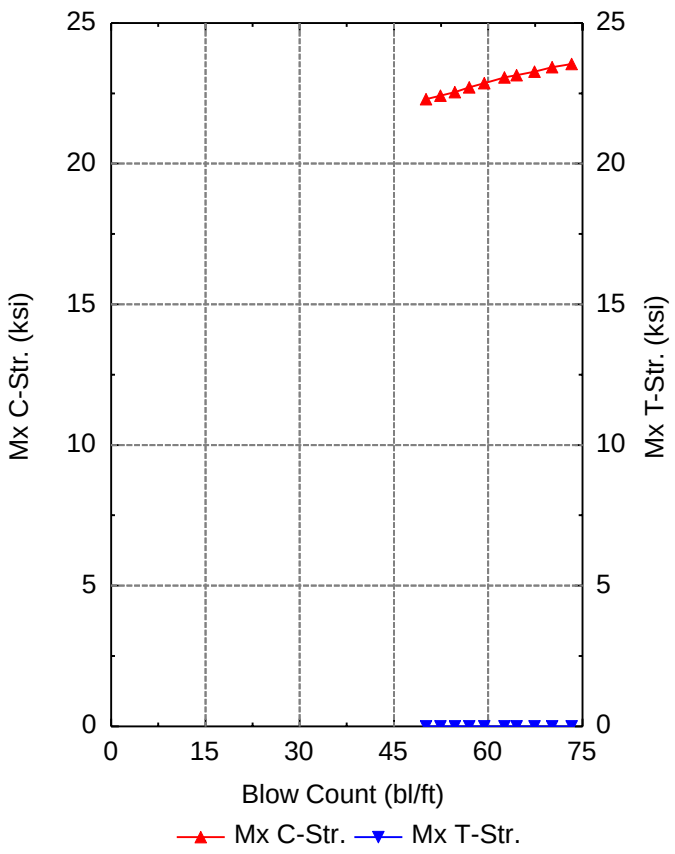
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

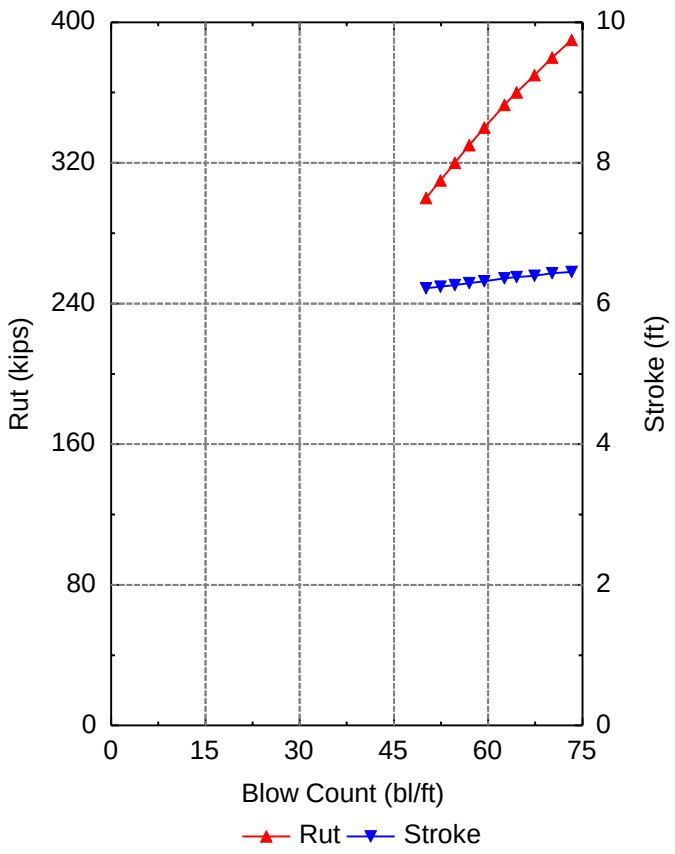
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

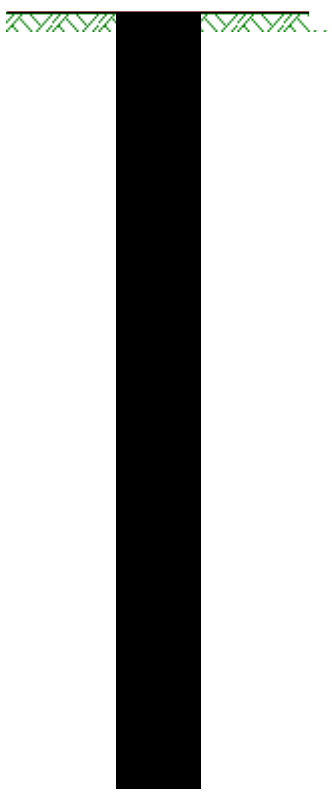
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



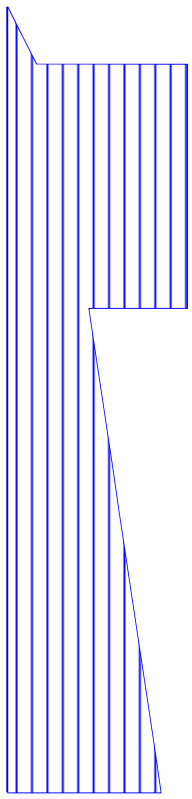
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	33.90	ft
Pile Penetration	33.90	ft
Pile Top Area	26.10	in²
RSA	No	





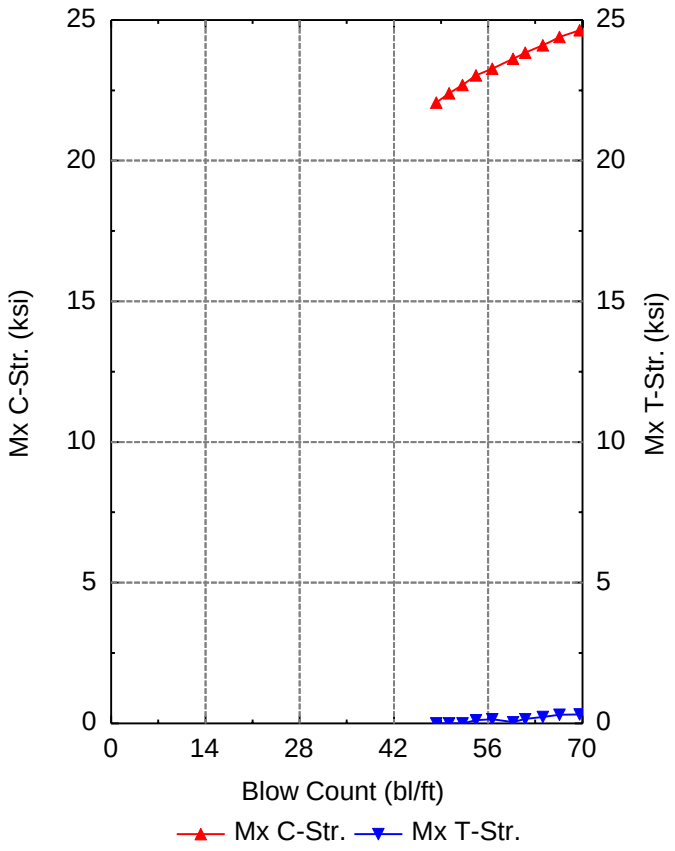
Pile Model



Shaft=77%
(Prop., SA)

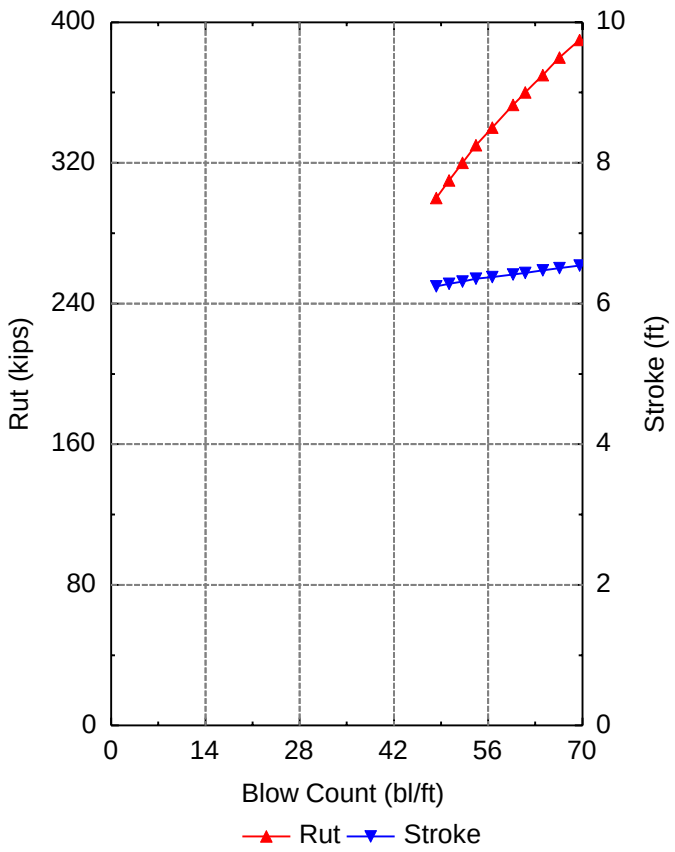
Bearing Graph Summary — APE D 25-42

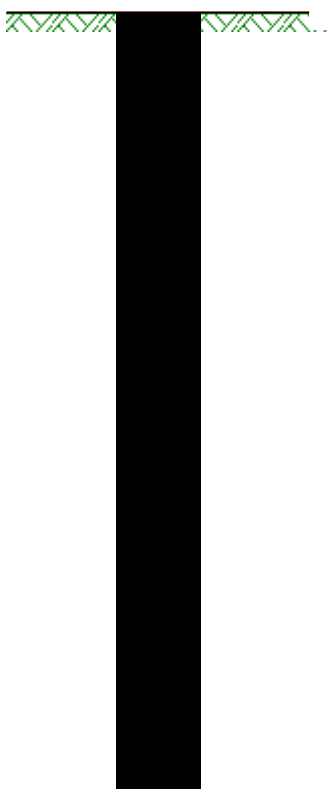
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
300.0	22.29	22.29	0.00	50.1	6.22	11.54	D 25-42	18.6
310.0	22.42	22.42	0.00	52.4	6.24	11.48	D 25-42	18.5
320.0	22.55	22.55	0.00	54.7	6.26	11.42	D 25-42	18.4
330.0	22.71	22.71	0.00	57.0	6.29	11.38	D 25-42	18.3
340.0	22.86	22.86	0.00	59.4	6.32	11.34	D 25-42	18.3
353.0	23.06	23.06	0.00	62.6	6.36	11.29	D 25-42	18.2
360.0	23.14	23.14	0.00	64.5	6.37	11.25	D 25-42	18.1
370.0	23.27	23.27	0.00	67.4	6.39	11.19	D 25-42	18.0
380.0	23.43	23.43	0.00	70.2	6.43	11.18	D 25-42	18.0
390.0	23.54	23.54	0.00	73.3	6.45	11.15	D 25-42	18.0



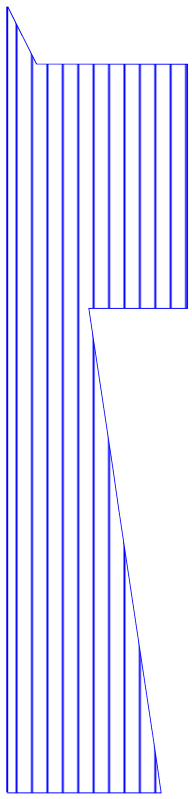
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	33.90	ft
Pile Penetration	33.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=23%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
300.0	22.07	21.79	0.00	48.3	6.24	12.17	D 25-42	19.6
310.0	22.39	21.95	0.00	50.2	6.28	12.18	D 25-42	19.6
320.0	22.69	22.09	0.00	52.2	6.31	12.20	D 25-42	19.7
330.0	23.03	22.27	0.11	54.2	6.36	12.26	D 25-42	19.8
340.0	23.27	22.34	0.15	56.6	6.37	12.23	D 25-42	19.7
353.0	23.62	22.53	0.04	59.7	6.41	12.25	D 25-42	19.8
360.0	23.83	22.60	0.16	61.5	6.44	12.26	D 25-42	19.8
370.0	24.10	22.76	0.23	64.1	6.47	12.28	D 25-42	19.8
380.0	24.39	22.89	0.30	66.6	6.51	12.33	D 25-42	19.9
390.0	24.63	23.01	0.31	69.6	6.54	12.33	D 25-42	19.9

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - Battered HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed battered H-piles at EB Abutment 2 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road Eastbound Bridge Sheet 7, Abutment No. 2 Footing Plan dated 4/2025.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road EB Bridge Abutment 2 H-Pile Analysis, dated 6/4/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on twenty (20) HP14x89 Grade 50 piles with a center-to-center pile spacing of 5.5 feet in the x-direction and 7 feet in the y-direction (Ref. 3). The piles are to be driven to bedrock.
2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at EB Abutment 2 (soil set 1).
3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - Battered HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2 = 0.18 ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in = 4.8 ft	
Pile head elevation =	95.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	60.6	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	36.0	ft	(Ref. 3, Sheet 13 of 53, includes additional foot for battered pile to reach bedrock)
Pile inclination =	14.0	degrees	(Ref. 6, piles battered at 3 on 12)

Ultimate Capacities:

Axial load on pile, P_u =	230	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	353	kips	

For soil above bedrock while driving pile to bedrock:

Nominal shaft resistance, R_s =	188	kips	(Ref. 6)
Maximum nominal toe resistance of soil, $R_{p\text{-}soil}$ =	57	kips	(Ref. 6)
Total soil resistance =	244	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.09	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at EB Bridge # 5823 Abutment 2 - Battered HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Shaft and Toe Resistance (during driving):

Nominal shaft resistance, R_s = 188 kips = 77 % of R_n (Ref. 6)
Nominal toe resistance, $R_{p\text{-}rock}$ = 57 kips = 23 % of R_n (Ref. 6)
Nominal resistance, R_n = 244 kips

Assume that once the pile has encountered bedrock, the resistance distribution will switch to 23% shaft resistance and 77% toe resistance. A second analysis will be performed to model these conditions.

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation = 95.5 ft
Groundwater elevation = 82.7 ft = 12.8 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Granular Borrow	94.0	93.0	1.5	2.5	125	32	-	-	-	Sand
Stiff Clay	93.0	82.5	2.5	13.0	122	-	1340	-	-	Clay
Glacial Till	82.5	61.6	13.0	33.9	130	35	-	-	-	Sand
Competent Bedrock	61.6	-	33.9	-	158	-	-	20.0	422.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	77%	23%	APE D25-42	45,892	Fuel Setting 2 (74%)	353	5.2	23.02	46%
Reaching tip elevation	23%	77%	APE D25-42	45,892	Fuel Setting 2 (74%)	353	5.0	23.57	47%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 2 (74% of the combustion pressure) while driving through soil and upon reaching bedrock should be able to overcome a soil resistance of 244 kips in order to drive the battered piles at EB Abutment 2 to bedrock and a nominal resistance of 353 kips while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

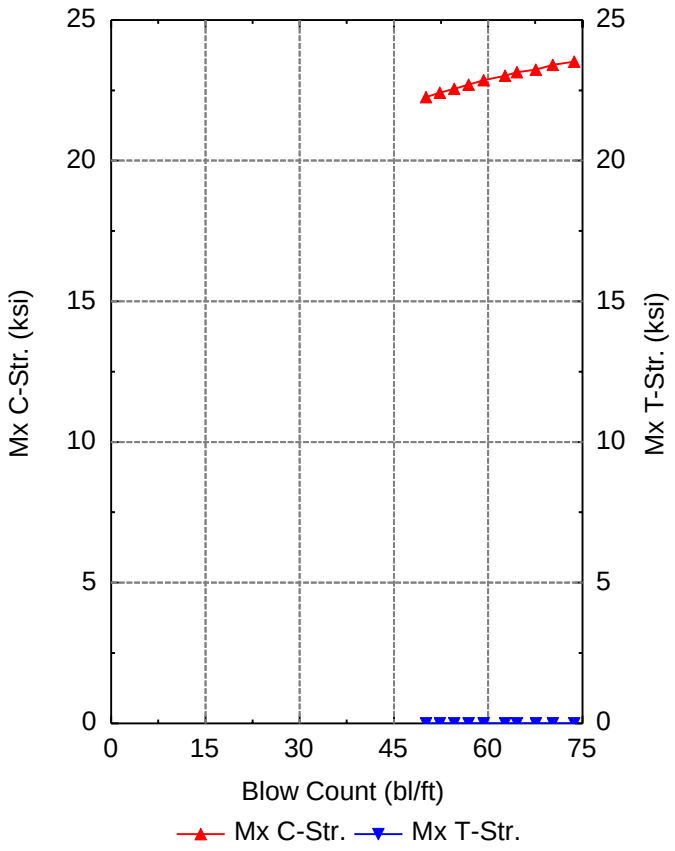
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

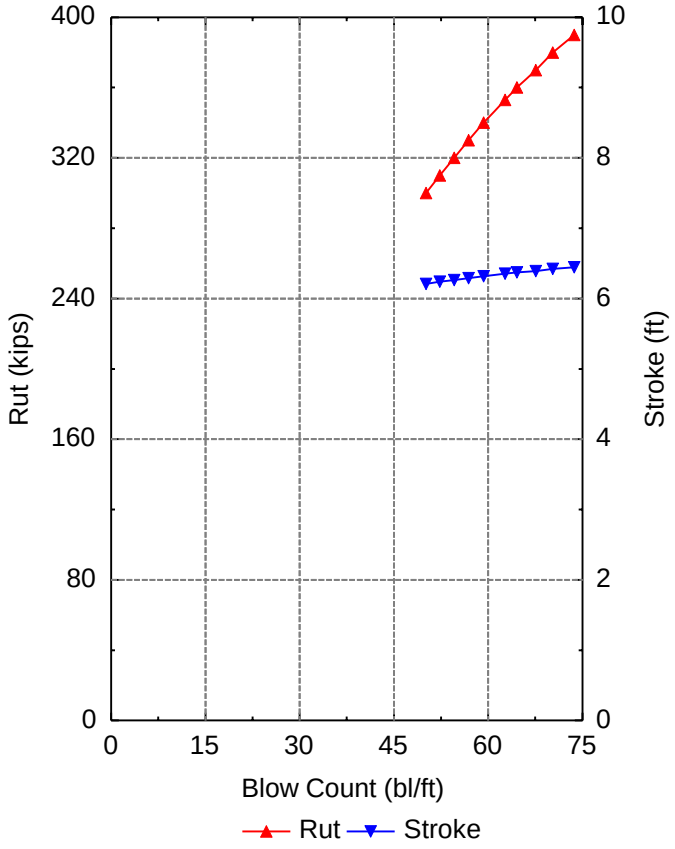
* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

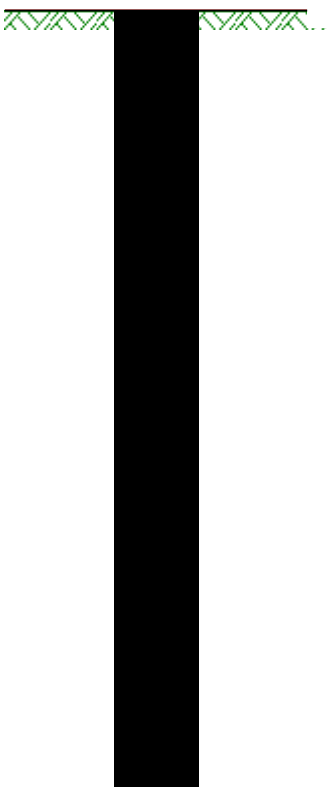
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



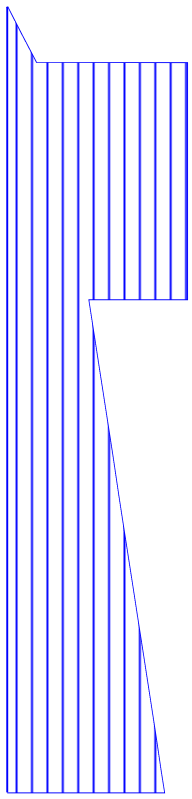


APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	34.90	ft
Pile Penetration	34.90	ft
Pile Top Area	26.10	in²
Pile Inclination	14	degree
RSA	No	



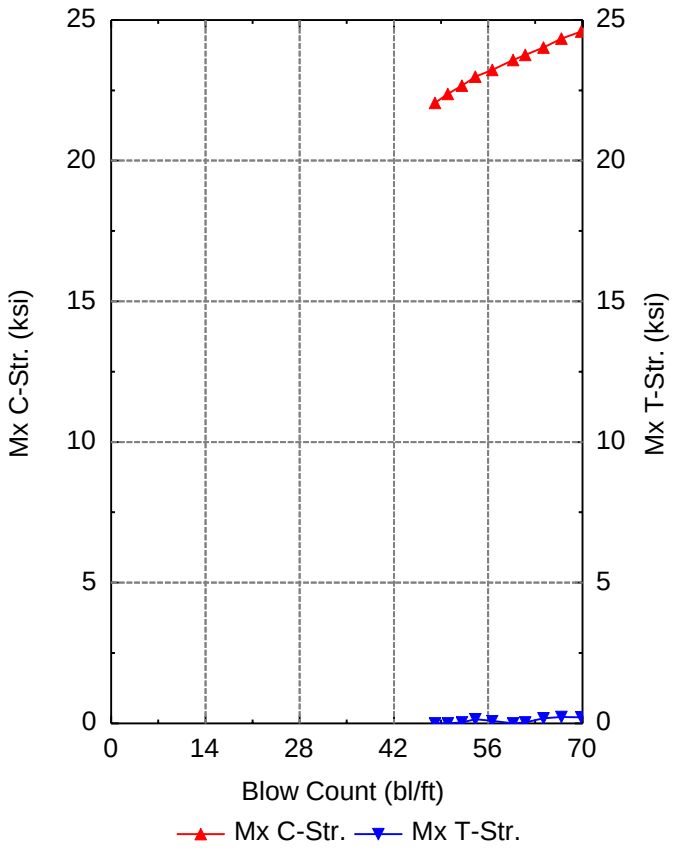
Pile Model



Shaft=77%
(Prop., SA)

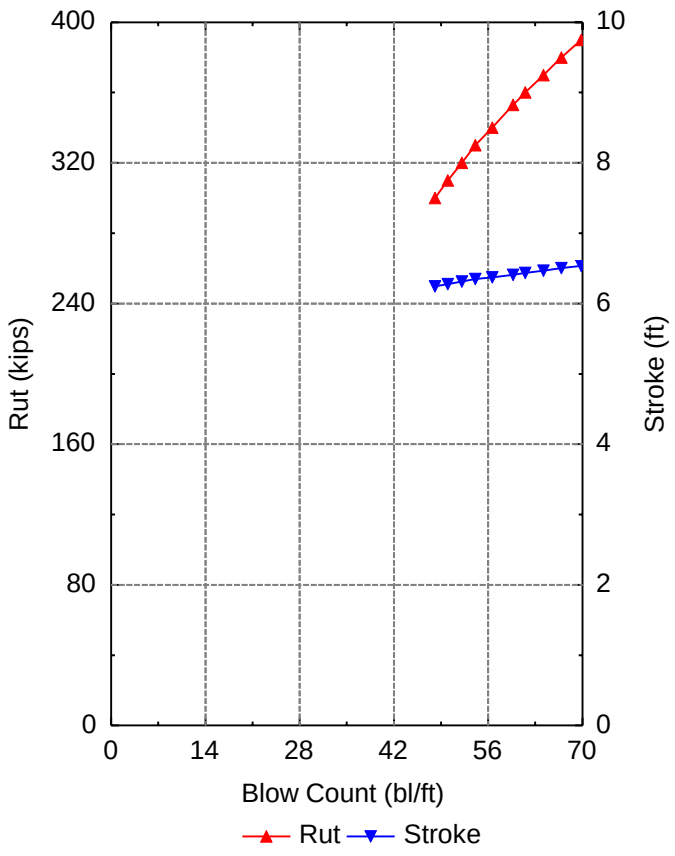
Bearing Graph Summary — APE D 25-42

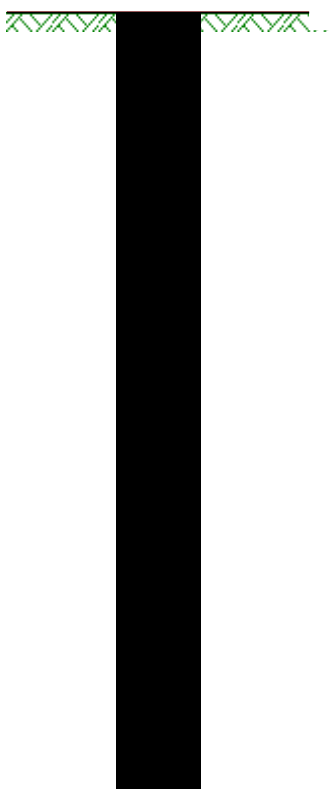
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
300.0	22.26	22.26	0.00	50.1	6.21	11.50	D 25-42	18.6
310.0	22.42	22.42	0.00	52.3	6.24	11.47	D 25-42	18.5
320.0	22.55	22.55	0.00	54.6	6.26	11.40	D 25-42	18.4
330.0	22.70	22.70	0.00	56.9	6.29	11.36	D 25-42	18.3
340.0	22.86	22.86	0.00	59.3	6.32	11.33	D 25-42	18.3
353.0	23.02	23.02	0.00	62.7	6.35	11.25	D 25-42	18.1
360.0	23.14	23.14	0.00	64.6	6.37	11.23	D 25-42	18.1
370.0	23.24	23.24	0.00	67.6	6.39	11.17	D 25-42	18.0
380.0	23.41	23.41	0.00	70.3	6.42	11.20	D 25-42	18.1
390.0	23.52	23.52	0.00	73.7	6.45	11.15	D 25-42	18.0



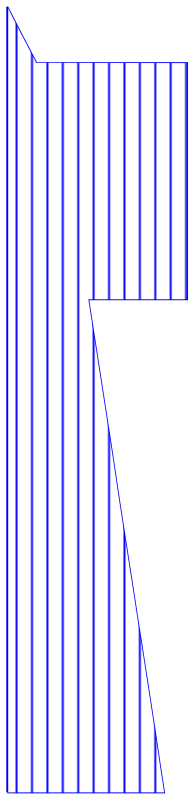
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.090	s/ft
Toe Damping	0.150	s/ft
Pile Length	34.90	ft
Pile Penetration	34.90	ft
Pile Top Area	26.10	in²
Pile Inclination	14	degree
RSA	No	





Pile Model



Shaft=23%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
300.0	22.05	21.78	0.00	48.1	6.24	12.19	D 25-42	19.7
310.0	22.37	21.94	0.00	50.0	6.28	12.22	D 25-42	19.7
320.0	22.67	22.09	0.03	52.1	6.31	12.25	D 25-42	19.8
330.0	22.98	22.25	0.14	54.1	6.35	12.29	D 25-42	19.8
340.0	23.23	22.34	0.09	56.6	6.37	12.26	D 25-42	19.8
353.0	23.57	22.51	0.00	59.7	6.41	12.29	D 25-42	19.8
360.0	23.76	22.60	0.03	61.5	6.43	12.31	D 25-42	19.8
370.0	24.02	22.73	0.18	64.2	6.47	12.32	D 25-42	19.9
380.0	24.34	22.87	0.23	66.9	6.50	12.36	D 25-42	19.9
390.0	24.59	23.00	0.21	69.8	6.53	12.37	D 25-42	19.9

APPENDIX J

Bridge 6787 Abutment Foundation Calculations

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Westbound (WB) (Bridge 6787) Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if proposed HP14x89 piles provide adequate support for the WB Abutment 1 based on the design load case 1 (controlling) (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at the WB Abutment 1 (Pile 24), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday December 2nd, 2024 with subject line "Bangor Hogan Road New WB Bridge - Abutment Bearing Loads for WSP Pile Foundation Evaluation and Layout/Size Recommendation".
15. WSP calculation package titled Lateral Earth Pressure at the proposed West Bound Bridge Abutments, dated 2/27/2025.
16. WSP calculation package titled Pile Drivability at Abutment 1 - HP14x89, dated 6/3/2025.
17. WSP Calculation package titled Hogan Road - Westbound Downdrag, dated 7/2/2025.

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Westbound (WB) (Bridge 6787) Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Case 1 output
4. FB-Multipier Case 2 output
5. Abutment loads

ASSUMPTIONS

1. The proposed Abutment 1 is assumed to be supported by twenty-four (24) HP 14x89 Grade 50 piles with center spacing of 8.5 feet in the x-direction and 6 feet in the y-direction shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) were applied at the bearings on the abutment stem wall.
4. All piles are assumed to have reinforced pile tips (Ref. 8).
5. All piles are plumb.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. Downdrag loading does develop along the length of the maximum axial loaded pile at WB Abutment 1 piles (Ref. 17). While downdrag develops along other piles at Abutment 1, the combined downdrag loading and maximum axial load at pile 24 controls.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added to the toe of the Abutment 1 pile cap to account for the granular borrow and crushed stone slope on the toe (Ref. 5), and a vertical earth load EV was added to the heal of the Abutment 1 pile cap to account for the backfill and traffic live load surcharge (Ref. 15). The EV loads on the toe and heal are added as point load spaced evenly along the pile cap between pile locations.
10. A horizontal earth load EH of 4,598 plf (Ref. 15) was added to the pile cap on the back side of the abutment within the area of the stem wall.
11. A φdyn of 0.65 is used to determine the factored pile resistance, based on the MaineDOT standard Specification, PDA testing will be performed on 2% of the production piles.
12. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Westbound (WB) (Bridge 6787) Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section	
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	0.455	in
Fillet area =	0.3	in^2		0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	19.3	in^2
Section area =	0.18	ft^2		0.13	ft^2
Moment of inertia about the x-axis, I_x =				653	in^4
Moment of inertia about the y-axis, I_y =				233	in^4
Radius of gyration about the x-axis, r_x =				5.82	in
Radius of gyration about the y-axis, r_y =				3.47	in
Elastic section modulus about the x-axis, S_x =				95.8	in^3
Elastic section modulus about the y-axis, S_y =				32.1	in^3
Plastic section modulus about the x-axis, Z_x =				105.7	in^3
Plastic section modulus about the y-axis, Z_y =				48.8	in^3

(ref. 13)

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	104.5	ft	Sheet B20 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	103.0	ft	
As-modeled pile cap midplane elevation =	103.0	ft	Sheet B20 (Ref. 5)
Base of pile cap elevation =	101.5	ft	Sheet B20 (Ref. 5)
Pile length =	30.8	ft	

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Westbound (WB) (Bridge 6787) Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Crushed Stone	1	Cohesionless	101.5	100.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Granular Borrow	2	Cohesionless	100.5	97.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Silt	3	Cohesionless	97.6	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	4	Cohesive	94.7	80.6	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	5	Cohesive	80.6	78.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	6	Cohesionless	78.1	73.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	7	Rock	73.2	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 210+25 in Profile B-B' (Sheet 4 Interpretive Subsurface Cross Section B-B').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

(Pile 24)
Load case 1

Maximum axial force in the piles (includes 21.9 kip downdrag load (Ref. 17), P_u =	236.6	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	133.9	kip-ft	=	1606.6	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	8.6	kip-ft	=	103.6	in-kips	
Maximum shear force in the piles, V_u =	31.8	kips				
Depth below pile head to fixity =	19.0	ft	=	Elev.	84.04	ft
Depth below pile cap to fixity =	17.5	ft	=	Elev.	84.04	ft

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Westbound (WB) (Bridge 6787) Abutment 1 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Crushed Stone	Granular Borrow	Silt	Glacial Till	
Friction angle, ϕ_f =	40	32	30	35	degrees (Ref. 6)
δ =	31	25	23	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.87	1.02	0.89	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.87	0.91	0.92	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'v =	0.053	0.286	0.649	1.890	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.044	0.112	0.209	0.941	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.340	1.340	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	0.95	0.95	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.179	1.179	0.844	0.844	ksf

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 56.4 in = 4.7 ft (from Part A)

	Crushed Stone	Granular Borrow	Silt	Stiff Clay	Medium Clay	Glacial Till	
Thickness =	1.0	2.9	2.9	14.1	2.5	4.9	ft (from stratigraphy in Part A
A_s =	4.7	13.6	13.6	66.2	11.7	23.0	ft^2
R_s per layer =	0.2	1.5	2.9	78.1	9.9	21.6	kips

Total R_s = 114.2 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD ≤ 70%
$q_p = 0.80 q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 1,279 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 422.1 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

$R_n = R_s + R_p$ Pile toe area, A_p = 0.13 ft^2 (from Part A)

Total R_p = 56.6 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 171 kips (bedrock)

CALCULATIONS

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$$R_r = \varphi_{stat} R_n$$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:
(Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method
φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method
φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Crushed Stone	Granular Borrow	Silt	Stiff Clay	Medium Clay	Glacial Till	Pile Tip (bedrock)	
R_s or R_p =	0.2	1.5	2.9	78.1	9.9	21.6	56.6	kips
φ_{stat} =	0.45	0.45	0.45	0.35	0.35	0.45	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	0.7	1.3	27.3	3.5	9.7	25.5	

Total R_r = 68 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 71% of the steel yield strength (Ref. 16):

R_n = 707 kips (Ref. 14)

φ_{dyn} = 0.65 (Assumption 11)

R_f = 460 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 237 kips (from Part B)

R_r = 460 kips > P_u = 237 kips OK

D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 0.65 (Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l = 209.58 in (from Part B)
Minimum radius of gyration, r_s = 3.47 in (from Part A)

Check: Kl/r_s = 39 < 120 OK

CALCULATIONS

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Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 12)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check: $\frac{b}{t}$ = 15.98 > 13.49 = $0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, l =	209.58	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

P_e (flexural) = 3,597 kips

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \leq 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

CALCULATIONS

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Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877 P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.27 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 863 kips

For compression member cross sections containing and slender elements, the nominal compressive resistance, P_n. Shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$P_n = F_{cr} A_{eff}$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$F_{cr} = \frac{P_{cr}}{A_g}$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

$P_{cr} =$	863	kips	Nominal compressive resistance (flexural)
$A_g =$	19.3	in^2	(from Part A)
$F_{cr} =$	44.7	ksi	

$A_{eff} = A_g - \sum (b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$If \ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$If \ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$ (Ref. 7, Eq. 6.9.4.2.2b-4)

CALCULATIONS

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where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	44.7	ksi	

$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = 14.3$$

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

c_1 =	0.22		(Ref. 7, Table 6.9.4.2.2b-1)
c_2 =	1.49		(Ref. 7, Table 6.9.4.2.2b-1)
c_3 =	0.00		(Ref. 7, Table 6.9.4.2.2b-1)
F_{el} =	79.09	ksi	(Ref. 7, Eq. 6.9.4.2.2b-4)
b_e =	6.84	in	(Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff} :

A_{eff} =	18.54	in^2	(Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n =	828.6	kips	(Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.50		(Ref. 7, Article 6.5.4.2))
P_r =	414	kips	

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.70		(Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$ =	580	kips	

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u =	237	kips	(from Part B)

$P_{r,\ axial}$ = 414 kips > P_u = 237 kips OK

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

CALCULATIONS

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Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	1607	in-kips	(from Part B)
M_{uy} =	104	in-kips	(from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	1607	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	104	in-kips	OK

F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \quad \text{if } P_u/P_{r,\text{ axial/ flexural}} < 0.2$$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \quad \text{if } P_u/P_{r,\text{ axial/ flexural}} \ge 0.2$$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.4 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.74 < 1.0 OK

CALCULATIONS

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G. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_p = 0.58 F_y d_w t_w$ (Ref. 7, 6.10.9.2-2))

$V_n = CV_p = V_{cr}$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$\frac{D}{t_w} \leq 1.12\sqrt{\frac{Ek}{F_y}}$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips	
V_n =	168	kips	
φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

$V_u \leq \varphi_v V_n$

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

V_u = 31.8 kips (from Part B)

V_r = 168 kips > V_u = 31.8 kips OK

CALCULATIONS

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H. Determine nominal pile driving resistance.

P_u = 237 kips (from Part B)

φ_{dyn} = 0.65 (Ref. 3, Table 7-2)

for driving criteria established by dynamic testing of at least two piles per site condition and no less than 2% of production piles

Nominal pile driving resistance, R_{ndr} = 364 kips

Confirm R_{ndr} does not exceed P_n:

R_{ndr}			P_n	(from Part D)	
364	kips	<	829	kips	OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 1, and an estimated pile length of 30.8 feet, a maximum axial force of 236.6 kips including downdrag developed at pile 24, the corresponding strong axis (x-axis) moment of 1,606 in-kips (133.9 ft-kips) occurs in the HP14x89 piles at Abutment 1 in Pile 24 of the proposed WB Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The WB Abutment 1 piles should be driven to a nominal resistance of 364 kips. The analysis indicates that the maximum top of pile deflection at the service limit state is less than 0.3 inches in the X and Y directions. This evaluation considers section loss for a 100 year design life. A summary table is presented below.

WB Abutment 1 Summary Table		
Pile Length (ft)		31
Depth below Pile Head to Fixity (ft)		19
Geotechnical Axial Resistance (kips)	Nominal	707
	Factored	460
Structural Axial Resistance (kips)	Nominal	863
	Factored	414
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.74
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

Figure 1b: Profile View of Pile Cap (Transverse)

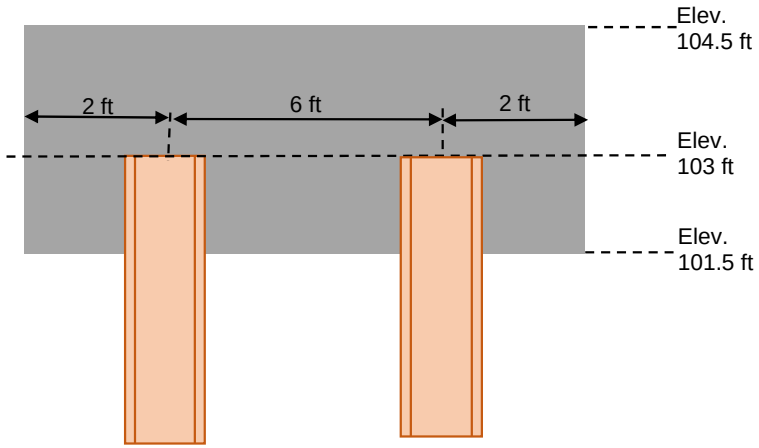


Figure 1c: 3D View of Pile Model

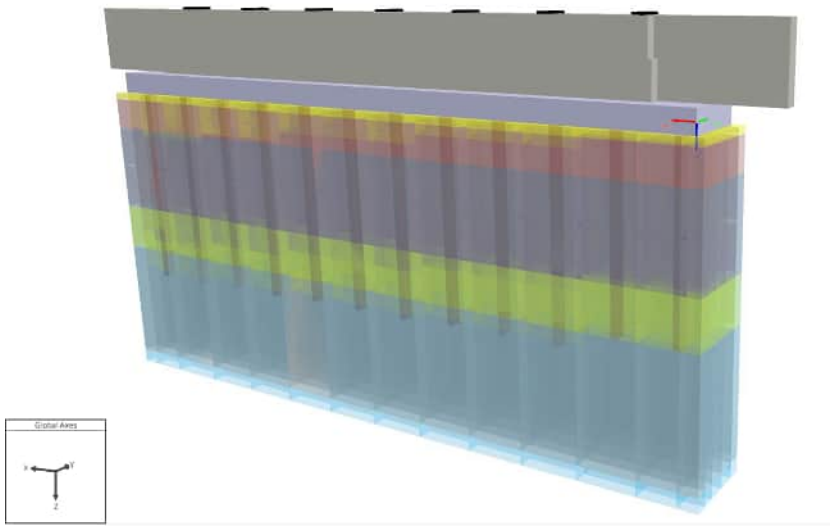
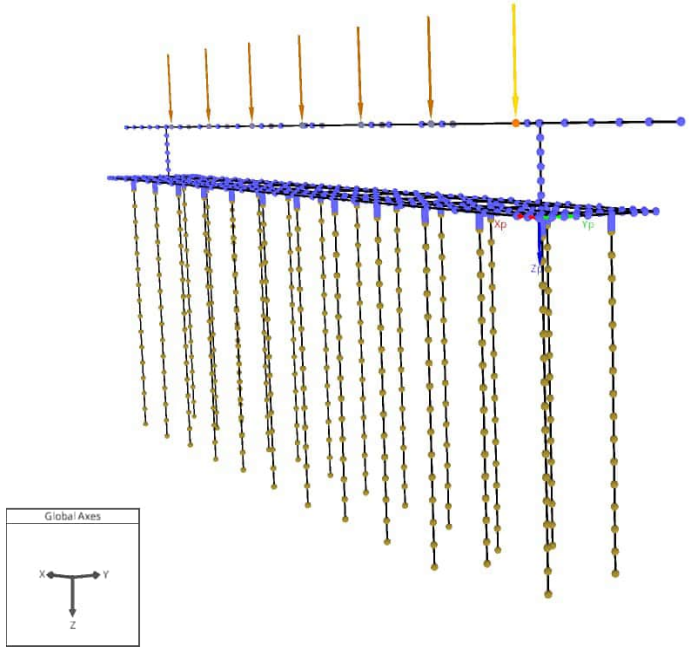


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Crushed Stone	1	Cohesionless	101.5	100.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	105	40	253	-		-	-	1.22		0.42	44		-	-	-
Granular Borrow	2	Cohesionless	100.5	97.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	112		-	-	-
Silt	3	Cohesionless	97.6	94.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	30	50	-		-	-	0.28		0.25	209		-	-	-
Stiff Clay	4	Cohesive	94.7	80.6	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medim Clay	5	Cohesive	80.6	78.1	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	6	Cohesionless	78.1	73.2	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	941		-	-	-
Bedrock	7	Rock	73.2	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3899	707	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on the WB Abutment 1 profile B-B' (Sheet 4).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf*) . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

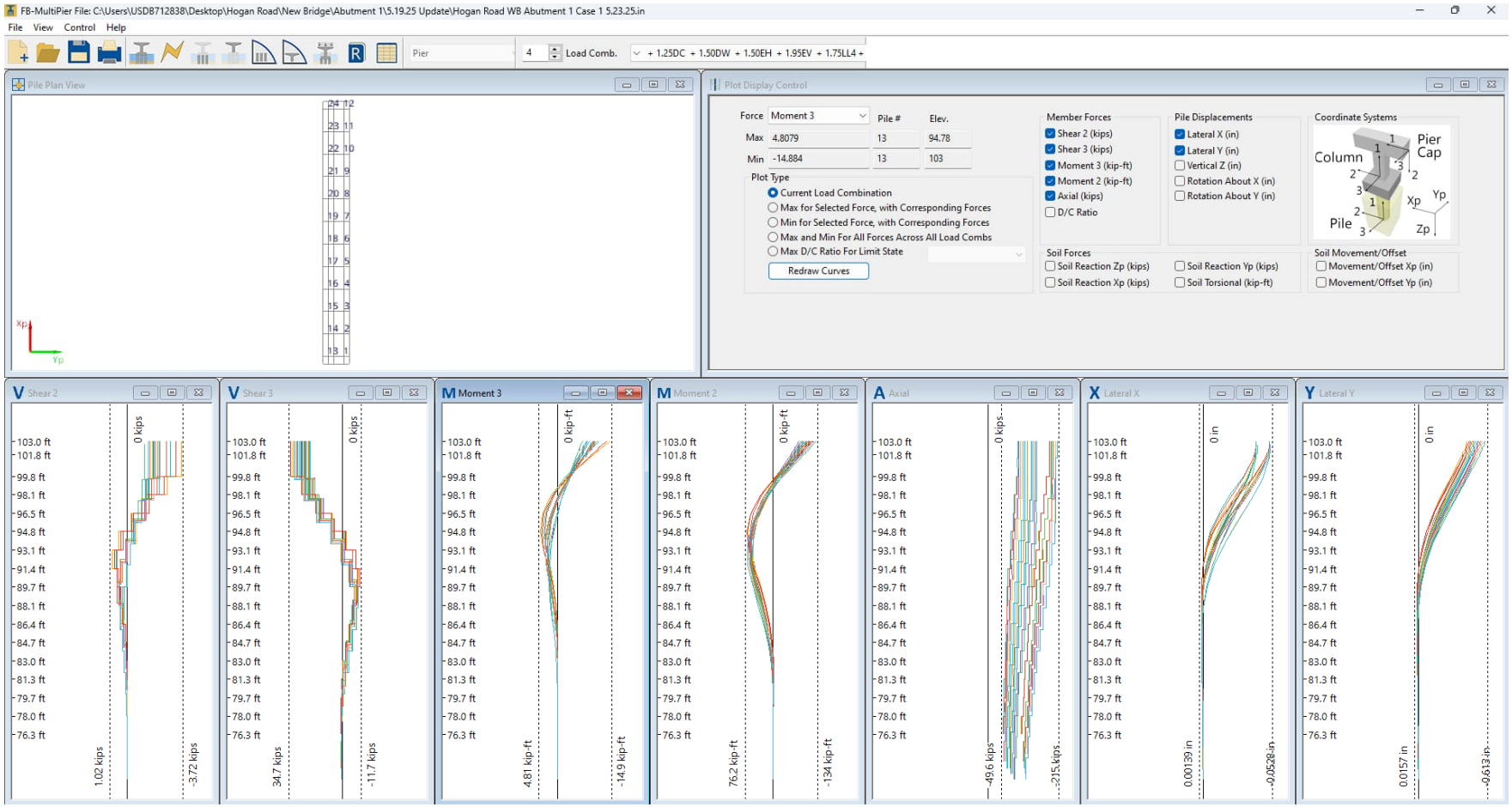
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

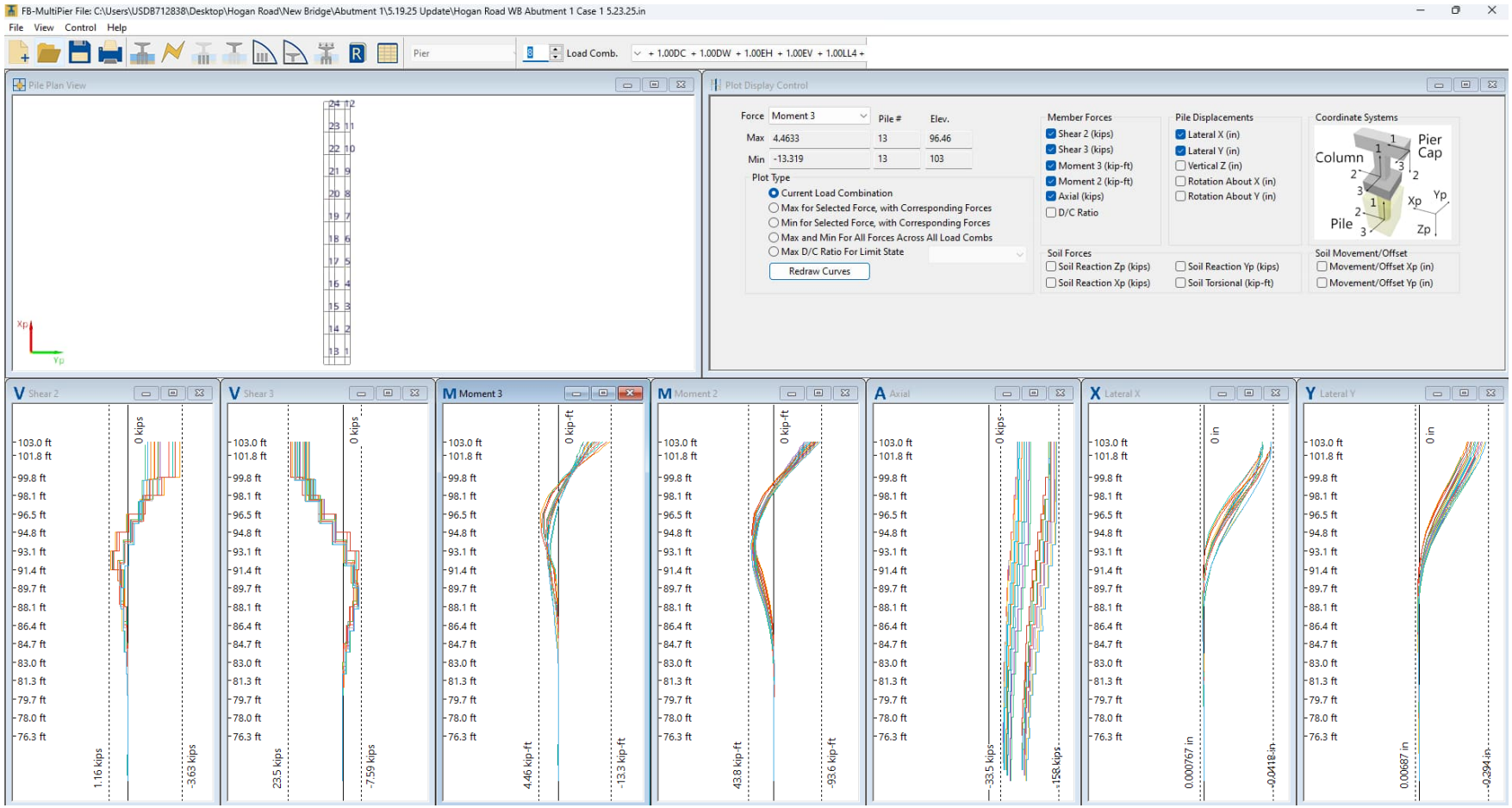
8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed:

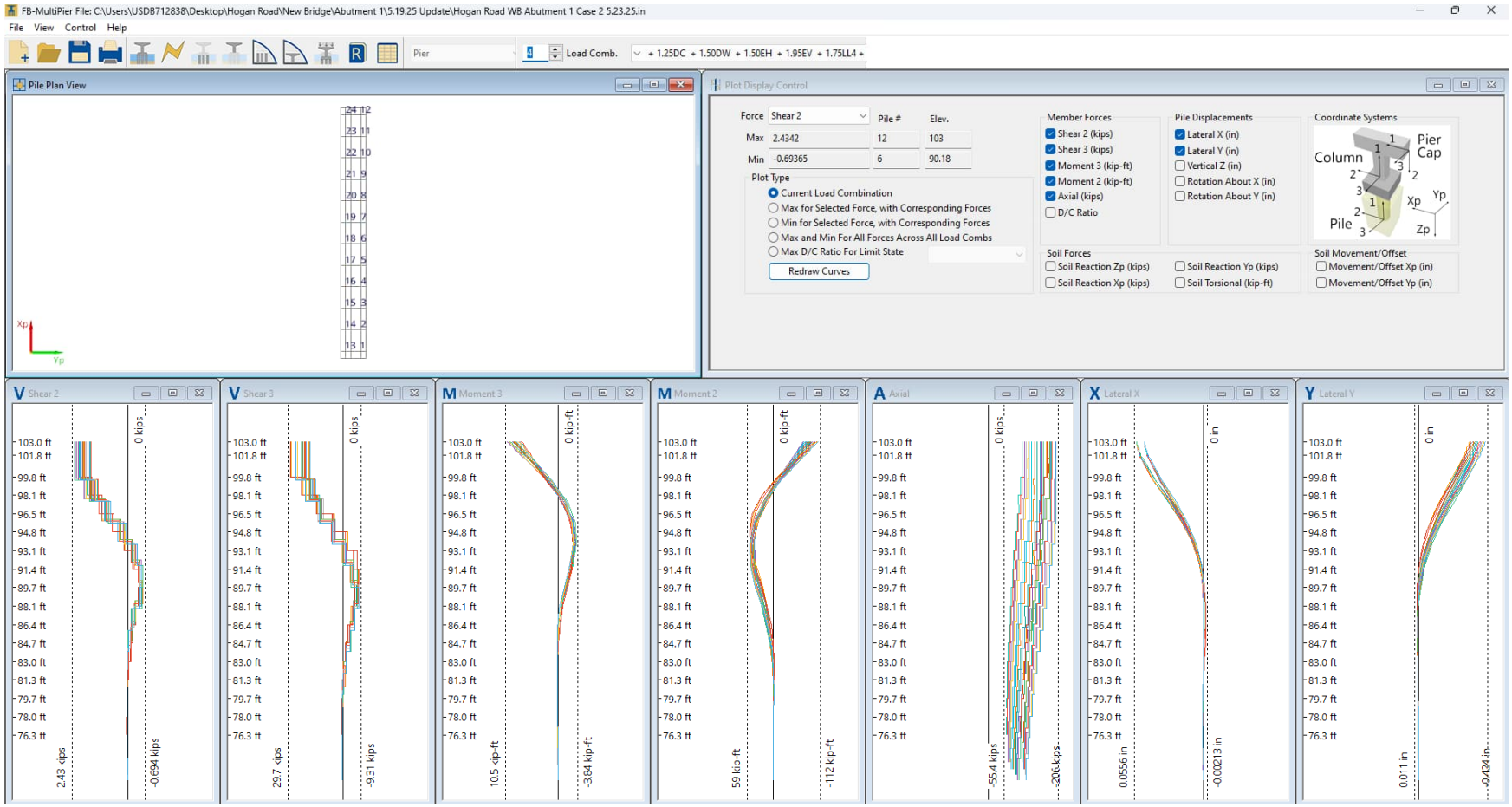
Case 1 Strength I



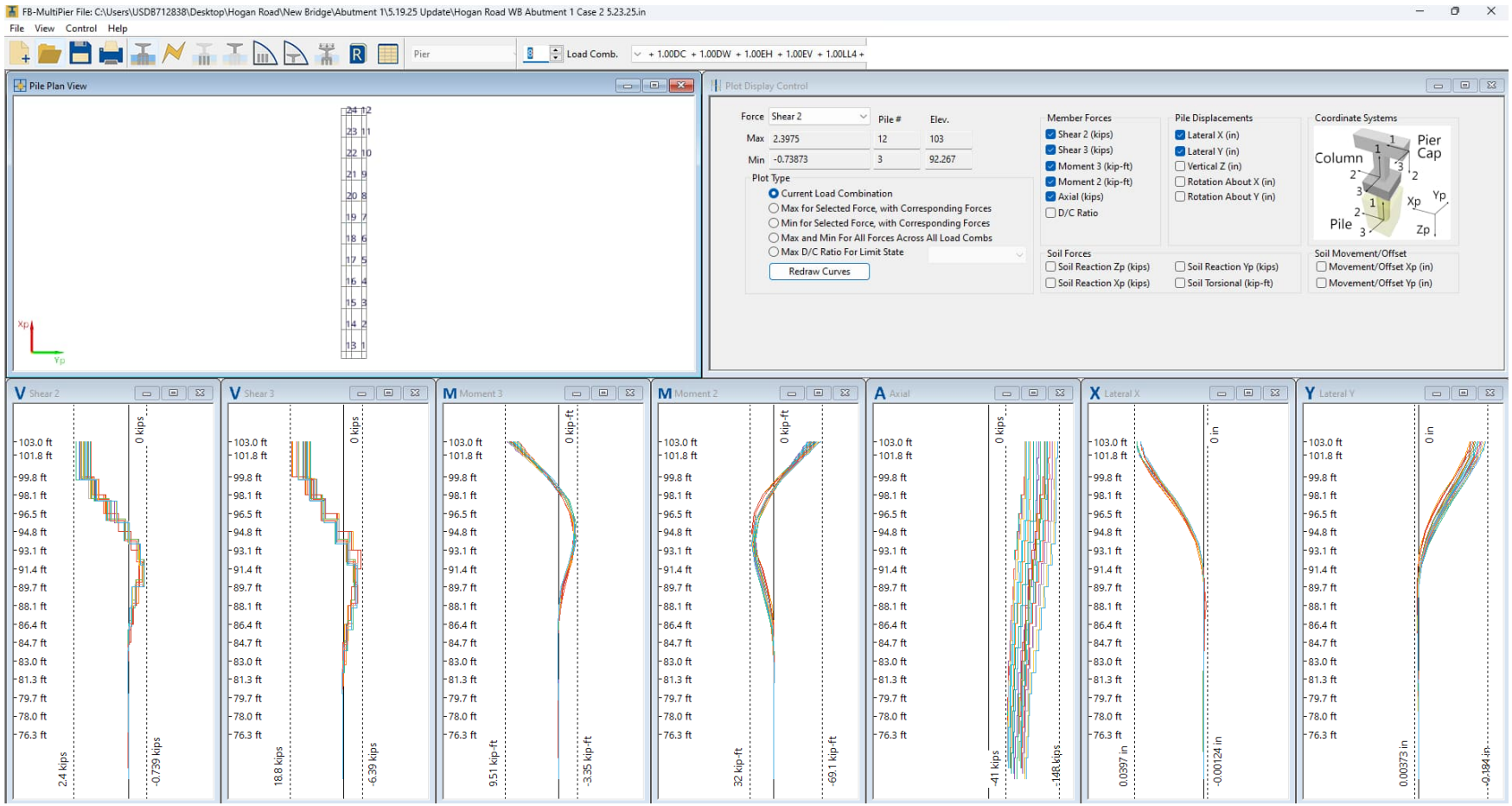
Case 1 Service I



Case 2 Strength I



Case 2 Service I



Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	11/12/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computations

Unfactored Bearing Loads, Abutment

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	G7
DC	80	80	80	80	80	80	80
DW	15	15	15	15	15	15	15
LL 1A	55	85	83	0	0	0	0
LL 1B	0	0	0	0	83	85	55
LL 2A	47	69	71	83	0	0	0
LL 2B	0	0	0	83	71	69	47
FR							

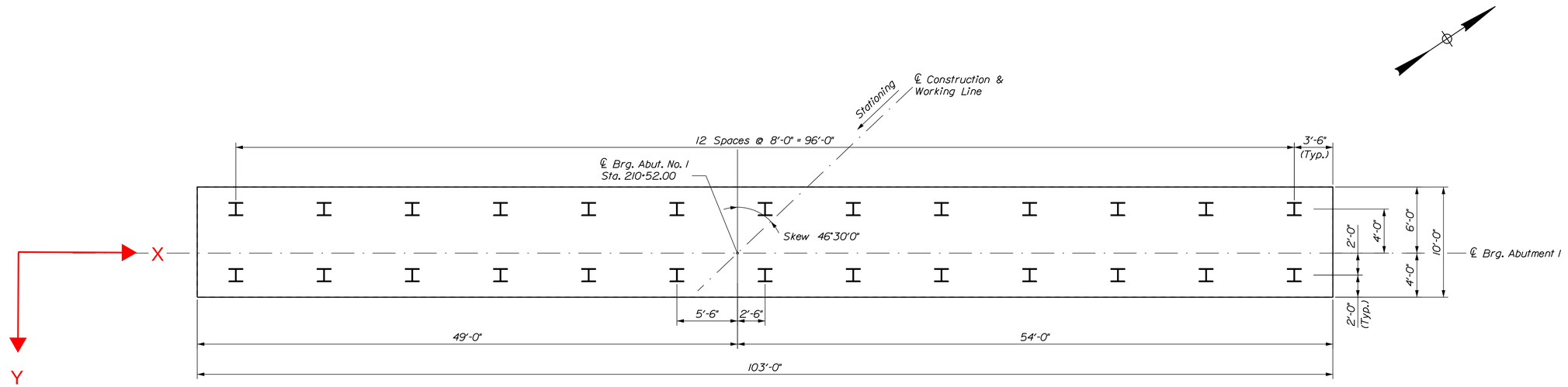
	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6	G7	G1	G2	G3	G4	G5	G6	G7
DC														
DW														
LL 1A														
LL 1B														
LL 2A														
LL 2B														
FR	7	7	7	7	7	7	7	7	7	7	7	7	7	7

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	G7
DC	80	80	80	80	80	80	80
DW	15	15	15	15	15	15	15
LL 1A	55	85	83	0	0	0	0
LL 1B	0	0	0	0	83	85	55
LL 2A	47	69	71	83	0	0	0
LL 2B	0	0	0	83	71	69	47
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6	G7	G1	G2	G3	G4	G5	G6	G7
DC														
DW														
LL 1A														
LL 1B														
LL 2A														
LL 2B														
FR	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7

NOTE TO REVIEWER:
1) Pile design is not complete and subject to change.



FOOTING PLAN & PILE LAYOUT

REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	JAH	8/31/24
CHECKED-REVIEWED	YP	SMH	8/31/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

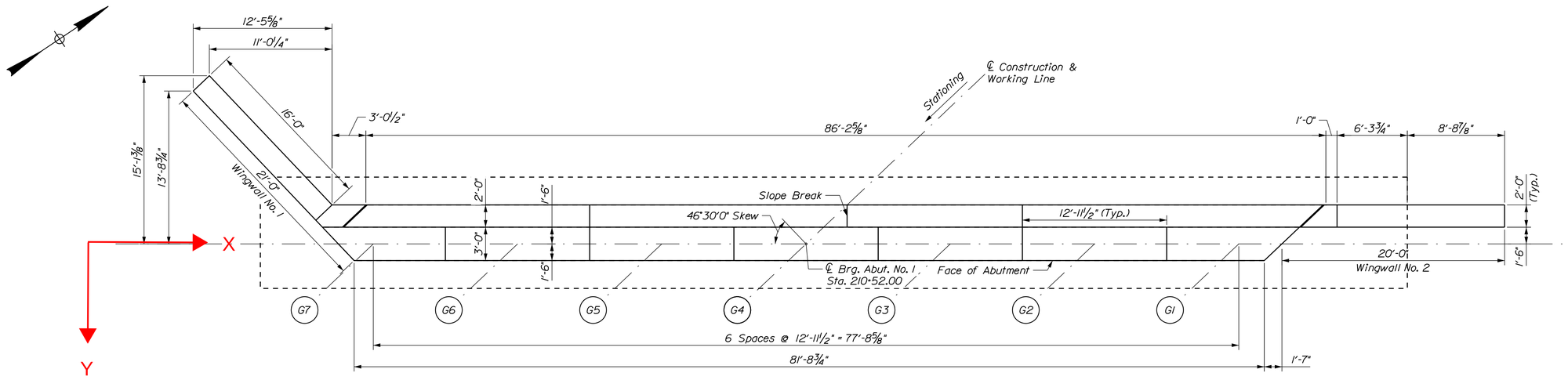
ABUTMENT NO.1
FOOTING PLAN

SHEET NUMBER

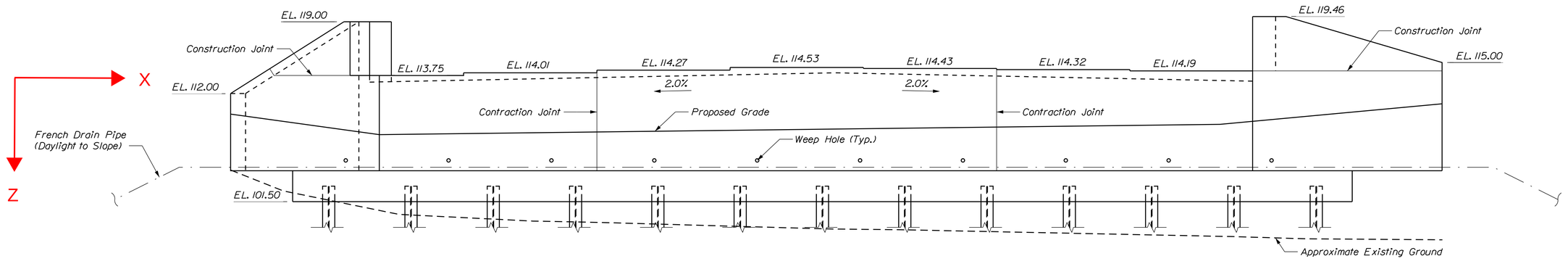
B19

OF B49

Filename: ...\B19_WB_Abutment_1_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



PLAN



ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 1
PLAN & ELEVATION

SHEET NUMBER

B20

OF B49

Filename: ...\B20_WB_Abutment_1_Detail.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the proposed HP 14x89 steel pile with the maximum loading (Pile 24) at Bridge #6787 Abutment 1 of the bridge carrying Hogan Road Westbound over I-95. This analysis uses estimated settlement for 75 years after construction.

METHOD

Use the Beta method (Ref. 6) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Maine Department of Transportation Bridge Program. 60% West Bound Bridge Plans, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Dated 8/30/2025.

2. WSP Settle3 West Bound model titled "Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z", dated 6/2025.

3. WSP Interpretive Subsurface Profile B- B', included as part of the Geotechnical Design Report.

4. WSP calculations of interpreted soil parameters based on field and laboratory data, named "Lab Field Data Summary Plots V4 31404817.000.xlsx".

5. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

6. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.

7.WSP FB-Multipier Soil properties for the proposed West Bound Bridge, named "New Bridge Abutment 1 FB-Multipier Soil Properties.xlsx", dated 5/20/2025.

8.WSP West Bound Abutment 1 Pile Analysis, named "Hogan Road New Bridge Abutment 1 H-Pile Analysis UPDATED Layout 5.19.25.xlsx", dated 6/3/2025.

9. Loading provided by VHB in Computation package titled "Bangor DDI Hogan Rd WB Bearing Loads 11/12/2024"

10. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.

ATTACHMENTS

A.1. Profile of estimated settlement beneath Abutment 1 75 years after construction

A.2. Profile of estimated settlement beneath Abutment 1 75 years after construction at the location of the pile with maximum axial loading

ASSUMPTIONS

1. The proposed Abutment 1 is supported on twenty-four HP14x89 steel piles, based on Ref. 8.The downdrag is evaluated for the pile with maximum axial loading based on the proposed abutment loads provided by VHB (Ref. 9) and distributed to the individual piles by WSP using FB-MultiPier (Ref. 8). The estimated pile length at this location per Ref. 8 is 29.3 feet below the base of the abutment.

2. The downdrag depth is based on the total settlement estimated to occurred 75 years after the construction of the embankment at the Abutment 1 at the maximum settlement location (Ref. 2).

3. At MaineDOT's request based on their internal research, the Beta method (Ref. 6, 10.7.3.8.6c) will be used to estimate shaft resistance, rather than the Alpha method on cohesive soils and the Nordlund method for cohesionless soils (FHWA method).

4. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 3 and Ref. 7 at the centerline of the Abutment 1, for soil set 5 which correspond to the soil set used in FB-Multipier model for Pile 24 (maximum loaded pile). The soil properties used in the downdrag analysis are taken from Ref. 4 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	101.5	feet	(Ref. 1, Sheet B20)
Water table elevation =	96.0	feet	(Ref. 7)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	Beta (β)[1]
	Top	Bottom	Top	Bottom				
Crushed Stone	101.5	100.5	0.0	1.0	105	40	-	0.58
Granular Borrow	100.5	97.6	1.0	3.9	125	32	-	0.30
Silt	97.6	94.7	3.9	6.8	125	30	-	0.33
Stiff Clay	94.7	80.6	6.8	20.9	122	-	1340	0.35
Medium Clay	80.6	78.1	20.9	23.4	121	-	888	0.27
Glacial Till	78.1	73.2	23.4	28.3	130	35	-	0.35
Bedrock	73.2	-	28.3	-	158	-	-	-

Note: [1]Beta values taken from Ref. 10, Table 7-9.

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	29.3	ft	(Ref. 8)
Pile section box perimeter =	57.0	in	(using pile flange width and depth from Ref. 5)
Pile section tip area =	26.1	in^2	(Ref. 5)
Pile nominal weight =	0.089	kip/ft	(Ref. 5)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 6 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 2 and shown in Attachment 1 the settlement beneath Abutment 1 after 75 years of construction for the maximum settlement point is estimated to be greater than 0.4 inches down to an elevation of 87.9 feet (depth of 14.0 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 16 feet of the pile below the base of the abutment.

Per Ref. 6 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use the Beta method to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 6, Tables 3.4.1-1 and 3.4.1-2)

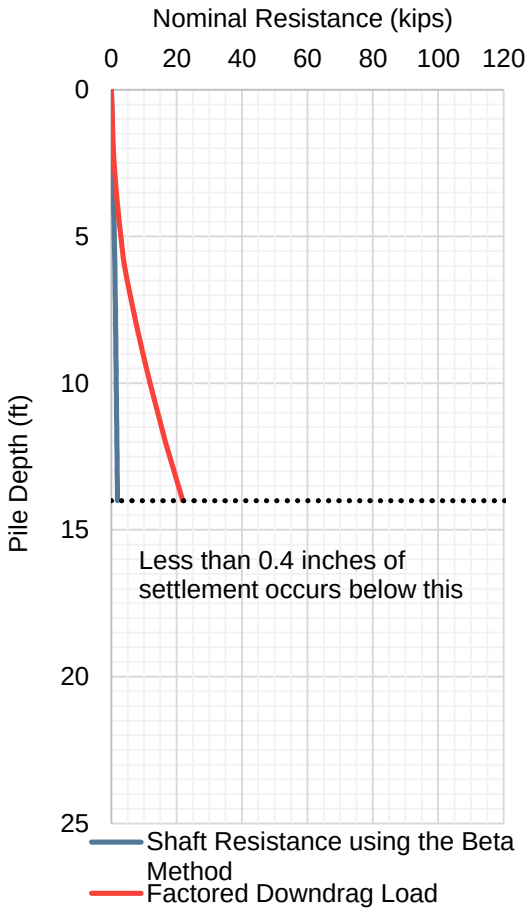
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	214.72	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type	Shaft Resistance using the Beta Method	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	214.7
1	Sand	0.3	0.3	215.1
2	Sand	0.3	0.6	215.5
3	Sand	0.5	1.2	216.2
4	Sand	0.7	1.9	217.0
5	Sand	0.9	2.9	218.1
6	Sand	1.1	4.1	219.3
7	Clay	1.2	5.8	221.2
8	Clay	1.3	7.7	223.1
9	Clay	1.4	9.7	225.2
10	Clay	1.5	11.9	227.5
11	Clay	1.6	14.2	229.9
12	Clay	1.7	16.6	232.4
13	Clay	1.8	19.2	235.1
14	Clay	1.9	21.9	237.9
15	Clay	2.0		216.1
16	Clay	2.1		216.1
17	Clay	2.2		216.2
18	Clay	2.3		216.3
19	Clay	2.4		216.4
20	Clay	2.5		216.5



*Strength I, including Substructure Loading, Downdrag, and Pile Weight

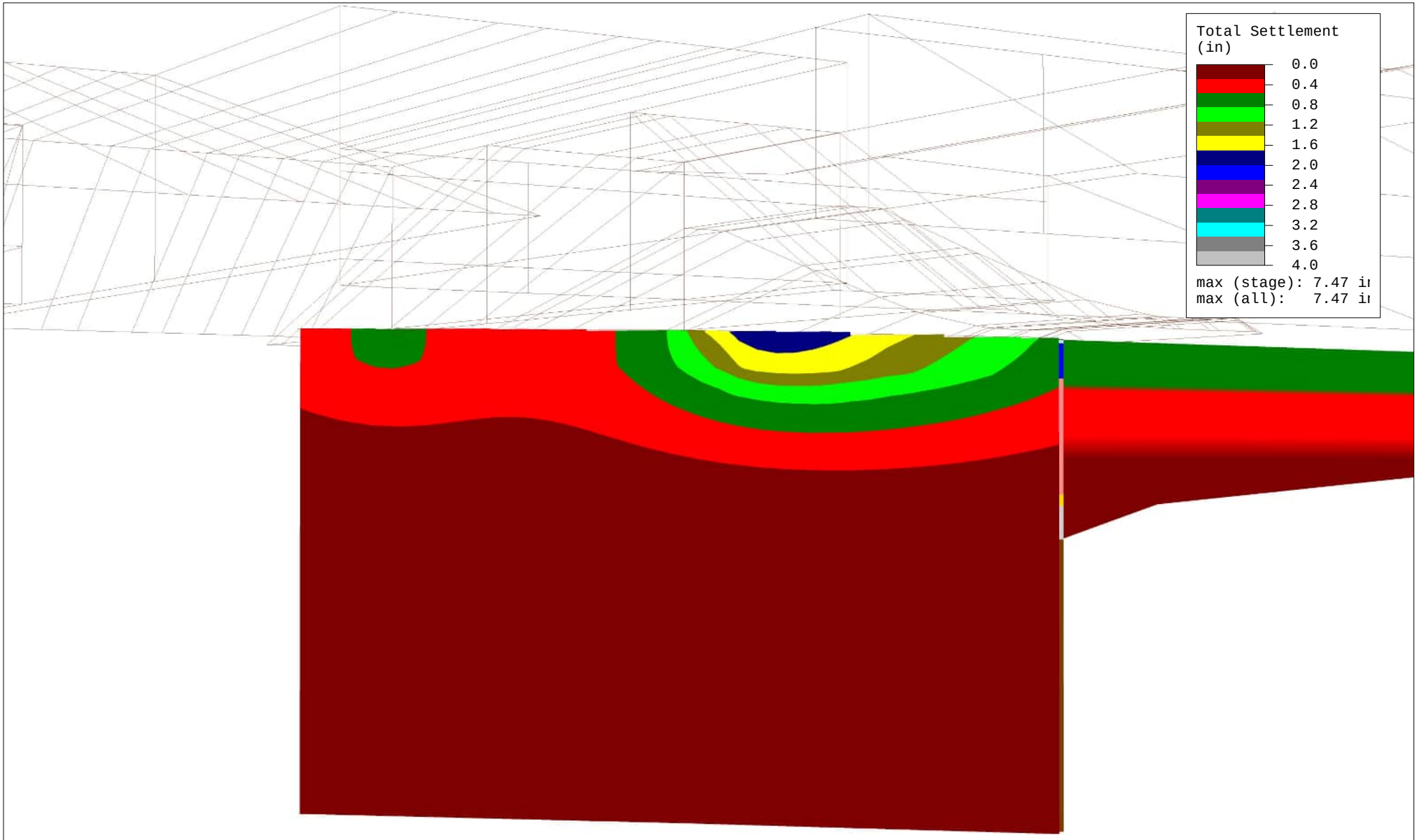
Strength I factored downdrag load = 21.9 kips per pile

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CONCLUSIONS

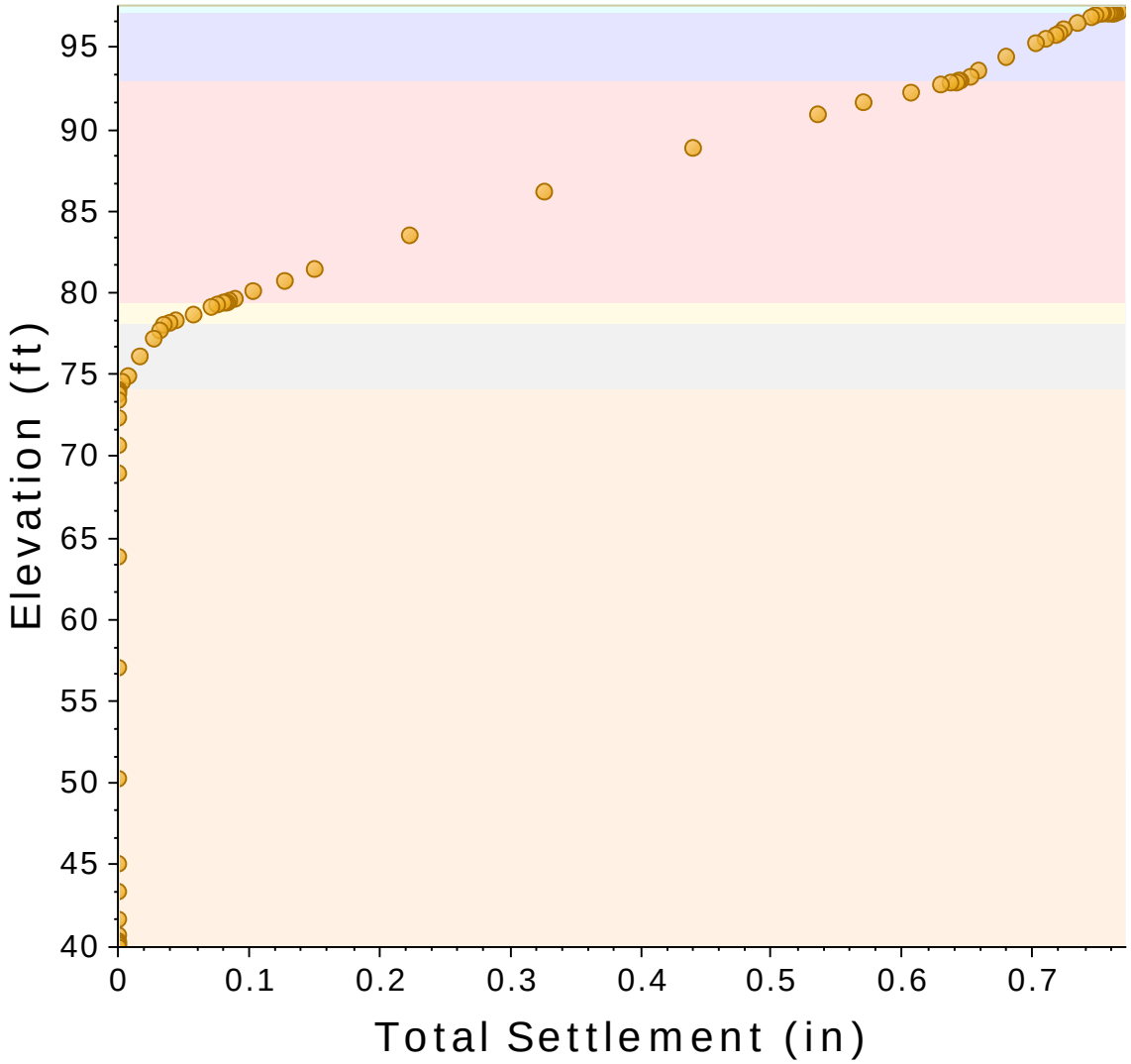
Based on the analysis of settlement for the proposed Abutment 1 of Bridge #6787 at the location of maximum loaded pile, downdrag is estimated to develop along the upper 14 feet of pile 24 below the base of the Abutment 1. A factored downdrag load of 21.9 kips per pile for the Strength I limit state was calculated.



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement along the proposed Abutment 1 after 75 years of construction			
Drawn By LMP	Checked By DEB	Reviewed By	Scale As shown	Figure A.1
Date 5/30/2025	File Name Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z			

SETTLE3 5.025

Total Settlement vs. Elevation



Max Load Pile Abutment 1 (Stage 10 = 75 y)

Reference Stage: None

wsp	*Project* MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
	Analysis Description Total Settlement at the location of the pile with maximum loading on the proposed Abutment 1 after 75 years of construction			
	Drawn By LMP	Checked By DEB	Reviewed By	Scale As shown
SETTLE3 5.025	Date 5/30/2025	File Name Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z		Figure A.2

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the proposed HP 14x89 steel pile closest to the maximum settlement point (Pile 8) at Bridge #6787 Abutment 1 of the bridge carrying Hogan Road Westbound over I-95. This analysis uses estimated settlement for 75 years after construction.

METHOD

Use the Beta method (Ref. 6) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Maine Department of Transportation Bridge Program. 60% West Bound Bridge Plans, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Dated 8/30/2025.

2. WSP Settle3 West Bound model titled "Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z", dated 6/2025.

3. WSP Interpretive Subsurface Profile B- B', included as part of the Geotechnical Design Report.

4. WSP calculations of interpreted soil parameters based on field and laboratory data, named "Lab Field Data Summary Plots V4 31404817.000.xlsx".

5. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

6. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.

7.WSP FB-Multipier Soil properties for the proposed West Bound Bridge, named "New Bridge Abutment 1 FB-Multipier Soil Properties.xlsx", dated 5/20/2025.

8.WSP West Bound Abutment 1 Pile Analysis, named "Hogan Road New Bridge Abutment 1 H-Pile Analysis UPDATED Layout 5.19.25.xlsx", dated 6/3/2025.

9. Loading provided by VHB in Computation package titled "Bangor DDI Hogan Rd WB Bearing Loads 11/12/2024"

10. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.

ATTACHMENTS

A.1. Profile of estimated settlement beneath Abutment 1 75 years after construction

A.2. Profile of estimated settlement beneath Abutment 1 75 years after construction at the location of the pile with maximum settlement.

ASSUMPTIONS

1. The proposed Abutment 1 is supported on twenty-four HP14x89 steel piles, based on Ref. 8. The downdrag is evaluated for the pile closest to the maximum settlement point. The estimated pile length at this location per Ref. 8 is 28.8 feet below the base of the abutment.

2. The downdrag depth is based on the total settlement estimated to occurred 75 years after the construction of the embankment at the Abutment 1 at the maximum settlement location (Ref. 3).

3. At MaineDOT's request based on their internal research, the Beta method (Ref. 6, 10.7.3.8.6c) will be used to estimate shaft resistance, rather than the Alpha methed on cohesive soils and the Nordlund method for cohesionless soils (FHWA method).

4. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 3 and Ref. 7 at the centerline of the Abutment 1, for soil set 4 which correspond to the soil set used in FB-Multipier model for Pile 8. The soil properties used in the downdrag analysis are taken from Ref. 5 and are based on correlations to field test and laboratory test data.

Base of Abutment 1 elevation =	101.5	feet	(Ref. 1, Sheet B20)
Water table elevation =	96.0	feet	(Ref. 7)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	Beta (β)[1]
	Top	Bottom	Top	Bottom				
Crushed Stone	101.5	100.5	0.0	1.0	105	40	-	0.58
Granular Borrow	100.5	98.1	1.0	3.4	125	32	-	0.30
Silt	98.1	96.0	3.4	5.5	125	30	-	0.33
Stiff Clay	96.0	83.1	5.5	18.4	59.6	-	1340	0.35
Medium Clay	83.1	79.2	18.4	22.3	58.6	-	888	0.27
Glacial Till	79.2	73.7	22.3	27.8	67.6	35	-	0.35
Bedrock	73.7	-	27.8	-	95.6	50	-	-

Note: [1]Beta values taken from Ref. 10, Table 7-9.

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	28.8	ft	(Ref. 8)
Pile section box perimeter =	57.0	in	(using pile flange width and depth from Ref. 6)
Pile section tip area =	26.1	in^2	(Ref. 5)
Pile nominal weight =	0.089	kip/ft	(Ref. 5)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 6 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 2 and shown in Attachment 1 the settlement beneath Abutment 1 after 75 years of construction for the maximum settlement point is estimated to be greater than 0.4 inches down to an elevation of 83.75 feet (depth of 18.0 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 18 feet of the pile below the base of the abutment.

Per Ref. 6 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use the Beta method to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 6, Tables 3.4.1-1 and 3.4.1-2)

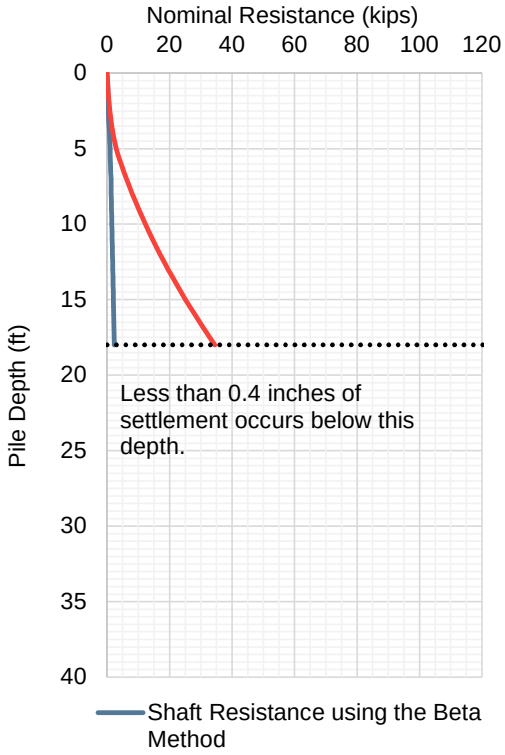
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	137.82	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type	Shaft Resistance using the Beta Method	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	137.8
1	Sand	0.3	0.3	138.2
2	Sand	0.3	0.6	138.6
3	Sand	0.5	1.2	139.3
4	Sand	0.7	1.9	140.1
5	Sand	0.9	2.9	141.2
6	Clay	1.1	4.5	142.8
7	Clay	1.2	6.2	144.6
8	Clay	1.3	8.1	146.6
9	Clay	1.4	10.1	148.7
10	Clay	1.5	12.3	151.0
11	Clay	1.6	14.6	153.4
12	Clay	1.7	17.0	155.9
13	Clay	1.8	19.6	158.5
14	Clay	1.9	22.3	161.3
15	Clay	2.0	25.1	164.3
16	Clay	2.1	28.1	167.3
17	Clay	2.2	31.2	170.5
18	Clay	2.3	34.5	173.9
19	Clay	2.4		139.5
20	Clay	2.5		139.6
21	Clay	2.0		139.7
22	Clay	2.1		139.8
23	Sand	2.2		139.9
24	Sand	2.9		140.0
25	Sand	3.0		140.0
26	Sand	3.1		140.1
27	Sand	3.3		140.2
28	Sand	3.4		140.3



*Strength I, including Substructure Loading, Downdrag, and Pile Weight

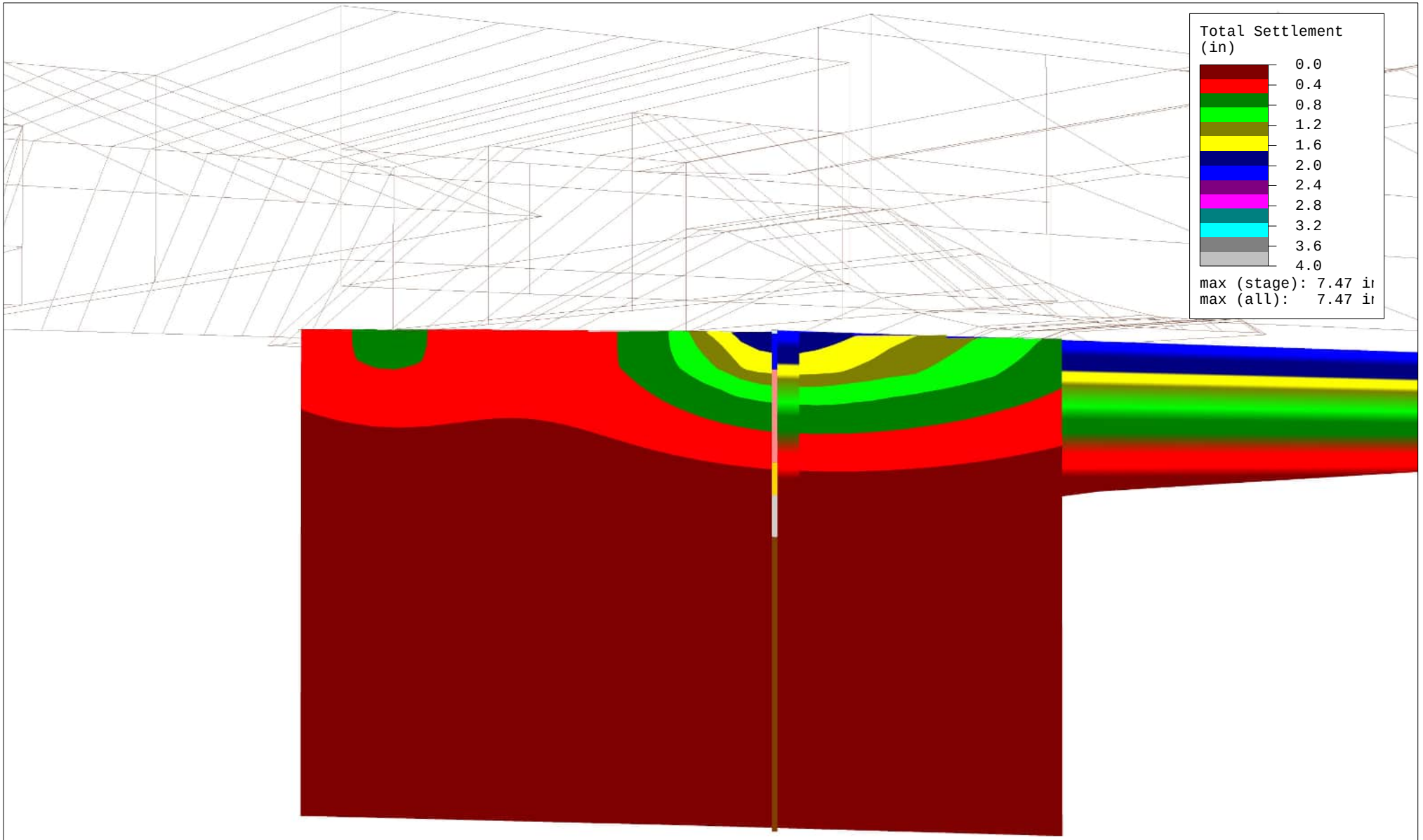
Strength I factored downdrag load = 34.5 kips per pile

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 1 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CONCLUSIONS

Based on the analysis of settlement for the proposed Abutment 1 of Bridge #6787 at the location of maximum settlement, downdrag is estimated to develop along the upper 18 feet of pile 8 below the base of the Abutment 1. A factored downdrag load of 34.5 kips per pile for the Strength I limit state was calculated.



Project

MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004

Analysis Description

Total Settlement along the proposed Abutment 1 after 75 years of construction

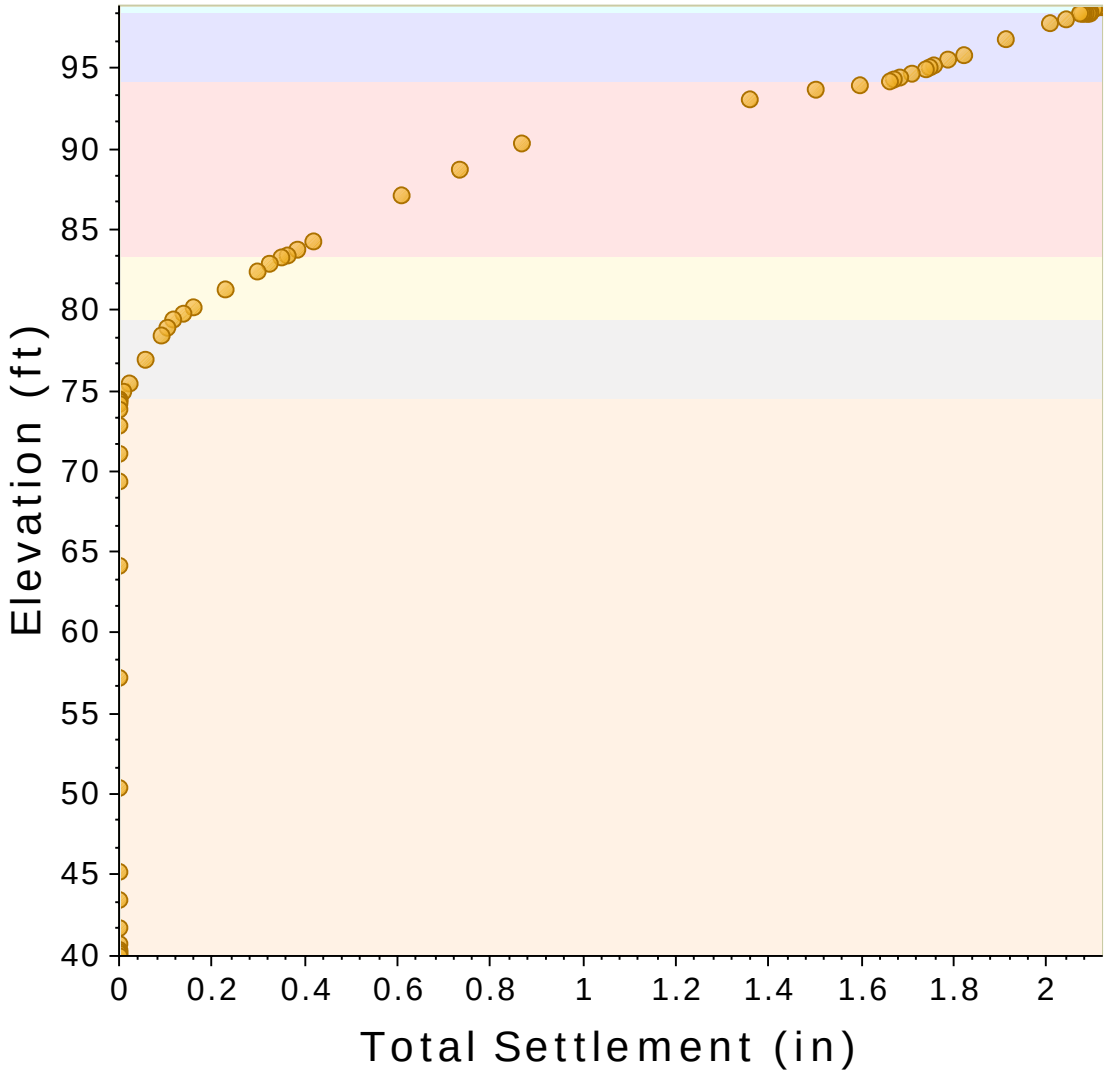
Drawn By	Checked By	Reviewed By	Scale
LMP	DEB		As shown

Date	File Name
5/30/2025	Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z

SETTLE3 5.025

Figure A.1

Total Settlement vs. Elevation



Max Settlement Abutment 1 (Stage 10 = 75 y)

Reference Stage: None

wsp	*Project* MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
	Analysis Description Total Settlement at the maximum settlement location on the proposed Abutment 1 after 75 years of construction			
	Drawn By LMP	Checked By DEB	Reviewed By	Scale As shown
SETTLE3 5.025	Date 5/30/2025	File Name Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z		Figure A.2

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 1 - HP14x89 - Resistance	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine the factored pile resistance of the proposed HP 14x89 piles at Abutment 1 based on the driving criteria outline in FHWA GEC 12 and the MaineDOT Standard Specification March 2020.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Abutment 1 H-Pile Analysis, dated 4/3/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
9. Federal Highway Administration (FHWA), FHWA-NHI-126-009 - Geotechnical Engineering Circular 12, Design and Construction of Driven Piles Vol II, July 2016
10. WSP boring logs.

ASSUMPTIONS

1. Pile tip elevations are based on an interpretation of the as-drilled top of bedrock elevations (Ref. 10).

2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 1 (soil set 1).

3. As per FHWA GEC 12 Section 14.2.6.C-8 (Ref. 9), absolute refusal during pile driving is defined as 20 blows for one inch or less or 10 blows for a half inch can be used to reduce the risk of pile damage. Driving stresses shall not exceed 90% of the pile yield strength per Maine Department of Transportation 2020 Standard Specifications Section 501 Foundation (Ref. 4).

4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

5. This analysis looks at resistance upon reaching the tip elevation in bedrock and does not consider the resistance during driving. A resistance distribution of 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.

6. A φdyn of 0.65 using (Ref. 8 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 1 - HP14x89 (Resistance)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2 = 0.18 ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in = 4.8 ft	
Pile head elevation =	103.0	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	76.3	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	26.7	ft	(Ref. 3, Sheet 13 of 53)

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.15	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation =	103.0	ft	
Groundwater elevation =	96.0	ft = 7.0	ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction (ksf)	End Bearing (ksf)	Soil Type
	Top	Bottom	Top	Bottom						
Crushed Stone	101.5	100.5	1.5	2.5	105	40	-	-	-	Sand
Stiff Clay	100.5	90.2	2.5	12.8	122	-	1340	-	-	Clay
Medium Clay	90.2	83.8	12.8	19.2	121	-	888	-	-	Clay
Glacial Till	83.8	77.3	19.2	25.7	130	35	-	-	-	Sand
Competent Bedrock	77.3	-	25.7	-	158	-	-	20.0	422.0	Rock Other

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 1 - HP14x89 (Resistance)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outline in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
Reaching tip elevation	10%	90%	APE D25-42	45,892	Fuel Setting 2 (74%)	707	19.9	35.70	71%

3. Determine nominal and factored toe resistance.

Nominal toe resistance = 707 kips (Intact section from GRLWEAP)

φ_{dyn} = 0.65 (Ref. 9, Table 10.5.5.2.3-1)

Factored toe resistance = 460 kips

CONCLUSIONS

This analysis indicates that an HP14x89 pile driven with an APE D25-42 hammer operated at 74% of the combustion pressure is expected to develop a nominal bearing resistance of 707 kips at WB Abutment 1 while limiting driving stresses to a maximum of 45 ksi. Blow counts were limited to 10 blows per 1/2" of penetration, in accordance with FHWA GEC 12 (Ref. 9) and Section 501 of the Maine Department of Transportation 2020 Standard Specifications for Construction (Ref. 4). The resulting factored toe resistance from this WEAP analysis is 460 kips.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

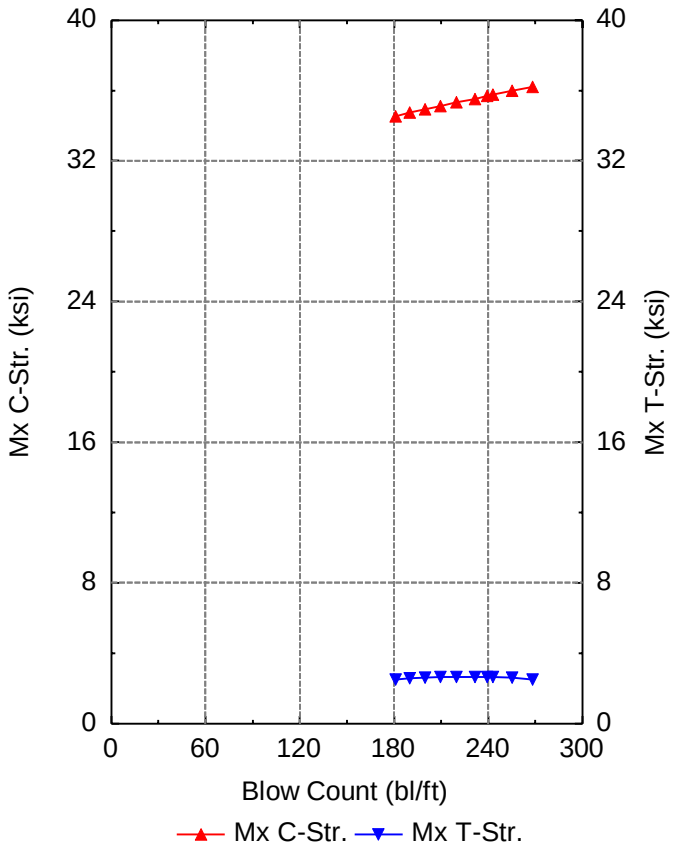
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

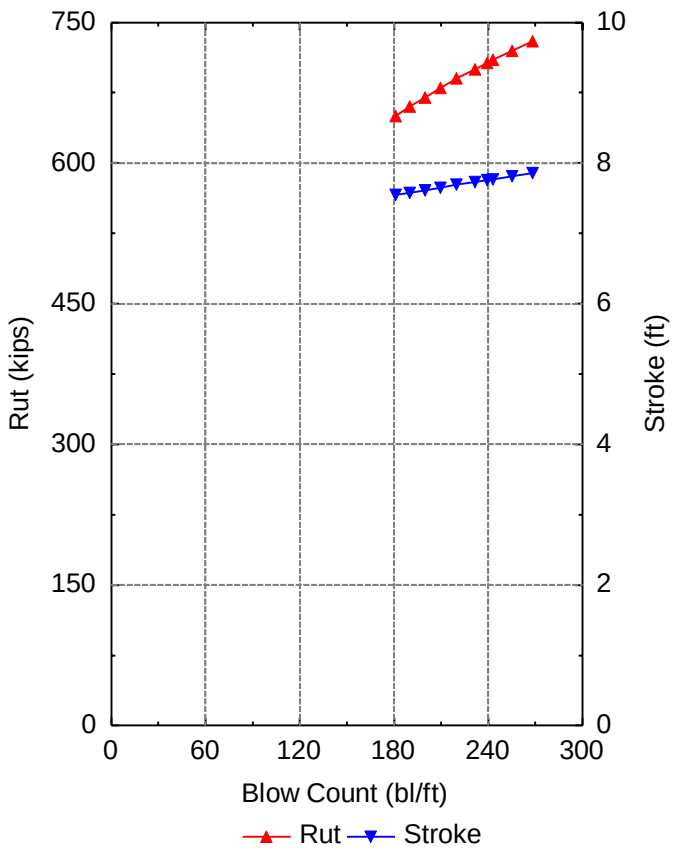
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

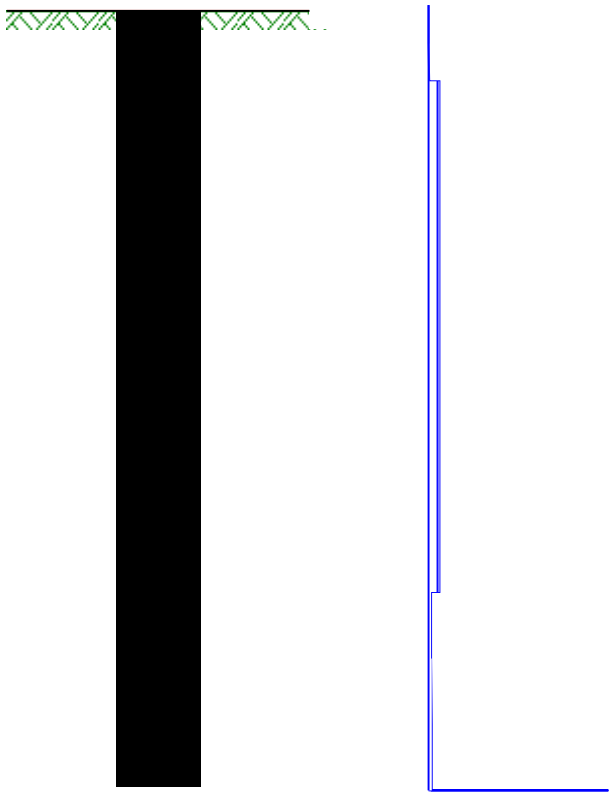
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.150	s/ft
Toe Damping	0.150	s/ft
Pile Length	25.70	ft
Pile Penetration	25.70	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model

Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
650.0	34.53	32.50	2.50	181.0	7.55	13.61	D 25-42	21.9
660.0	34.73	32.77	2.55	189.8	7.58	13.64	D 25-42	22.0
670.0	34.91	33.05	2.60	199.6	7.62	13.67	D 25-42	22.1
680.0	35.10	33.34	2.64	209.7	7.65	13.72	D 25-42	22.1
690.0	35.34	33.60	2.65	219.8	7.69	13.79	D 25-42	22.2
700.0	35.50	33.88	2.65	231.5	7.73	13.83	D 25-42	22.3
707.0	35.70	34.08	2.66	239.2	7.75	13.90	D 25-42	22.4
710.0	35.78	34.15	2.63	242.9	7.77	13.92	D 25-42	22.4
720.0	35.99	34.43	2.60	255.3	7.81	14.00	D 25-42	22.6
730.0	36.21	34.73	2.50	268.3	7.86	14.09	D 25-42	22.7

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at Bridge 6787 WB Abutment 1 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed H-piles at WB Abutment 1 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Abutment 1 H-Pile Analysis, dated 4/3/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
9. Federal Highway Administration (FHWA), FHWA-NHI-126-009 - Geotechnical Engineering Circular 12, Design and Construction of Driven Piles Vol II, July 2016
10. WSP boring logs.

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on twenty-four (24) HP14x89 Grade 50 piles with a center-to-center pile spacing of 8.5 feet in the x-direction and 6 feet in the y-direction (Ref. 3). The piles are to be driven to bedrock.

2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 1 (soil set 1).

3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.

4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at Bridge 6787 WB Abutment 1 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2 = 0.18 ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = $(d \times 2) + (b_f \times 2)$ =	57.0	in = 4.8 ft	
Pile head elevation =	103.0	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	76.3	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	26.7	ft	(Ref. 3, Sheet 13 of 53)

Ultimate Capacities:

Axial load on pile, P_u =	237	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	364	kips	

For soil above bedrock while driving pile to bedrock:

Nominal shaft resistance, R_s =	114	kips	(Ref. 6)
Maximum nominal toe resistance of soil, $R_{p\text{-}soil}$ =	57	kips	(Ref. 6)
Total soil resistance =	171	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.15	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at Bridge 6787 WB Abutment 1 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Shaft and Toe Resistance (during driving):

Nominal shaft resistance, R_s = 114 kips = 67 % of R_n (from APile analysis, Attachment 3)
Nominal toe resistance, $R_{p\text{-}rock}$ = 57 kips = 33 % of R_n (from APile analysis, Attachment 3)
Nominal resistance, R_n = 171 kips

Assume that once the pile has encountered bedrock, the resistance distribution will switch to 33% shaft resistance and 67% toe resistance. A second analysis will be performed to model these conditions.

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation = 103.0 ft
Groundwater elevation = 96.0 ft = 7.0 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Crushed Stone	101.5	100.5	1.5	2.5	105	40	-	-	-	Sand
Stiff Clay	100.5	90.2	2.5	12.8	122	-	1340	-	-	Clay
Medium Clay	90.2	83.8	12.8	19.2	121	-	888	-	-	Clay
Glacial Till	83.8	77.3	19.2	25.7	130	35	-	-	-	Sand
Competent Bedrock	77.3	-	25.7	-	158	-	-	20.0	422.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	67%	33%	APE D25-42	45,892	Fuel Setting 2 (74%)	364	6.7	23.7	47%
Reaching tip elevation	33%	67%	APE D25-42	45,892	Fuel Setting 2 (74%)	364	5.8	23.2	46%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 2 (74% of the combustion pressure) while driving through soil and upon reaching bedrock should be able to overcome a soil resistance of 171 kips in order to drive the piles at WB Abutment 1 to bedrock and a nominal resistance of 364 kips while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

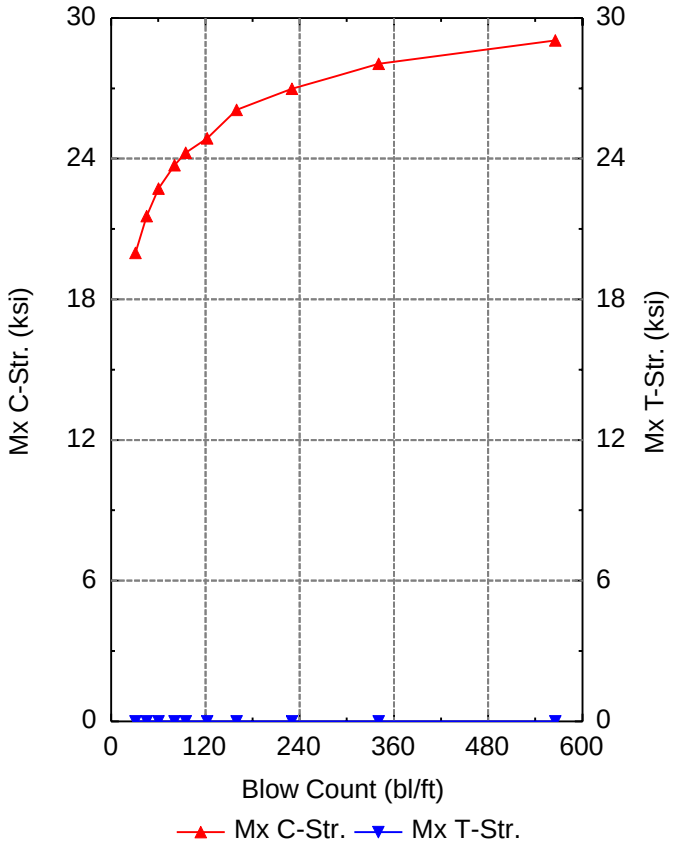
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

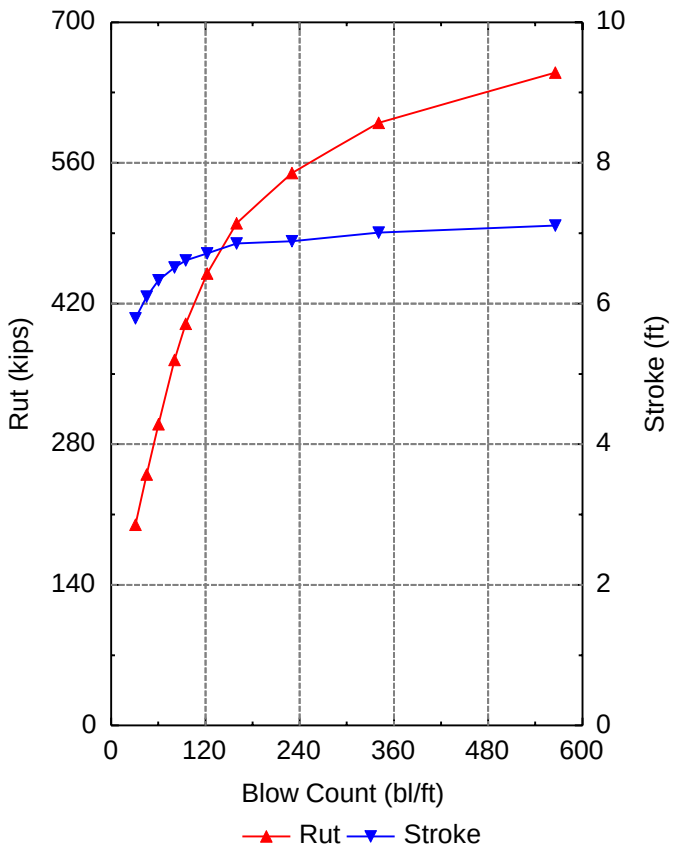
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

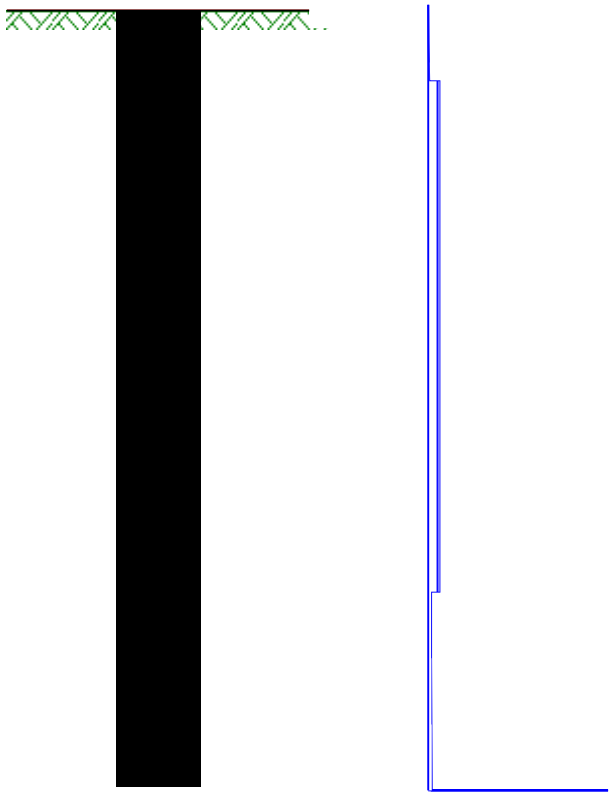
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.150	s/ft
Toe Damping	0.150	s/ft
Pile Length	25.70	ft
Pile Penetration	25.70	ft
Pile Top Area	26.10	in²
RSA	No	



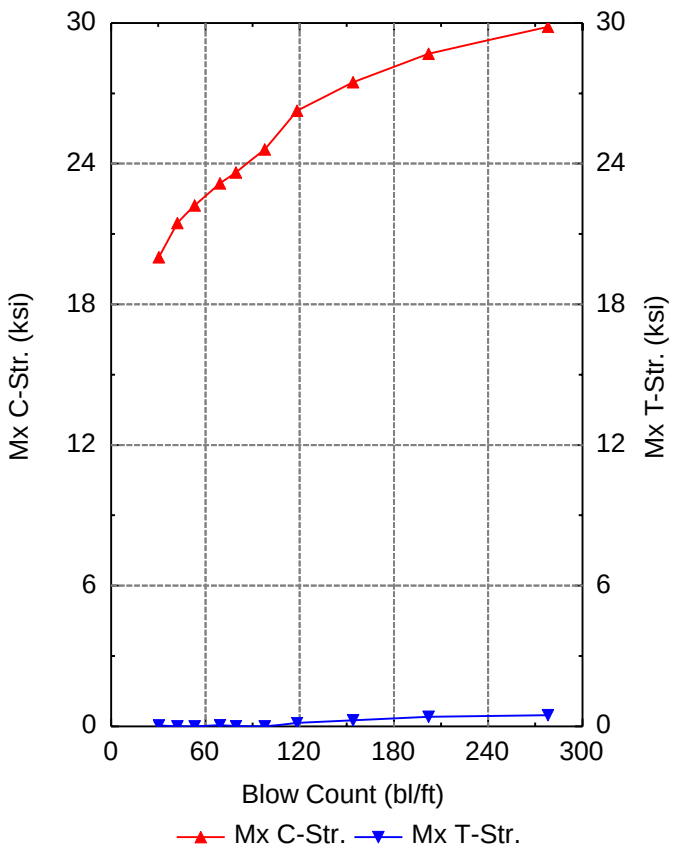


Pile Model

Shaft=67%
(Prop., SA)

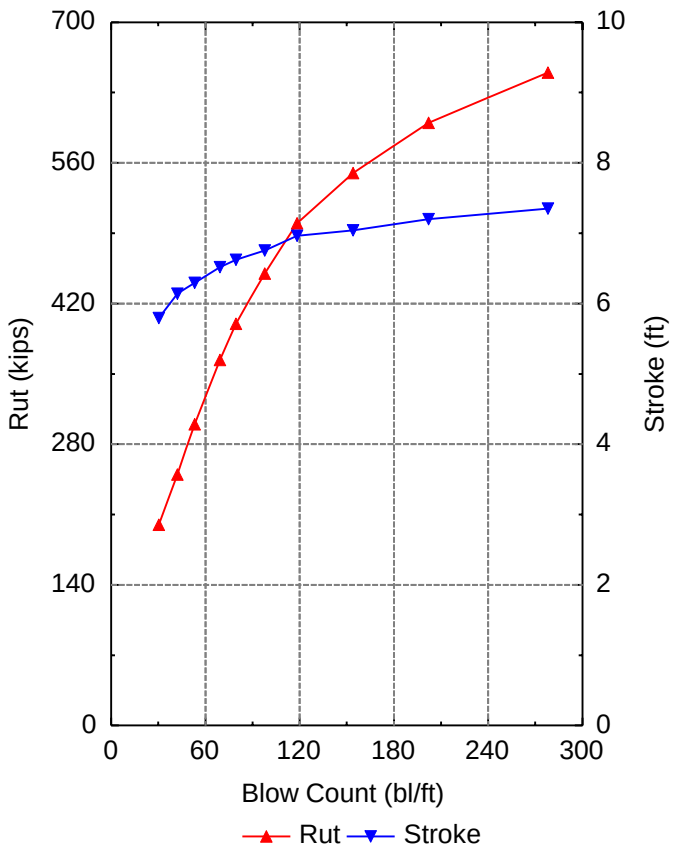
Bearing Graph Summary — APE D 25-42

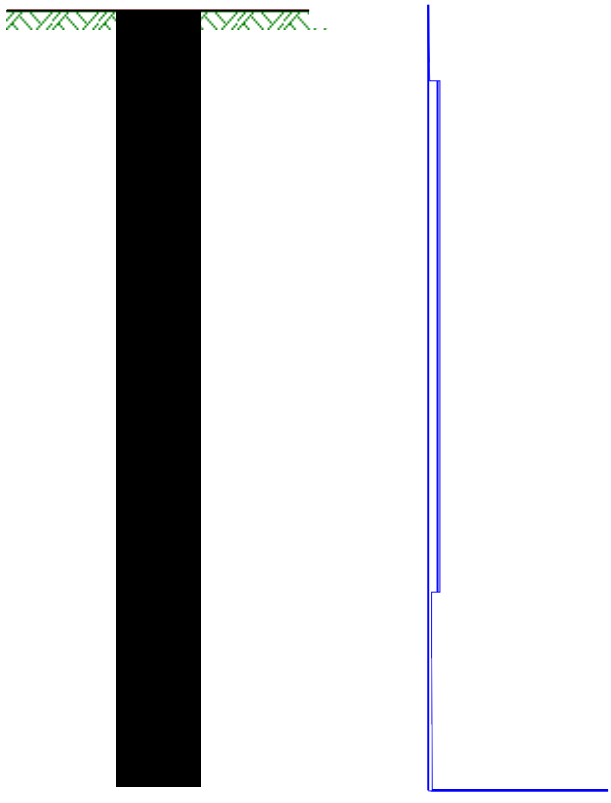
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
200.0	19.97	19.97	0.00	31.0	5.79	12.02	D 25-42	19.4
250.0	21.55	21.55	0.00	45.3	6.10	11.63	D 25-42	18.8
300.0	22.72	22.72	0.00	60.4	6.33	11.55	D 25-42	18.6
364.0	23.71	23.71	0.00	80.8	6.51	11.21	D 25-42	18.1
400.0	24.25	24.25	0.00	94.8	6.61	11.22	D 25-42	18.1
450.0	24.86	24.86	0.00	122.3	6.71	11.23	D 25-42	18.1
500.0	26.08	26.08	0.00	159.6	6.86	11.38	D 25-42	18.3
550.0	26.98	26.98	0.00	230.1	6.89	11.22	D 25-42	18.1
600.0	28.04	28.04	0.00	340.5	7.01	11.31	D 25-42	18.2
650.0	29.05	29.05	0.00	565.5	7.11	11.37	D 25-42	18.3



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.150	s/ft
Toe Damping	0.150	s/ft
Pile Length	25.70	ft
Pile Penetration	25.70	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model

Shaft=33%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
200.0	20.00	20.00	0.04	30.4	5.79	12.25	D 25-42	19.7
250.0	21.47	21.47	0.00	42.1	6.14	12.09	D 25-42	19.5
300.0	22.22	22.22	0.00	53.1	6.30	11.82	D 25-42	19.1
364.0	23.16	23.16	0.05	69.3	6.52	11.68	D 25-42	18.8
400.0	23.63	23.63	0.01	79.4	6.62	11.75	D 25-42	19.0
450.0	24.61	24.18	0.00	97.7	6.75	11.79	D 25-42	19.0
500.0	26.25	25.00	0.14	118.4	6.96	12.12	D 25-42	19.5
550.0	27.47	26.05	0.25	154.1	7.04	12.03	D 25-42	19.4
600.0	28.69	27.26	0.40	202.0	7.20	12.18	D 25-42	19.6
650.0	29.83	28.36	0.48	278.1	7.35	12.42	D 25-42	20.0

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

OBJECTIVE

Determine if proposed HP14x89 piles provide adequate support for the WB Abutment 2 based on the design load case 2 (controlling) (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Abutment 2 (Pile 12), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday December 2nd, 2024 with subject line "Bangor Hogan Road New WB Bridge - Abutment Bearing Loads for WSP Pile Foundation Evaluation and Layout/Size Recommendation".
15. WSP calculation package titled Lateral Earth Pressure at the proposed West Bound Bridge Abutments, dated 2/27/2025.
16. WSP calculation package titled Pile Drivability at Abutment 2 - HP14x89, dated 4/15/2025.
17. WSP calulation package titled Hogan Road - WB Downdrag, dated 9/15/2025.

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Case 1 output
4. FB-Multipier Case 2 output
5. Abutment loads

CALCULATIONS

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ASSUMPTIONS

1. The proposed Abutment 2 is assumed to be supported by twenty-four (24) HP 14x89 Grade 50 piles with center spacing of 8.5 feet in the x-direction and 6 feet in the y-direction shown in Ref. 5.
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) were applied at the bearings on the abutment stem wall.
4. All piles are assumed to have reinforced pile tips (Ref. 8).
5. All piles are plumb.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. A Downdrag load of 10.3 kips (factored) develops along the length of the maximum axial loaded pile and is included in this anlaysis. (Ref. 17).
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added to the toe of the Abutment 2 pile cap to account for the granular borrow and crushed stone slope on the toe (Ref. 5), and a vertical earth load EV was added to the heal of the Abutment 2 pile cap to account for the backfill and traffic live load surcharge (Ref. 15). The EV loads on the toe and heal are added as point loads spaced evenly along the pile cap between pile locations.
10. A horizontal earth load EH of 4,041 plf (Ref. 15) was added to the pile cap on the back side of the abutment within the area of the stem wall.
11. A φdyn of 0.65 is used to determine the factored pile resistance, based on the MaineDOT standard Specification, PDA testing will be performed on 2% of the production piles.
12. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

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CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	13.64	in	
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	14.5	in	
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Fillet area =	0.3	in^2		0.3	in^2	
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	19.3	in^2	
Section area =	0.18	ft^2		0.13	ft^2	
Moment of inertia about the x-axis, I_x =				653	in^4	
Moment of inertia about the y-axis, I_y =				233	in^4	
Radius of gyration about the x-axis, r_x =				5.82	in	
Radius of gyration about the y-axis, r_y =				3.47	in	
Elastic section modulus about the x-axis, S_x =				95.8	in^3	
Elastic section modulus about the y-axis, S_y =				32.1	in^3	
Plastic section modulus about the x-axis, Z_x =				105.7	in^3	
Plastic section modulus about the y-axis, Z_y =				48.8	in^3	

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	102.0	ft	Sheet B23 (Ref. 5)
Pile cap thickness =	3	ft	Sheet B23 (Ref. 5)
Pile cap midplane elevation =	100.5	ft	Sheet B23 (Ref. 5)
elevation =	100.5	ft	
Base of pile cap elevation =	99.0	ft	Sheet B23 (Ref. 5)
Pile length =	33.6	ft	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Crushed Stone	1	Cohesionless	99	98	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
FGA	2	Cohesionless	98	81	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	3	Cohesive	81.0	78.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	4	Cohesive	78.0	67.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	67.9	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Sta. 113+13 in Profile F-F' (Sheet X Interpretive Subsurface Cross Section G-G').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 12)					
Maximum axial force in the piles (includes 10.3 kip downdrag load (Ref. 17), P_u =	229.2	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	144.1	kip-ft	=	1728.8	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	6.9	kip-ft	=	83.3	in-kips	
Maximum shear force in the piles, V_u =	26.0	kips				
Depth below pile head to fixity =	21.0	ft	=	Elev.	79.50	ft
Depth below pile cap to fixity =	19.5	ft	=	Elev.	79.50	ft

wsp

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction and a wave equation analysis.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/φ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for φ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for φ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for φ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Crushed Stone	FGA	
Friction angle, φ_f =	40	35.5	degrees (Ref. 6)
δ =	31	28	degrees (based on δ/φ_f from above)
K_δ, interpolated =	1.87	1.29	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.87	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.053	0.258	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.044	0.137	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.340	1.340	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	0.95	0.95	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.179	1.179	0.844	0.844	ksf

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 56.4 in = 4.7 ft (from Part A)

	Crushed Stone	FGA	Stiff Clay	Medium Clay		
Thickness =	1	17	3.0	10.1	ft	(from stratigraphy in Part A)
A_s =	4.7	79.8	14.1	47.4	ft^2	
R_s per layer =	0.2	10.9	16.6	40.0	kips	

Total R_s = 67.8 kips

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD $\leq$ 70%
$q_p = 0.80 q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 908 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 299.6 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

Pile toe area, A_p = 0.13 ft^2 (from Part A)

Total R_p = 40.2 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 108 kips (bedrock)

CALCULATIONS

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$R_r = \varphi_{stat} R_n$ (Ref. 7, Eq. 10.7.3.8.6a-1)

where:
(Ref. 7 Tbl. 10.5.5.2.3-1)

φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method
φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method
φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Crushed Stone	FGA	Stiff Clay	Medium Clay	Pile Tip (Bedrock)	
R_s or R_p =	0.2	10.9	16.6	40.0	40.2	kips
φ_{stat} =	0.45	0.45	0.35	0.35	0.45	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	4.9	5.8	14.0	18.1	

Total R_r = 43 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obained from GRWEAP at 90% of the steel yield strength (Ref. 16):

R_n = 697 kips (Ref. 16)

φ_{dyn} = 0.65 (Assumption 11)

R_f = 453 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 229 kips (from Part B)

R_r = 453 kips > P_u = 229 kips OK

CALCULATIONS

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D. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, *l* =	233.95	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 44 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 9)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$$\frac{b}{t} = 15.98 > 13.49 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1)

Equivalent nominal yield resistance, P_o = 966 kips

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	0.65		(Ref. 7, Table C4.6.2.5-1; fixed rotation and translation condition at pile head due to embedment into pile cap)
Unbraced length, *l* =	233.95	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	2,886	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877 P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.33 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 840 kips

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

For compression member cross sections containing and slender elements, the nominal compressive resistance, P_n. Shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	840	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	43.5	ksi	

A_{eff} = $A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	43.5	ksi	

$\lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ = 14.5

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

c_1 = 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
c_2 = 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
c_3 = 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

F_{el} = 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

b_e = 6.90 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff}:

A_{eff} = 18.64 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n = 810.3 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
φ_c = 0.50 (Ref. 7, Article 6.5.4.2))
P_r = 405 kips

$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
φ_c = 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$ = 567 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 229 kips (from Part B)

$P_{r,\ axial}$ = 405 kips > P_u = 229 kips OK

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

E. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	1729	in-kips (from Part B)
M_{uy} =	83	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	1729	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	83	in-kips	OK

wsp

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
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F. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.4 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.75 < 1.0 OK

G. Determine the nominal and factored structural resistance of a single pile in shear.

$V_n = CV_p = V_{cr}$

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_p = 0.58F_y d_w t_w$ (Ref. 7, 6.10.9.2-2))

$V_n = CV_p = V_{cr}$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

Date:	9/15/2025	**Made by:**	ATM
Project No.:	31404817.004	**Checked by:**	KAR
Subject:	New Bridge Abutment 2 H-Pile Analysis	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 26.0 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 26.0 kips OK

H. Determine nominal pile driving resistance.

$P_u =$ 229 kips (from Part B)

$\varphi_{dyn} =$ 0.65 (Ref. 3, Table 7-2)

for driving criteria established by dynamic testing of at least two piles per site condition and no less than 2% of production piles

Nominal pile driving resistance, $R_{ndr} =$ 353 kips

Confirm R_{ndr} does not exceed P_n: R_{ndr} P_n (from Part D)

353 kips < 840 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, an estimated pile length of 33.6 feet, and downdrag loading a maximum axial force of 229.2 kips, the corresponding strong axis (x-axis) moment of 1,729 in-kips (144.1 ft-kips) occurs in the proposed HP14x89 piles at Abutment 2 in Pile 12 of the proposed WB Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The analysis indicates that the maximum top of pile deflection at the service limit state is 0.37 inches in the y-direction. The WB Abutment 2 piles should be driven to a nominal resistance of 353 kips. This evaluation considers section loss for an additional 100 year design life. A summary table is presented below.

Pile 12 Summary Table		
Pile Length (ft)		34
Depth below Pile Head to Fixity (ft)		21
Geotechnical Axial Resistance (kips)	Nominal	697
	Factored	453
Structural Axial Resistance (kips)	Nominal	810
	Factored	405
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.75
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

Figure 1b: Profile View of Pile Cap (Transverse)

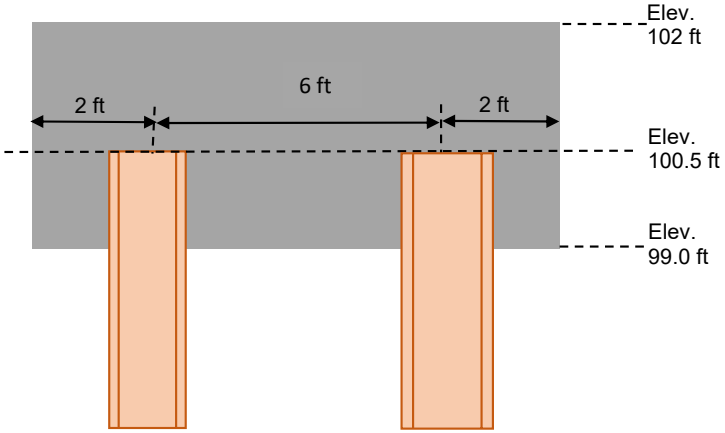


Figure 1c: 3D View of Pile Model

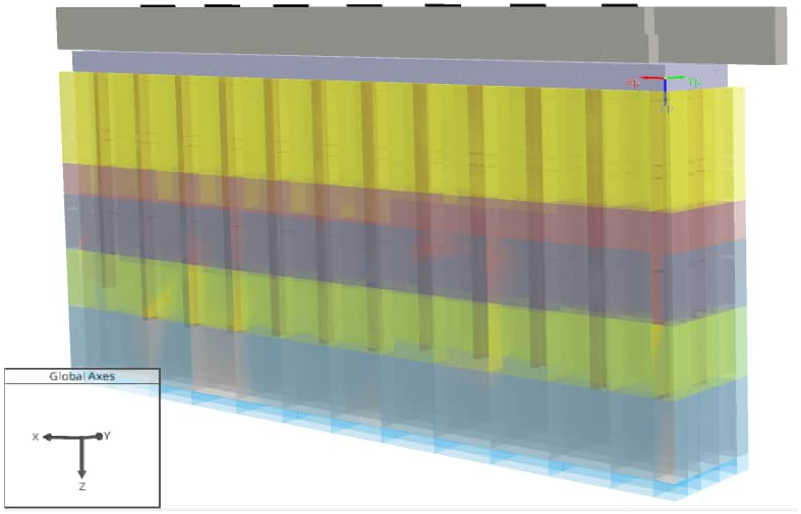
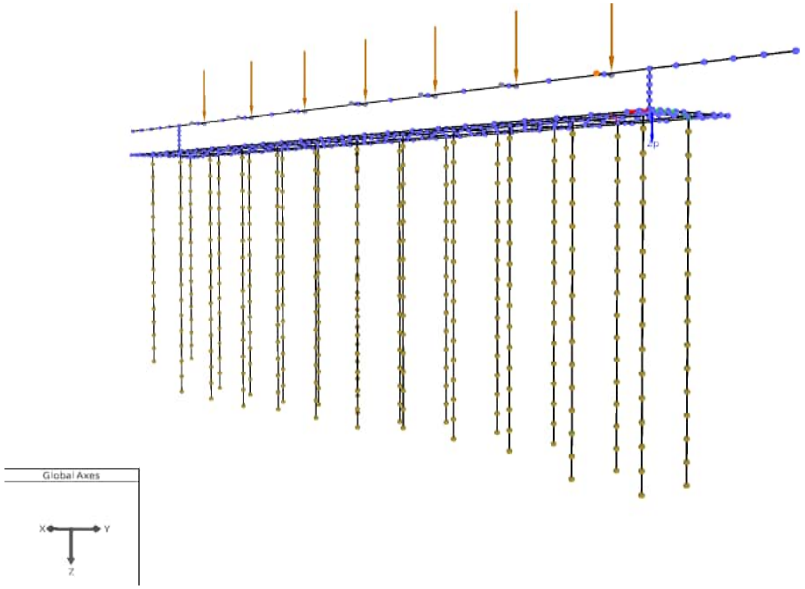


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Crushed Stone	1	Cohesionless	99.0	98.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	105	40	253	-		-	-	1.22		0.42	44		-	-	-
Granular Borrow	2	Cohesionless	98.0	90.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	219		-	-	-
Existing Fill	3	Cohesionless	90.7	89.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	34	127	-		-	-	0.65		0.35	512		-	-	-
Silt	4	Cohesionless	89.3	85.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	30	50	-		-	-	0.28		0.25	459		-	-	-
Stiff Clay	5	Cohesive	85.6	78.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medim Clay	6	Cohesive	78.8	72.9	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	7	Cohesionless	72.9	64.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	1378		-	-	-
Bedrock	8	Rock	64.4	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		3844	697	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on the WB Abutment 2 profile C-C' (Sheet 5).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

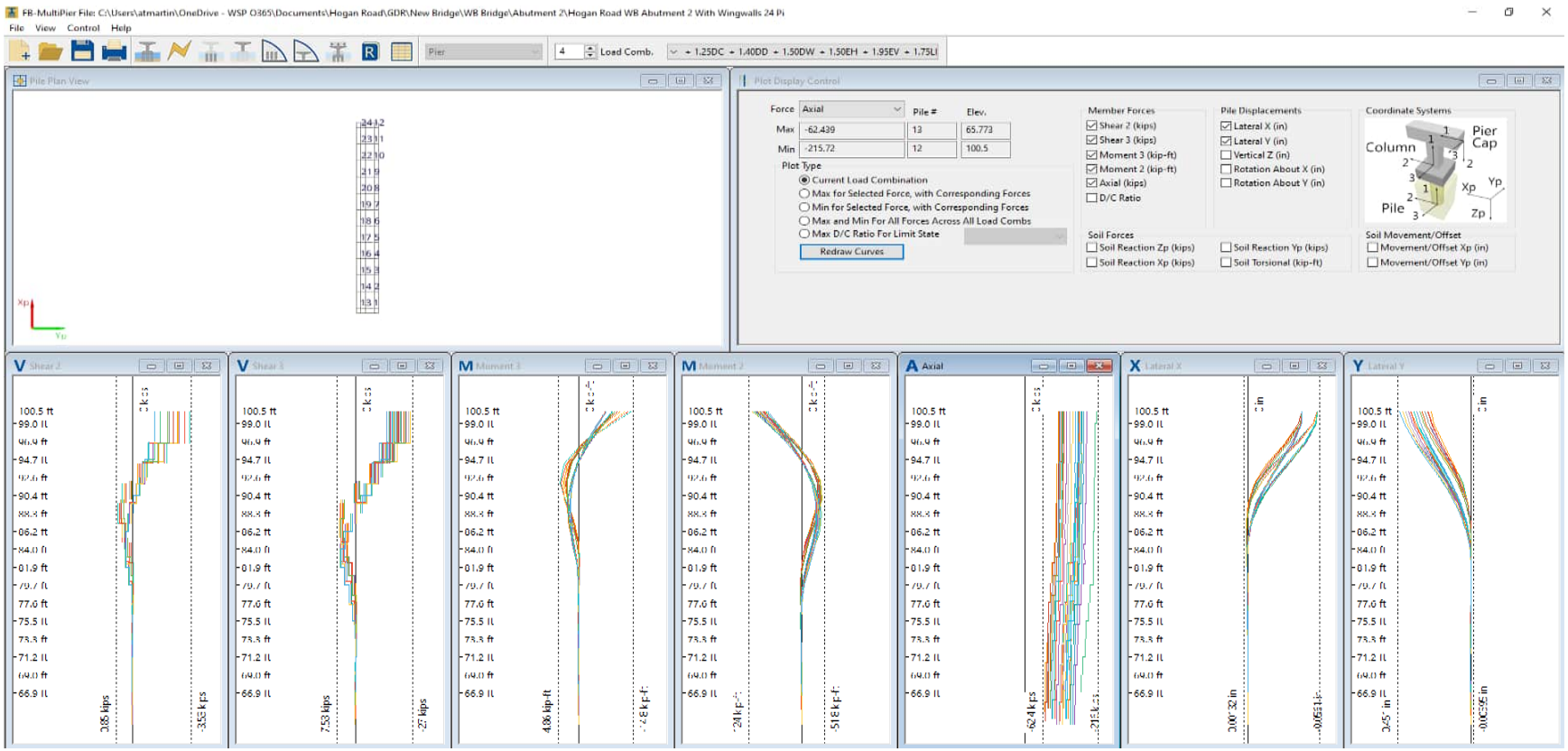
6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

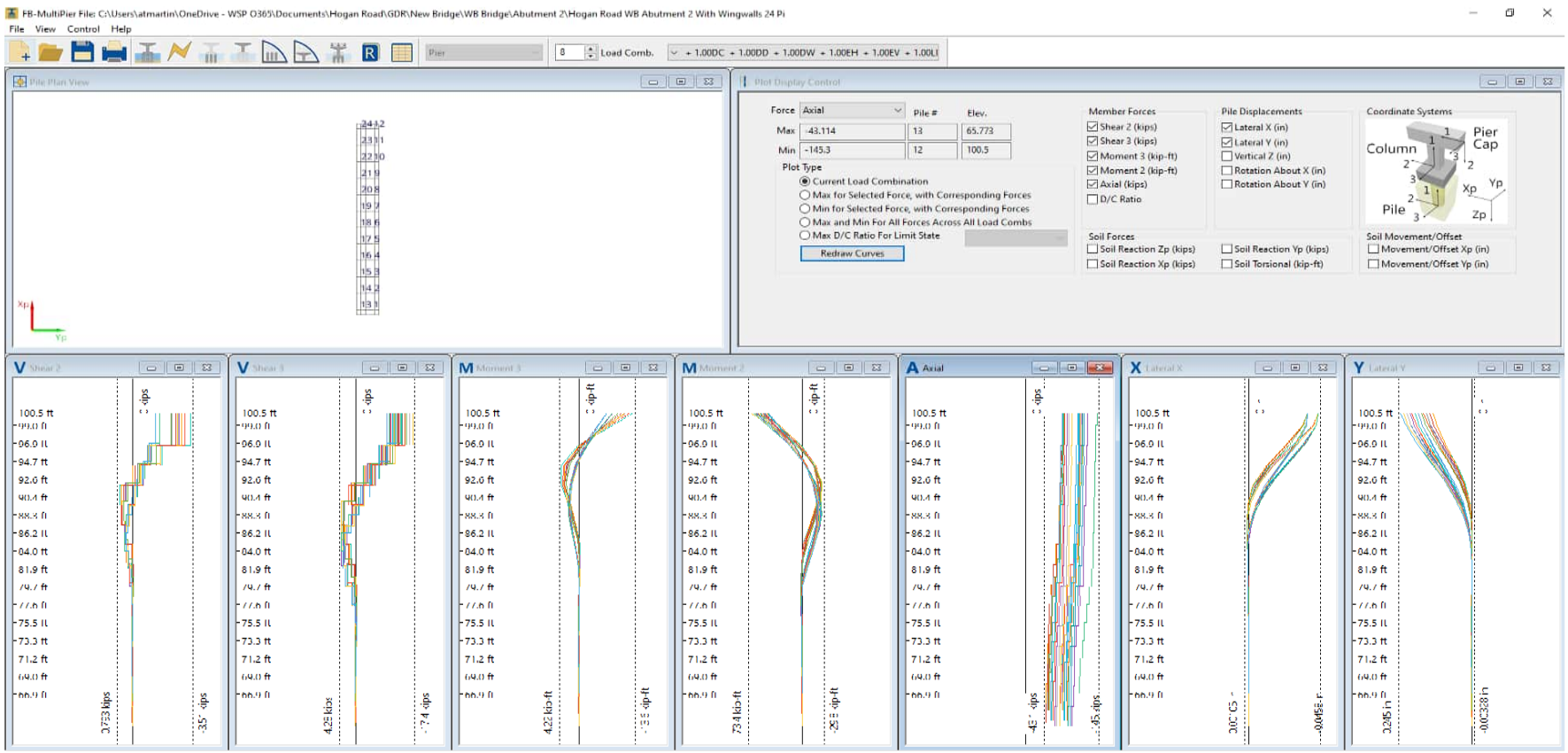
8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

Prepared: DEB
Checked: ATM
Reviewed:

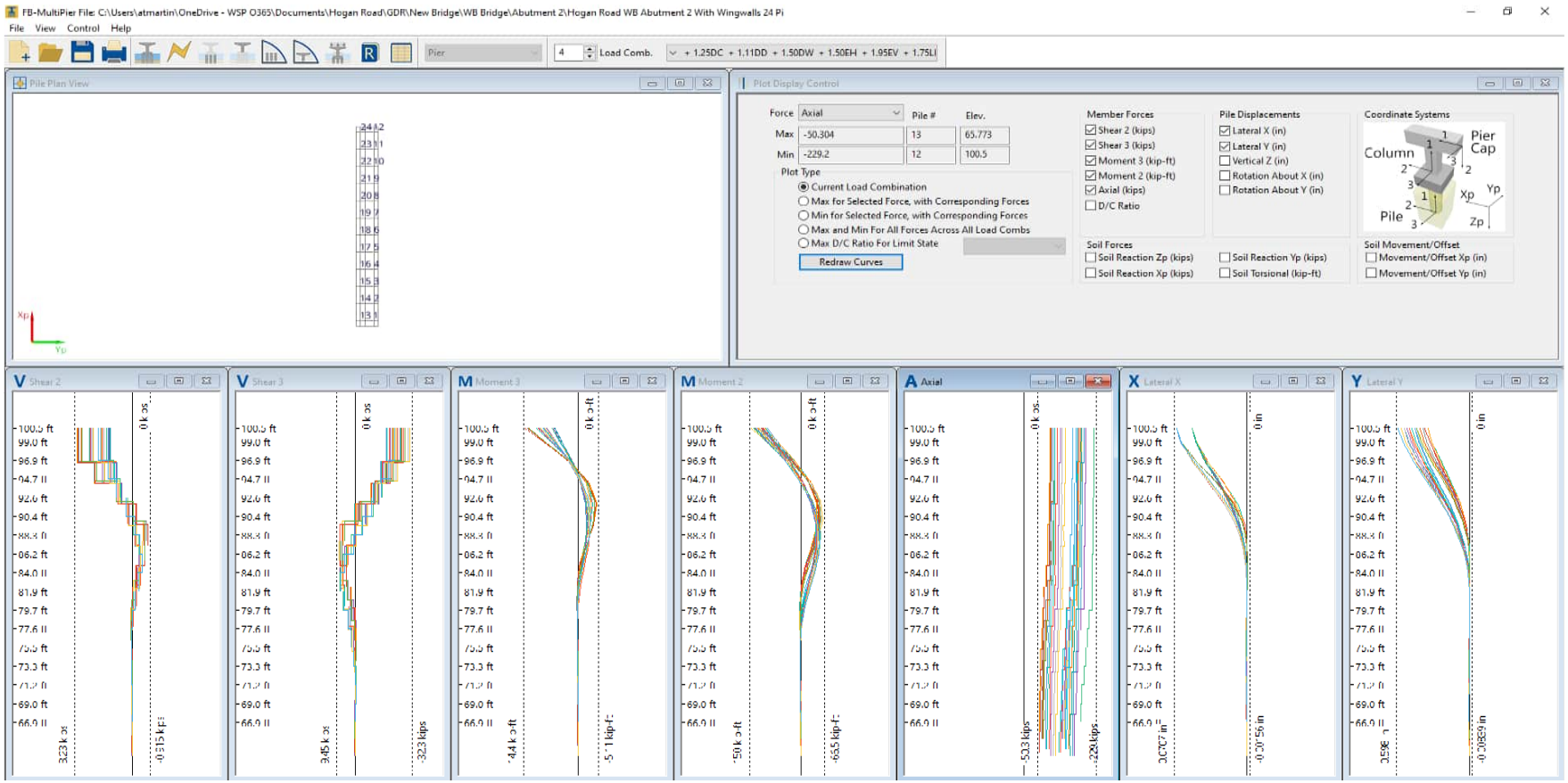
Case 1 Strength I



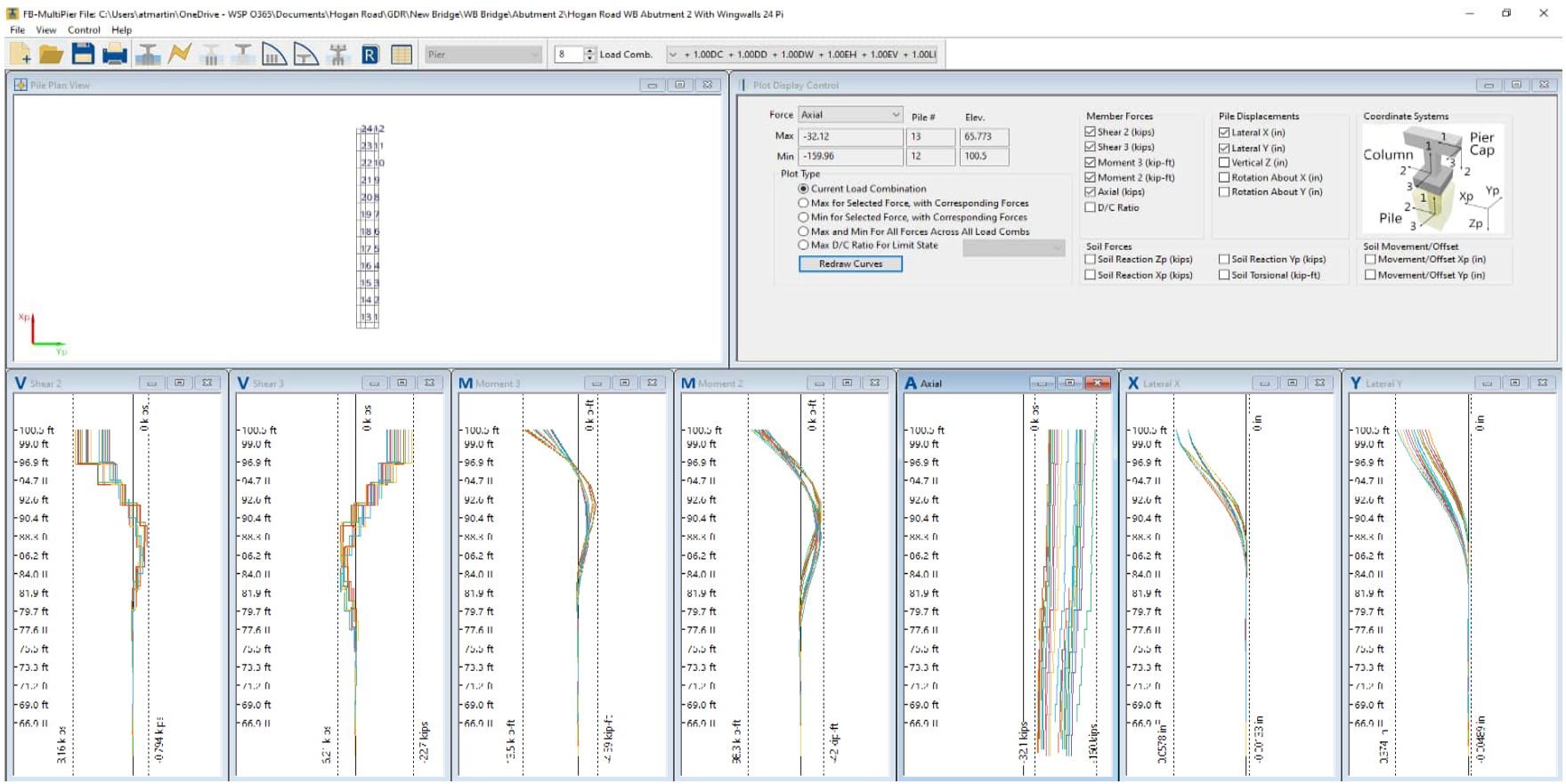
Case 1 Service I



Case 2 Strength I



Case 2 Service I



Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	11/12/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computations

Unfactored Bearing Loads, Abutment

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	G7
DC	80	80	80	80	80	80	80
DW	15	15	15	15	15	15	15
LL 1A	55	85	83	0	0	0	0
LL 1B	0	0	0	0	83	85	55
LL 2A	47	69	71	83	0	0	0
LL 2B	0	0	0	83	71	69	47
FR							

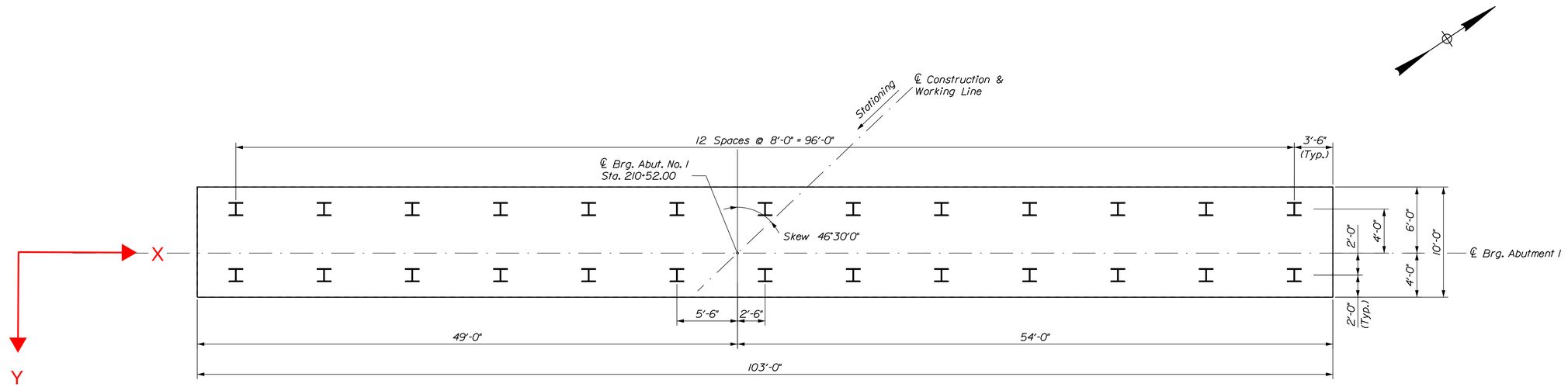
	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6	G7	G1	G2	G3	G4	G5	G6	G7
DC														
DW														
LL 1A														
LL 1B														
LL 2A														
LL 2B														
FR	7	7	7	7	7	7	7	7	7	7	7	7	7	7

Abutment Bearing Reactions

	Fz (kip), Vertical						
Case	G1	G2	G3	G4	G5	G6	G7
DC	80	80	80	80	80	80	80
DW	15	15	15	15	15	15	15
LL 1A	55	85	83	0	0	0	0
LL 1B	0	0	0	0	83	85	55
LL 2A	47	69	71	83	0	0	0
LL 2B	0	0	0	83	71	69	47
FR							

	Fy (kip)							Fx (kip)						
	G1	G2	G3	G4	G5	G6	G7	G1	G2	G3	G4	G5	G6	G7
DC														
DW														
LL 1A														
LL 1B														
LL 2A														
LL 2B														
FR	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7	-7

NOTE TO REVIEWER:
1) Pile design is not complete and subject to change.



FOOTING PLAN & PILE LAYOUT

REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	JAH	8/31/24
CHECKED-REVIEWED	YP	SMH	8/31/24
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

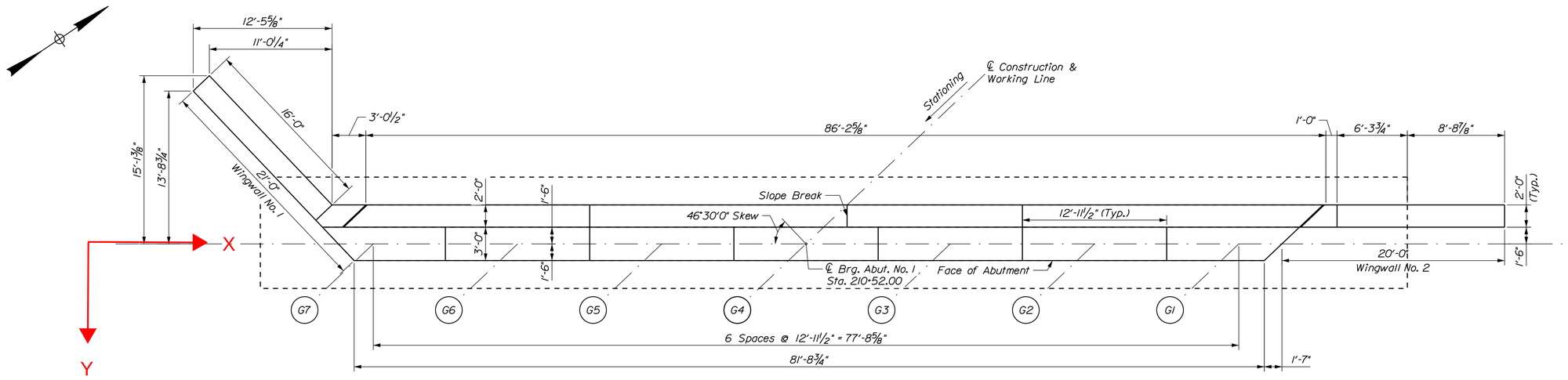
ABUTMENT NO.1
FOOTING PLAN

SHEET NUMBER

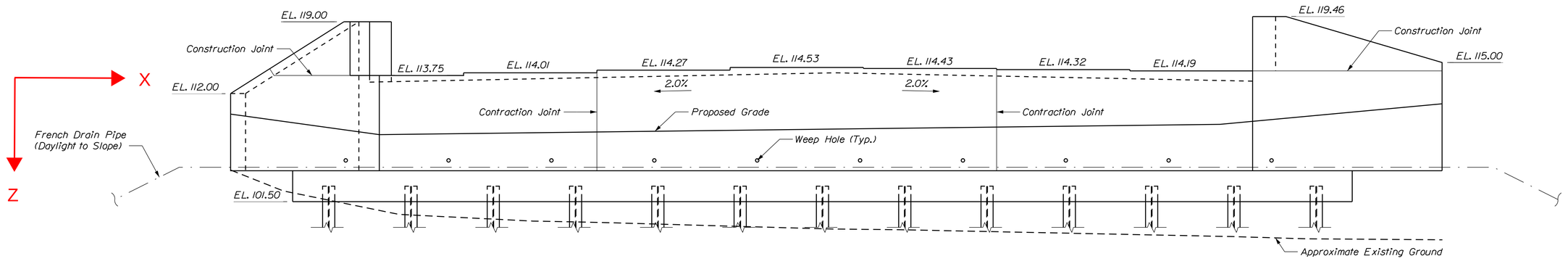
B19

OF B49

Filename: ...\B19_WB_Abutment_1_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



PLAN



ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

ABUTMENT NO. 1
PLAN & ELEVATION

SHEET NUMBER

B20

OF B49

Filename: ...\B20_WB_Abutment_1_Detail.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

CALCULATIONS

Date:	9/15/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the proposed HP 14x89 steel pile with the maximum loading (Pile 12) at Bridge #6787 Abutment 2 of the bridge carrying Hogan Road Westbound over I-95. This analysis uses estimated settlement for 75 years after construction.

METHOD

Use the Beta method (Ref. 6) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Maine Department of Transportation Bridge Program. 60% West Bound Bridge Plans, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Dated 8/30/2025.

2. WSP Settle3 West Bound model titled "Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z", dated 6/2025.

3. WSP Interpretive Subsurface Profile B- B', included as part of the Geotechnical Design Report.

4. WSP calculations of interpreted soil parameters based on field and laboratory data, named "Lab Field Data Summary Plots V4 31404817.000.xlsx".

5. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

6. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.

7.WSP FB-Multipier Soil properties for the proposed West Bound Bridge, named "New Bridge Abutment 2 FB-Multipier Soil Properties.xlsx", dated 5/20/2025.

8.WSP West Bound Abutment 2 Pile Analysis, named "Hogan Road New Bridge Abutment 2 H-Pile Analysis 24 Piles", dated 9/15/2025.

9. Loading provided by VHB in Computation package titled "Bangor DDI Hogan Rd WB Bearing Loads 11/12/2024"

10. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.

11. Westbound Bridge Plans provided by VB in Progress Prints titled "WB_Structures_Plot_09_11_2025smh.pdf", reeceived on 9/11/2025.

ATTACHMENTS

A.1. Profile of estimated settlement beneath Abutment 2 75 years after construction

A.2. Profile of estimated settlement beneath Abutment 2 75 years after construction at the location of the pile with maximum axial loading

ASSUMPTIONS

1. The proposed Abutment 2 is supported on twenty-four HP14x89 steel piles, based on Ref. 11. The downdrag is evaluated for the pile with maximum axial loading based on the proposed abutment loads provided by VHB (Ref. 10) and distributed to the individual piles by WSP using FB-MultiPier (Ref. 8). The estimated pile length at this location per Ref. 8 is 33.6 feet below the base of the abutment.

2. The downdrag depth is based on the total settlement estimated to occurred 75 years after the construction of the embankment at the Abutment 2 at the location of the pile with maximum loading (Ref. 3).

CALCULATIONS

Date:	9/15/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 3 and Ref. 7 at the centerline of the Abutment 2, for soil set 5 which correspond to the soil set used in FB-Multipier model for Pile 12. The soil properties used in the downdrag analysis are taken from Ref. 4 and are based on correlations to field test and laboratory test data.

Base of Abutment 2 elevation =	99.0	feet	(Ref. 1, Sheet B23)
Water table elevation =	84.4	feet	(Ref. 7)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	
	Top	Bottom	Top	Bottom			Top	Bottom
Crushed Stone	99.0	98	0.0	1.0	105	40	-	-
FGA	98.0	84.4			18	35.5	-	-
FGA	84.4	81.0	14.6	18.0	-44.4	35.5	-	-
Stiff Clay	81.0	78.0	18.0	21.0	59.6	-	1340	1340
Medium Clay	78.0	67.9	21.0	31.1	58.6	-	888	888
Bedrock	67.9	-	31.1	-	95.6	50	-	-

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	33.6	ft	(Ref. 8)
Pile section box perimeter =	57.0	in	(using pile flange width and depth from Ref. 5)
Pile section tip area =	26.1	in^2	(Ref. 5)
Pile nominal weight =	0.089	kip/ft	(Ref. 5)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 6 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 2 and shown in Attachment 1 the settlement beneath Abutment 2 after 75 years of construction for the maximum settlement point is estimated to be greater than 0.4 inches down to an elevation of 77.72 feet (depth of 22.0 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 22 feet of the pile below the base of the abutment.

Per Ref. 6 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	9/15/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use the Beta method to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 6, Tables 3.4.1-1 and 3.4.1-2)

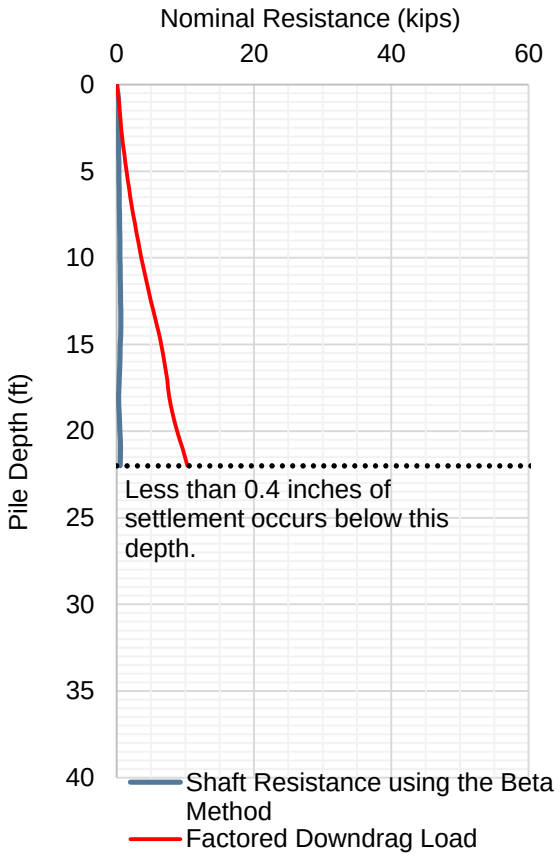
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	219.36	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type	Shaft Resistance using the Beta Method	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	219.4
1	Sand	0.3	0.3	219.7
2	Sand	0.2	0.5	220.1
3	Sand	0.3	0.8	220.4
4	Sand	0.3	1.1	220.8
5	Sand	0.3	1.4	221.2
6	Sand	0.3	1.8	221.7
7	Sand	0.4	2.2	222.2
8	Sand	0.4	2.6	222.7
9	Sand	0.4	3.1	223.3
10	Sand	0.5	3.6	223.8
11	Sand	0.5	4.1	224.5
12	Sand	0.5	4.7	225.1
13	Sand	0.6	5.3	225.8
14	Sand	0.6	5.9	226.5
15	Sand	0.5	6.5	227.2
16	Sand	0.4	7.0	227.7
17	Sand	0.4	7.3	228.2
18	Sand	0.3	7.6	228.6
19	Clay	0.4	8.2	229.2
20	Clay	0.5	8.8	229.9
21	Clay	0.6	9.6	230.8
22	Clay	0.5	10.3	231.6
23	Clay	0.6		221.4
24	Clay	0.7		221.5
25	Clay	0.7		221.6
26	Clay	0.8		221.7
27	Clay	0.9		221.8



CALCULATIONS

Date:	9/15/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum loading	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

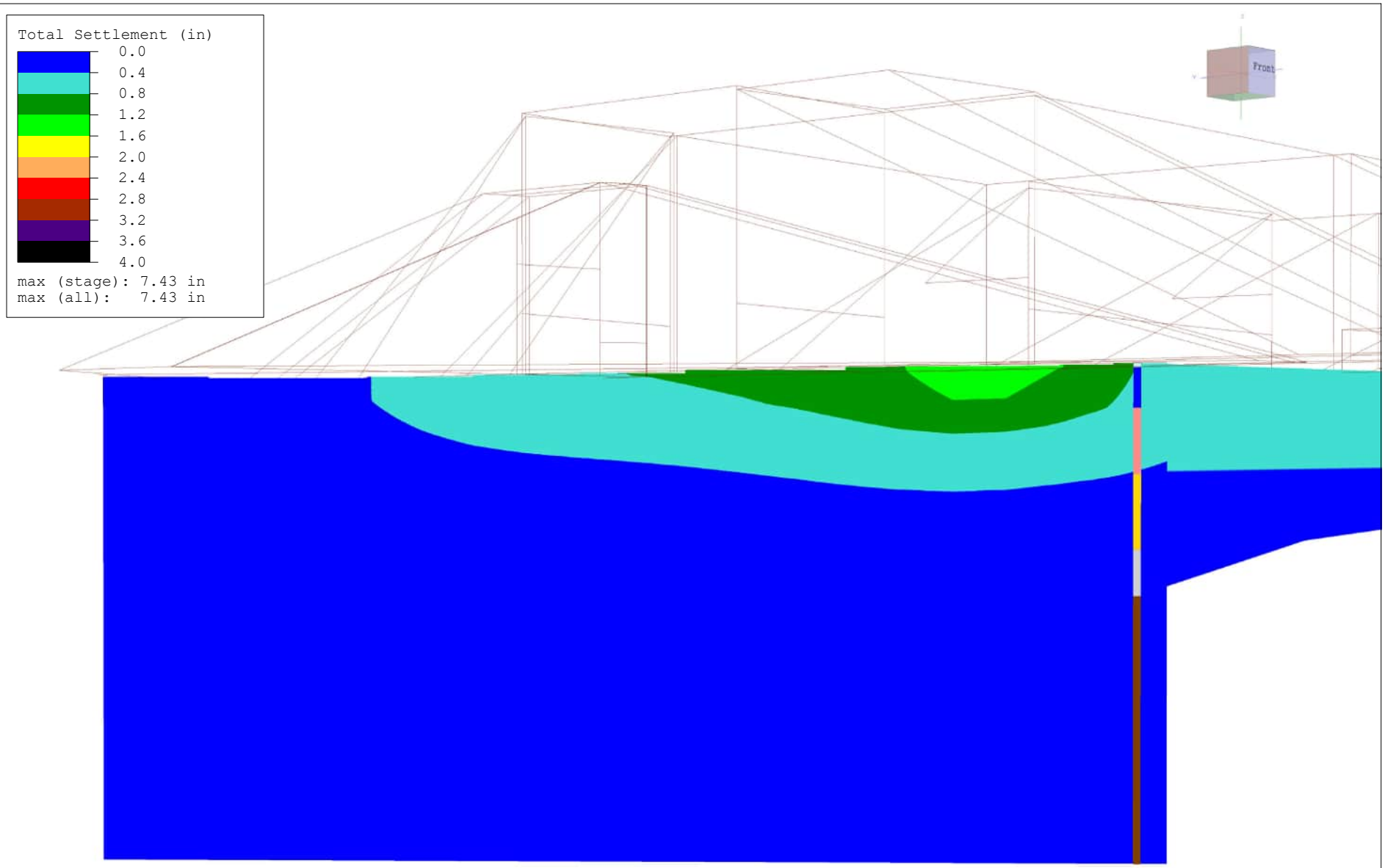
28	Clay	1.0		221.9
29	Sand	1.0		221.9
30	Sand	1.1		222.0
31	Sand	1.2		222.1
32	Sand	1.2		222.2
33	Sand	0.0		222.3
34	Sand	0.0		222.4
35	Sand	0.0		222.5
36	Sand	0.0		222.6

*Strength I, including Substructure Loading, Downdrag, and Pile Weight

Strength I factored downdrag load = 10.3 kips per pile

CONCLUSIONS

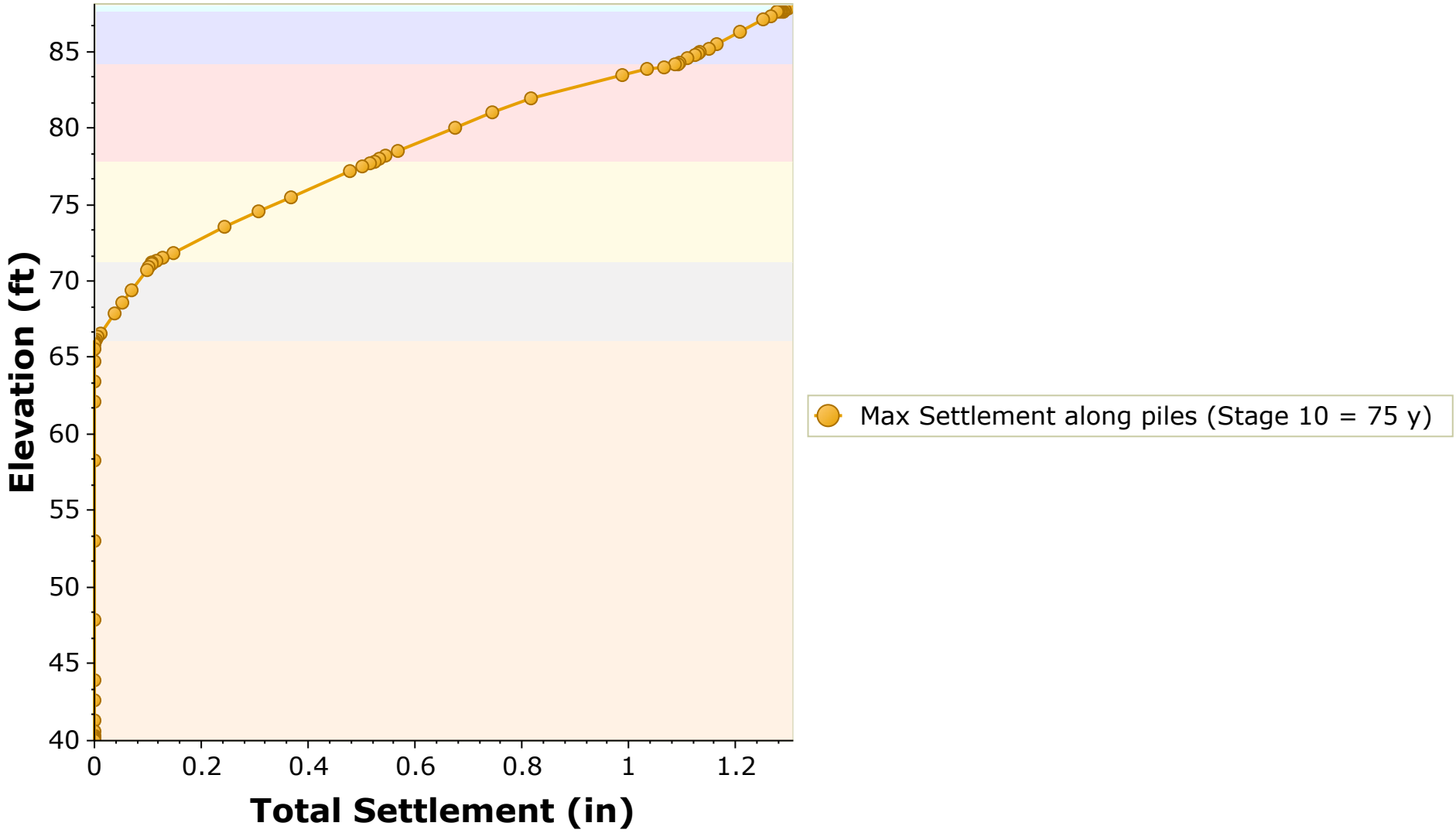
Based on the analysis of settlement for the proposed Abutment 2 of Bridge #6787 at the location of Pile 12, downdrag is estimated to develop along the upper 22 feet of pile 12 below the base of the Abutment 2. A factored downdrag load of 10.3 kips per pile for the Strength I limit state was calculated.



Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
Analysis Description	Total Settlement at Westbound Bridge Abutment 2 at 75 Years after Construction			
Drawn By ATM	Checked By KAR	Reviewed By CCB	Scale as shown	**Figure** A.1
Date 9/15/2025	File Name Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z			

SETTLE3 5.025

Total Settlement vs. Elevation



Reference Stage: None

WSP	*Project* MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
	Analysis Description: Total Settlement at Proposed Abutment 2 Pile with Maximum Loading 75 Afters Construction			
	Drawn By: ATM	Checked By: KAR	Reviewed By: CCB	Scale: as shown
SETTLE3 5.025	*Date*: 9/15/2025	File Name: Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z		**Figure** A.2

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

OBJECTIVE

Evaluate the downdrag load acting on the proposed HP 14x89 steel pile closest to the maximum settlement point (Pile 10) at Bridge #6787 Abutment 2 of the bridge carrying Hogan Road Westbound over I-95. This analysis uses estimated settlement for 75 years after construction.

METHOD

Use the Beta method (Ref. 6) to estimate the shaft resistance contributing to downdrag.

REFERENCES

1. Maine Department of Transportation Bridge Program. 60% West Bound Bridge Plans, Bangor, Maine, Federal Project #1859510, WIN 018595.10. Dated 8/30/2025.

2. WSP Settle3 West Bound model titled "Hogan Rd Highway Interchange Phase2 LWF June2025 w slab & staging & grading.s3z", dated 6/2025.

3. WSP Interpretive Subsurface Profile B- B', included as part of the Geotechnical Design Report.

4. WSP calculations of interpreted soil parameters based on field and laboratory data, named "Lab Field Data Summary Plots V4 31404817.000.xlsx".

5. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx

6. AASHTO LRFD Bridge Design Specifications, 9th Edition. 2020.

7.WSP FB-Multipier Soil properties for the proposed West Bound Bridge, named "New Bridge Abutment 2 FB-Multipier Soil Properties.xlsx", dated 5/20/2025.

8.WSP West Bound Abutment 2 Pile Analysis, named "Hogan Road New Bridge Abutment 2 H-Pile Analysis UPDATED Layout 5.21.25.xlsx", dated 5/21/2025.

9. Loading provided by VHB in Computation package titled "Bangor DDI Hogan Rd WB Bearing Loads 11/12/2024"

10. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.

ATTACHMENTS

A.1. Profile of estimated settlement beneath Abutment 2 75 years after construction

A.2. Profile of estimated settlement beneath Abutment 2 75 years after construction at the location of the pile with maximum settlement.

ASSUMPTIONS

1. The proposed Abutment 2 is supported on twenty-six HP14x89 steel piles, based on Ref. 8. The downdrag is evaluated for the pile closest to the maximum settlement point. The estimated pile length at this location per Ref. 8 is 36 feet below the base of the abutment.

2. The downdrag depth is based on the total settlement estimated to occurred 75 years after the construction of the embankment at the Abutment 2 at the maximum settlement location (Ref. 2).

3. At MaineDOT's request based on their internal research, the Beta method (Ref. 6, 10.7.3.8.6c) will be used to estimate shaft resistance, rather than the Alpha methed on cohesive soils and the Nordlund method for cohesionless soils (FHWA method).

4. Downdrag is calculated with an uncorroded pile section to represent the maximum possible downdrag load.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CALCULATION

1. Establish soil profile and pile properties.

The soil profile used in the downdrag analysis is based on Ref. 3 and Ref. 7 at the centerline of the Abutment 2, for soil set 4 which correspond to the soil set used in FB-Multipier model for Pile 10. The soil properties used in the downdrag analysis are taken from Ref. 4 and are based on correlations to field test and laboratory test data.

Base of Abutment 2 elevation =	99.0	feet	(Ref. 1, Sheet B23)
Water table elevation =	84.4	feet	(Ref. 7)

Soil Layer	Elevation (feet)		Depth below Abutment (feet)		Effective Unit Weight (pcf)	Friction Angle (°)	Undrained Shear Strength (psf)	Beta (β)[1]
	Top	Bottom	Top	Bottom				
Crushed Stone	99.0	98	0.0	1.0	105	40	-	0.58
FGA	98	84.4	1.0	14.6	18	35.5	-	0.56
FGA	84.4	81.0	14.6	18.0	-44.4	35.5	-	0.56
Stiff Clay	81.0	78.0	18.0	21.0	59.6	-	1340	0.35
Medium Clay	78.0	71.4	21.0	27.6	58.6	-	888	0.27
Glacial Till	71.4	64.4	27.6	34.6	67.6	35	-	0.35
Bedrock	64.4	-	34.6	-	95.6	50	-	-

Note: [1]Beta values taken from Ref. 10, Table 7-9.

Pile steel elastic modulus =	29,000,000	psi	(typical value for steel)
Pile length =	36.0	ft	(Ref. 8)
Pile section box perimeter =	57.0	in	(using pile flange width and depth from Ref. 7)
Pile section tip area =	26.1	in^2	(Ref. 5)
Pile nominal weight =	0.089	kip/ft	(Ref. 5)

2. Identify the depth below the abutment to which settlement is large enough to fully develop downdrag loading.

Per Ref. 6 Article 3.11.8, downdrag can be assumed to fully develop for the length of pile with settlement equal to or greater than 0.4 inches. As calculated in Ref. 2 and shown in Attachment 1 the settlement beneath Abutment 2 after 75 years of construction for the maximum settlement point is estimated to be greater than 0.4 inches down to an elevation of 75.53 feet (depth of 23.5 feet below the base of the abutment). Thus, downdrag loading is assumed to be fully developed for the upper 24 feet of the pile below the base of the abutment.

Per Ref. 6 Article 10.7.1.6.2 for piles end bearing on rock downdrag shall be considered at the strength and extreme limit state.

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

3. Use the Beta method to estimate shaft resistance and total downdrag load.

Downdrag Load Factors:
(Ref. 6, Tables 3.4.1-1 and 3.4.1-2)

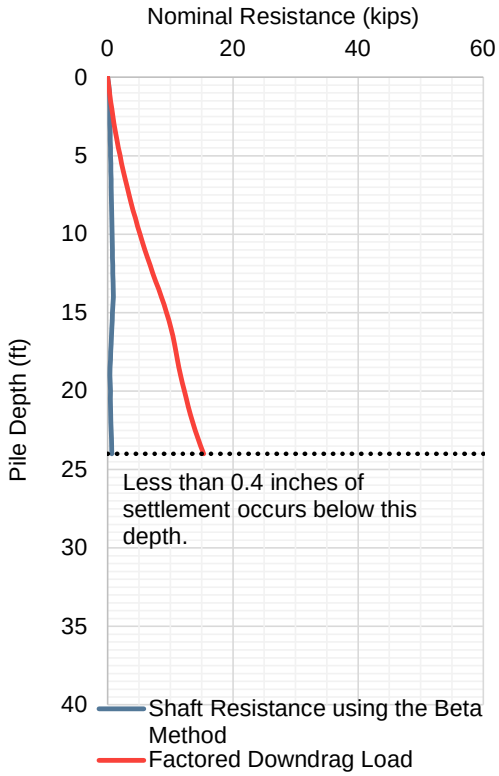
Soil Type	Strength I
Sand	1.05
Clay	1.40

Abutment Loads:

	Strength I		
Per Pile	151.15	kips	(Ref. 8)

Shaft resistance and downdrag load for each foot of pile length:

Pile Depth below Base of Abutment	Soil Type	Shaft Resistance using the Beta Method	Factored Downdrag Load	Total Factored Axial Load*
(feet)		(kips)	(kips)	(kips)
0	Sand	0.0	0.0	151.2
1	Sand	0.3	0.3	151.5
2	Sand	0.3	0.6	152.0
3	Sand	0.4	1.0	152.4
4	Sand	0.4	1.5	153.0
5	Sand	0.5	2.0	153.6
6	Sand	0.5	2.5	154.2
7	Sand	0.6	3.1	154.9
8	Sand	0.6	3.7	155.6
9	Sand	0.7	4.4	156.4
10	Sand	0.7	5.2	157.2
11	Sand	0.8	5.9	158.1
12	Sand	0.8	6.8	159.0
13	Sand	0.8	7.7	160.0
14	Sand	0.9	8.6	161.0
15	Sand	0.8	9.4	161.9
16	Sand	0.7	10.1	162.7
17	Sand	0.5	10.7	163.3
18	Sand	0.4	11.1	163.9
19	Clay	0.4	11.6	164.5
20	Clay	0.5	12.3	165.2
21	Clay	0.4	12.9	165.9
22	Clay	0.5	13.6	166.7
23	Clay	0.6	14.4	167.6
24	Clay	0.7	15.3	168.6
25	Clay	0.7		153.4
26	Clay	0.8		153.5
27	Clay	0.9		153.6
28	Clay	1.0		153.6
29	Sand	1.4		153.7
30	Sand	1.5		153.8



*Strength I, including Substructure Loading, Downdrag, and Pile Weight

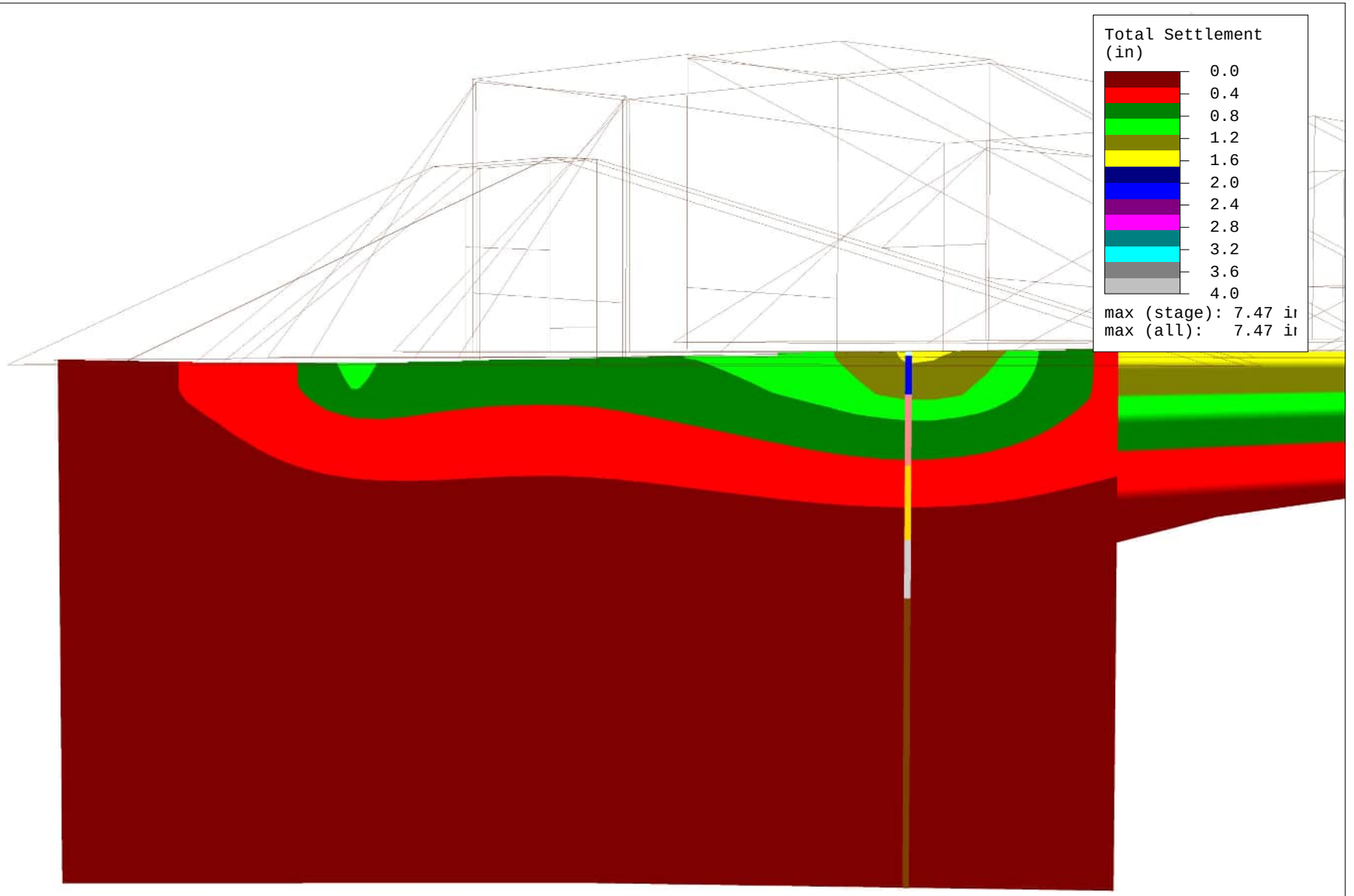
Strength I factored downdrag load = 15.3 kips per pile

CALCULATIONS

Date:	7/2/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Downdrag at the proposed Bridge #6787 Abutment 2 - Pile with maximum settlement	**Reviewed by:**	CCB
Project Short Title:	MaineDOT Bangor Hogan Road Highway Diverging Diamond Interchange		

CONCLUSIONS

Based on the analysis of settlement for the proposed Abutment 2 of the West Bound Bridge at the location of maximum settlement, downdrag is estimated to develop along the upper 24 feet of pile 10 below the base of the Abutment 2. A factored downdrag load of 15.3 kips per pile for the Strength I limit state was calculated.



wsp

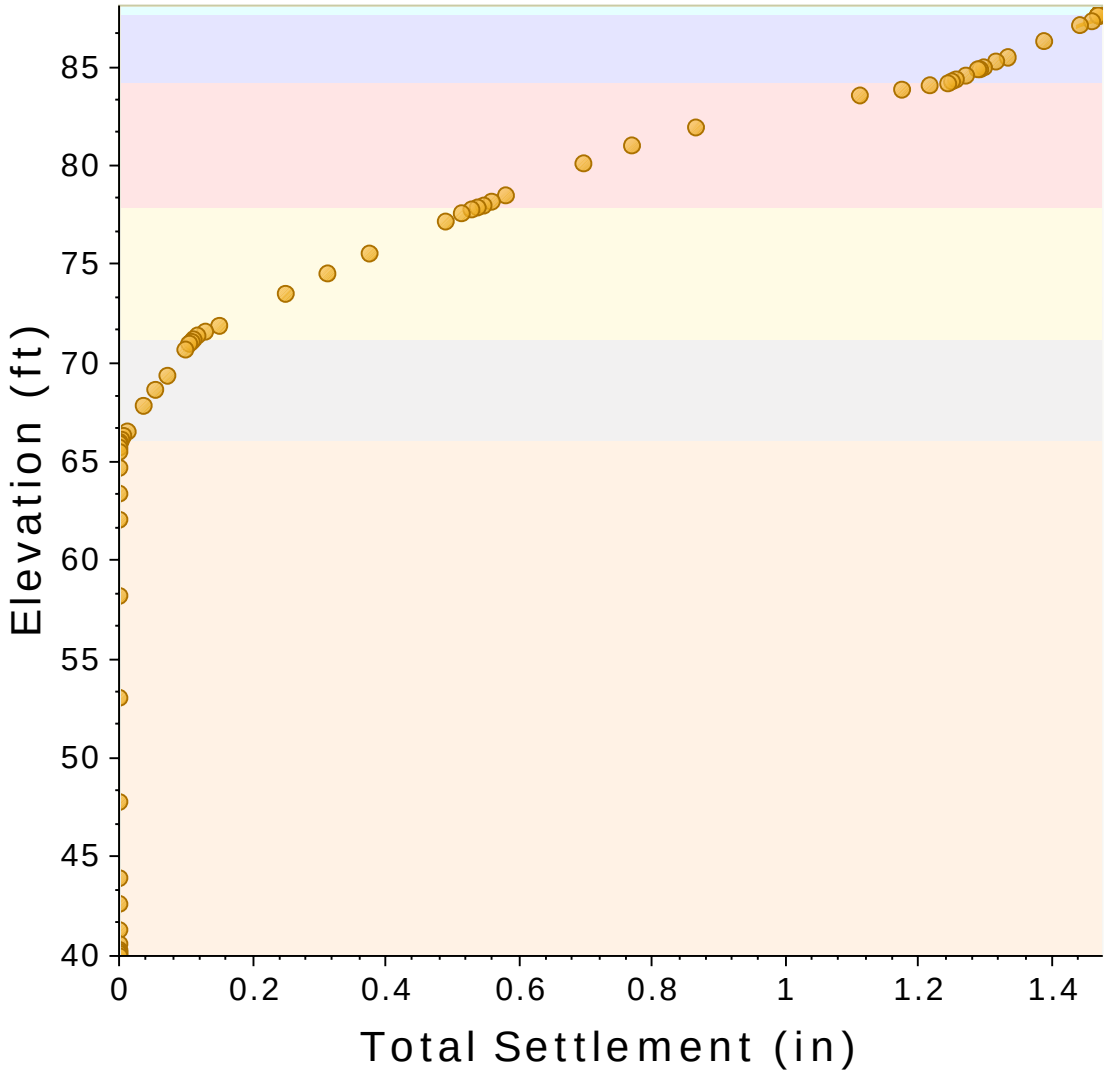
SETTLE3 5.025

Project	MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004
Analysis Description	Total Settlement along the proposed Abutment 2 after 75 years of construction

Drawn By	Checked By	Reviewed By	Scale
LMP	DEB		As shown
Date 5/30/2025	**File Name** Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z		

Figure A.1

Total Settlement vs. Elevation



Max Settlement Abutment 2 (Stage 10 = 75 y)

Reference Stage: None

wsp	*Project* MaineDOT Bangor Hogan Road Bridge Replacement, Project #31404817.004			
	Analysis Description Total Settlement at the maximum settlement location on the proposed Abutment 2 after 75 years of construction			
	Drawn By LMP	Checked By DEB	Reviewed By	Scale As shown
	Date 5/30/2025	File Name Hogan Rd Highway Interchange Phase2 LWF May2025 w slab & staging & grading (1).s3z		Figure A.2

SETTLE3 5.025

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Resistance)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine the factored pile resistance of the proposed HP 14x89 piles at Abutment 2 based on the driving criteria outline in FHWA GEC 12 and the MaineDOT Standard Specification March 2020.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Abutment 2 H-Pile Analysis, dated 4/3/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
9. Federal Highway Administration (FHWA), FHWA-NHI-126-009 - Geotechnical Engineering Circular 12, Design and Construction of Driven Piles Vol II, July 2016
10. WSP boring logs.

ASSUMPTIONS

1. Pile tip elevations are based on an interpretation of the as-drilled top of bedrock elevations (Ref. 10).

2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 2 (soil set 5).

3. As per FHWA GEC 12 Section 14.2.6.C-8 (Ref. 9), absolute refusal during pile driving is defined as 20 blows for one inch or less or 10 blows for a half inch can be used to reduce the risk of pile damage. Driving stresses shall not exceed 90% of the pile yield strength per Maine Department of Transportation 2020 Standard Specifications Section 501 Foundation (Ref. 4).

4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

5. This analysis looks at resistance upon reaching the tip elevation in bedrock and does not consider the resistance during driving. A resistance distribution of 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.

6. A φdyn of 0.65 using (Ref. 8 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Resistance)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2	= 0.18 ft^2 (Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in	= 4.8 ft
Pile head elevation =	100.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	66.9	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	33.6	ft	(Ref. 3, Sheet 13 of 53)

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.11	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation =	100.5	ft	
Groundwater elevation =	84.4	ft	= 16.1 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Crushed Stone	99.0	98.0	1.5	2.5	105	40	-	-	-	Sand
FGA	98.0	81.0	2.5	19.5	18	35.5	-	-	-	Sand
Stiff Clay	81.0	78.0	19.5	22.5	122	-	1340	-	-	Clay
Medium Clay	78.0	67.9	22.5	32.6	121	-	888	-	-	Clay
Competent Bedrock	67.9	-	32.6	-	158	-	-	20.0	422.0	Rock Other

CALCULATIONS

Date:	6/3/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Resistance)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outline in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
Reaching tip elevation	10%	90%	APE D25-42	45,892	Fuel Setting 2 (74%)	697	20.0	37.62	75%

3. Determine nominal and factored toe resistance.

Nominal toe resistance = 697 kips (Intact section from GRLWEAP)

φ_{dyn} = 0.65 (Ref. 9, Table 10.5.5.2.3-1)

Factored toe resistance = 453 kips

CONCLUSIONS

This analysis indicates that an HP14x89 pile driven with an APE D25-42 hammer operated at 74% of the combustion pressure is expected to develop a nominal bearing resistance of 697 kips at WB Abutment 2 while limiting driving stresses to a maximum of 45 ksi. Blow counts were limited to 10 blows per 1/2" of penetration, in accordance with FHWA GEC 12 (Ref. 9) and Section 501 of the Maine Department of Transportation 2020 Standard Specifications for Construction (Ref. 4). The resulting factored toe resistance from this WEAP analysis is 453 kips.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

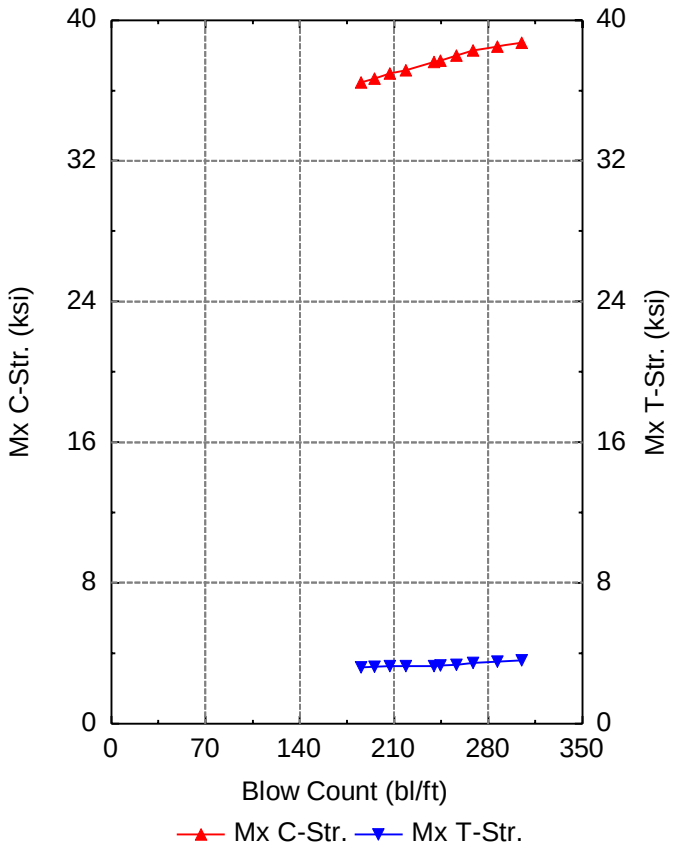
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

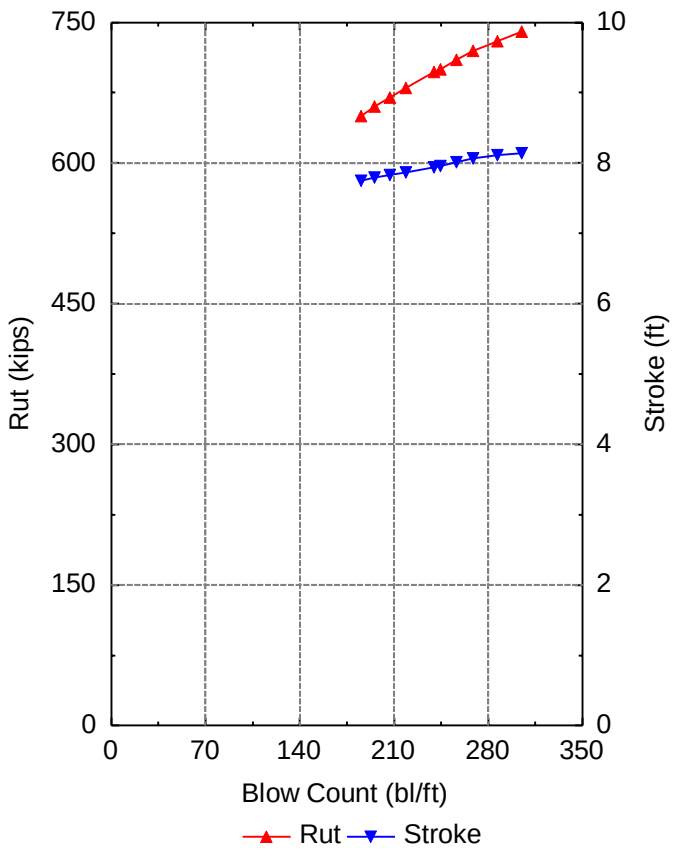
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

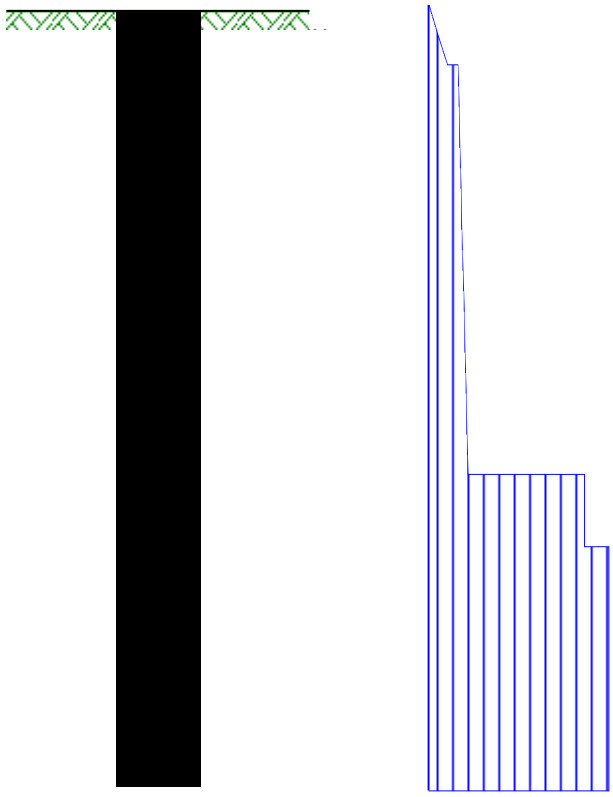
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.110	s/ft
Toe Damping	0.150	s/ft
Pile Length	32.60	ft
Pile Penetration	32.60	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model

Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
650.0	36.45	31.20	3.19	185.4	7.75	15.18	D 25-42	24.5
660.0	36.68	31.51	3.23	195.5	7.79	15.30	D 25-42	24.7
670.0	36.95	31.85	3.26	207.1	7.83	15.40	D 25-42	24.8
680.0	37.17	32.15	3.27	218.7	7.87	15.52	D 25-42	25.0
697.0	37.62	32.68	3.26	239.9	7.94	15.74	D 25-42	25.4
700.0	37.68	32.77	3.28	244.6	7.95	15.76	D 25-42	25.4
710.0	38.00	33.13	3.35	256.4	8.01	15.93	D 25-42	25.7
720.0	38.29	33.47	3.44	268.9	8.07	16.11	D 25-42	26.0
730.0	38.51	33.75	3.51	286.7	8.11	16.20	D 25-42	26.1
740.0	38.72	34.03	3.59	305.0	8.14	16.31	D 25-42	26.3

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Driving)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed H-piles at WB Abutment 2 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Abutment 2 H-Pile Analysis, dated 6/3/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

ASSUMPTIONS

1. The proposed abutment is assumed to be supported on twenty-six (26) HP14x89 Grade 50 piles with a center-to-center pile spacing of 8 feet in the x-direction and 6 feet in the y-direction (Ref. 3). The piles are to be driven to bedrock.

2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 2 (soil set 5).

3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.

4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Driving)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using Micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2	= 0.18 ft^2 (Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = $(d \times 2) + (b_f \times 2)$ =	57.0	in	= 4.8 ft
Pile head elevation =	100.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	66.9	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	33.6	ft	(Ref. 3, Sheet 13 of 53)

Ultimate Capacities:

Axial load on pile, P_u =	208	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	320	kips	

For soil above bedrock while driving pile to bedrock:

Nominal shaft resistance, R_s =	135	kips	(Ref. 6)
Maximum nominal toe resistance of soil, $R_{p\text{-}soil}$ =	57	kips	(Ref. 6)
Total soil resistance =	192	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.11	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

CALCULATIONS

Date:	7/7/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Abutment 2 - HP14x89 (Driving)	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Shaft and Toe Resistance (during driving):

Nominal shaft resistance, R_s = 135 kips = 70 % of R_n (Ref. 6)
Nominal toe resistance, $R_{p\text{-}rock}$ = 57 kips = 30 % of R_n (Ref. 6)
Nominal resistance, R_n = 192 kips

Assume that once the pile has encountered bedrock, the resistance distribution will switch to 30% shaft resistance and 70% toe resistance. A second analysis will be performed to model these conditions.

Soil Profile Input, to calculate Variable Resistance Distribution:
(Ref. 8)

Pile head elevation = 100.5 ft
Groundwater elevation = 84.4 ft = 16.1 ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Crushed Stone	99.0	98.0	1.5	2.5	105	40	-	-	-	Sand
FGA	98.0	81.0	2.5	19.5	18	35.5	-	-	-	Sand
Stiff Clay	81.0	78.0	19.5	22.5	122	-	1340	-	-	Clay
Medium Clay	78.0	67.9	22.5	32.6	121	-	888	-	-	Clay
Competent Bedrock	67.9	-	32.6	-	158	-	-	20.0	422.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at R_{ndr} for the abutment.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	70%	30%	APE D25-41	45,892	Fuel Setting 2 (74%)	320	4.6	21.7	43%
Reaching tip elevation	30%	70%	APE D25-42	45,892	Fuel Setting 2 (74%)	320	4.3	22.3	45%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 2 (74% of the combustion pressure) while driving through soil and upon reaching bedrock should be able to overcome a soil resistance of 192 kips in order to drive the piles at WB Abutment 2 to bedrock and a nominal resistance of 320 kips while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

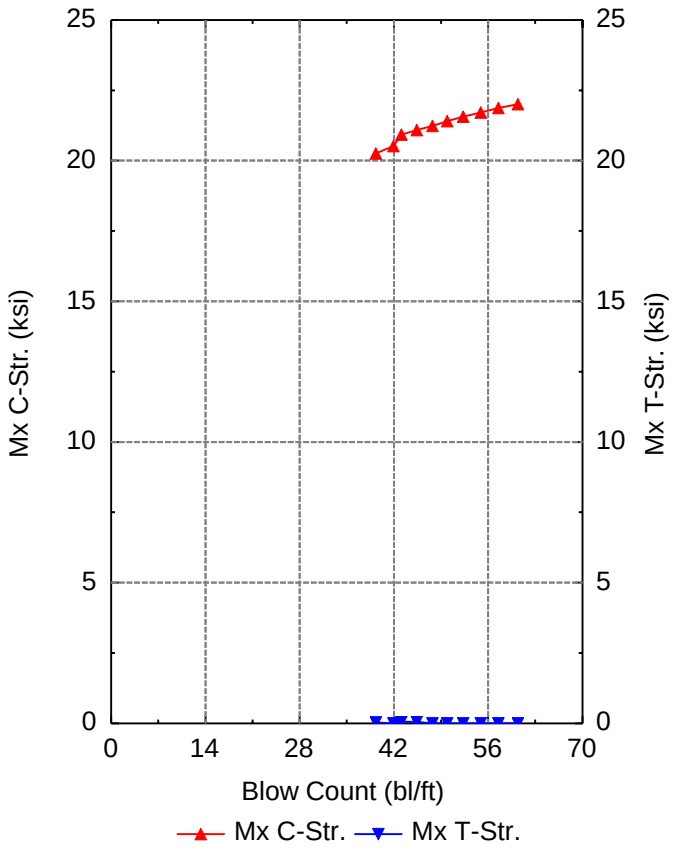
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

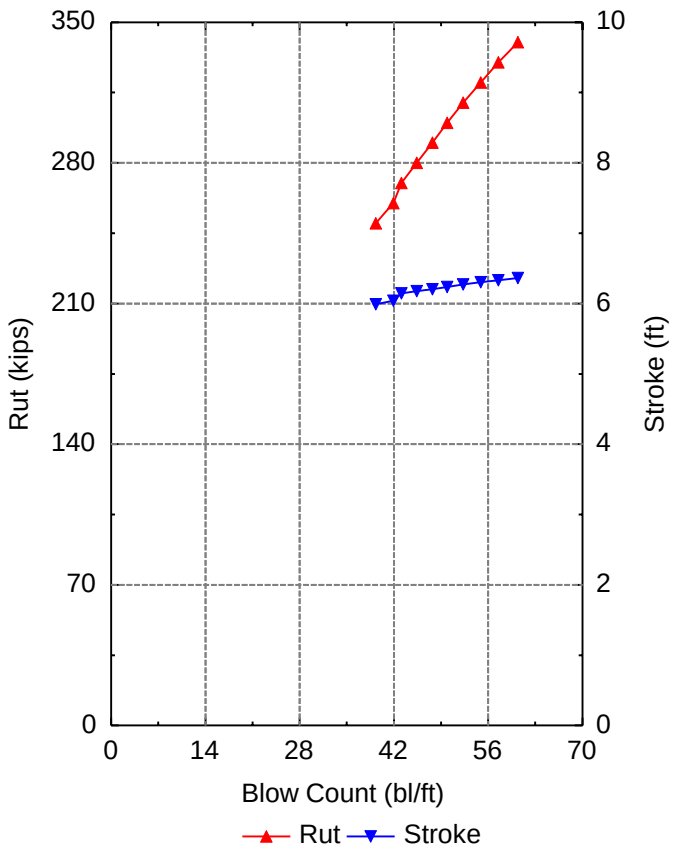
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

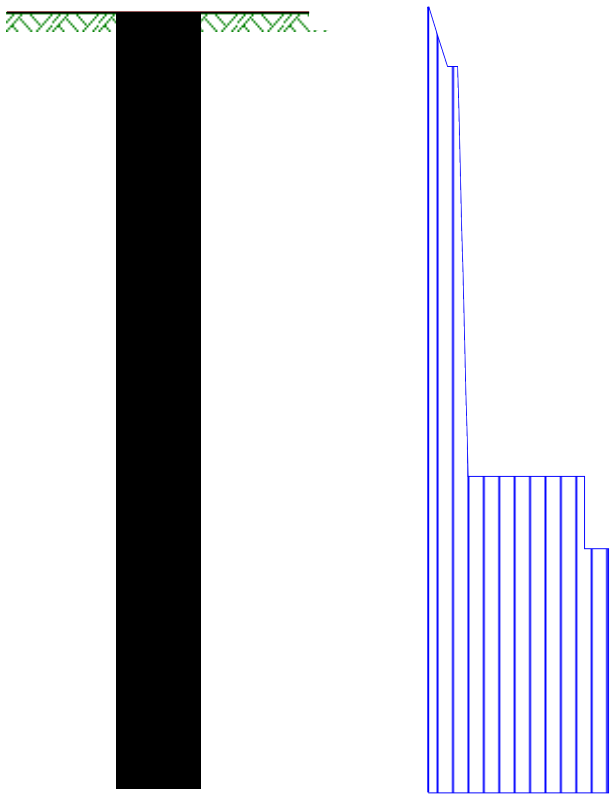
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.110	s/ft
Toe Damping	0.150	s/ft
Pile Length	32.60	ft
Pile Penetration	32.60	ft
Pile Top Area	26.10	in²
RSA	No	



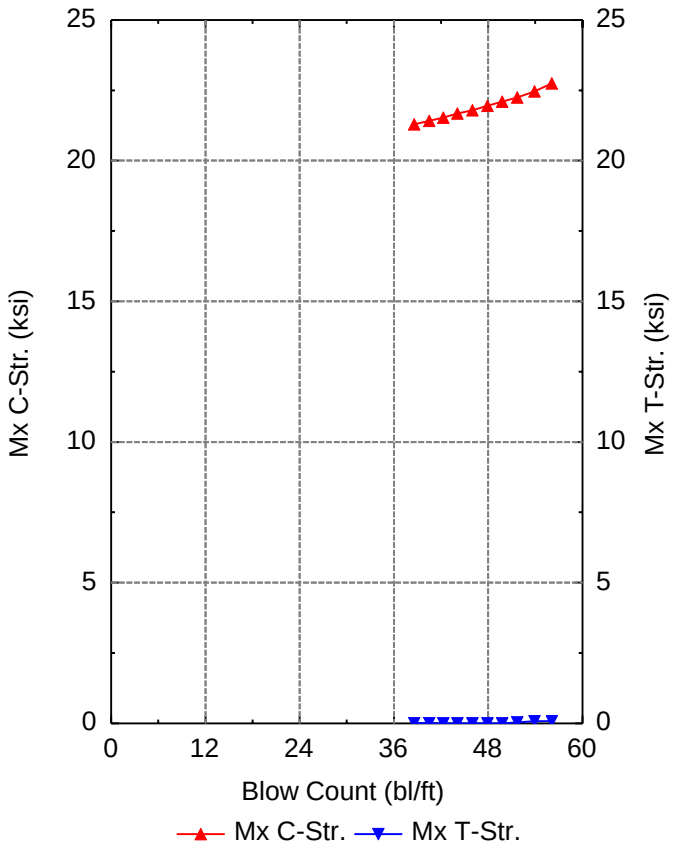


Pile Model

Shaft=70%
(Prop., SA)

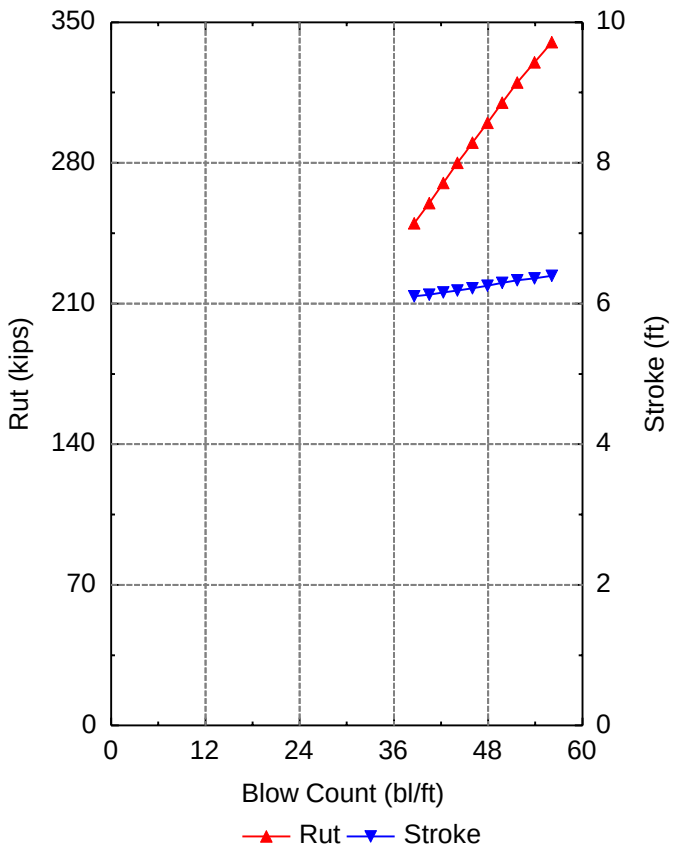
Bearing Graph Summary — APE D 25-42

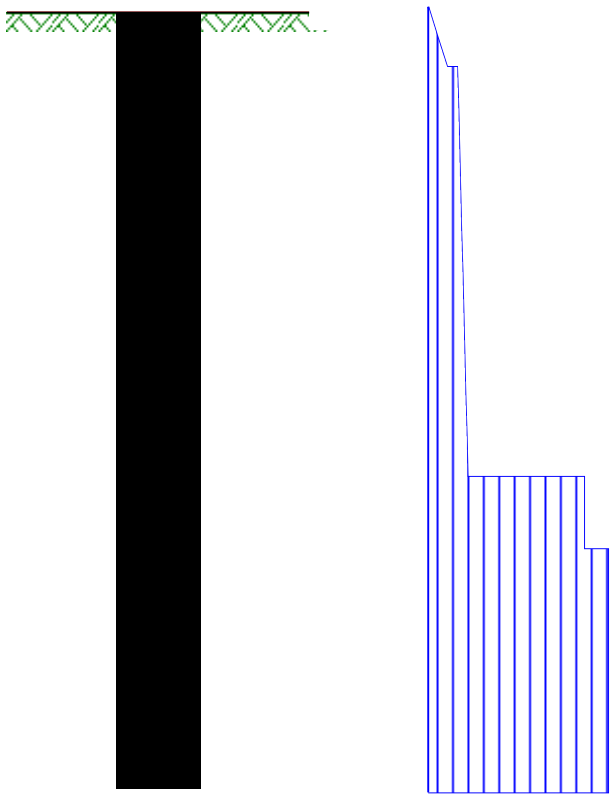
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
250.0	20.25	20.25	0.03	39.3	5.99	12.46	D 25-42	20.1
260.0	20.51	20.51	0.00	41.9	6.04	12.45	D 25-42	20.1
270.0	20.93	20.93	0.05	43.1	6.14	12.64	D 25-42	20.4
280.0	21.09	21.09	0.05	45.4	6.18	12.57	D 25-42	20.3
290.0	21.24	21.24	0.00	47.7	6.21	12.54	D 25-42	20.2
300.0	21.40	21.40	0.00	49.9	6.24	12.55	D 25-42	20.2
310.0	21.56	21.56	0.00	52.3	6.27	12.56	D 25-42	20.3
320.0	21.71	21.71	0.00	54.9	6.30	12.54	D 25-42	20.2
330.0	21.87	21.87	0.00	57.5	6.33	12.52	D 25-42	20.2
340.0	22.00	22.00	0.00	60.4	6.36	12.49	D 25-42	20.1



APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.110	s/ft
Toe Damping	0.150	s/ft
Pile Length	32.60	ft
Pile Penetration	32.60	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model

Shaft=30%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
250.0	21.29	20.90	0.01	38.6	6.10	12.68	D 25-42	20.4
260.0	21.42	21.02	0.01	40.5	6.13	12.54	D 25-42	20.2
270.0	21.53	21.13	0.00	42.3	6.16	12.49	D 25-42	20.1
280.0	21.68	21.29	0.00	44.1	6.19	12.52	D 25-42	20.2
290.0	21.79	21.41	0.00	46.0	6.22	12.48	D 25-42	20.1
300.0	21.95	21.55	0.00	47.9	6.26	12.49	D 25-42	20.1
310.0	22.10	21.72	0.00	49.8	6.29	12.50	D 25-42	20.2
320.0	22.25	21.88	0.03	51.7	6.33	12.52	D 25-42	20.2
330.0	22.46	21.98	0.07	53.9	6.36	12.50	D 25-42	20.2
340.0	22.75	22.13	0.07	56.1	6.39	12.51	D 25-42	20.2

APPENDIX K

Bridge 6787 Pier Foundation Calculations

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 1A based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 1A (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
16. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

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ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier 1A Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 1A is assumed to be supported by eight (8) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in each direction (ref. 5)

2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).

3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.

4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.

5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.

6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.

7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.

8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.

9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 91.8 feet and the top of Pier 1A footing of El. 82.5 feet, resulting in 6.3 feet of overburden soil. Both pile caps for Pier 1 are located at the same footing elevation of 82.5 feet which is at the calculated frost depth of 7.3 feet (Ref. 16) below the lowest adjacent ground surface along Pier 1. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.

10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assumed to begin 1-foot below the top of bedrock surface.

11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	85.5	ft	Sheet B12 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.0	ft	
As-modeled pile cap midplane elevation =	84.0	ft	Sheet B12 (Ref. 5)
Base of pile cap elevation =	82.5	ft	Sheet B12 (Ref. 5)
Pile length =	14.1	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	4.0	ft	
Bottom of socket El. =	69.4	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	82.5	81.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	81.5	79.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	79.0	73.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	73.4	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 1 (Sheet X Interpretive Subsurface Cross Section X-X').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 1)					
Maximum axial force in the piles, P_u =	407.1	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	34.9	kip-ft	=	419.3	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	25.3	kip-ft	=	303.5	in-kips	
Maximum shear force in the piles, V_u =	9.3	kips				
Depth below pile head to fixity =	13.8	ft	=	Elev.	70.23	ft
Depth below pile cap to fixity =	12.3	ft	=	Elev.	70.23	ft

wsp

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.031	0.401	ksf (at midpoint of soil layer using stratigraphy in Part A)
f_s =	0.012	0.200	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	4	ft	Grout below pile toe:	0.5	ft (Assumption 7)
Socket diameter:	30	in	(Assumption 4)		
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)				

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

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Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.4	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 15 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	67	ksf
Unit tip resistance q_p =	543	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	848	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	424	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
424	>	407	OK

CALCULATIONS

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Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	407	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	147.27	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 51 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check: $\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

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Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	147.27	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	2,137	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

CALCULATIONS

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Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877 P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.45 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 800 kips

For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$P_n = F_{cr} A_{eff}$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$F_{cr} = \frac{P_{cr}}{A_g}$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	800	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	41.4	ksi	

$A_{eff} = A_g - \sum (b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$If \ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$If \ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$ (Ref. 7, Eq. 6.9.4.2.2b-2)

CALCULATIONS

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$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	41.4	ksi	
$\lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ =	14.8		

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

c_1 =	0.22		(Ref. 7, Table 6.9.4.2.2b-1)
c_2 =	1.49		(Ref. 7, Table 6.9.4.2.2b-1)
c_3 =	0.00		(Ref. 7, Table 6.9.4.2.2b-1)
F_{el} =	79.09	ksi	(Ref. 7, Eq. 6.9.4.2.2b-4)
b_e =	6.99	in	(Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff}:

A_{eff} =	18.82	in^2	(Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n =	778.8	kips	(Ref. 7, Eq. 6.9.4.2.2a-1)
$P_{r_{axial}} = \varphi_c P_n$			(Ref. 7 Eq. 6.9.2.1-1)
φ_c =	0.60		(Ref. 7, Article 6.5.4.2))
P_r =	467	kips	

$P_{r_{axial/flexural}} = \varphi_c P_n$			(Ref. 7 Eq. 6.9.2.1-1)
φ_c =	0.70		(Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$ =	545	kips	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 407 kips (from Part B)

$P_{r,\,axial}$ = 467 kips > P_u = 407 kips OK

F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3) $\lambda_f = \frac{b_f}{2t_f}$ λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4) $\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$ λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5) $\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$ λr_f = 19.99

Nominal and factored structural resistance in flexure:

$M_n = M_p = 1.5F_yS_y$ if $\lambda_f \le \lambda_{pf}$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 419 in-kips (from Part B)

M_{uy} = 303 in-kips (from Part B)

M_{rx} = 4,973 in-kips > M_{ux} = 419 in-kips OK

M_{ry} = 1,913 in-kips > M_{uy} = 303 in-kips OK

wsp

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.96 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00	(Ref. 7, Article 6.5.4.2)
V_r =	168	kips

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.3 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.3 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2 (Strength I), and an estimated pile length of 14.1 feet, a maximum axial force of 407.1 kips , the corresponding strong axis (x-axis) moment of 419 in-kips (34.9 ft-kips) occurs in the HP14x89 piles at Pier 1A in Pile 1 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 3.5-foot pile embedment into a 4.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 1A Summary Table		
Pile Length (ft)		14
Depth below Pile Head to Fixity (ft)		14
Geotechnical Axial Resistance (kips)	Nominal	848
	Factored	424
Structural Axial Resistance (kips)	Nominal	800
	Factored	467
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.96
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

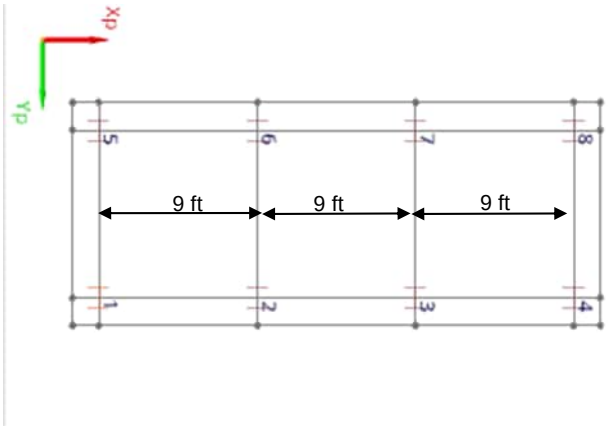


Figure 1b: Profile View of Pile Cap (Transverse)

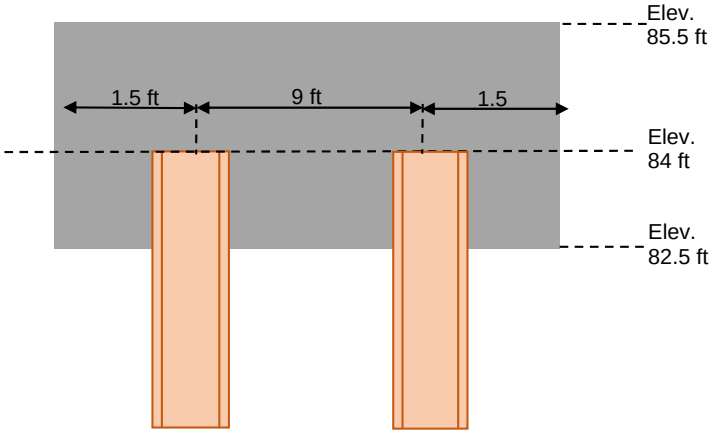


Figure 1c: 3D View of Pile Model

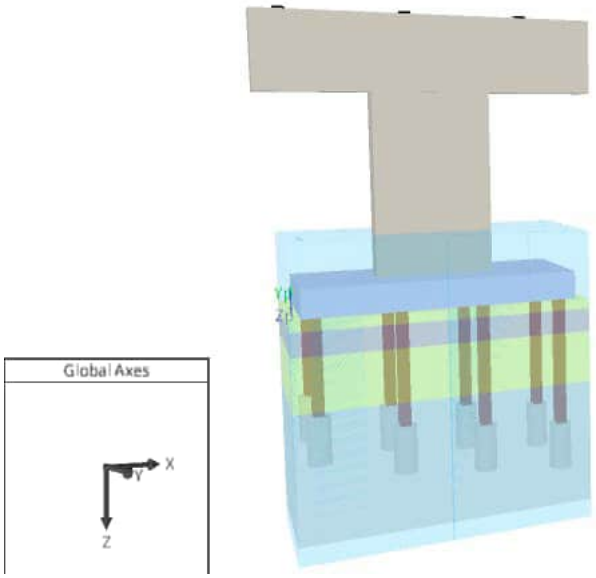
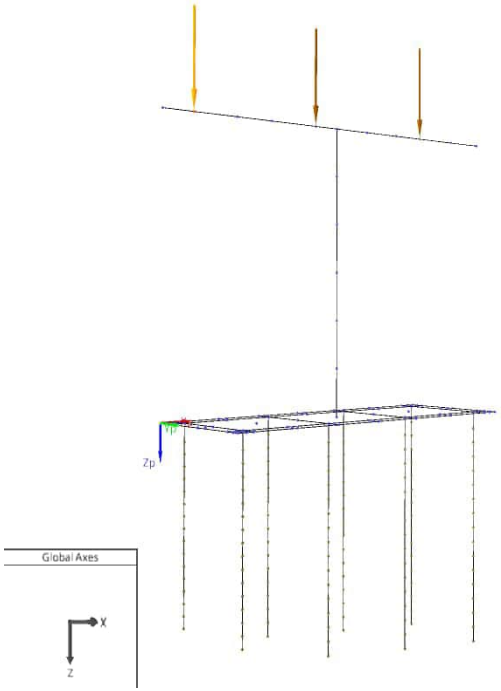


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	82.5	81.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	3	Cohesive	81.5	79.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	4	Cohesionless	79.0	73.4	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	200		-	-	-
Bedrock	5	Rock	73.4	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6327.0	848.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 1 profile (Sheet B12 Pier 1 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

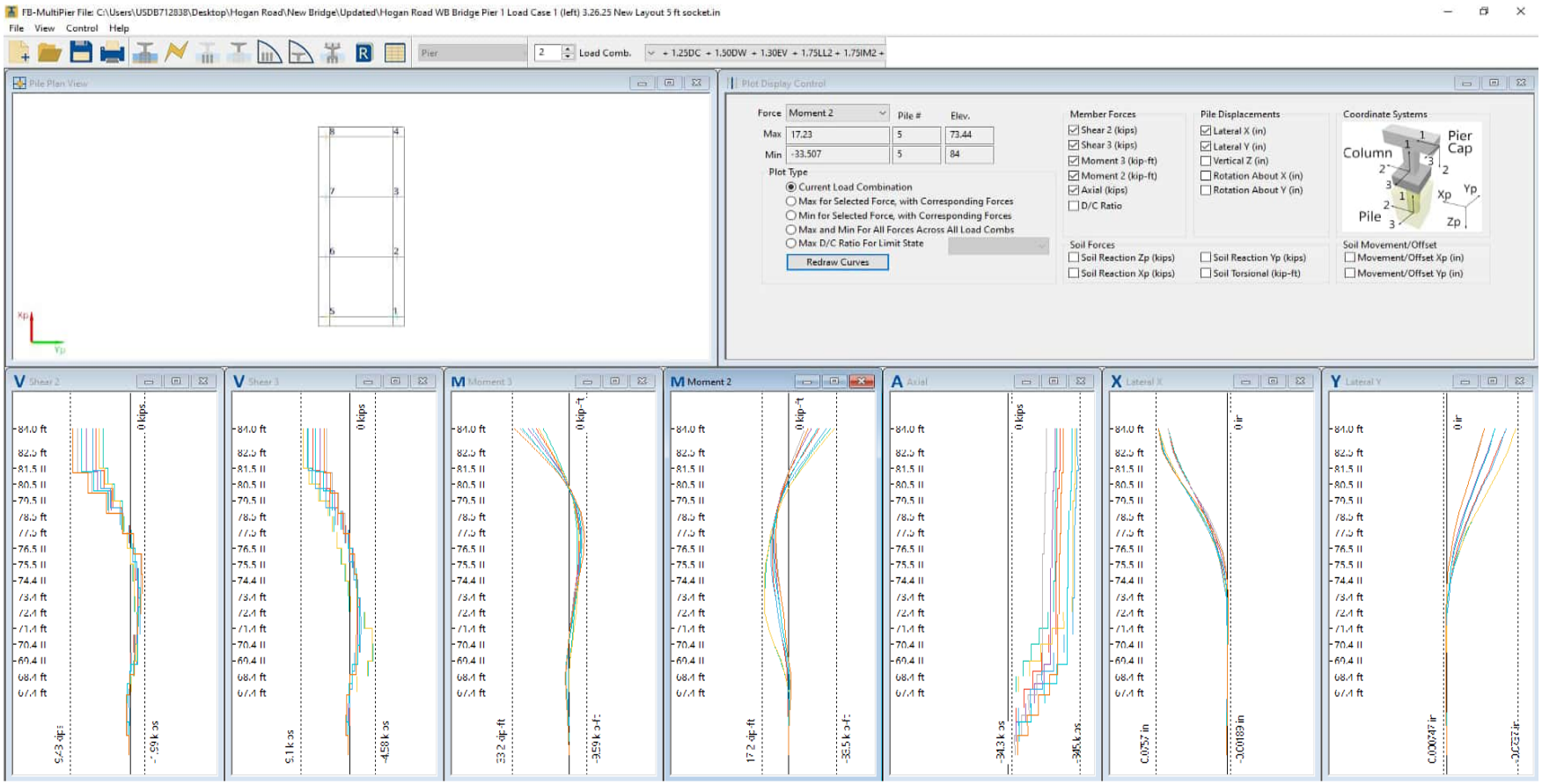
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

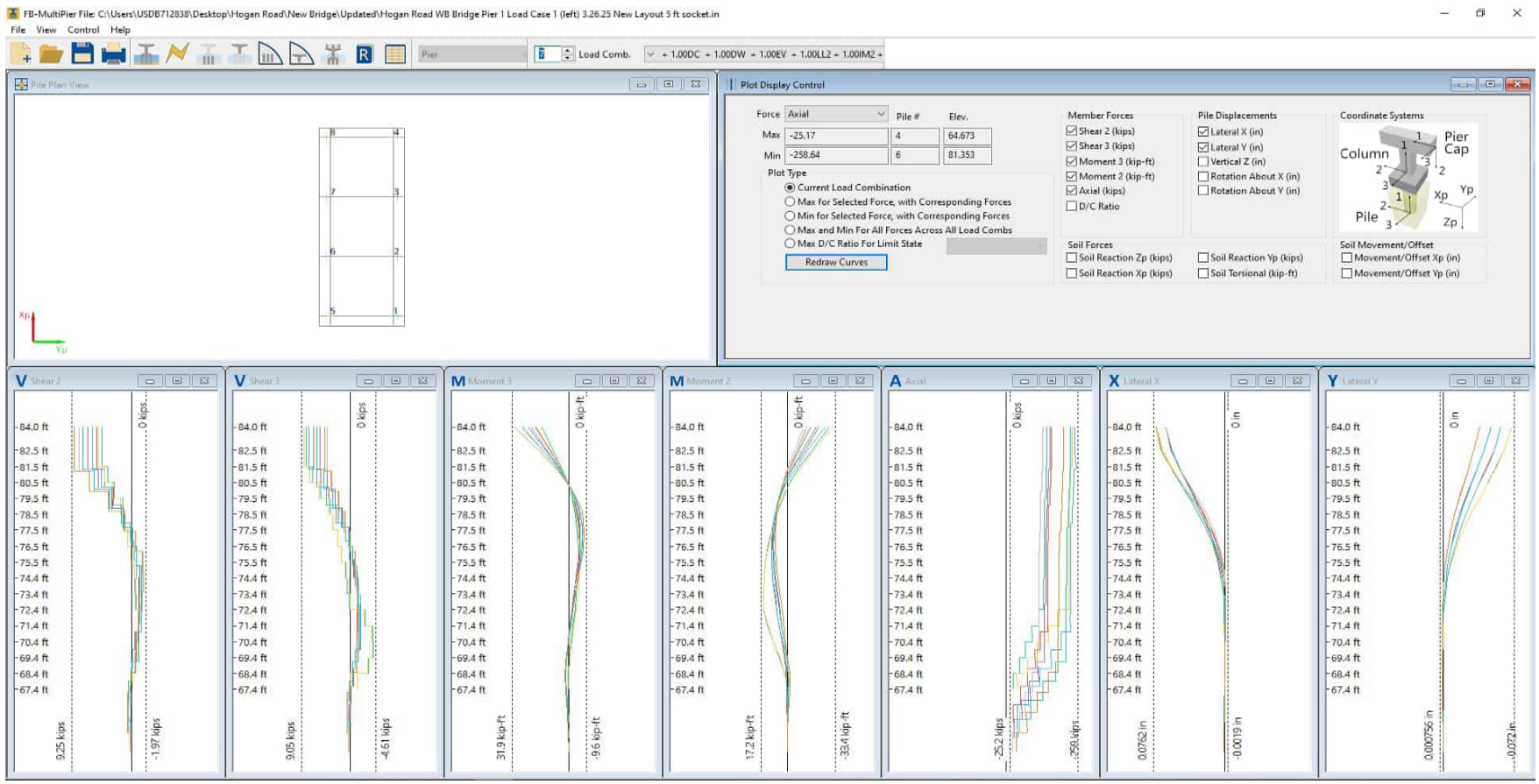
Prepared: DEB
Checked: ATM
Reviewed:

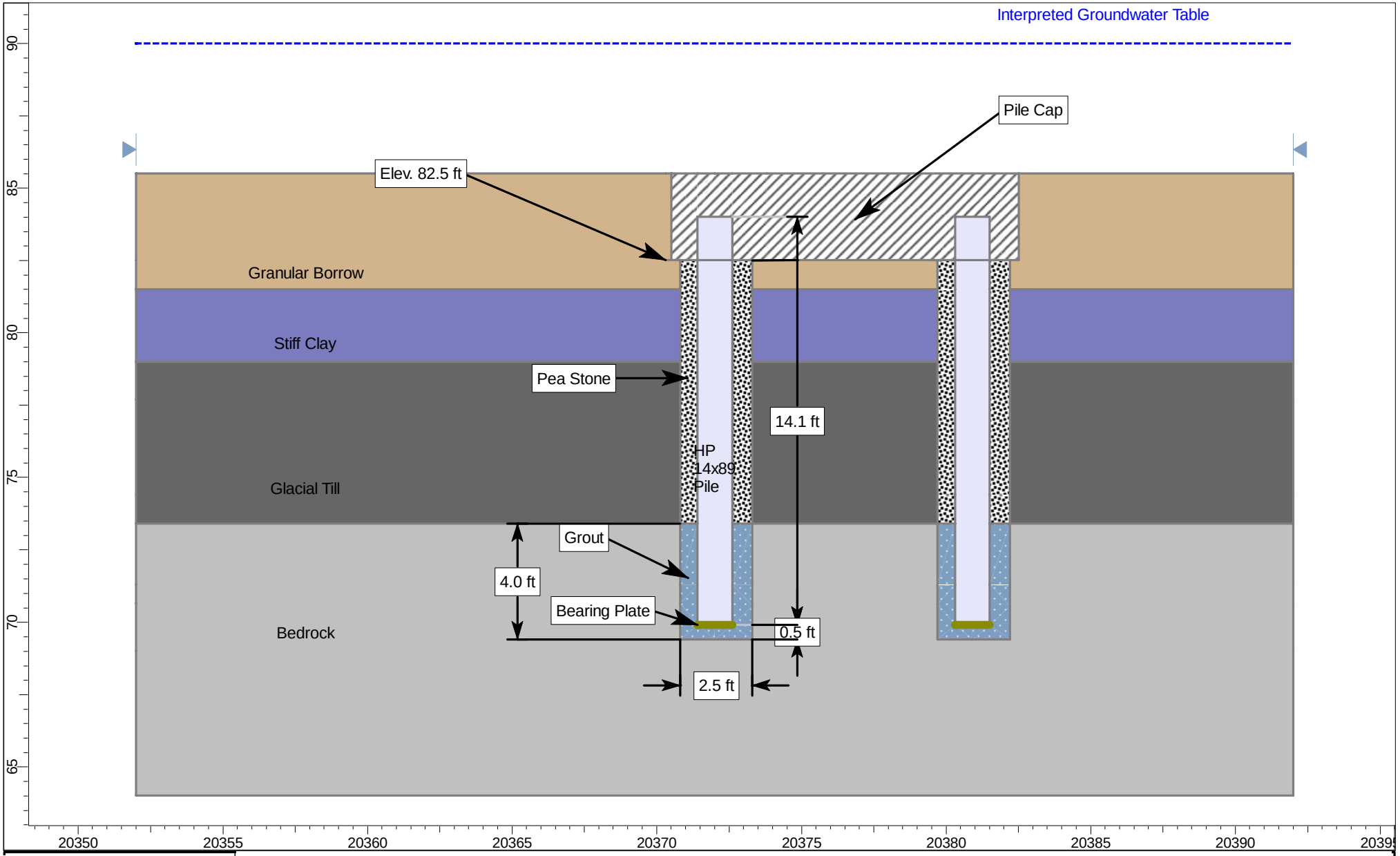
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 1 Strength I



Case 1 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 1A- Design Typical Section			
Drawn By: DEB	Checked By: ATM	Reviewed By: CCB	Scale: 1:55	Attachment 4
Date: 5/8/2025	File Name: Pier 1A sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

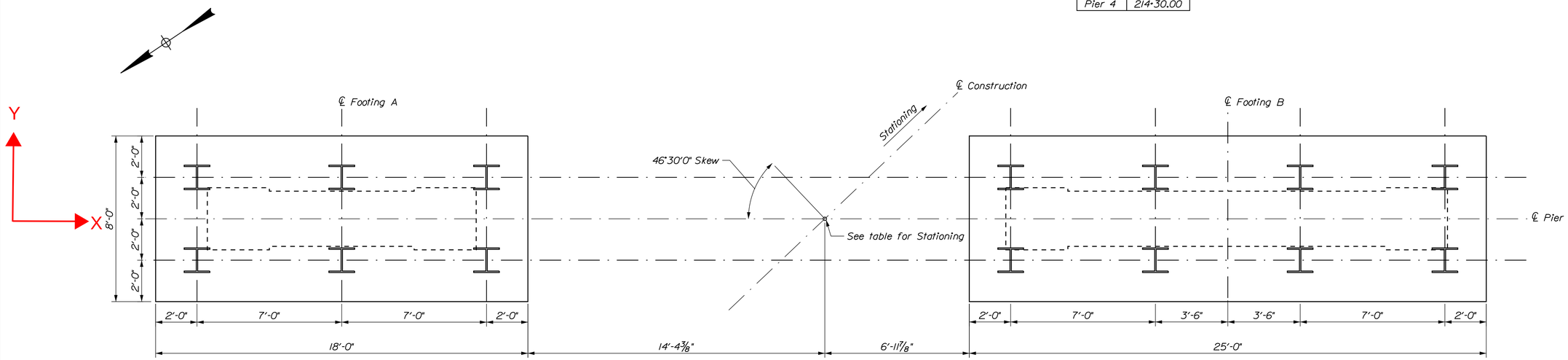
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

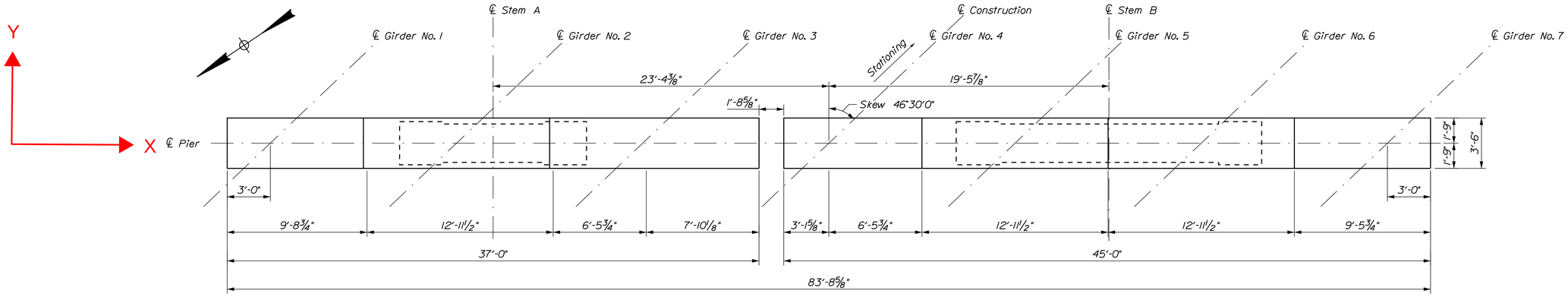
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

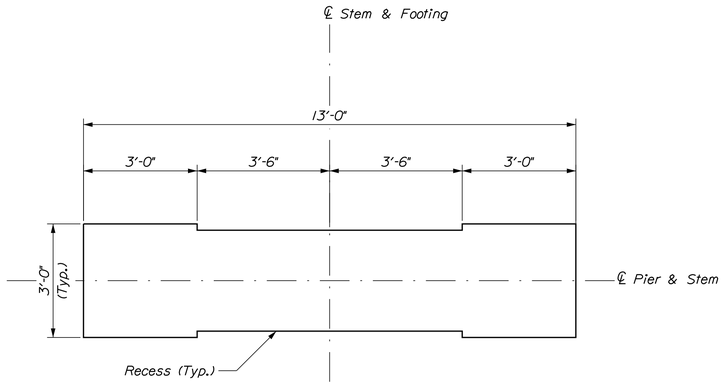
SHEET NUMBER

B25

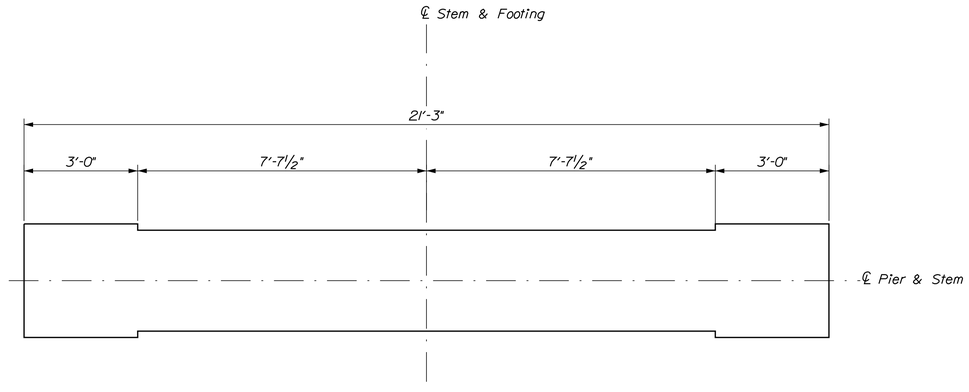
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1B H-Pile Analysis Load Case 1	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 1B based on the design load Case 1 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 1B (Pile 10), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
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7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
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10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
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12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1B H-Pile Analysis Load Case 1	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I output
4. Pier 1B Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 1 (right) is assumed to be supported by ten (10) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 6.75 feet in the x-direction (ref. 5)
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 90.6 feet and the top of Pier 1 (right) footing of El. 82.5 feet, resulting in 5.2 feet of overburden soil. Both pile caps for Pier 1 are located at the same footing elevation of 82.5 feet which is at the calculated frost depth of 7.3 feet (Ref. 16) below the lowest adjacent ground surface along Pier 1. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assuemd to begin 1-foot below the top of bedrock surface.
11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

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Subject:	WB Bridge # 6787 Pier 1B H-Pile Analysis Load Case 1	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	85.5	ft	Sheet B12 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.0	ft	
as-modeled pile cap midplane elevation =	84.0	ft	Sheet B12 (Ref. 5)
Base of pile cap elevation =	82.5	ft	Sheet B12 (Ref. 5)
Pile length =	19.5	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	5.0	ft	
Bottom of socket El. =	64.0	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	82.5	81.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	81.5	74.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	74.4	69.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	69.0	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 1 (Sheet X Interpretive Subsurface Cross Section X-X').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 10)

Maximum axial force in the piles, P_u =	385.2	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	26.5	kip-ft	=	317.6	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	31.2	kip-ft	=	374.0	in-kips	
Maximum shear force in the piles, V_u =	9.1	kips				
Depth below pile head to fixity =	12.3	ft	=	Elev.	71.74	ft
Depth below pile cap to fixity =	10.8	ft	=	Elev.	71.74	ft

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.031	0.668	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.012	0.333	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	5	ft	Grout below pile toe:	0.5	ft	(Assumption 7)
Socket diameter:	30	in	(Assumption 4)			
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)					

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

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Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.5	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 15 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	70	ksf
Unit tip resistance q_p =	553	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	864	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	432	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
432	>	385	OK

CALCULATIONS

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Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	385	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	129.15	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 45 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$$\frac{b}{t} = 15.98 > 13.49 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

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Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	129.15	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	2,779	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.35 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 835 kips

CALCULATIONS

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For compression member cross sections containing and slender elements, the nominal compressive resistance, P_n shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	835	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	43.2	ksi	

A_{eff} = $A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	43.2	ksi	

wsp

CALCULATIONS

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$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = \quad 14.5$$

check $b/t =$ 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

$c_1 =$ 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
$c_2 =$ 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
$c_3 =$ 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

$F_{el} =$ 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

$b_e =$ 6.91 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff} :

$A_{eff} =$ 18.66 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

$P_n =$ 806.8 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.60 (Ref. 7, Article 6.5.4.2))
$P_r =$ 484 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural} =$ 565 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 385 kips (from Part B)

$P_{r,\ axial} =$ 484 kips > $P_u =$ 385 kips OK

CALCULATIONS

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F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	318	in-kips (from Part B)
M_{uy} =	374	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	318	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	374	in-kips	OK

wsp

CALCULATIONS

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G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.91 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 1B H-Pile Analysis Load Case 1	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.1 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.1 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 1, and an estimated pile length of 19.5 feet, a maximum axial force of 385.2 kips , the corresponding strong axis (y-axis) moment of 374.0 in-kips (31.2 ft-kips) occurs in the HP14x89 piles at Pier 1B in Pile 10 of the proposed Westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity in the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 4.5-foot pile embedment into a 5.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 1B Summary Table		
Pile Length (ft)		20
Depth below Pile Head to Fixity (ft)		12
Geotechnical Axial Resistance (kips)	Nominal	864
	Factored	432
Structural Axial Resistance (kips)	Nominal	835
	Factored	484
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.91
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

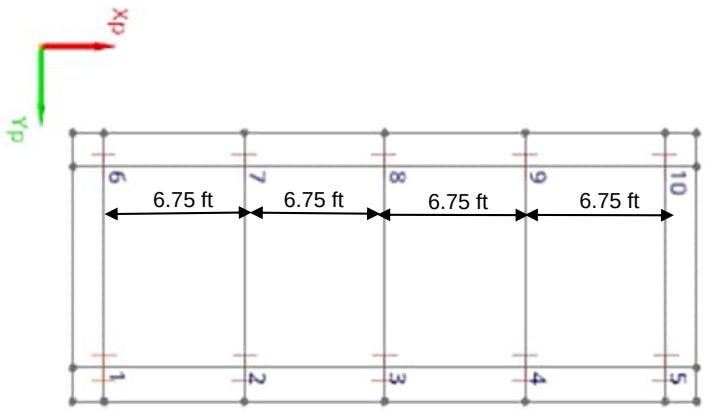


Figure 1b: Profile View of Pile Cap (Transverse)

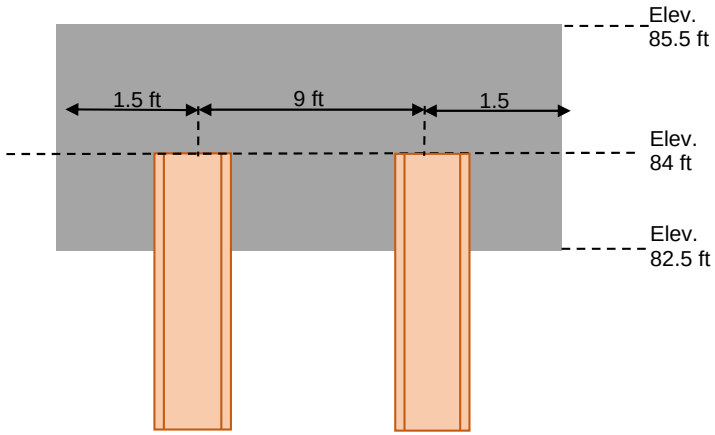


Figure 1c: 3D View of Pile Model

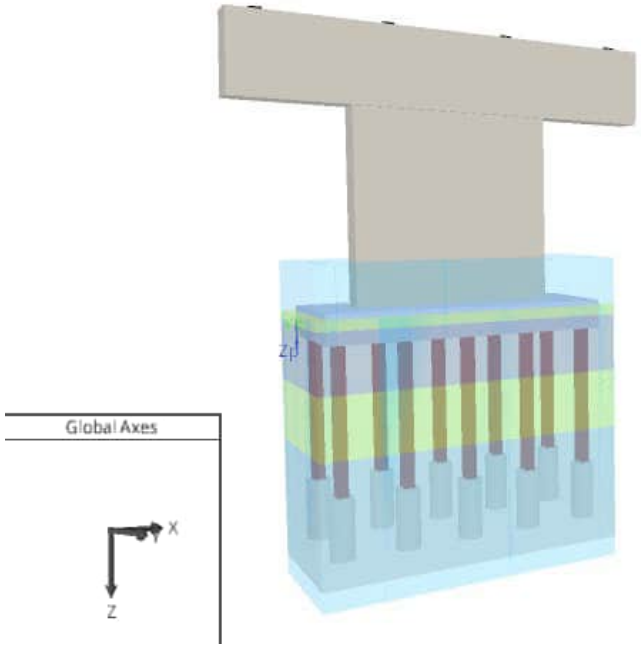
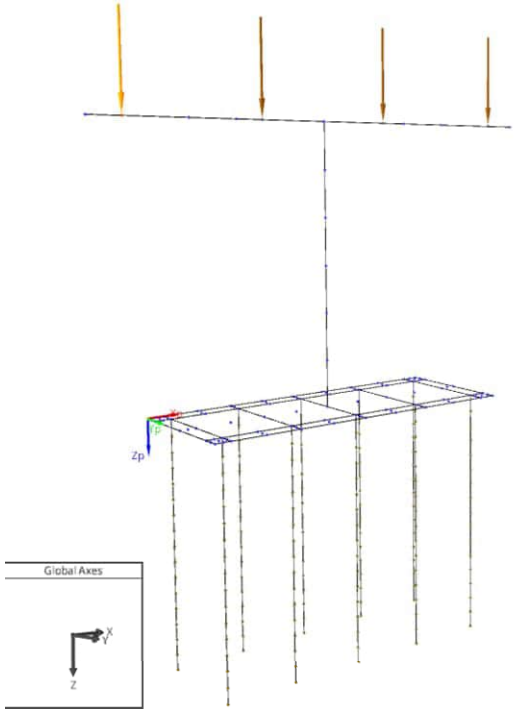


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	82.5	81.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	2	Cohesive	81.5	74.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	3	Cohesionless	74.4	69.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	333		-	-	-
Bedrock	4	Rock	69.0	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6446.4	864.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 1 profile (Sheet B12 Pier 1 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf*) . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

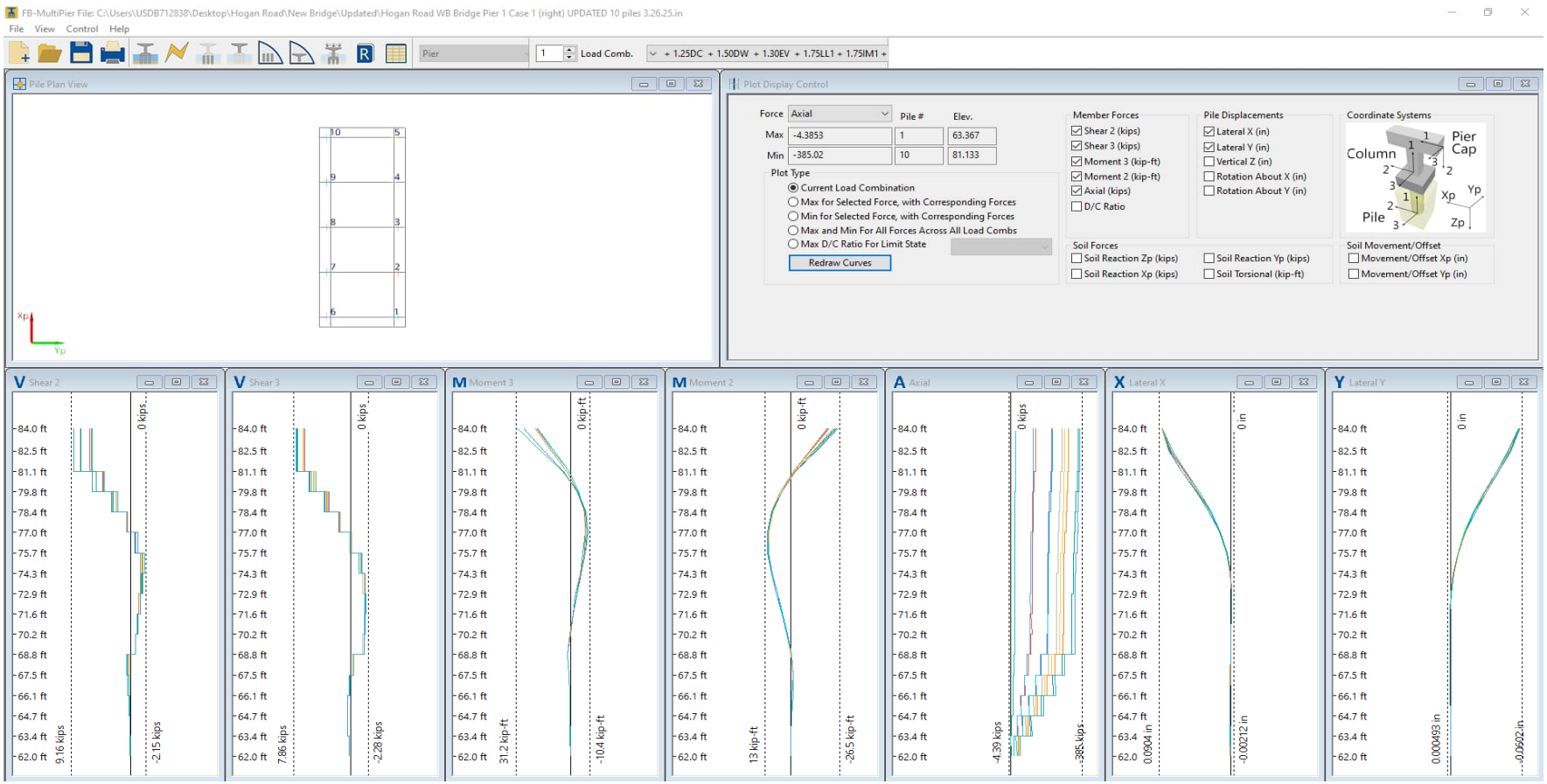
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

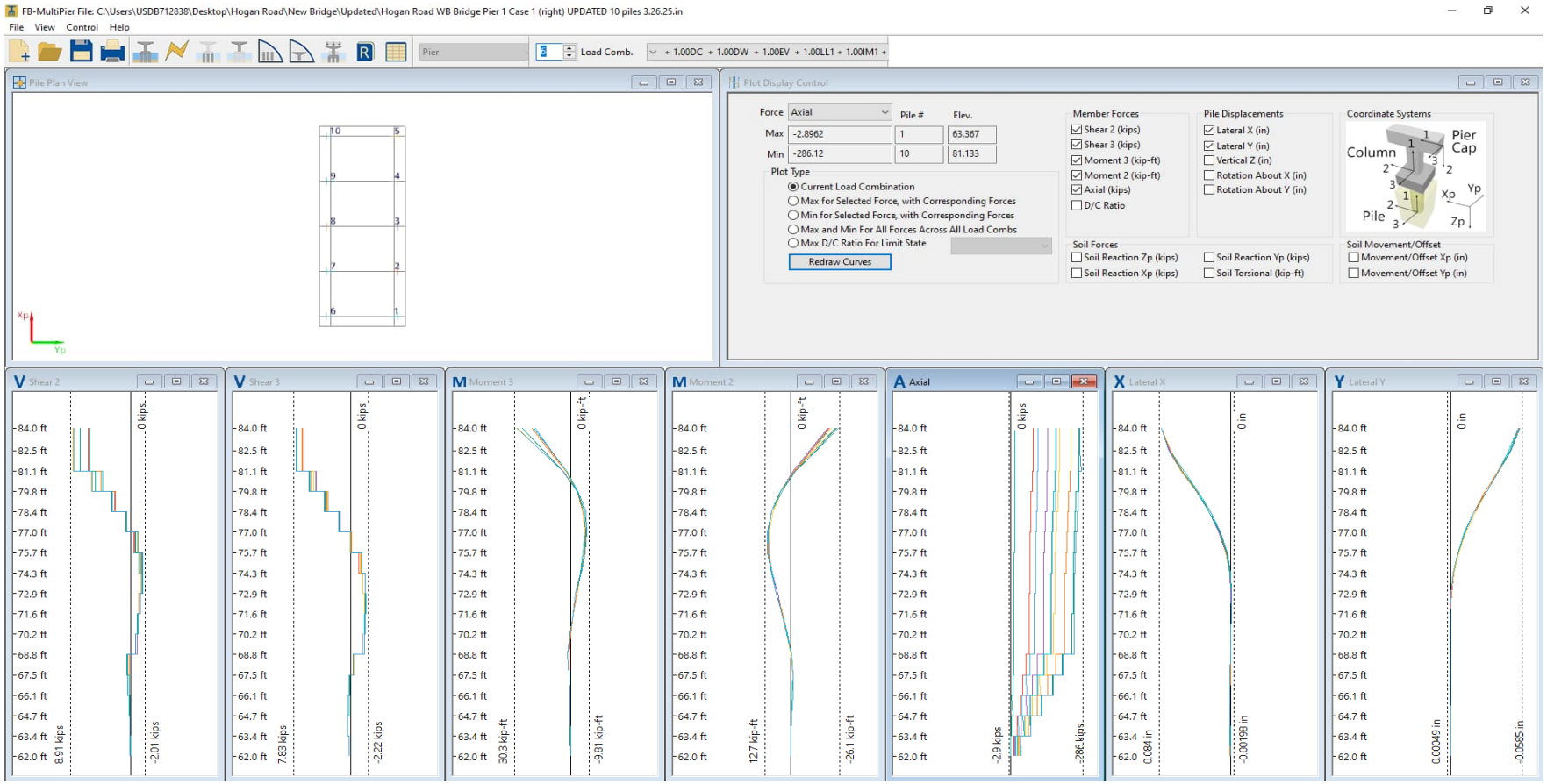
Prepared: DEB
Checked: ATM
Reviewed:

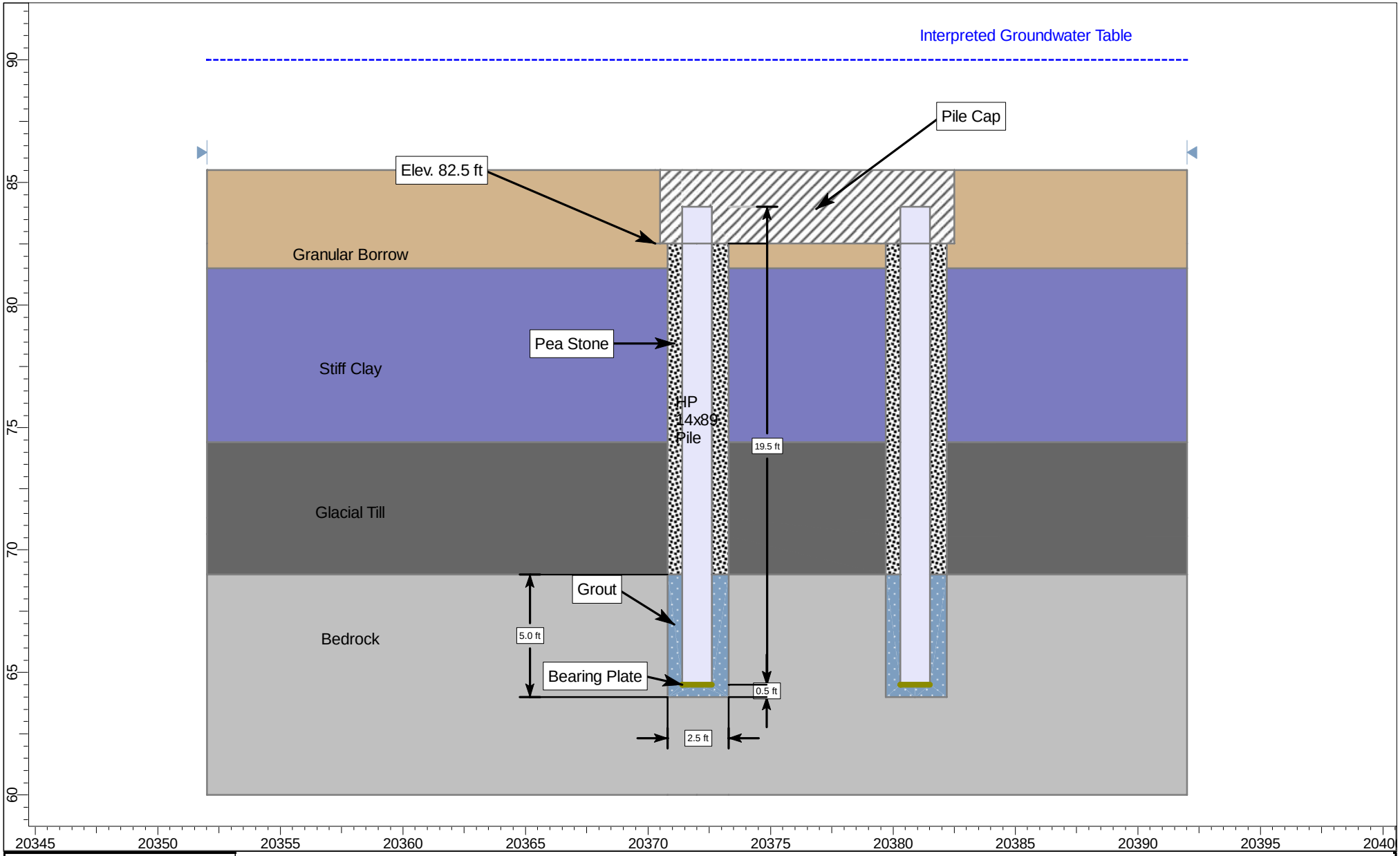
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 1 Strength I



Case 1 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 1B - Design Typical Section			
Drawn By: DEB	Checked By: ATM	Reviewed By: CCB	Scale: 1:65	Attachment 4
Date: 5/6/2025	File Name: Pier 1B sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

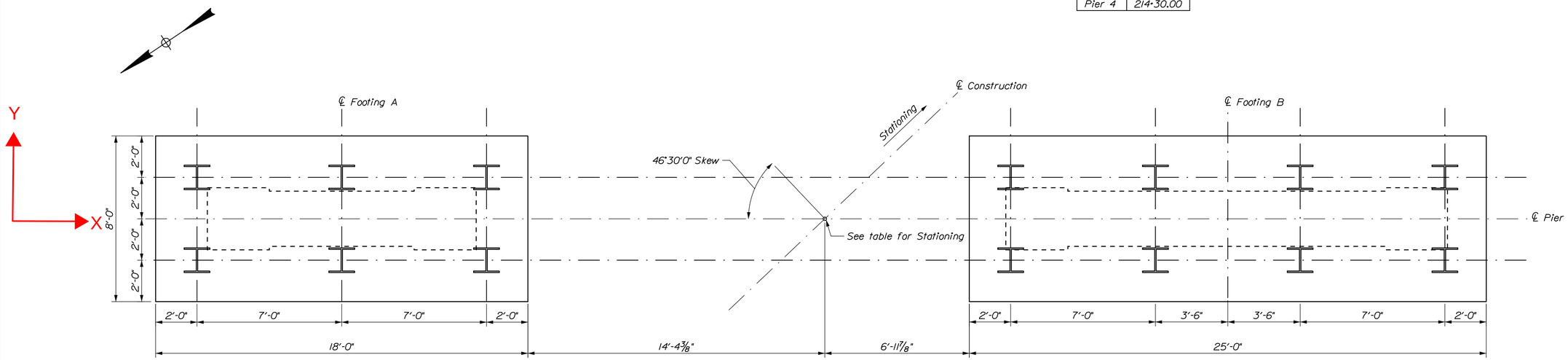
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

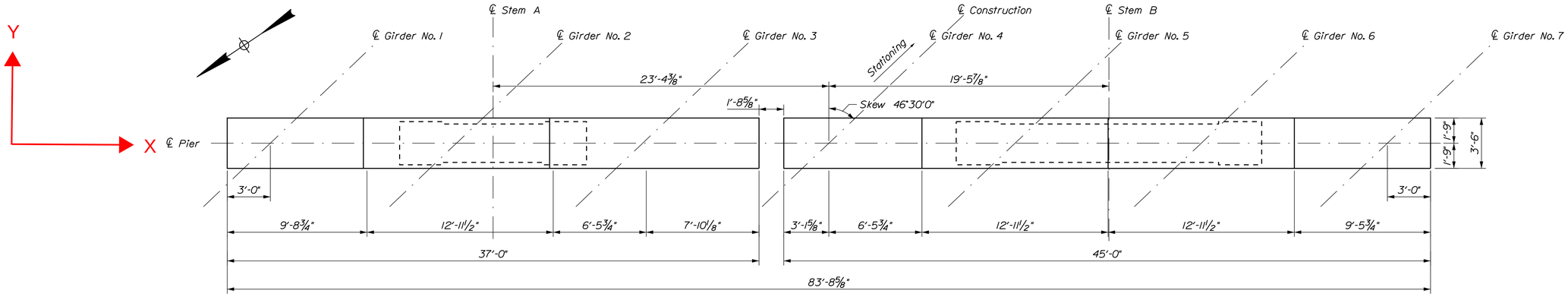
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

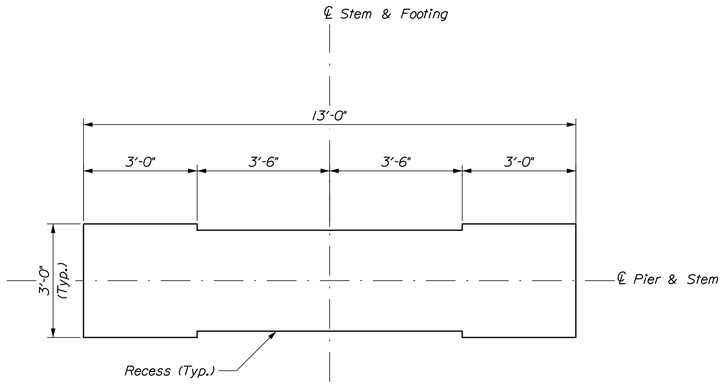
SHEET NUMBER

B25

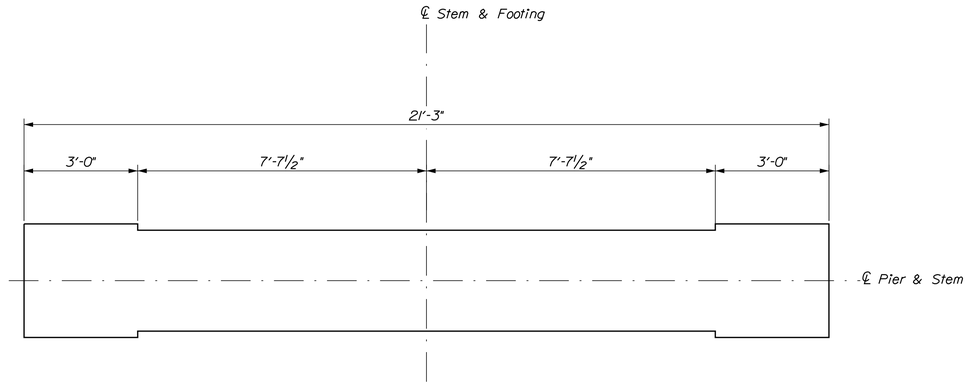
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

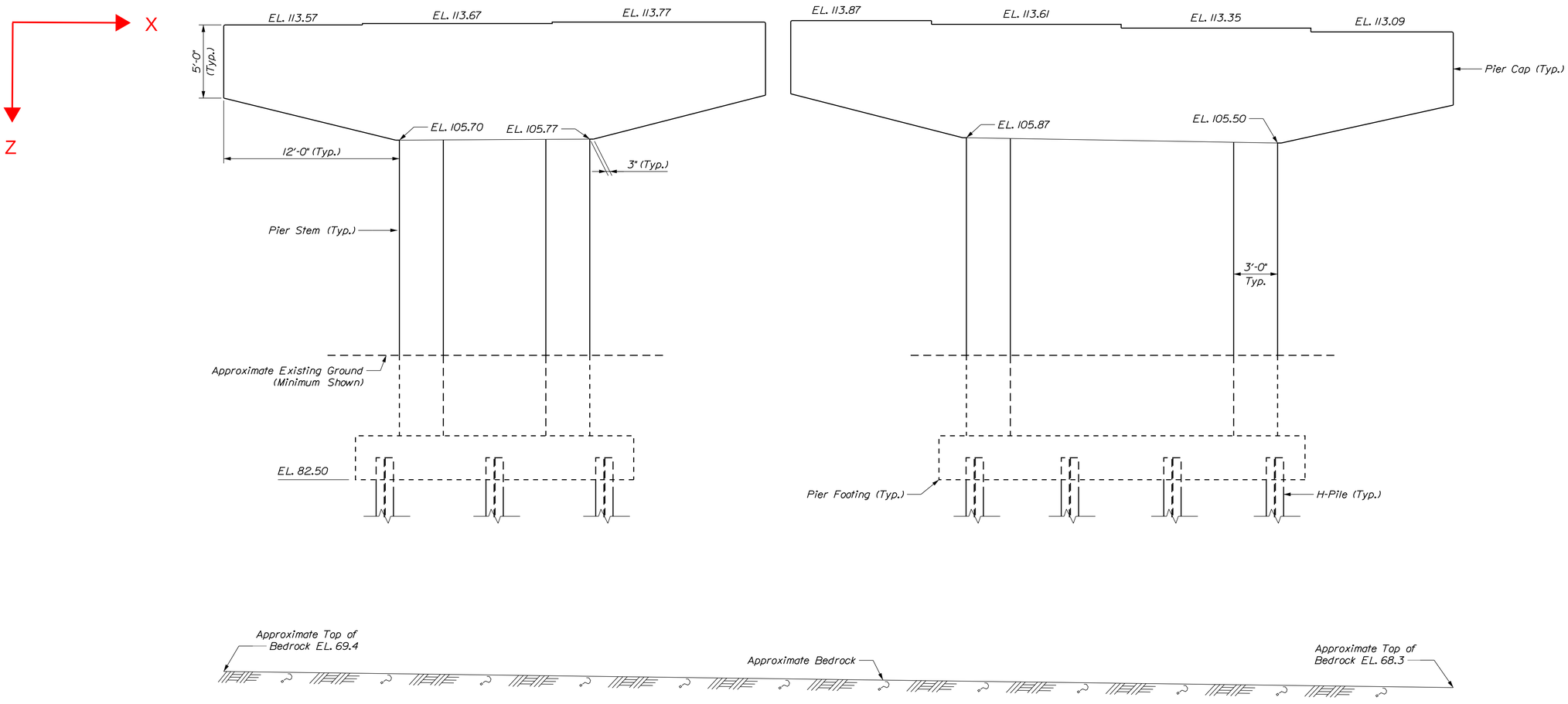
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	4/30/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 2A based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 2A (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

Date:	4/30/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier 2A Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 2A is assumed to be supported by eight (8) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 9 feet in the x-direction (ref. 5)
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 93.4 feet and the top of Pier 2A footing of El. 86 feet, resulting in 7.4 feet of overburden soil. Both pile caps for Pier 2 are located at the same footing elevation of 83 feet which is at the calculated frost depth of 7.3 feet (Ref. 15) below the lowest adjacent ground surface along Pier 2. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assumed to begin 1-foot below the top of bedrock surface.
11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

Date:	4/30/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	86.0	ft	Sheet B13 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.5	ft	
As-modeled pile cap midplane elevation =	84.5	ft	Sheet B13 (Ref. 5)
Base of pile cap elevation =	83.0	ft	Sheet B13 (Ref. 5)
Pile length =	14.7	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	3.0	ft	
Bottom of socket El. =	69.3	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

Date:	4/30/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	82.0	78.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	78.8	72.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	72.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 2 (Sheet X Interpretive Subsurface Cross Section X-X').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 1)					
Maximum axial force in the piles, P_u =	398.5	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	33.7	kip-ft	=	404.1	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	25.2	kip-ft	=	302.0	in-kips	
Maximum shear force in the piles, V_u =	9.3	kips				
Depth below pile head to fixity =	14.2	ft	=	Elev.	70.28	ft
Depth below pile cap to fixity =	12.7	ft	=	Elev.	70.28	ft

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'v =	0.031	0.473	ksf (at midpoint of soil layer using stratigraphy in Part A)
$f_s = \alpha s_u$			
f_s =	0.012	0.235	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	3	ft	Grout below pile toe:	0.5	ft	(Assumption 7)
Socket diameter:	30	in	(Assumption 4)			
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)					

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

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Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u \left[m_b \left(\frac{A}{q_u} \right) + s \right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u \left[m_b \frac{(\sigma'_{vb})}{q_u} + s \right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.3	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 15 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	65	ksf
Unit tip resistance q_p =	532	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	831	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	416	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
416	>	399	OK

CALCULATIONS

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Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	399	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	152.67	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 53 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$$\frac{b}{t} = 15.98 > 13.49 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

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Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	152.67	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	1,989	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.49 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 788 kips

CALCULATIONS

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For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	788	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	40.8	ksi	

$A_{eff} = A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	40.8	ksi	

CALCULATIONS

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$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = \quad 14.9$$

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

c_1 = 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
c_2 = 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
c_3 = 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

F_{el} = 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

b_e = 7.02 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff} :

A_{eff} = 18.87 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n = 769.9 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
φ_c = 0.60 (Ref. 7, Article 6.5.4.2))
P_r = 462 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
φ_c = 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$ = 539 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

P_u = 399 kips (from Part B)

$P_{r,\ axial}$ = 462 kips > P_u = 399 kips OK

CALCULATIONS

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F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	404	in-kips (from Part B)
M_{uy} =	302	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	404	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	302	in-kips	OK

wsp

CALCULATIONS

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G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \qquad \text{if } P_u/P_{r,\text{ axial/ flexural}} < 0.2 \qquad \text{(Ref. 7, Eq. 6.9.2.2.1-1)}$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \qquad \text{if } P_u/P_{r,\text{ axial/ flexural}} \ge 0.2 \qquad \text{(Ref. 7, Eq. 6.9.2.2.1-2)}$$

$P_u/P_{r,\text{ axial/ flexural}}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.95 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w \qquad \text{(Ref. 7, 6.10.9.2-2))}$$

$$V_n = CV_p = V_{cr} \qquad \text{(Ref. 7, 6.10.9.2-1)}$$

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}} \qquad \text{then C = 1.0} \qquad \text{(Ref. 7, Eq. 6.10.9.3.2-4)}$$

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

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Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.3 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.3 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 15.2 feet, a maximum axial force of 398.5 kips , the corresponding weak axis (x-axis) moment of 404.1 in-kips (33.7 ft-kips) occurs in the HP14x89 piles at Pier 2A in Pile 1 of the proposed Westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 2.5-foot pile embedment into a 3.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 2A Summary Table		
Pile Length (ft)		14.7
Depth below Pile Head to Fixity (ft)		14.2
Geotechnical Axial Resistance (kips)	Nominal	831
	Factored	416
Structural Axial Resistance (kips)	Nominal	788
	Factored	462
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.95
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

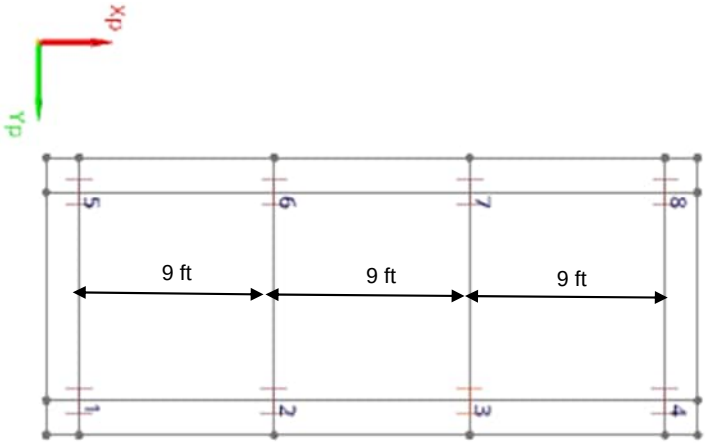


Figure 1b: Profile View of Pile Cap (Transverse)

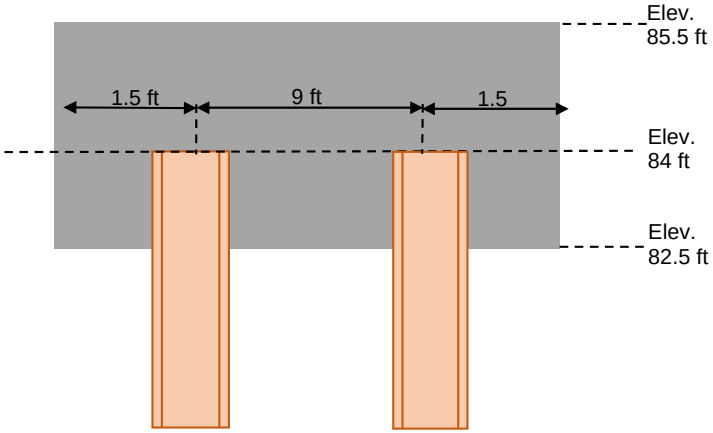


Figure 1c: 3D View of Pile Model

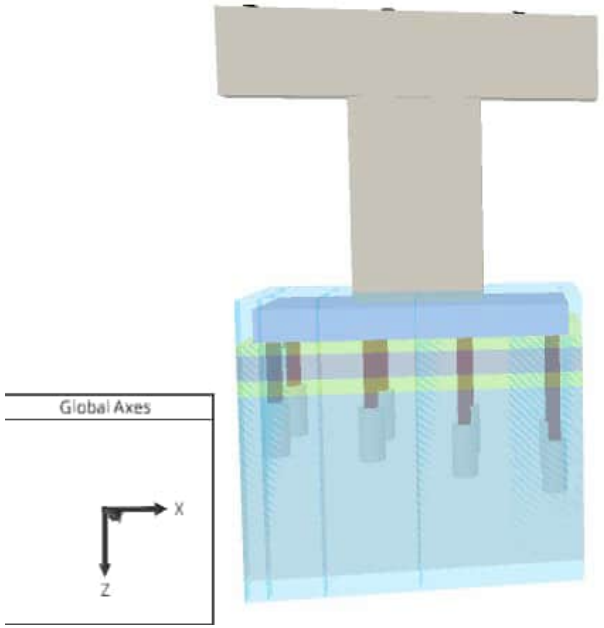
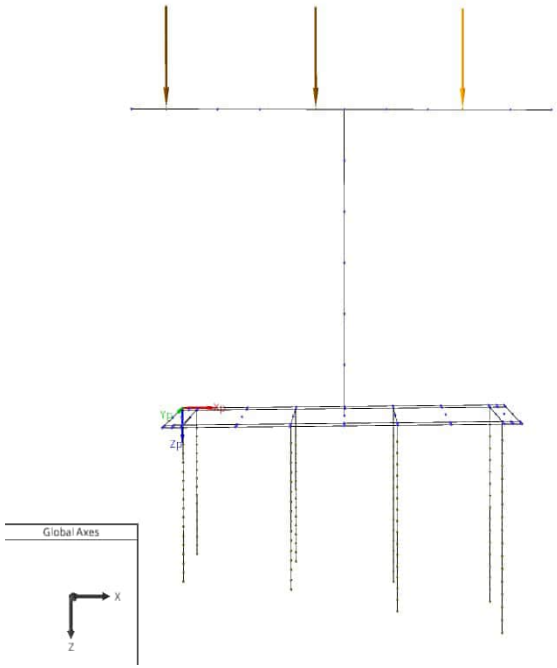


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	2	Cohesive	82.0	78.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	3	Cohesionless	78.8	72.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	235		-	-	-
Bedrock	4	Rock	72.3	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6200	831	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 3 profile (Sheet B14 Pier 3 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

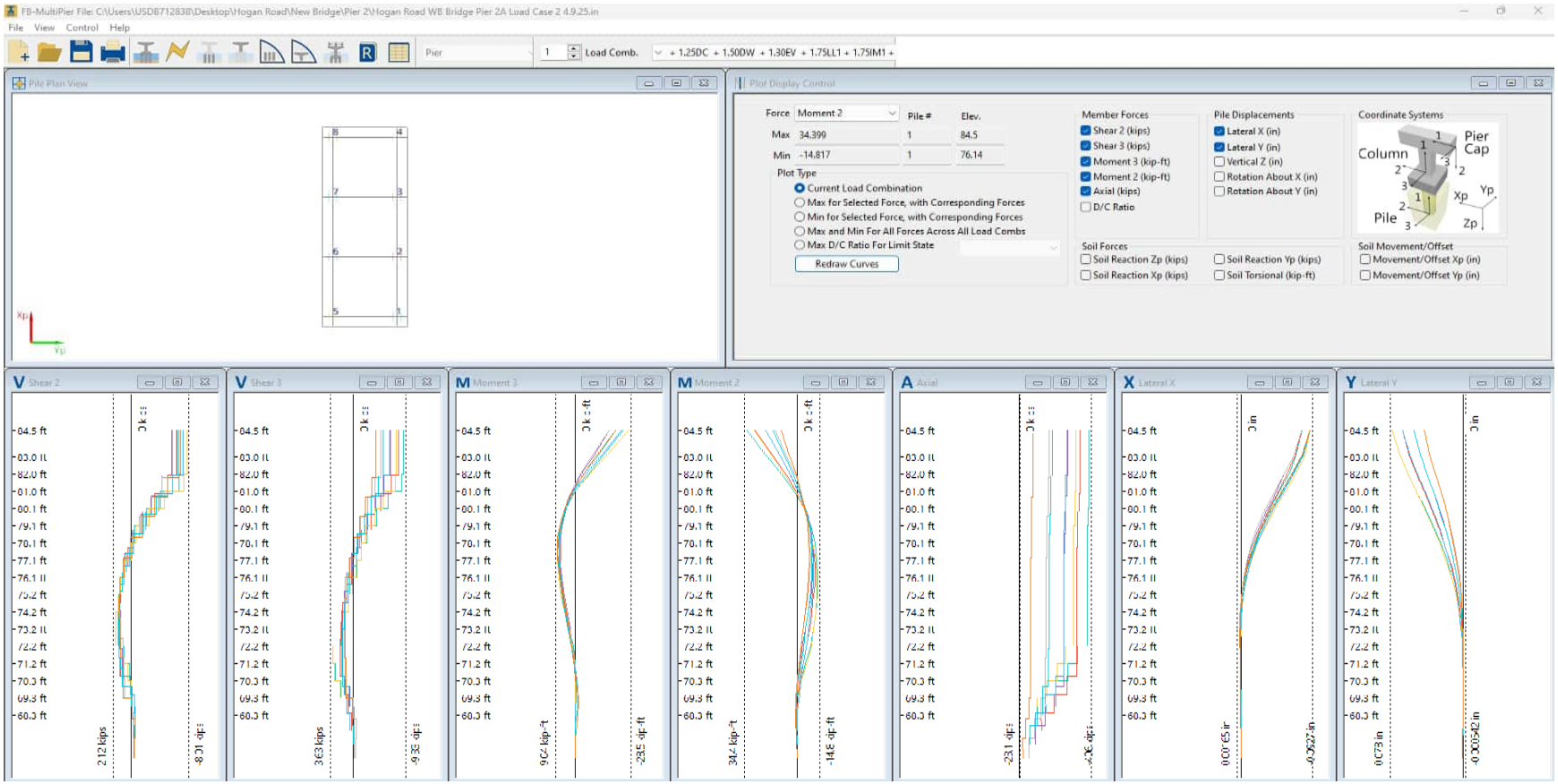
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

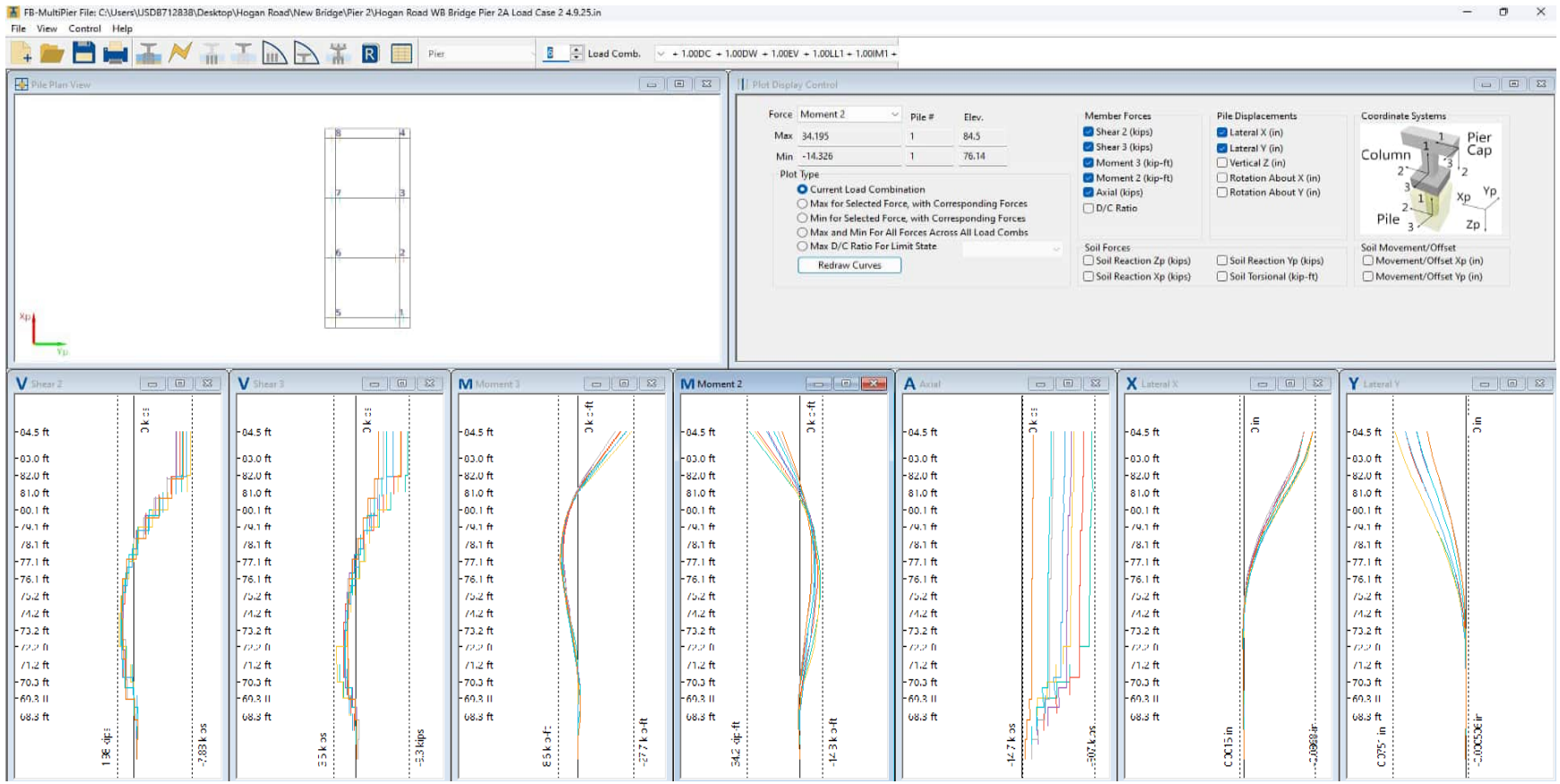
Prepared: DEB
Checked: ATM
Reviewed:

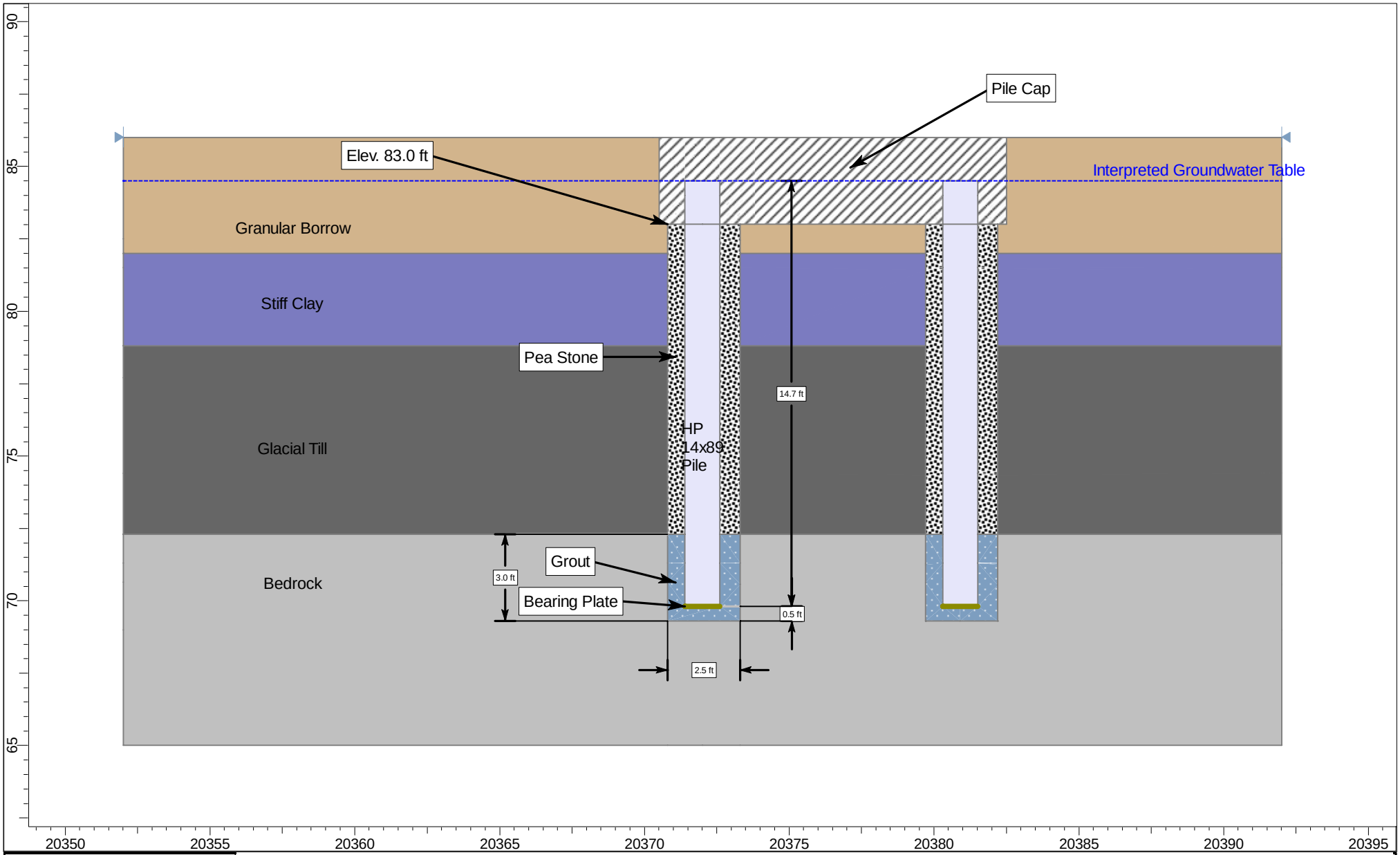
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 2A - Design Typical Section			
Drawn By DEB	Checked By ATM	Reviewed By CCB	Scale 1:55	Attachment 4
Date 5/6/2025	File Name Pier 2A sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

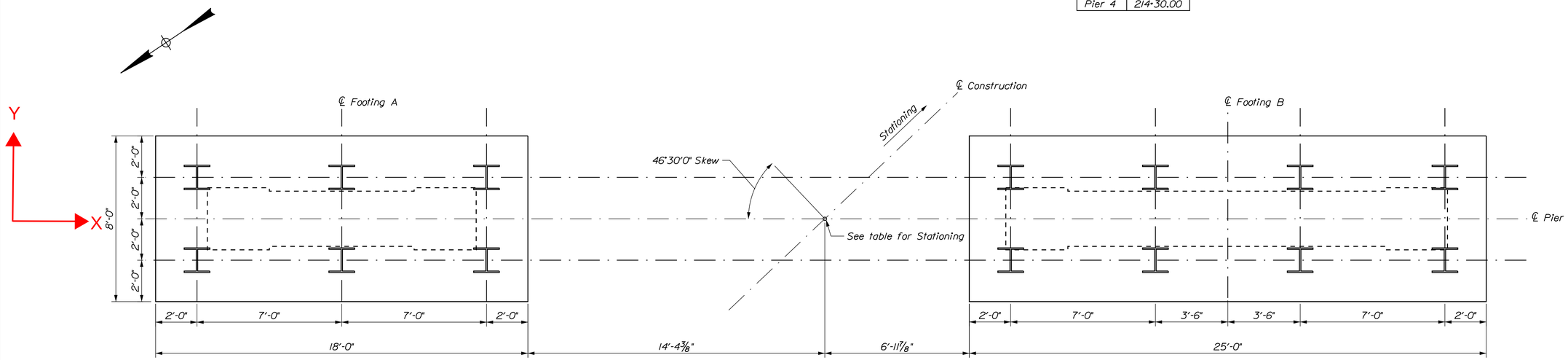
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

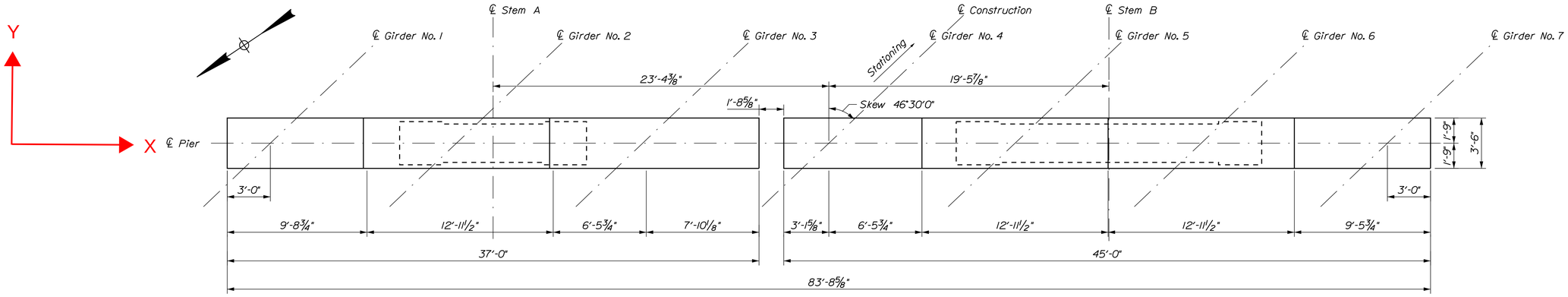
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

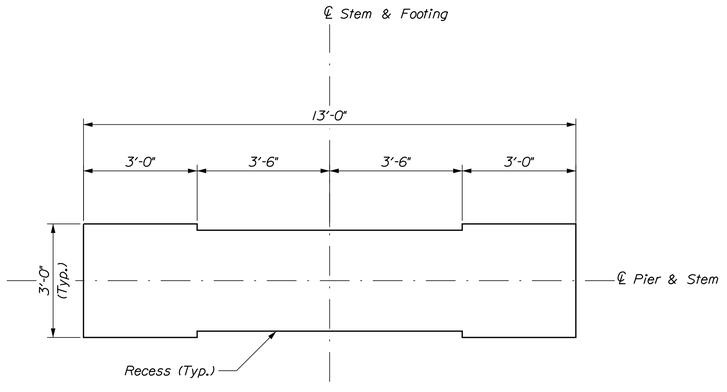
SHEET NUMBER

B25

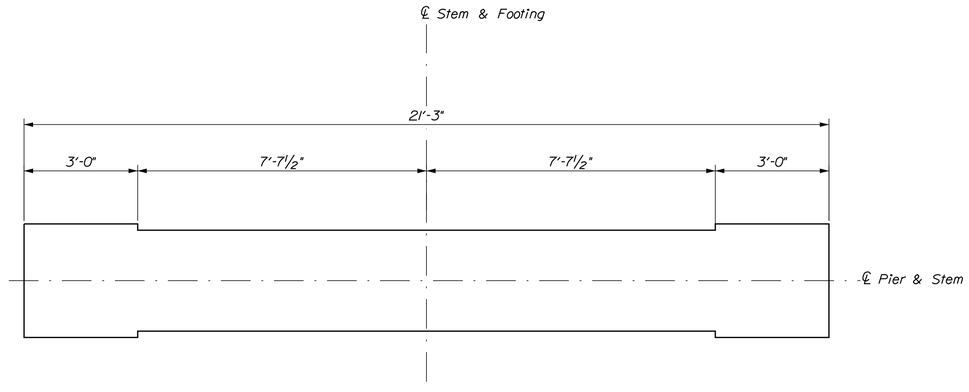
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

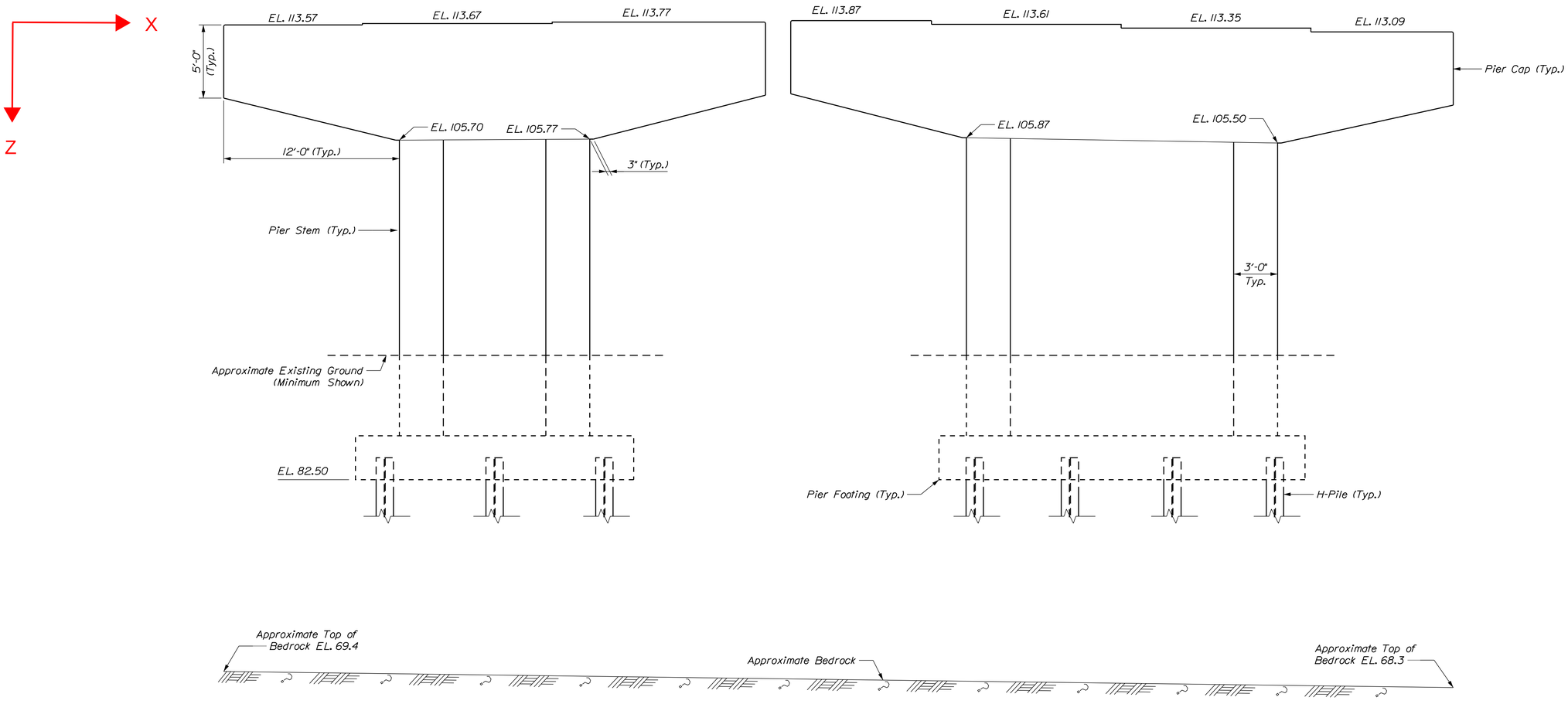
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 2B based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 2B (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier 2B Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 1 (right) is assumed to be supported by ten (10) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 6.75 feet in the x-direction (ref. 5)
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 92.3 feet and the top of Pier 2B footing of El. 86 feet, resulting in 6.3 feet of overburden soil. Both pile caps for Pier 2 are located at the same footing elevation of 83 feet which is at the calculated frost depth of 7.3 feet (Ref. 16) below the lowest adjacent ground surface along Pier 2. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assuemd to begin 1-foot below the top of bedrock surface.
11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	13.64	in	
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	14.5	in	
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Fillet area =	0.3	in^2		0.3	in^2	
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	19.3	in^2	
Section area =	0.18	ft^2		0.13	ft^2	
Moment of inertia about the x-axis, I_x =				653	in^4	
Moment of inertia about the y-axis, I_y =				233	in^4	
Radius of gyration about the x-axis, r_x =				5.82	in	
Radius of gyration about the y-axis, r_y =				3.47	in	
Elastic section modulus about the x-axis, S_x =				95.8	in^3	
Elastic section modulus about the y-axis, S_y =				32.1	in^3	
Plastic section modulus about the x-axis, Z_x =				105.7	in^3	
Plastic section modulus about the y-axis, Z_y =				48.8	in^3	

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	86.0	ft	Sheet B13, Ref. 5
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.5	ft	Sheet B13, Ref. 5
As-modeled pile cap midplane elevation =	84.5	ft	
Base of pile cap elevation =	83.0	ft	Sheet B13, Ref. 5
Pile length =	20.4	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	5.0	ft	
Bottom of socket El. =	63.6	ft, includes upper 1-foot of bedrock (Assumption 10).	

wsp

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	82.0	78.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	78.0	75.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	4	Cohesive	75.0	74.5	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	5	Cohesionless	74.5	68.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	6	Rock	68.6	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 2 (Sheet B13 Pier 2 Elevation (Ref. 5)).

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 1)

Maximum axial force in the piles, P_u =	385.7	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	25.5	kip-ft	=	306.4	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	31.7	kip-ft	=	380.7	in-kips	
Maximum shear force in the piles, V_u =	9.2	kips				
Depth below pile head to fixity =	12.6	ft	=	Elev.	71.87	ft
Depth below pile cap to fixity =	11.1	ft	=	Elev.	71.87	ft

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 2B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.031	0.706	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.012	0.351	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	5	ft	Grout below pile toe:	0.5	ft	(Assumption 7)
Socket diameter:	30	in	(Assumption 4)			
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)					

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.5	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 15 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	70	ksf
Unit tip resistance q_p =	553	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	864	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	432	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
432	>	386	OK

wsp

CALCULATIONS

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Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	386	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	133.54	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 46 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

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Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	133.54	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	2,599	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.37 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 827 kips

CALCULATIONS

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For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	827	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	42.8	ksi	

A_{eff} = $A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	42.8	ksi	

CALCULATIONS

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$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = \quad 14.6$$

check $b/t =$ 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

$c_1 =$ 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
$c_2 =$ 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
$c_3 =$ 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

$F_{el} =$ 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

$b_e =$ 6.93 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff} :

$A_{eff} =$ 18.70 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

$P_n =$ 800.3 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.60 (Ref. 7, Article 6.5.4.2))
$P_r =$ 480 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural} =$ 560 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 386 kips (from Part B)

$P_{r,\ axial} =$ 480 kips > $P_u =$ 386 kips OK

CALCULATIONS

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F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	306	in-kips (from Part B)
M_{uy} =	381	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	306	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	381	in-kips	OK

CALCULATIONS

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G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \quad \text{if } P_u/P_{r,\ axial/\ flexural} < 0.2 \quad \text{(Ref. 7, Eq. 6.9.2.2.1-1)}$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \quad \text{if } P_u/P_{r,\ axial/\ flexural} \ge 0.2 \quad \text{(Ref. 7, Eq. 6.9.2.2.1-2)}$$

$P_u/P_{r,\ axial/\ flexural}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.92 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w \quad \text{(Ref. 7, 6.10.9.2-2))}$$

$$V_n = CV_p = V_{cr} \quad \text{(Ref. 7, 6.10.9.2-1)}$$

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}} \quad \text{then C = 1.0} \quad \text{(Ref. 7, Eq. 6.10.9.3.2-4)}$$

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00	(Ref. 7, Article 6.5.4.2)
V_r =	168	kips

CALCULATIONS

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Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.2 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.2 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 20.4 feet, a maximum axial force of 385.7 kips , the corresponding strong axis (y-axis) moment of 380.7 in-kips (31.7 ft-kips) occurs in the HP14x89 piles at Pier 2B in Pile 1 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 4.5-foot pile embedment into a 5.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 2B Summary Table		
Pile Length (ft)		20.4
Depth below Pile Head to Fixity (ft)		12.6
Geotechnical Axial Resistance (kips)	Nominal	864
	Factored	432
Structural Axial Resistance (kips)	Nominal	827
	Factored	480
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.92
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

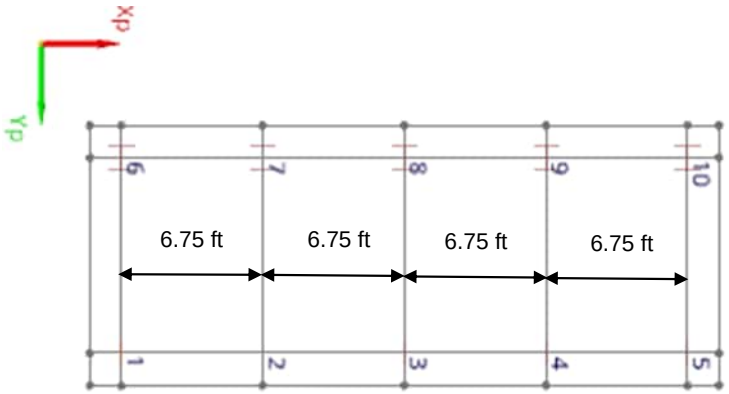


Figure 1b: Profile View of Pile Cap (Transverse)

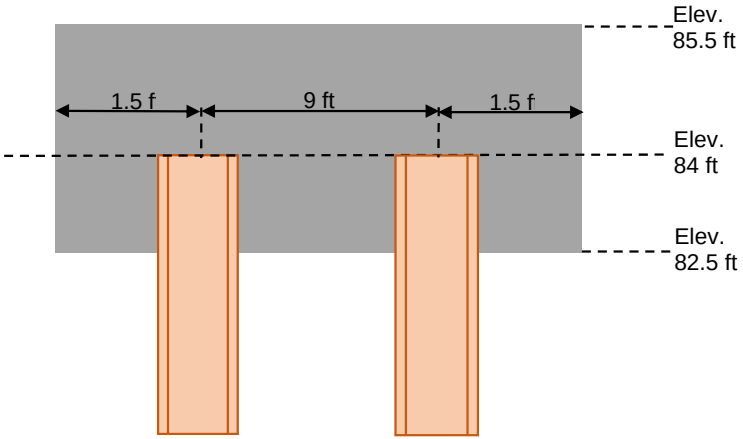


Figure 1c: 3D View of Pile Model

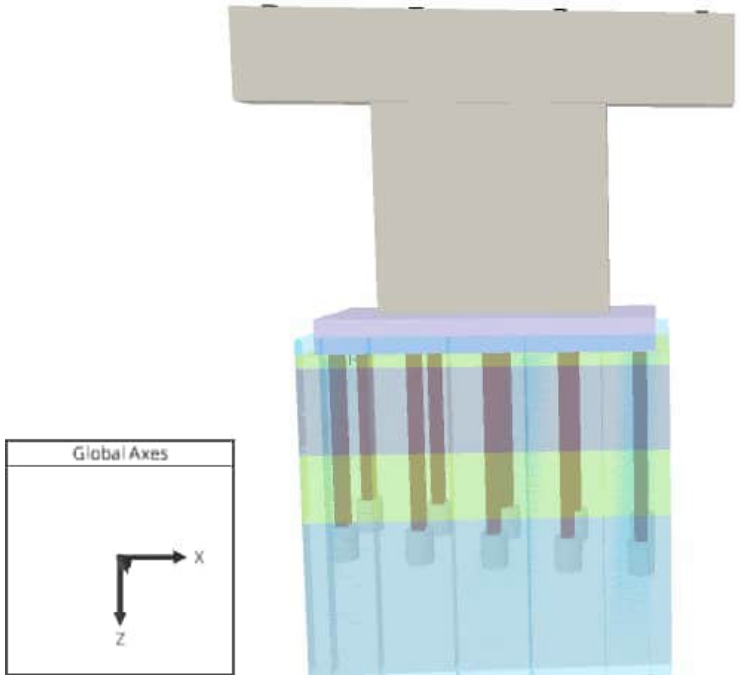
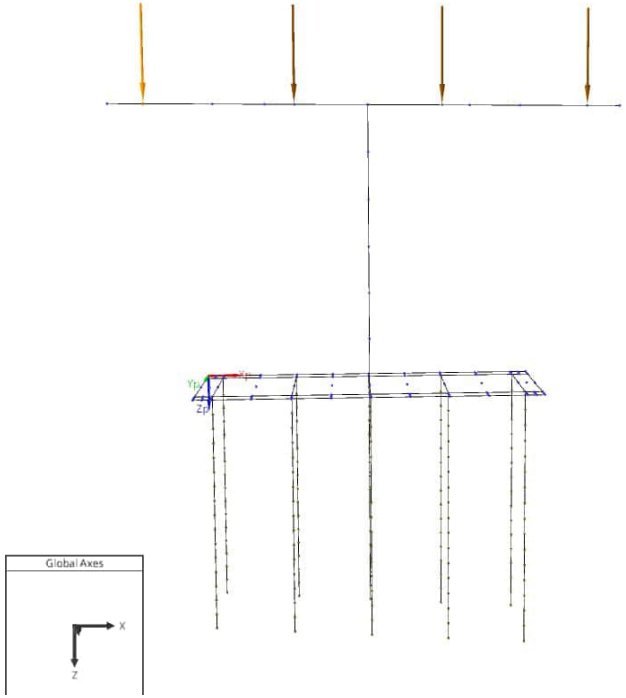


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	2	Cohesive	82.0	78.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medium Clay	3	Cohesive	78.0	75.0	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Stiff Clay	4	Cohesive	75.0	74.5	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	5	Cohesionless	74.5	68.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	351		-	-	-
Bedrock	6	Rock	68.6	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6200.2	831.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 2 profile (Sheet B13 Pier 2 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

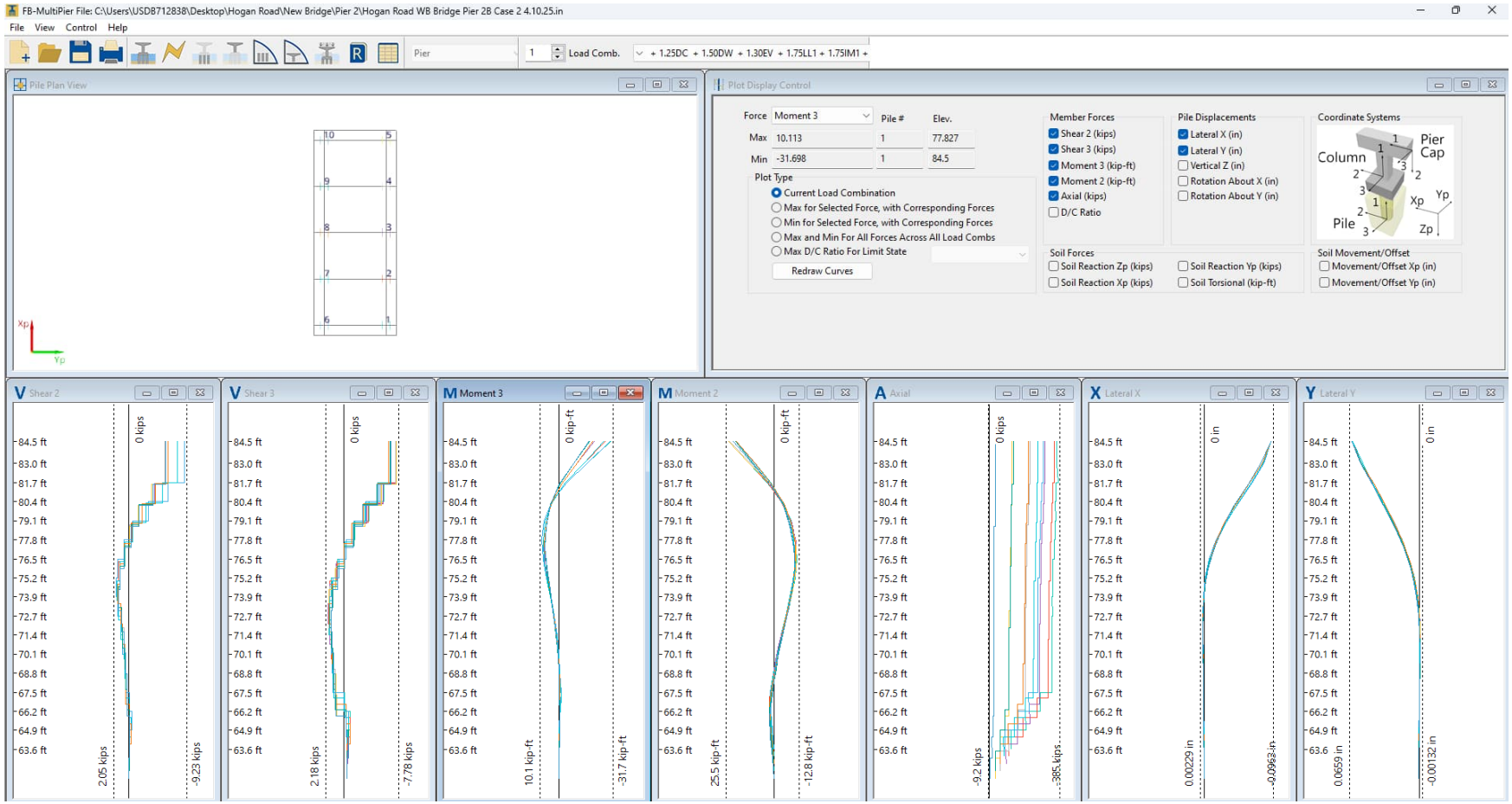
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

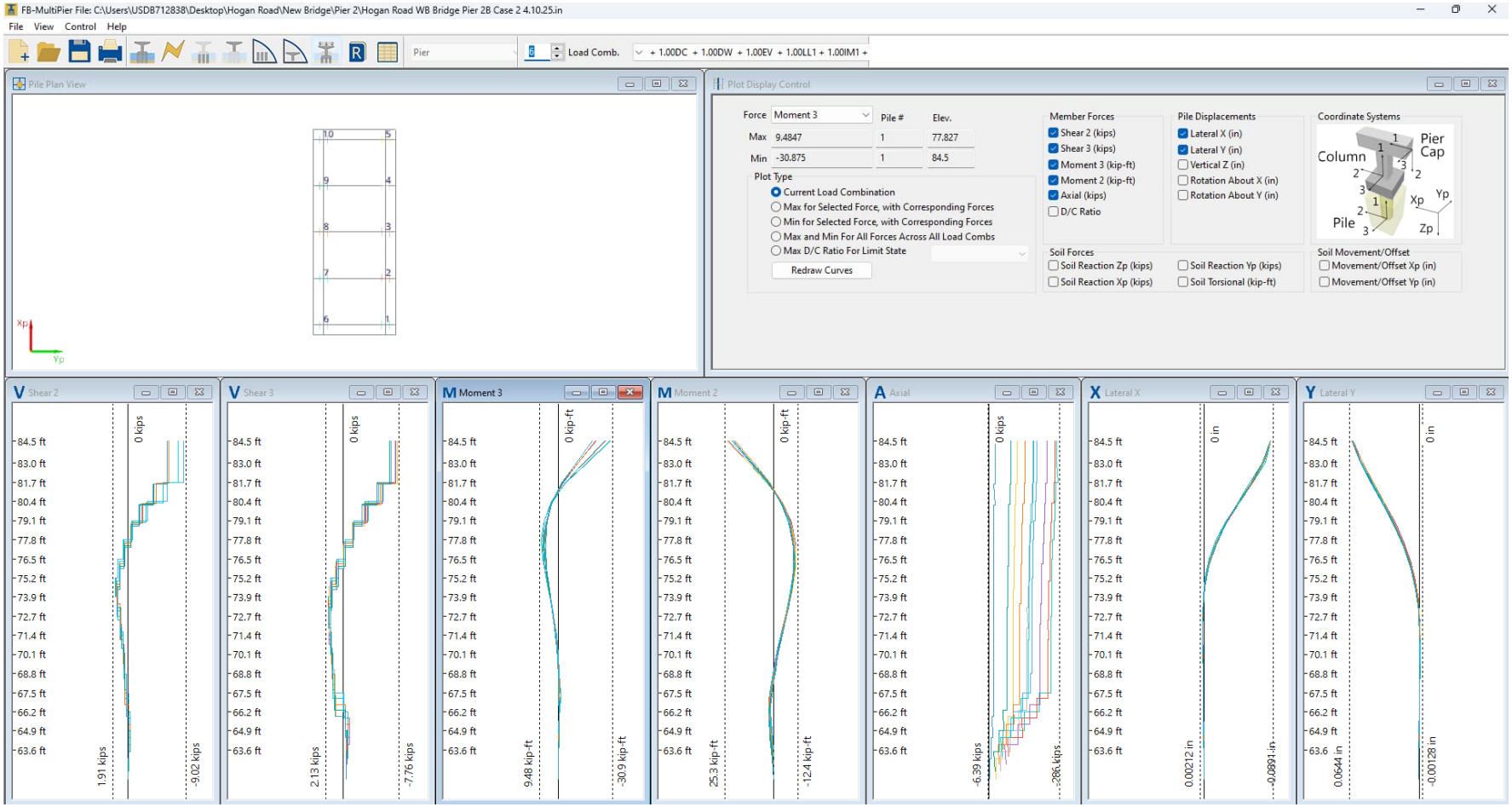
Prepared: DEB
Checked: ATM
Reviewed:

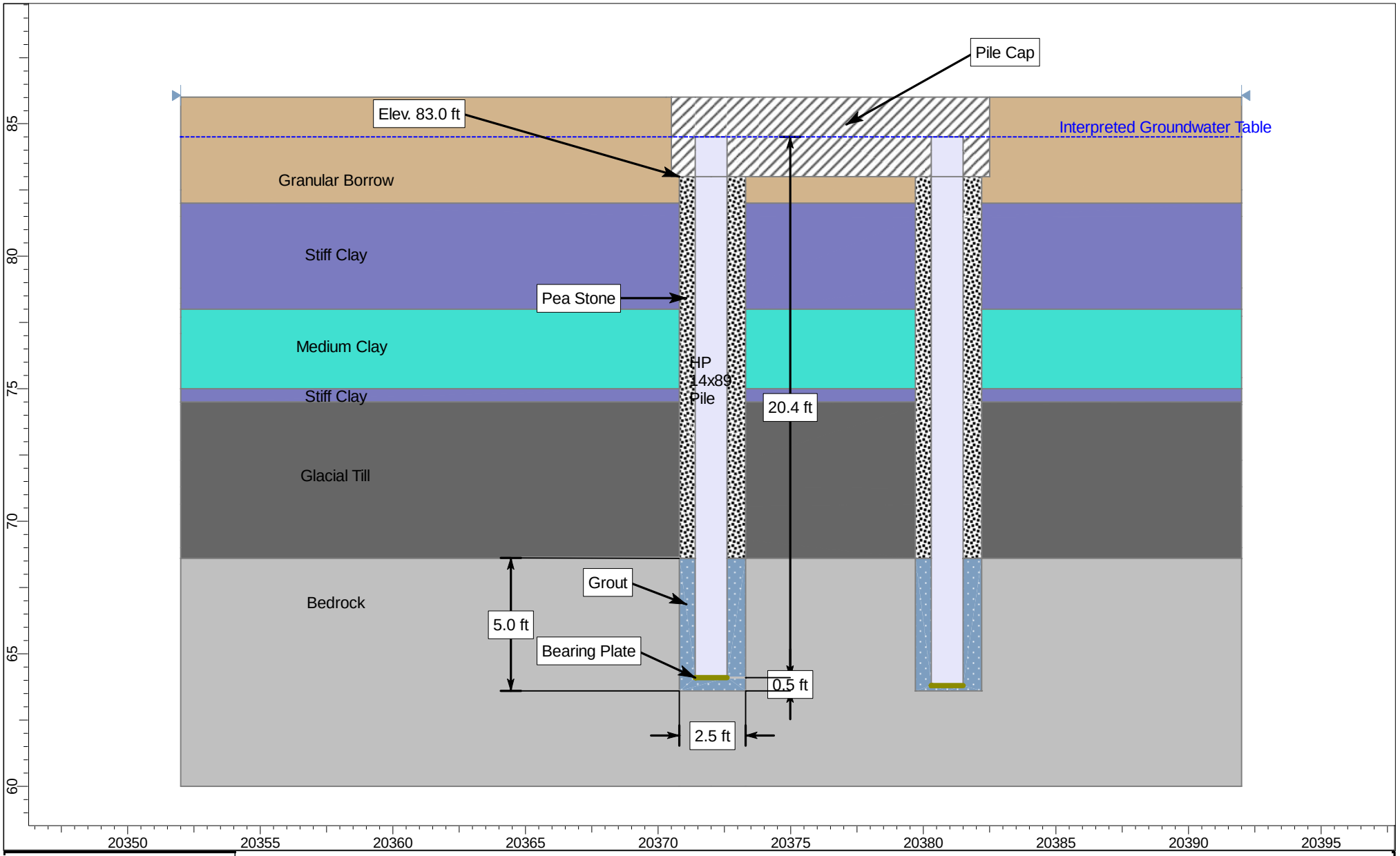
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 2B - Design Typical Section			
Drawn By DEB	Checked By ATM	Reviewed By CCB	Scale 1:60	Attachment 4
Date 5/6/2025	File Name Pier 2B sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

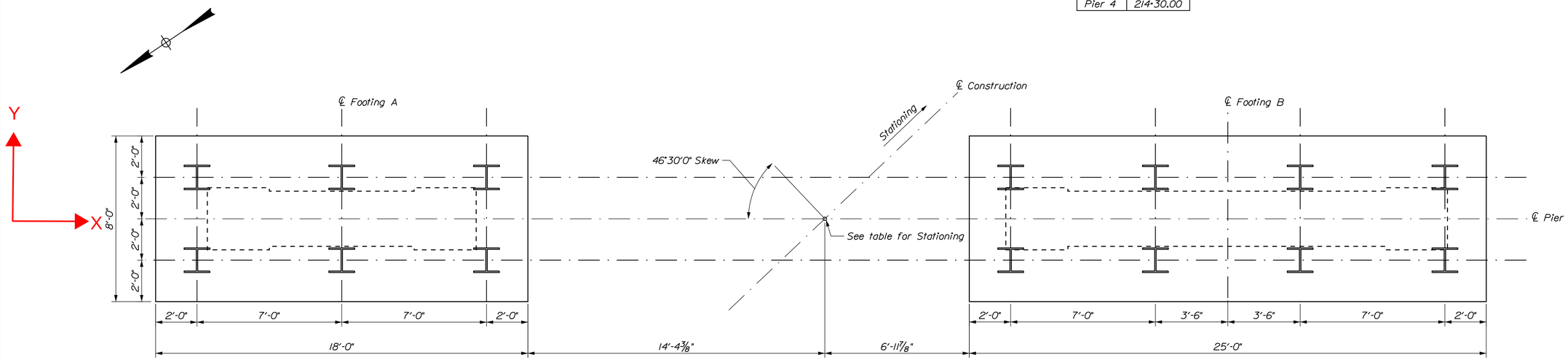
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

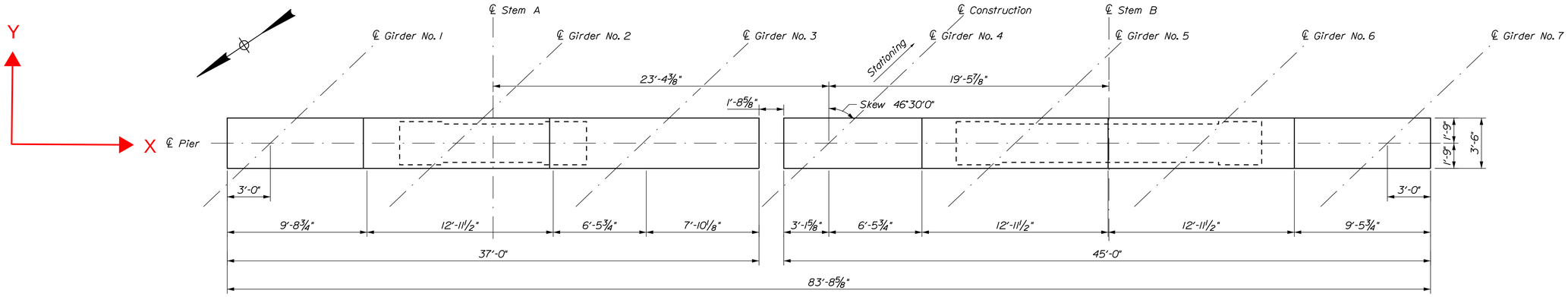
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

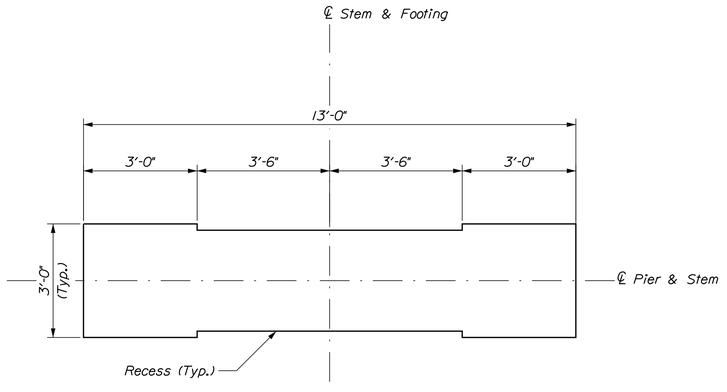
SHEET NUMBER

B25

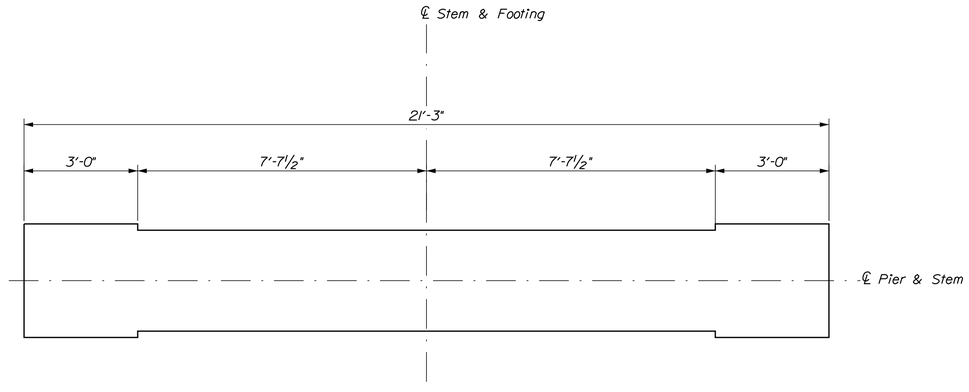
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

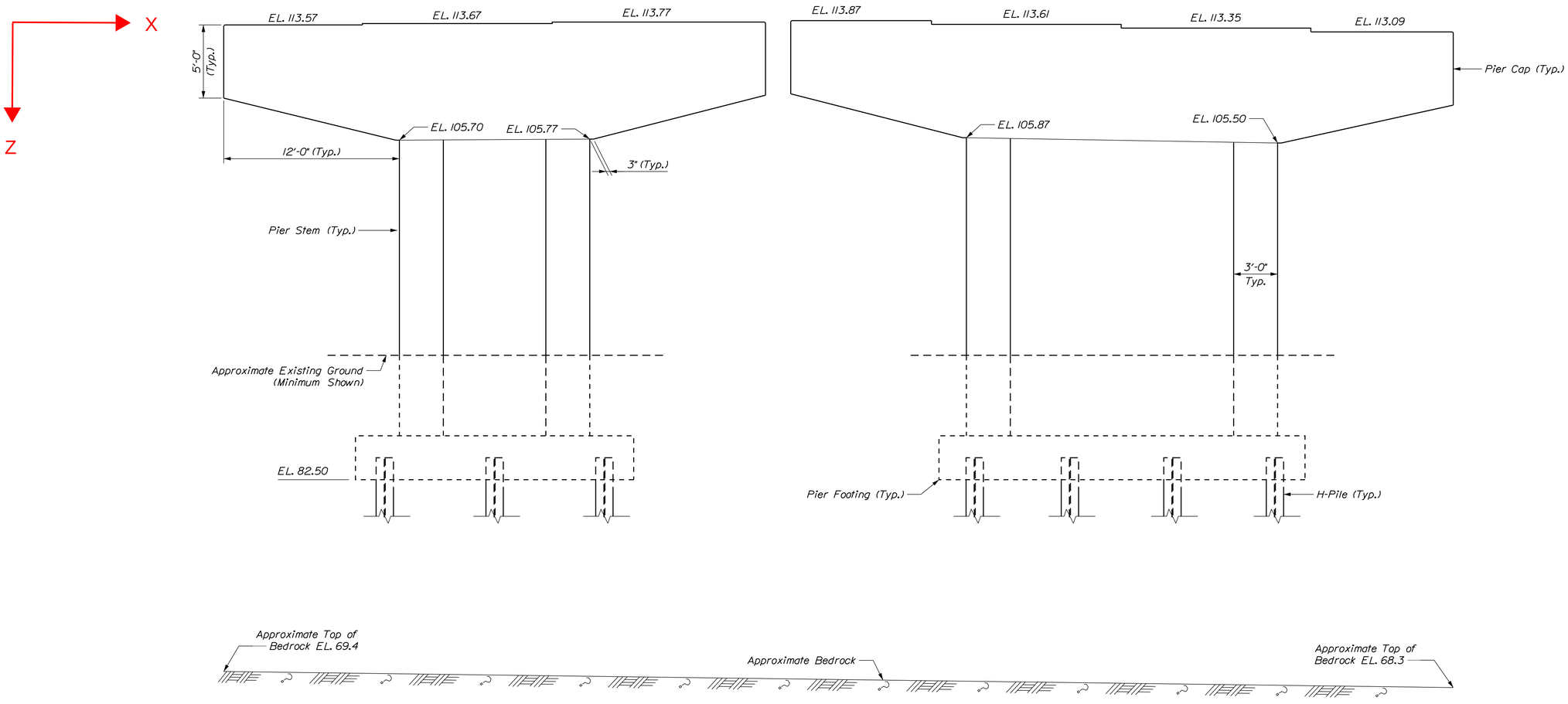
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 3A based on the preliminary design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 3A (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier 3A Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 3A is assumed to be supported by eight (8) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 9 feet in the x-direction (Attachment 1)
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 90.5 feet and the top of Pier 3A footing of El. 84.5 feet, resulting in 6 feet of overburden soil, both pile caps for Pier 3 are located at the same footing elevation of 81.5 feet which is at the calculated frost depth of 7.3 feet (Ref. 15) below the lowest adjacent ground surface along Pier 3. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assumed to begin 1-foot below the top of bedrock surface.
11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	84.5	ft	Sheet B14 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	83.0	ft	
As-modeled pile cap midplane elevation =	83.0	ft	Sheet B14 (Ref. 5)
Base of pile cap elevation =	81.5	ft	Sheet B14 (Ref. 5)
Pile length =	10.7	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	5.0	ft	
Bottom of socket El. =	71.8	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	81.5	80.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	80.5	78.2	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	78.2	76.8	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	76.8	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 3 (Sheet B14, Ref. 5).

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 1)					
Maximum axial force in the piles, P_u =	376.3	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	29.0	kip-ft	=	347.9	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	31.0	kip-ft	=	372.1	in-kips	
Maximum shear force in the piles, V_u =	9.5	kips				
Depth below pile head to fixity =	10.4	ft	=	Elev.	72.58	ft
Depth below pile cap to fixity =	8.9	ft	=	Elev.	72.58	ft

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.031	0.247	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.012	0.123	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	5	ft	Grout below pile toe:	0.5	ft	(Assumption 7)
Socket diameter:	30	in	(Assumption 4)			
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)					

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.5	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 14 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	70	ksf
Unit tip resistance q_p =	553	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	864	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	432	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
432	>	376	OK

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	376	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	107.01	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 37 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check: $\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	107.01	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	4,048	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.24 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 874 kips

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	874	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	45.2	ksi	

A_{eff} = $A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	45.2	ksi	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

$$\lambda_r\sqrt{\frac{F_y}{F_{cr}}} = \quad 14.2$$

check $b/t =$ 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

$c_1 =$ 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
$c_2 =$ 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
$c_3 =$ 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

$F_{el} =$ 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

$b_e =$ 6.82 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff}:

$A_{eff} =$ 18.50 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

$P_n =$ 836.9 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.60 (Ref. 7, Article 6.5.4.2))
$P_r =$ 502 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural} =$ 586 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 376 kips (from Part B)

$P_{r,\ axial} =$ 502 kips > $P_u =$ 376 kips OK

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	348	in-kips (from Part B)
M_{uy} =	372	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	348	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	372	in-kips	OK

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.6 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.88 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.5 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.5 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 10.7 feet, a maximum axial force of 376.3 kips , the corresponding strong axis (y-axis) moment of 372.1 in-kips (31.0 ft-kips) occurs in the HP14x89 piles at Pier 3A in Pile 1 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 4.5-foot pile embedment into a 5.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 3A Summary Table		
Pile Length (ft)		10.7
Depth below Pile Head to Fixity (ft)		10.4
Geotechnical Axial Resistance (kips)	Nominal	864
	Factored	432
Structural Axial Resistance (kips)	Nominal	874
	Factored	502
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.88
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

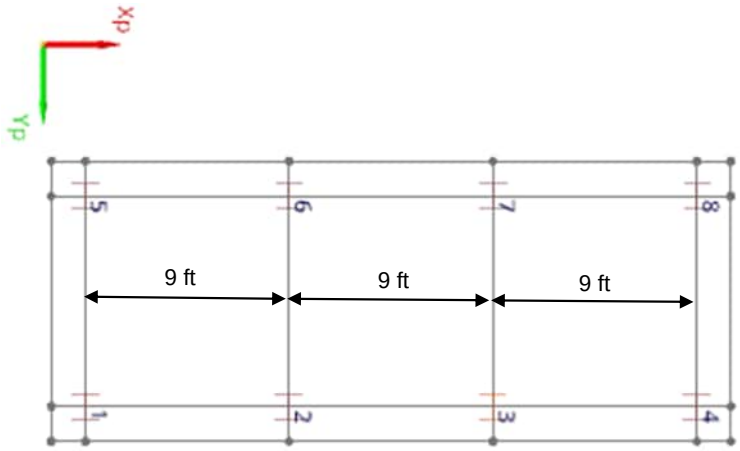


Figure 1b: Profile View of Pile Cap (Transverse)

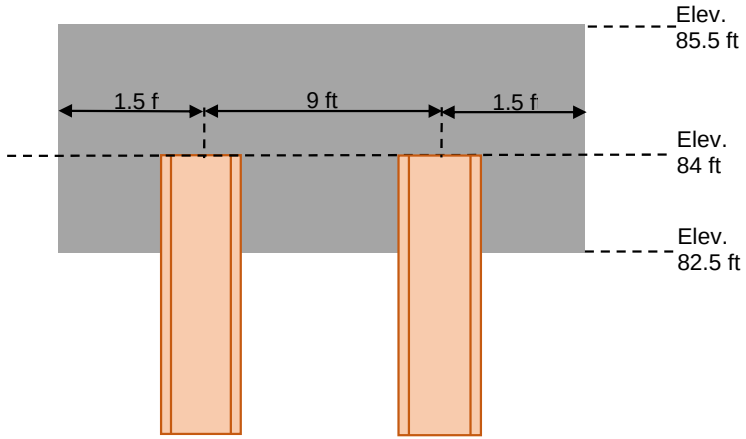


Figure 1c: 3D View of Pile Model

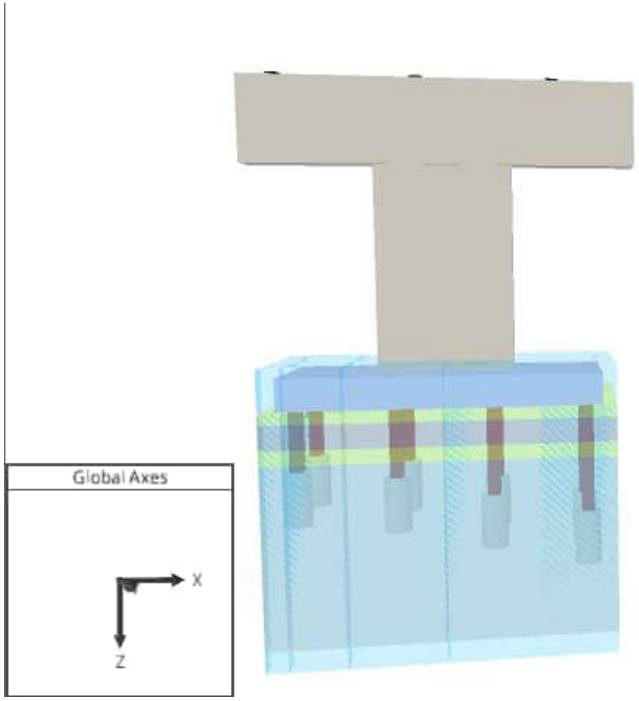
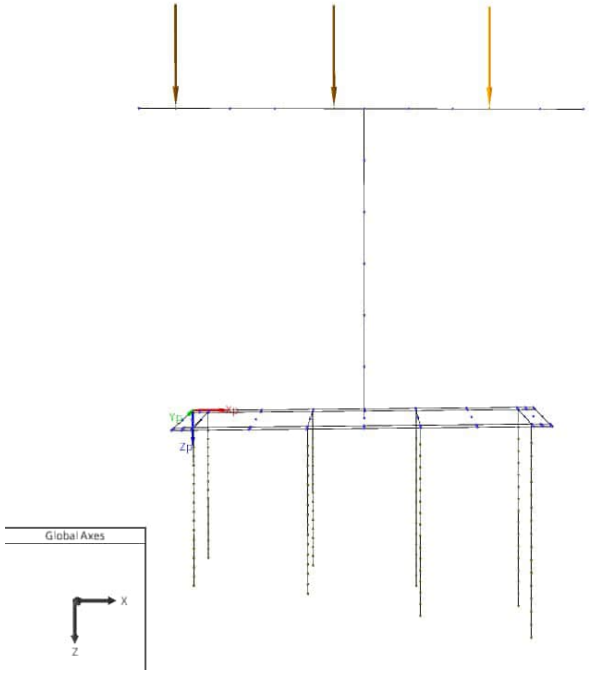


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	81.5	80.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	2	Cohesive	80.5	78.2	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Glacial Till	3	Cohesionless	78.2	76.8	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	123		-	-	-
Bedrock	4	Rock	76.8	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6565.8	880.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 3 profile (Sheet B14 Pier 3 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf*) . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

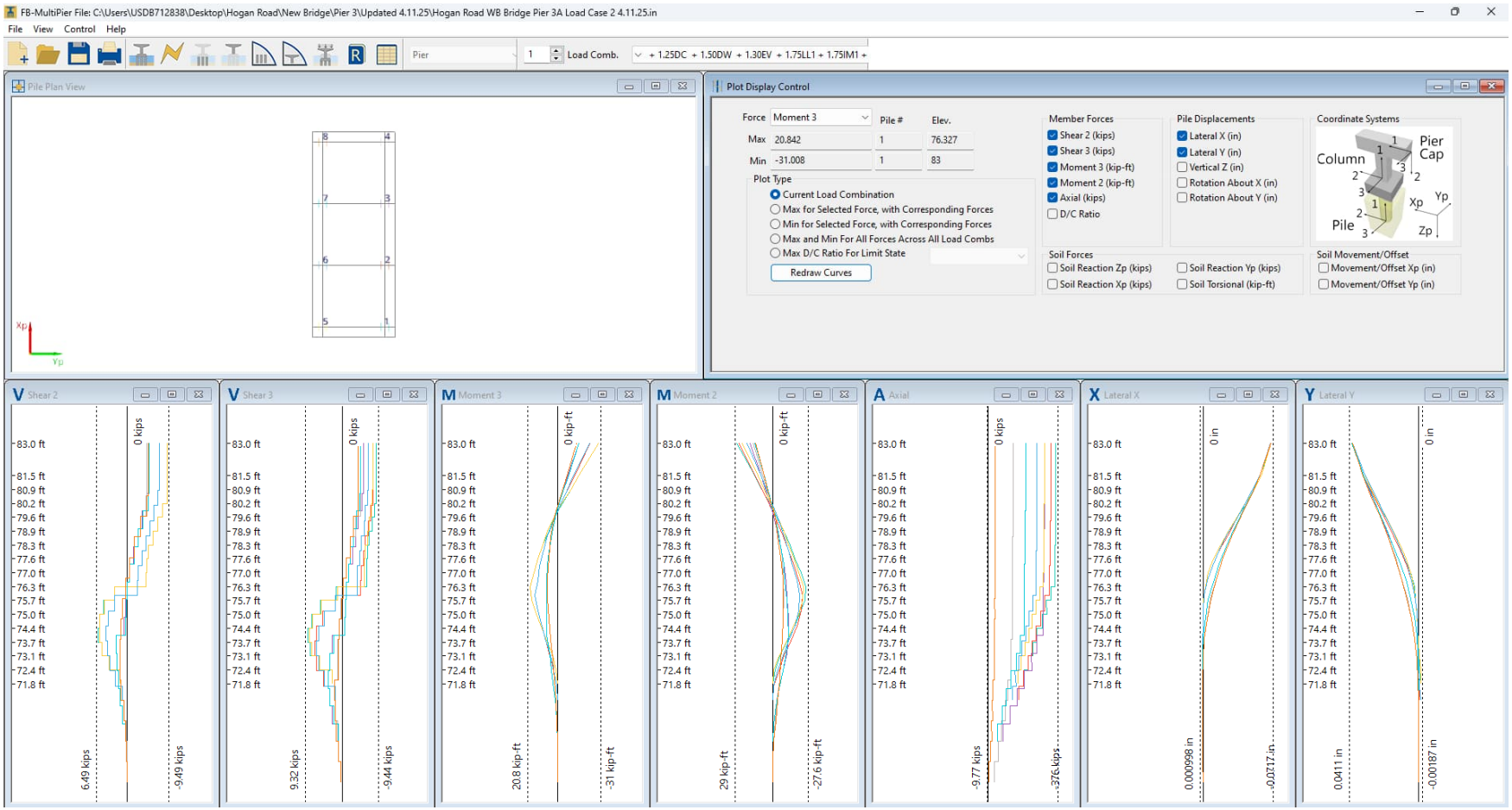
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

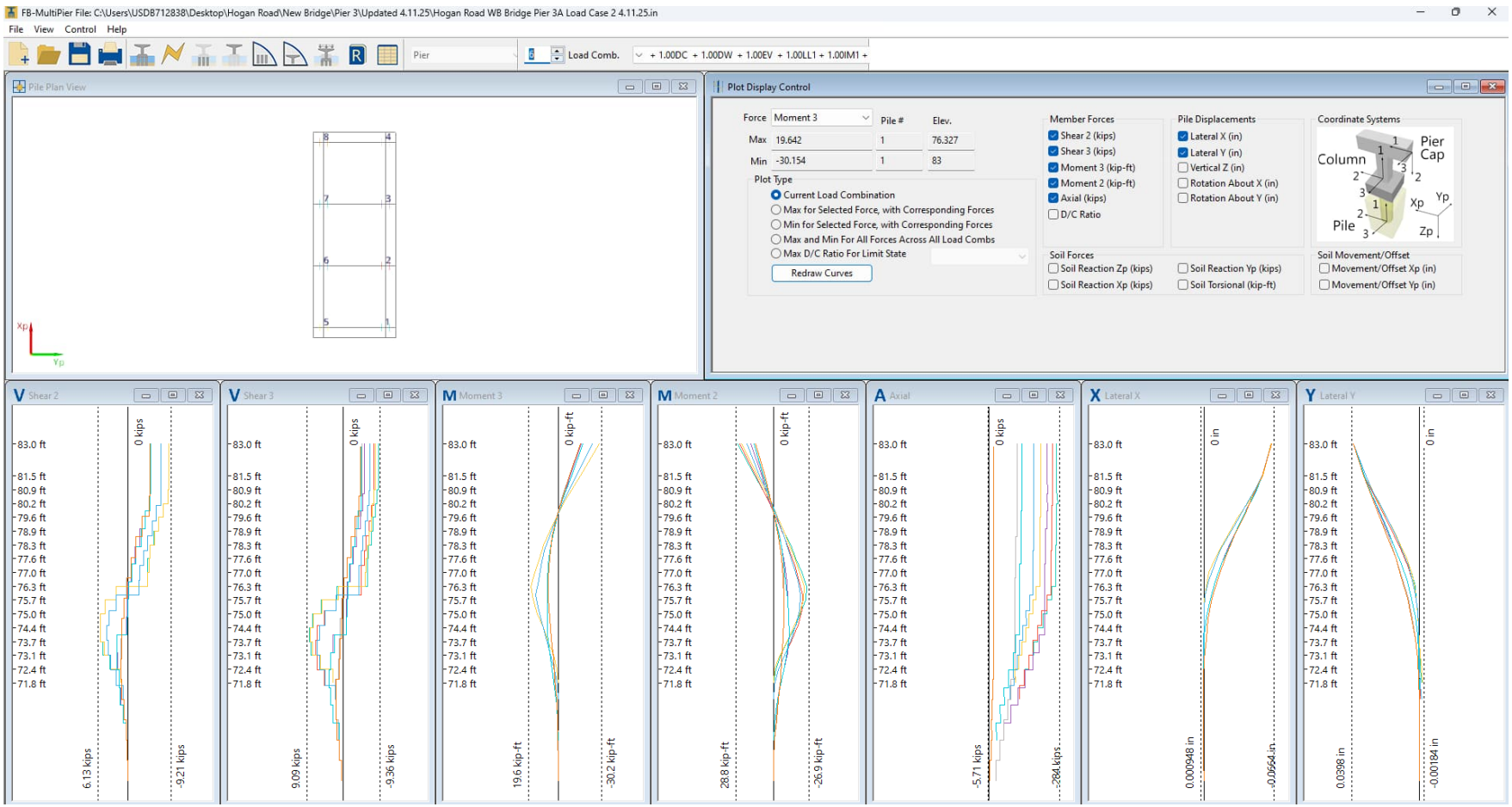
Prepared: DEB
Checked: ATM
Reviewed:

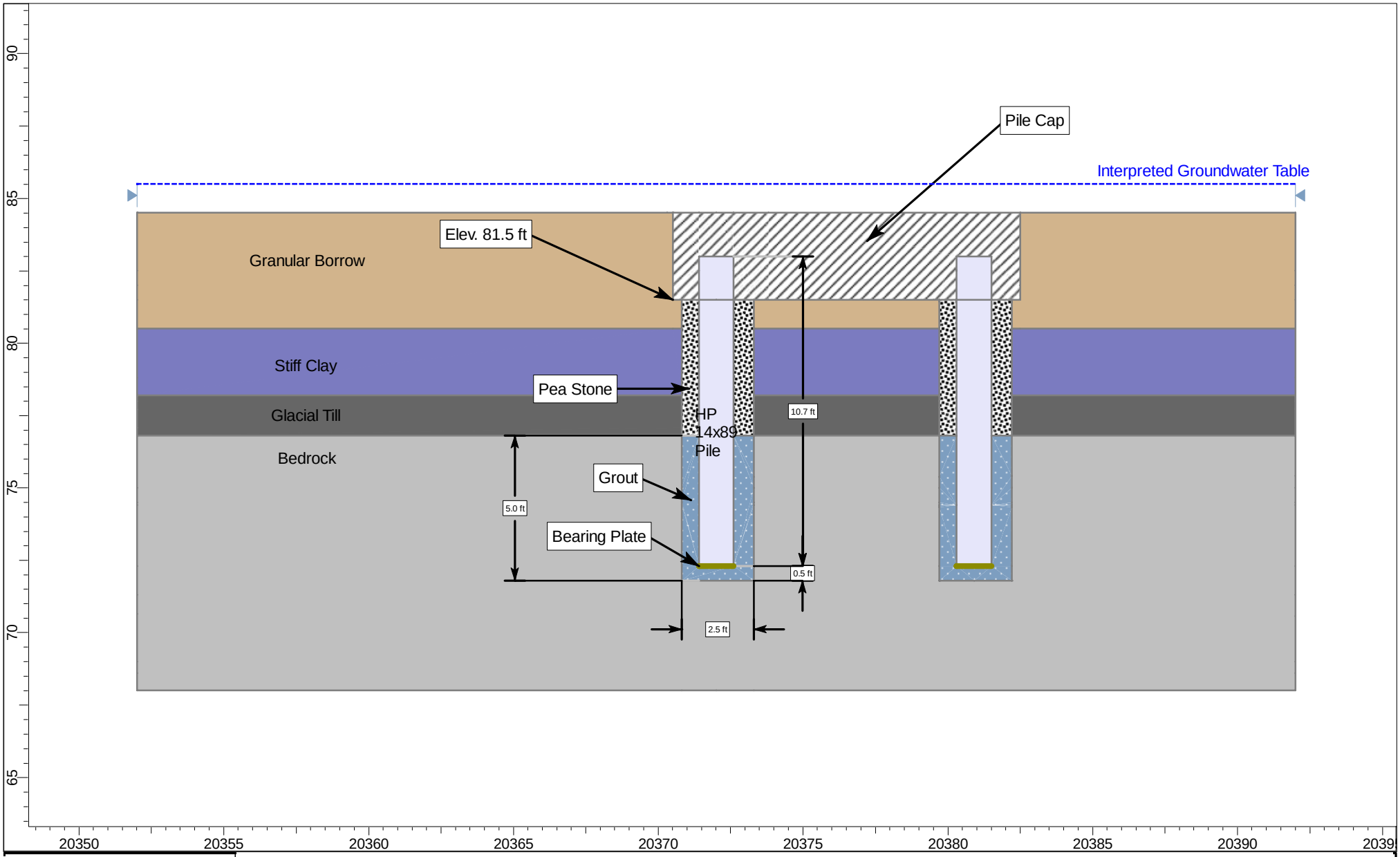
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 3A - Design Typical Section			
Drawn By: DEB	Checked By: ATM	Reviewed By: CCB	Scale: 1:55	Attachment 4
Date: 5/7/2025	File Name: Pier 3A sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

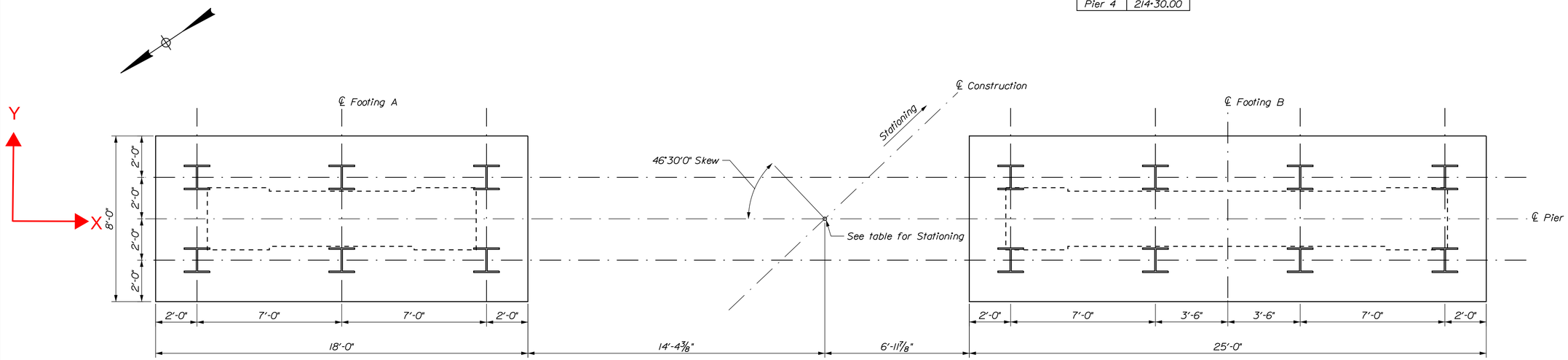
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

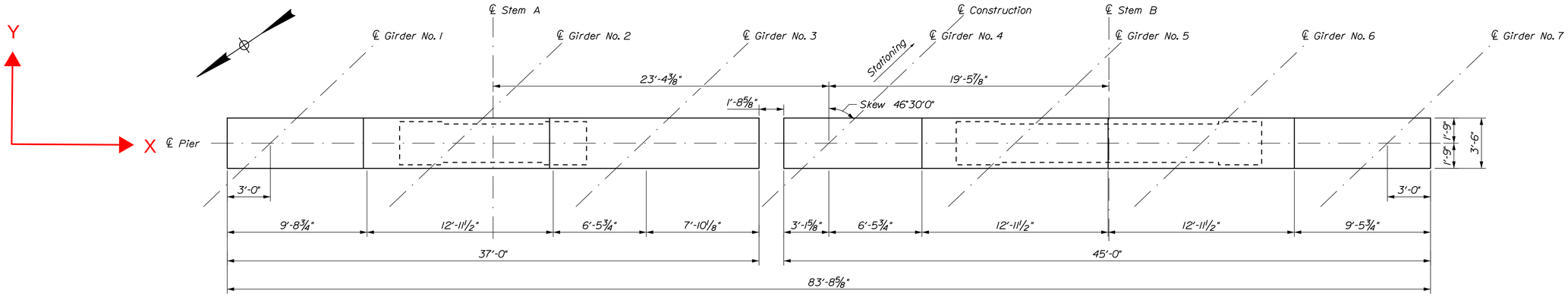
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

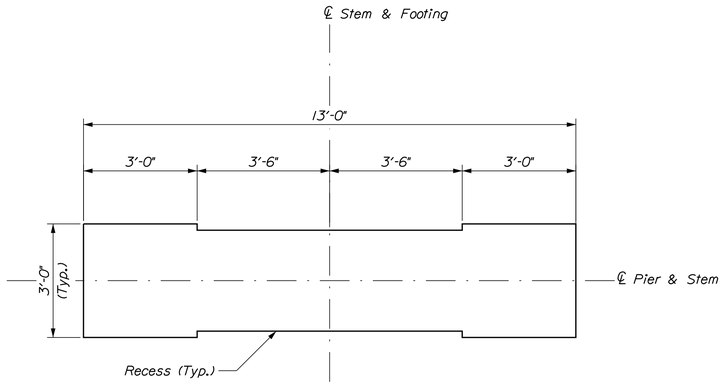
SHEET NUMBER

B25

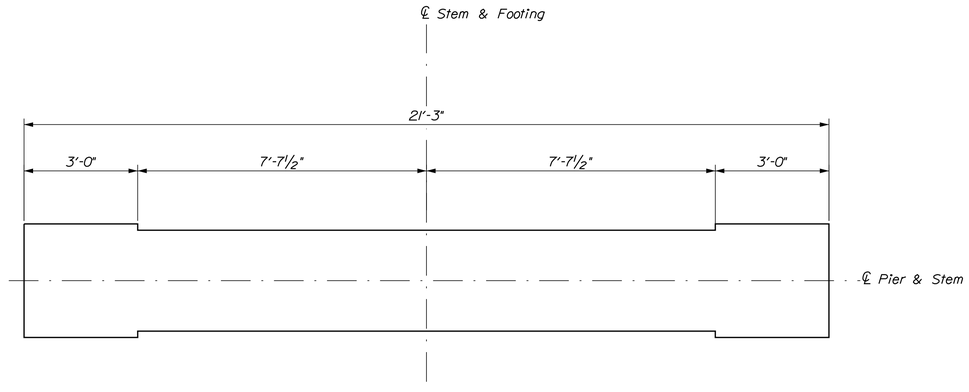
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

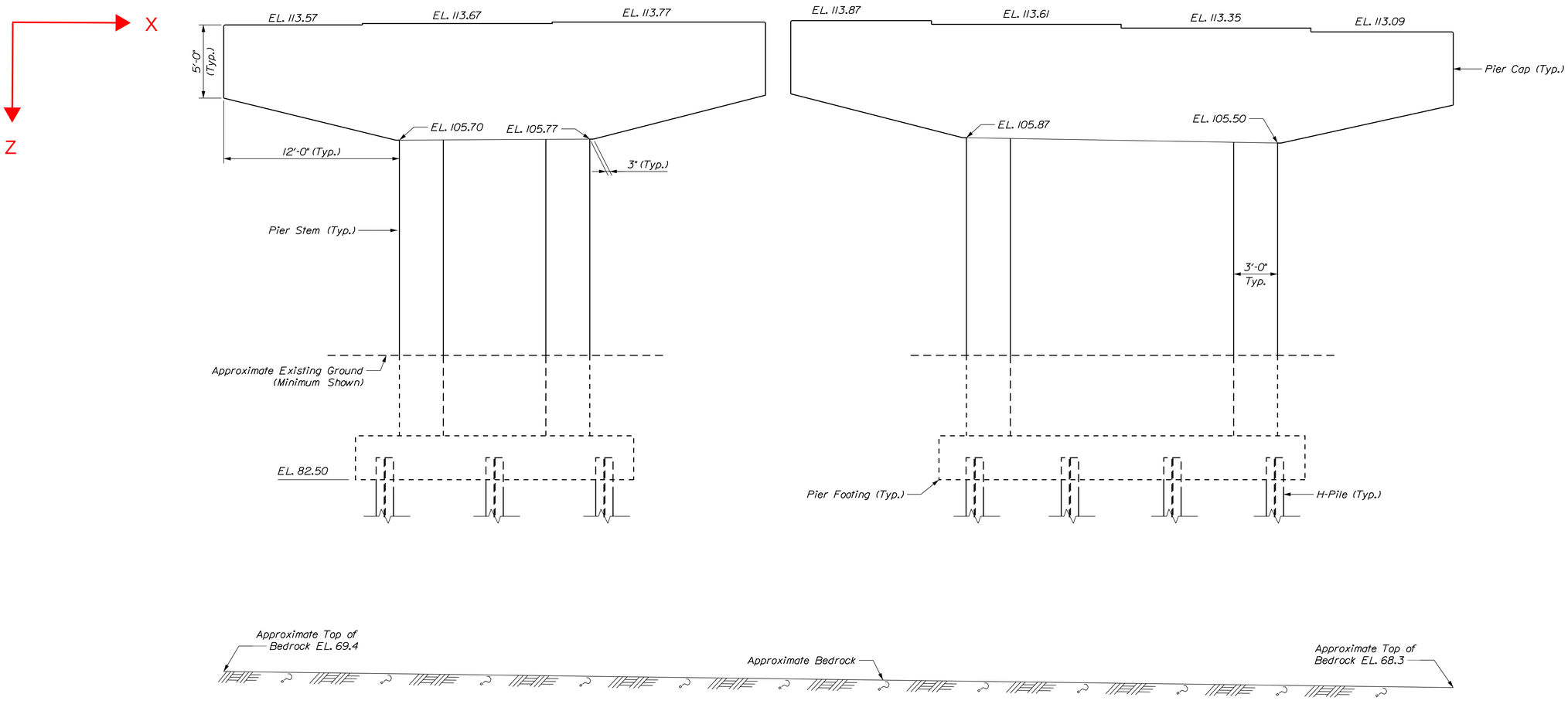
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

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BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles grouted into drilled rock sockets will provide adequate support for Pier 3B based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 3B (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

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ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier 3B Rock Socket Design Typical Section
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 3B is assumed to be supported by ten (10) HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 6.75 feet in the x-direction (Attachment 1).
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. A rock socket diameter of 30 inches (approximately twice the pile flange width) is assumed in order to permit pile installation and grouting.
5. A design compressive strength of 6,000 psi is assumed for the grout backfill in the annular space between the pile and the rock socket wall, consistent with the grout compressive strength specified in the MaineDOT Special Provision for Rock-Socketed H-Pile Foundation (Ref. 15). The annular space above the rock socket is assumed to be backfilled with Backfill Type C (703.22) and is modeled with a soil lateral resistance.
6. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
7. The pile design with grouted rock socket is assumed to include a steel plate welded to the tip of the pile in order to promote load transfer to bedrock. This steel shoe plate is assumed to have dimensions 15 inches by 15 inches (to accommodate the pile flange width) and a steel yield stress of 50 ksi. The grouted rock socket is assumed to extend 6 inches below the base of the shoe plate, with grout backfill between base of shoe plate and base of rock socket, in order to permit full contact with the bedrock surface.
8. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
9. A vertical earth load EV was added based on the difference between the existing ground surface El. of 89.5 feet and the top of Pier 3B footing of El. 84.5 feet, resulting in 5.0 feet of overburden soil. Both pile caps for Pier 3 are located at the same footing elevation of 81.5 feet which is at the calculated frost depth of 7.3 feet (Ref. 15) below the lowest adjacent ground surface along Pier 3. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
10. The upper 1-foot of bedrock is neglected in the rock socket depth due to poor RQD, the socket depth is assumed to begin 1-foot below the top of bedrock surface.
11. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

CALCULATIONS

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CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	13.64	in	
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	14.5	in	
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	0.455	in	
Fillet area =	0.3	in^2		0.3	in^2	
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	19.3	in^2	
Section area =	0.18	ft^2		0.13	ft^2	
Moment of inertia about the x-axis, I_x =				653	in^4	
Moment of inertia about the y-axis, I_y =				233	in^4	
Radius of gyration about the x-axis, r_x =				5.82	in	
Radius of gyration about the y-axis, r_y =				3.47	in	
Elastic section modulus about the x-axis, S_x =				95.8	in^3	
Elastic section modulus about the y-axis, S_y =				32.1	in^3	
Plastic section modulus about the x-axis, Z_x =				105.7	in^3	
Plastic section modulus about the y-axis, Z_y =				48.8	in^3	

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	84.5	ft	Sheet B14 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	83.0	ft	
As-modeled pile cap midplane elevation =	83.0	ft	Sheet B14 (Ref. 5)
Base of pile cap elevation =	81.5	ft	Sheet B14 (Ref. 5)
Pile length =	17.8	ft, includes a 6 in reduction for the shoe plate (Ref. 7).	
Socket length =	4.0	ft	
Bottom of socket El. =	64.7	ft, includes upper 1-foot of bedrock (Assumption 10).	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	81.5	80.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	80.5	75.7	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	75.7	73.5	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	4	Cohesionless	73.5	68.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	68.7	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 3 (Sheet B14 (Ref. 5)).

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 1)					
Maximum axial force in the piles, P_u =	390.0	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	25.0	kip-ft	=	300.6	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	31.1	kip-ft	=	373.5	in-kips	
Maximum shear force in the piles, V_u =	9.0	kips				
Depth below pile head to fixity =	16.1	ft	=	Elev.	66.86	ft
Depth below pile cap to fixity =	14.6	ft	=	Elev.	66.86	ft

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.031	0.640	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.012	0.319	ksf

D. Calculate tip resistance for the rock socket.

Rock Socket Properties

Socket length:	4	ft	Grout below pile toe:	0.5	ft	(Assumption 7)
Socket diameter:	30	in	(Assumption 4)			
Socket fill material:	grout with compressive strength of 6,000 psi (Assumption 7)					

Additional Rock Properties

Unit weight of rock =	158		pcf (based on rock type, Ref. 12)
Hoek-Brown m_i =	19		(Ref. 7, Table 10.4.6.4-1, for rock type metasandstone)
GSI =	45		(based on rock core field observations)
Uniaxial compressive strength of rock, qu =	1,278,785	psf	(from rock lab test results, Ref. 10)

CALCULATIONS

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Calculate the unit tip resistance q_p for the rock socket. Use the method for drilled shafts in rock (Ref. 1, Article 10.8.3.5.4c). Based on the boring logs for this site, the rock below the proposed base of the rock socket is closely jointed; thus Equations 10.8.3.5.4c-2 and 10.8.3.5.4c-3 (Ref. 1) will be used to determine the unit tip resistance.

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-2)

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{vb})}{q_u} + s\right]^a$$ (Ref. 7, Eq. 10.8.3.5.4c-3)

$$s = e^{\left(\frac{GSI-100}{9-3D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-2)

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$ (Ref. 7, Eq. 10.4.6.4-3)

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)}$$ (Ref. 7, Eq. 10.4.6.4-4)

where:

Uniaxial compressive strength of rock, q_u =	1,279	ksf (from Step D)
Unit weight of rock =	158	pcf (from Step D)
Vertical effective stress at tip elevation, σ'_{vb} =	0.4	ksf (from stratigraphy in Step 3)
Geological Strength Index, GSI =	45	(from Step D)
Hoek-Brown material index constant, m_i =	19	(from Step D)
Disturbance factor, D =	0	(Ref. 14 Ch. 5 Fig. 9, assume no blasting, mechanical excavation)
s =	0.0022	
a =	0.508	
m_b =	2.665	
A =	67	ksf
Unit tip resistance q_p =	543	ksf
Base area of pile shoe plate =	1.6	ft^2 (from Assumption 7)
Nominal tip resistance at base of rock socket =	848	kips
Resistance factor =	0.50	(Ref. 7, Table 10.5.5.2.4-1)
Factored tip resistance at base of rock socket =	424	kips

Compare tip resistance to axial load on pile:

Factored tip resistance (kips):		Strength I factored axial load per pile (kips):	
424	>	390	OK

CALCULATIONS

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Check that grout compressive strength is sufficient for axial load on pile:

Grout compressive strength =	6,000	psi	=	864	ksf (Assumption 8)
Grout resistance beneath pile shoe plate =	1,350	kips			
Resistance factor =	0.70	for bearing on concrete (Ref. 7, Article 5.5.4.2)			
Factored grout resistance beneath shoe plate =	945	kips			

Compare grout resistance to axial load on pile:

Factored grout resistance (kips):		Strength I factored axial load per pile (kips):	
945	>	390	OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	175.69	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)

Check: Kl/r_s = 61 < 120 OK

Local buckling and equivalent nominal yield resistance :

$$\frac{b}{t} \leq \lambda_r$$ (Ref. 7, Eq. 6.9.4.2.1-1) $$0.56\sqrt{\frac{E_{st}}{F_y}}$$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 11)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$$\frac{b}{t} = 15.98 > 13.49 = 0.56\sqrt{\frac{E_{st}}{F_y}}$$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

CALCULATIONS

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Calculate equivalent nominal yield resistance:

$$P_o = F_y A_g$$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	175.69	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	1,502	kips	

Elastic and flexural torsional buckling resistance:

$$\frac{D}{t_w} \leq 150$$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$$P_n = P_o 0.658^{P_o/P_e}$$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$$P_n = 0.877 P_e$$ (Ref. 7, Eq. 6.9.4.1.1-2)

P_o / P_e = 0.64 < 2.25, so use Eq. 6.9.4.1.1-1

P_n = 738 kips

CALCULATIONS

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For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	738	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	38.2	ksi	

A_{eff} = $A_g - \sum(b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If\ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If\ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	38.2	ksi	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

$$\lambda_r\sqrt{\frac{F_y}{F_{cr}}} = \quad 15.4$$

check $b/t =$ 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

$c_1 =$ 0.22 (Ref. 7, Table 6.9.4.2.2b-1)
$c_2 =$ 1.49 (Ref. 7, Table 6.9.4.2.2b-1)
$c_3 =$ 0.00 (Ref. 7, Table 6.9.4.2.2b-1)

$F_{el} =$ 79.09 ksi (Ref. 7, Eq. 6.9.4.2.2b-4)

$b_e =$ 7.15 in (Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff}:

$A_{eff} =$ 19.10 in^2 (Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

$P_n =$ 729.7 kips (Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.60 (Ref. 7, Article 6.5.4.2))
$P_r =$ 438 kips

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)
$\varphi_c =$ 0.70 (Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural} =$ 511 kips

Check that the structural axial resistance is sufficient to support axial load on pile:

$P_u =$ 390 kips (from Part B)

$P_{r,\ axial} =$ 438 kips > $P_u =$ 390 kips OK

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	301	in-kips (from Part B)
M_{uy} =	374	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	301	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	374	in-kips	OK

wsp

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.8 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.99 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

Date:	5/8/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 3B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.0 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.0 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 17.8 feet, a maximum axial force of 390.0 kips , the corresponding strong axis (y-axis) moment of 373.5 in-kips (31.1 ft-kips) occurs in the HP14x89 piles at Pier 3B in Pile 1 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity within the grouted rock socket. The analysis indicates that the maximum top of pile deflection at the serivce limit state is less than 0.1 inch in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag. The results indicate that 3.5-foot pile embedment into a 4.0-foot rock socket backfilled with grout of compressive strength 6,000 psi is sufficient for the pile to achieve fixity. The annular space above the rock socket should be backfilled with pea stone. Based on the corrosion rates used in the analysis, HP 14x89 piles with 15" by 15" bearing plates welded to the tips are adequately sized to accommodate the anticipated design loads. It is assumed that the bearing plate thickness and weld will be designed by others. A summary table is presented below.

Pier 1 Right Summary Table		
Pile Length (ft)		17.8
Depth below Pile Head to Fixity (ft)		16.1
Geotechnical Axial Resistance (kips)	Nominal	848
	Factored	424
Structural Axial Resistance (kips)	Nominal	738
	Factored	438
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.99
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

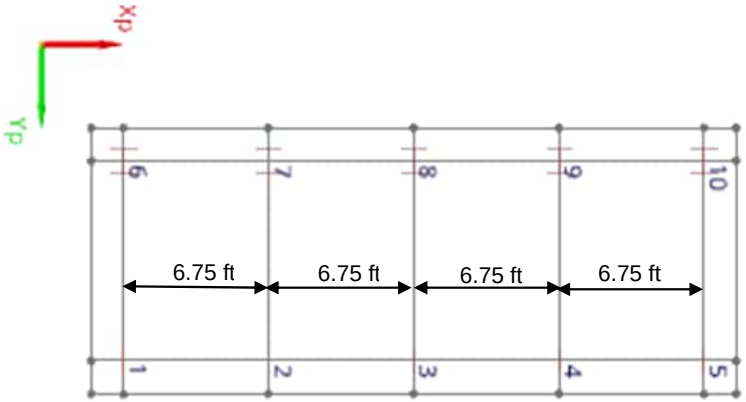


Figure 1b: Profile View of Pile Cap (Transverse)

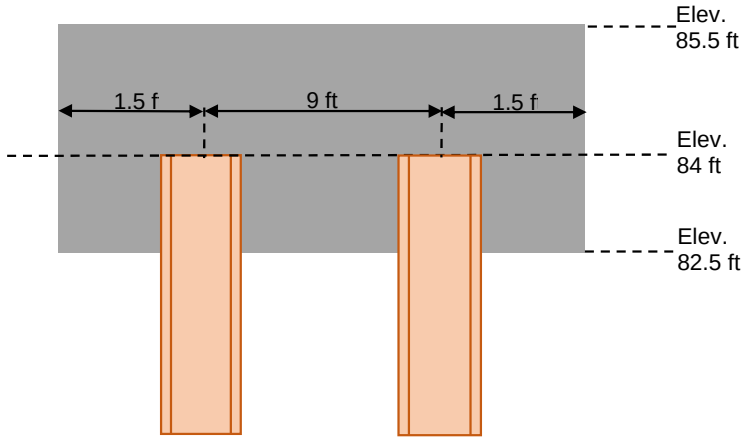


Figure 1c: 3D View of Pile Model

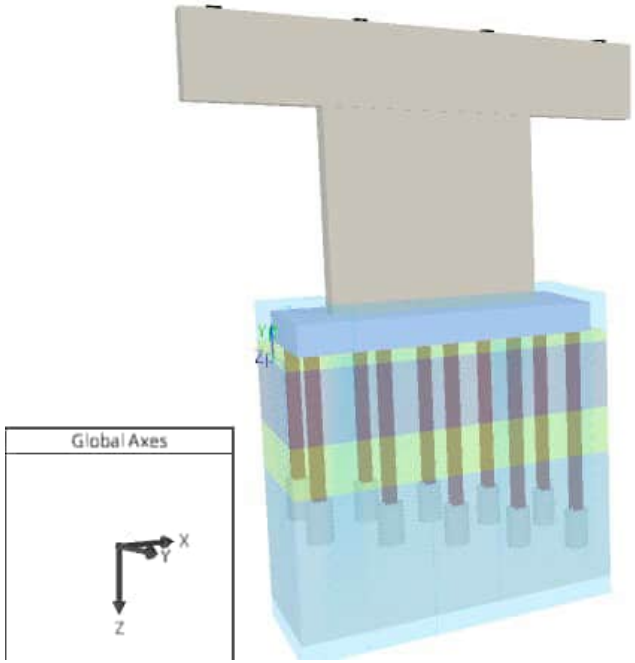
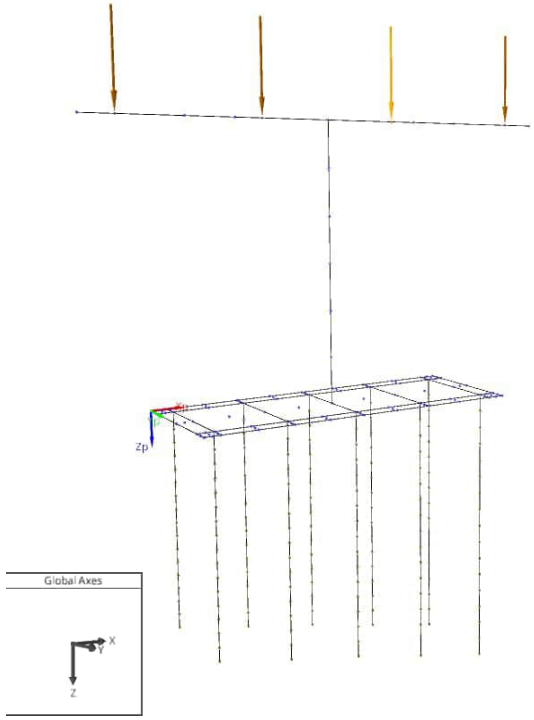


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	81.5	80.5	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	53	-		-	-	0.46		0.30	12		-	-	-
Stiff Clay	2	Cohesive	80.5	75.7	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medium Clay	3	Cohesive	75.7	73.5	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	4	Cohesionless	73.5	68.7	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	319		-	-	-
Bedrock	5	Rock	68.7	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		6678	895	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 3 profile (Sheet B14 Pier 3 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

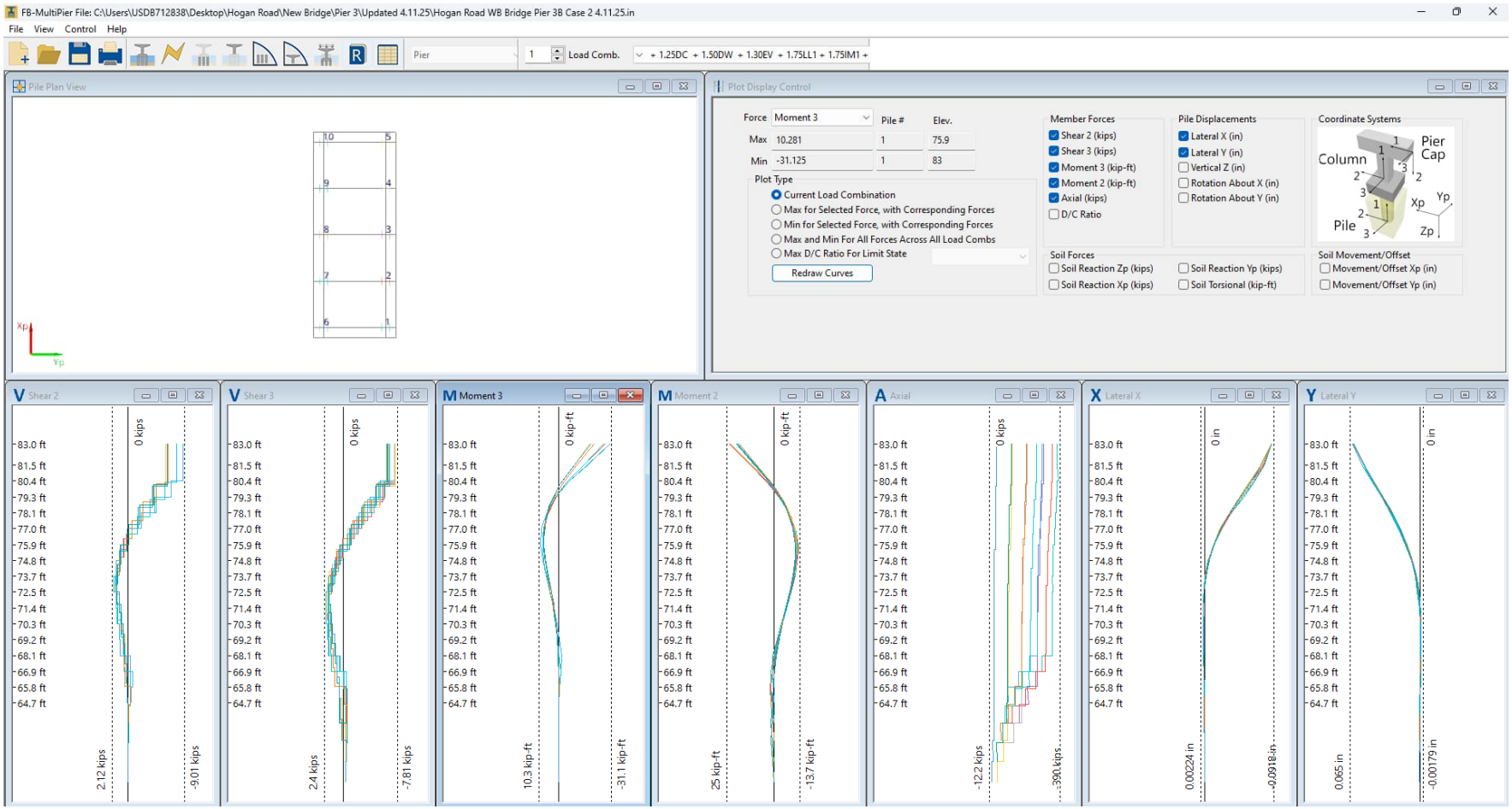
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

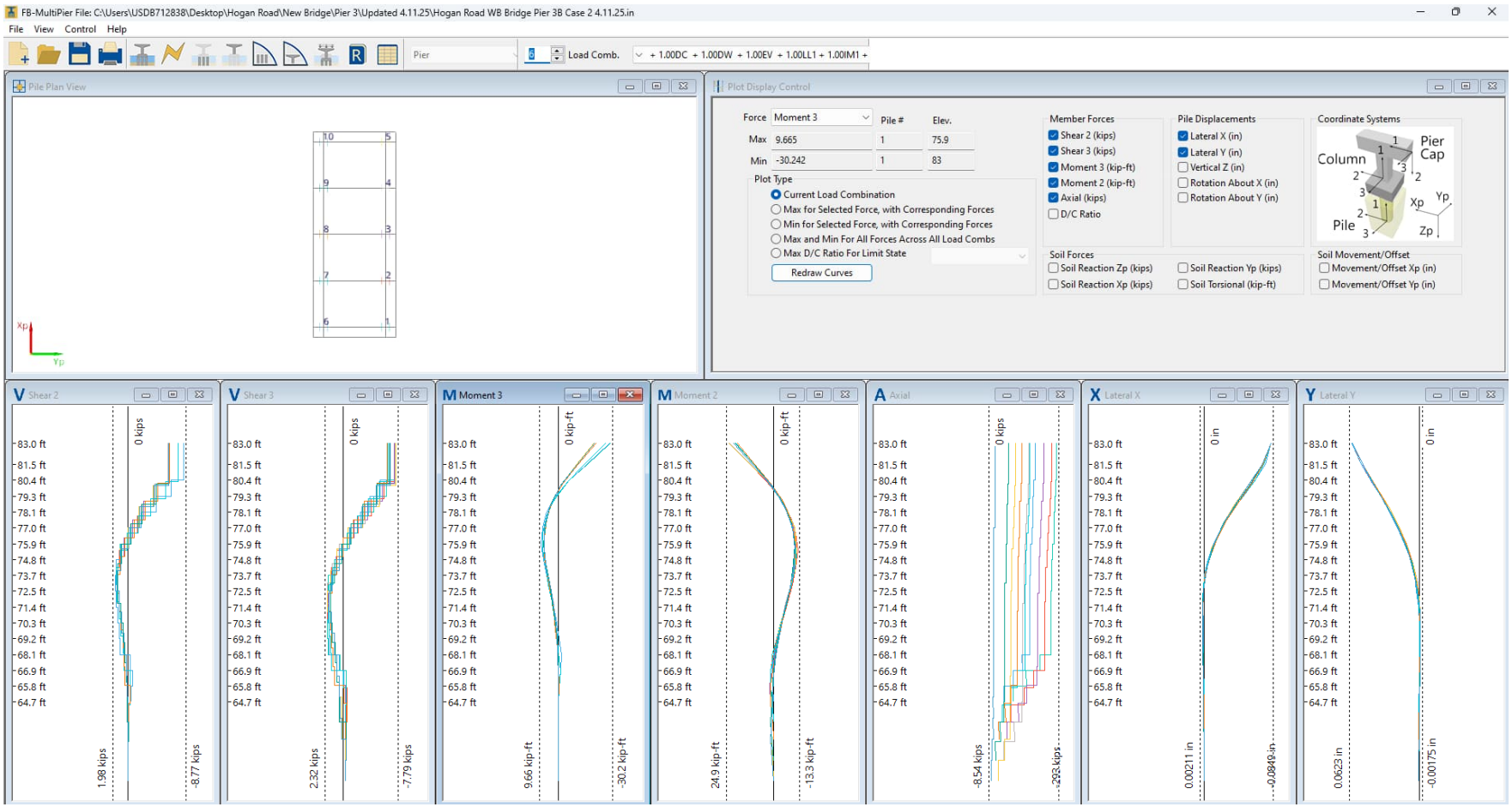
Prepared: DEB
Checked: ATM
Reviewed:

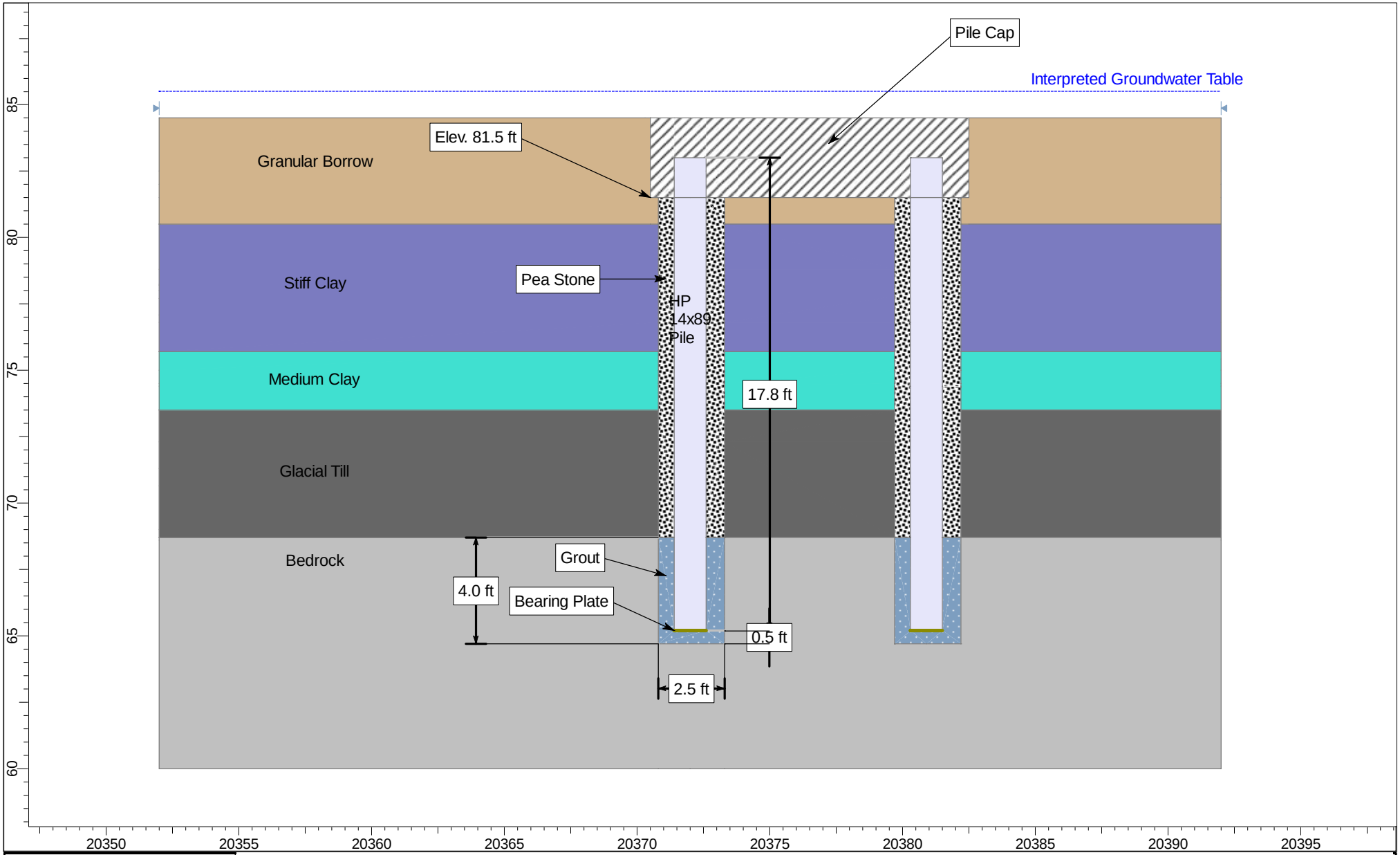
Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I





Project	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME			
Analysis Description	HP 14x89 Grouted Pile at WB Pier 3B - Design Typical Section			
Drawn By DEB	Checked By ATM	Reviewed By CCB	Scale 1:60	Attachment 4
Date 5/7/2025	File Name Pier 3B sketch.slmd			

SLIDE 9.037

Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

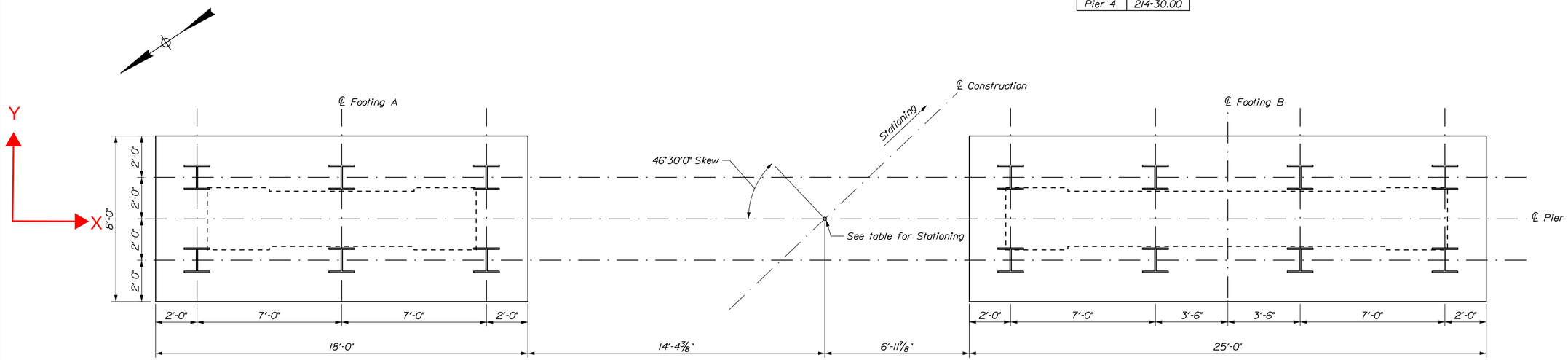
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

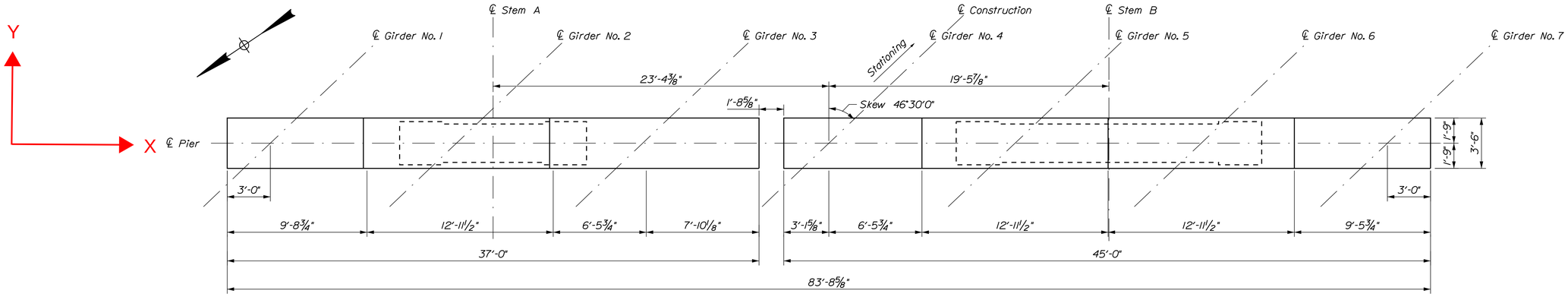
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

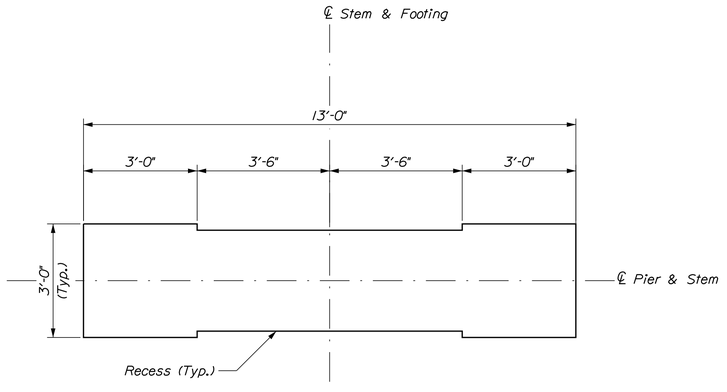
SHEET NUMBER

B25

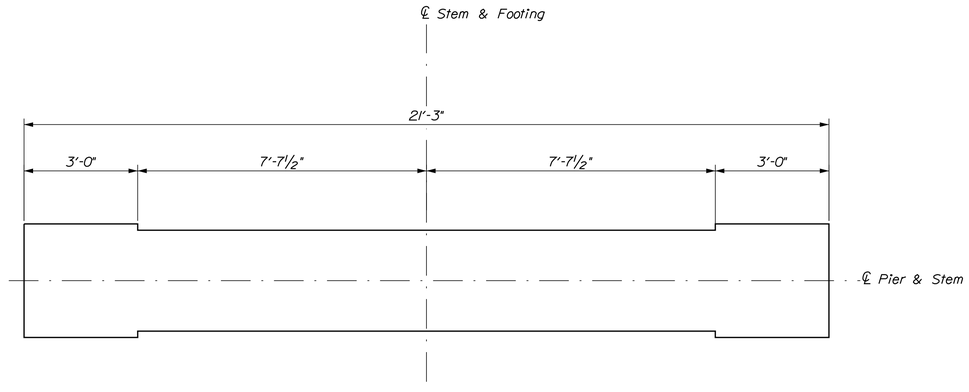
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

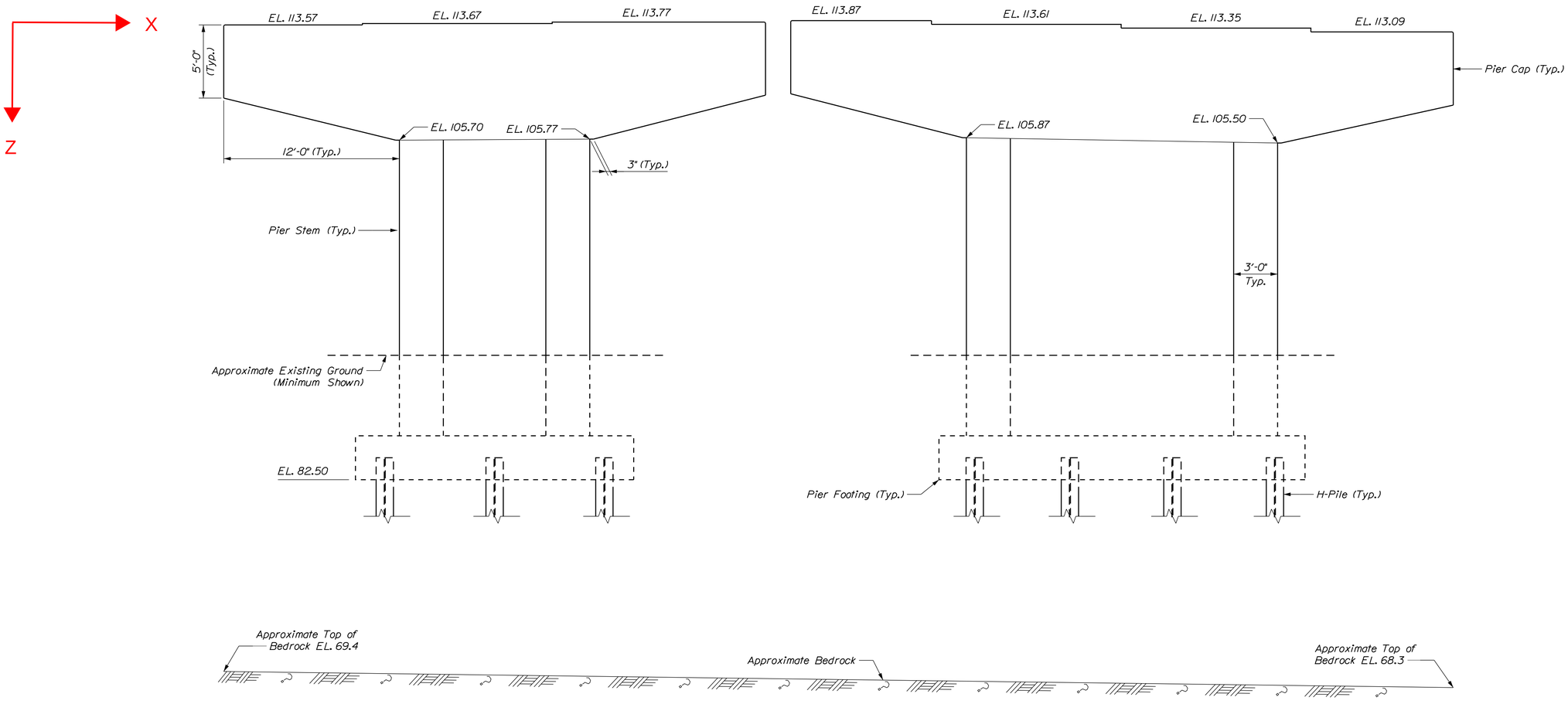
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge 6787 Pier 4A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles provide adequate support for WB Pier 4A based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 4A (Pile 1), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. WSP calculation package titled Pile Drivability at WB Pier 4 - HP14x89, dated 5/9/2025.
16. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

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ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
4. Pier Loads

ASSUMPTIONS

1. The proposed Pier 3A is assumed to be supported by eight (8) plumb HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 9 feet in the x-direction (Attachment 1).
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.
4. All piles are assumed to have reinforced pile tips (Ref. 8).
5. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.
6. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.
7. A vertical earth load EV was added based on the difference between the existing ground surface El. of 93.4 feet and the top of Pier 2A footing of El. 86 feet, resulting in 7.4 feet of overburden soil, both pile caps for Pier 2 are located at the same footing elevation of 83 feet which is at the calculated frost depth of 7.3 feet (Ref. 16) below the lowest adjacent ground surface along Pier 2. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.
8. The upper 1-foot of bedrock is neglected due to poor RQD, piles are assumed to penetrate 1-foot in to bedrock.
9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.
10. A φdyn of 0.65 using (Ref. 9 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

CALCULATIONS

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CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	86.0	ft	Sheet B15 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.5	ft	
As-modeled pile cap midplane elevation =	84.5	ft	Sheet B15 (Ref. 5)
Base of pile cap elevation =	83.0	ft	Sheet B15 (Ref. 5)
Pile length =	21.2	ft, includes upper 1-foot of bedrock (Assumption 8).	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	2	Cohesive	82.0	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	3	Cohesionless	81.4	64.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	4	Rock	64.3	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 4 (Sheet 15 (Ref. 5)).

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model: (Pile 1)

Maximum axial force at the top of pile, P_{u_top} =	379.8	kips				
Maximum axial force at the pile tip, P_{u_tip} =	344.7	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	28.1	kip-ft	=	337.4	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	25.9	kip-ft	=	310.6	in-kips	
Maximum shear force in the piles, V_u =	9.8	kips				
Depth below pile head to fixity =	16.0	ft	=	Elev.	68.54	ft
Depth below pile cap to fixity =	14.5	ft	=	Elev.	68.54	ft

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.063	0.988	ksf (at midpoint of soil layer using stratigraphy in Part A)

$f_s = \alpha s_u$

f_s =	0.024	0.492	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Medium Clay		
	Top	Bottom	
Undrained shear strength, s_u =	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	0.95	0.95	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	0.844	0.844	ksf

wsp

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 56.4 in = 4.7 ft (from Part A)

	Granular Borrow	Medium Clay	Glacial Till		
Thickness =	1.0	0.6	17.1	ft	(from stratigraphy in Part A)
A_s =	4.7	2.8	80.3	ft^1	
R_s per layer =	0.1	2.4	39.5	kips	

Total R_s = 42.0

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD ≤ 70%
$q_p = 0.80 q_u$ for RQD = 100% (Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 1,279 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 422.1 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

$R_n = R_s + R_p$ Pile toe area, A_p = 0.13 ft^2 (from Part A)

Total R_p = 56.6 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 99 kips (bedrock)

$R_r = \varphi_{stat} R_n$ (Ref. 7, Eq. 10.7.3.8.6a-1)

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where: φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method

(Ref. 7 Tbl. 10.5.5.2.3-1) φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Granular Borrow	Medium Clay	Glacial Till	Pile Tip (bedrock)	
R_s or R_p =	0.1	2.4	39.5	56.6	kips
φ_{stat} =	0.45	0.35	0.45	0.5	(Ref. 7, Table 10.5.5.2.3-
R_r per layer =	0.1	0.8	17.8	25.5	

Total R_r = 44 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 89% of the steel yield strength (Ref. 16):

R_n = 800 kips (Ref. 14)

φ_{dyn} = 0.65 (Assumption 10)

R_f = 520 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 379.8 kips (from Part B)

R_r = 520 kips > P_u = 379.8 kips OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 1.2 (Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)

Unbraced length, l = 173.49 in (from Part B)

Minimum radius of gyration, r_s = 3.47 in (from Part A)

Check: Kl/r_s = 60 < 120 OK

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Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 9)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check: $\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	173.49	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	1,540	kips	

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \leq 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

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Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877 P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.63 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 743 kips

For compression member cross sections containing and slender elements, the nominal compressive resistance, $P_{n,}$ shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$P_n = F_{cr} A_{eff}$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$F_{cr} = \frac{P_{cr}}{A_g}$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	743	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	38.5	ksi	

$A_{eff} = A_g - \sum (b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$If \ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$If \ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ then $b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$ (Ref. 7, Eq. 6.9.4.2.2b-2)

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$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	38.5	ksi	

$\lambda_r \sqrt{\frac{F_y}{F_{cr}}}$ = 15.4

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

c_1 =	0.22		(Ref. 7, Table 6.9.4.2.2b-1)
c_2 =	1.49		(Ref. 7, Table 6.9.4.2.2b-1)
c_3 =	0.00		(Ref. 7, Table 6.9.4.2.2b-1)
F_{el} =	79.09	ksi	(Ref. 7, Eq. 6.9.4.2.2b-4)
b_e =	7.14	in	(Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff} :

A_{eff} =	19.08	in^2	(Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n =	733.7	kips	(Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.50		(Ref. 7, Article 6.5.4.2))
P_r =	367	kips	

$P_{r_{axial\,/\,flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.70		(Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$=	514	kips	

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Check that the structural axial resistance is sufficient to support axial load on pile:

P_{u_tip}= 345 kips (from Part B)

$P_{r,\ axial}$ = 367 kips > P_{u_tip}= 345 kips OK

F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3) $\lambda_f = \frac{b_f}{2t_f}$ λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4) $\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$ λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5) $\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$ λr_f = 19.99

Nominal and factored structural resistance in flexure:

$M_n = M_p = 1.5F_yS_y$ if $\lambda_f \le \lambda_{pf}$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} = 337 in-kips (from Part B)

M_{uy} = 311 in-kips (from Part B)

M_{rx} = 4,973 in-kips > M_{ux} = 337 in-kips OK

M_{ry} = 1,913 in-kips > M_{uy} = 311 in-kips OK

wsp

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge 6787 Pier 4A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.7 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.88 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00	(Ref. 7, Article 6.5.4.2)
V_r =	168	kips

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge 6787 Pier 4A H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 9.8 kips (from Part B)

$V_r =$ 168 kips > $V_u =$ 9.8 kips OK

CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 21.2 feet, a maximum axial force of 379.9 kips at the top of the pile, the corresponding weak axis (x-axis) moment of 342.8 in-kips (28.6 ft-kips) occurs in the HP14x89 piles at Pier 4A in Pile 1 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The analysis indicates that the maximum top of pile deflection at the service limit state is less than 0.15 inches in the X and Y directions. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag as none is anticipated. A summary table is presented below.

Pier 4A Summary Table		
Pile Length (ft)		21.2
Depth below Pile Head to Fixity (ft)		16.0
Geotechnical Axial Resistance (kips)	Nominal	800
	Factored	520
Structural Axial Resistance (kips)	Nominal	743
	Factored	367
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.88
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

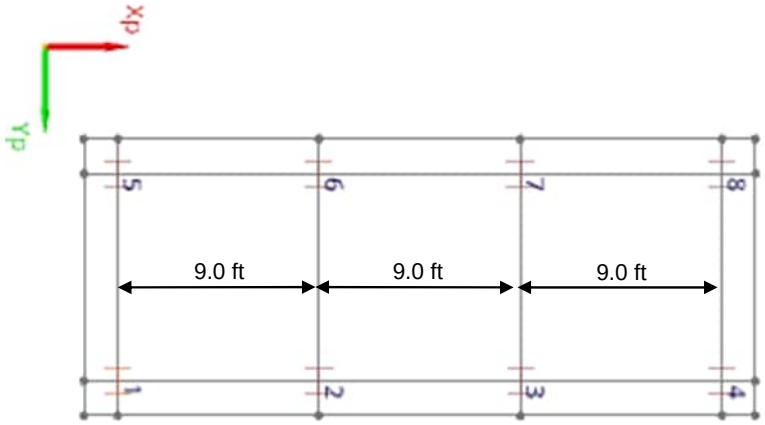


Figure 1b: Profile View of Pile Cap (Transverse)

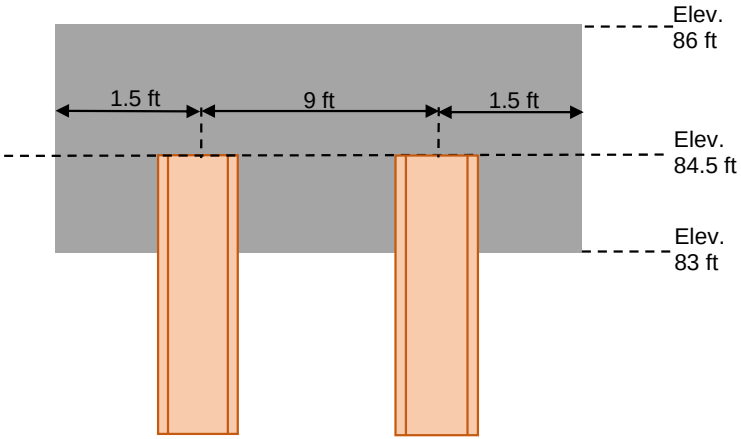


Figure 1c: 3D View of Pile Model

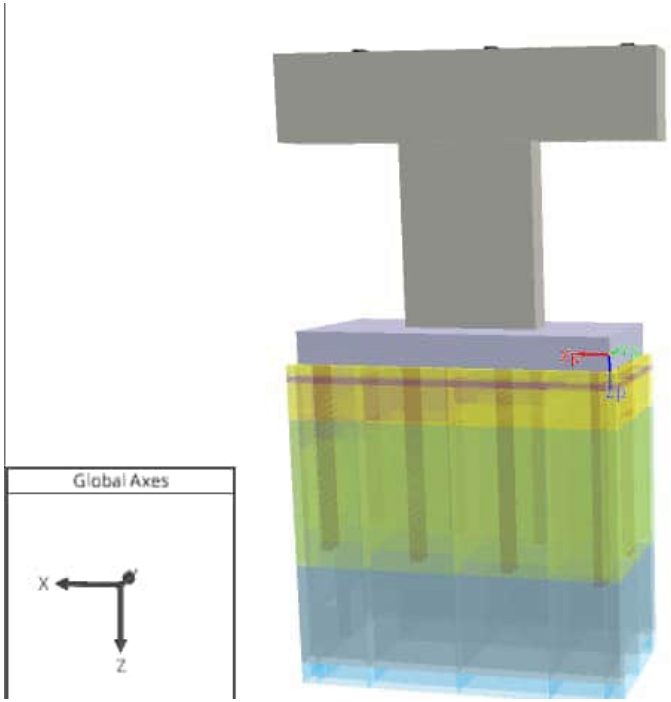
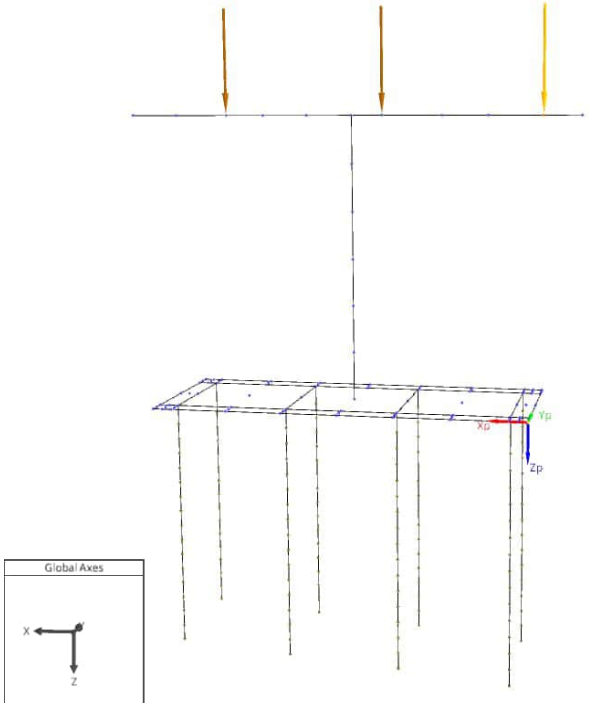


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	24		-	-	-
Medim Clay	2	Cohesive	82.0	81.4	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	3	Cohesionless	81.4	78.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	146	-		-	-	0.74		0.38	208		-	-	-
Glacial Till	4	Cohesionless	78.0	64.3	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	549		-	-	-
Bedrock	5	Rock	64.3	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		4411.8	800.0	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 4 profile (Sheet B15 Pier 4 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

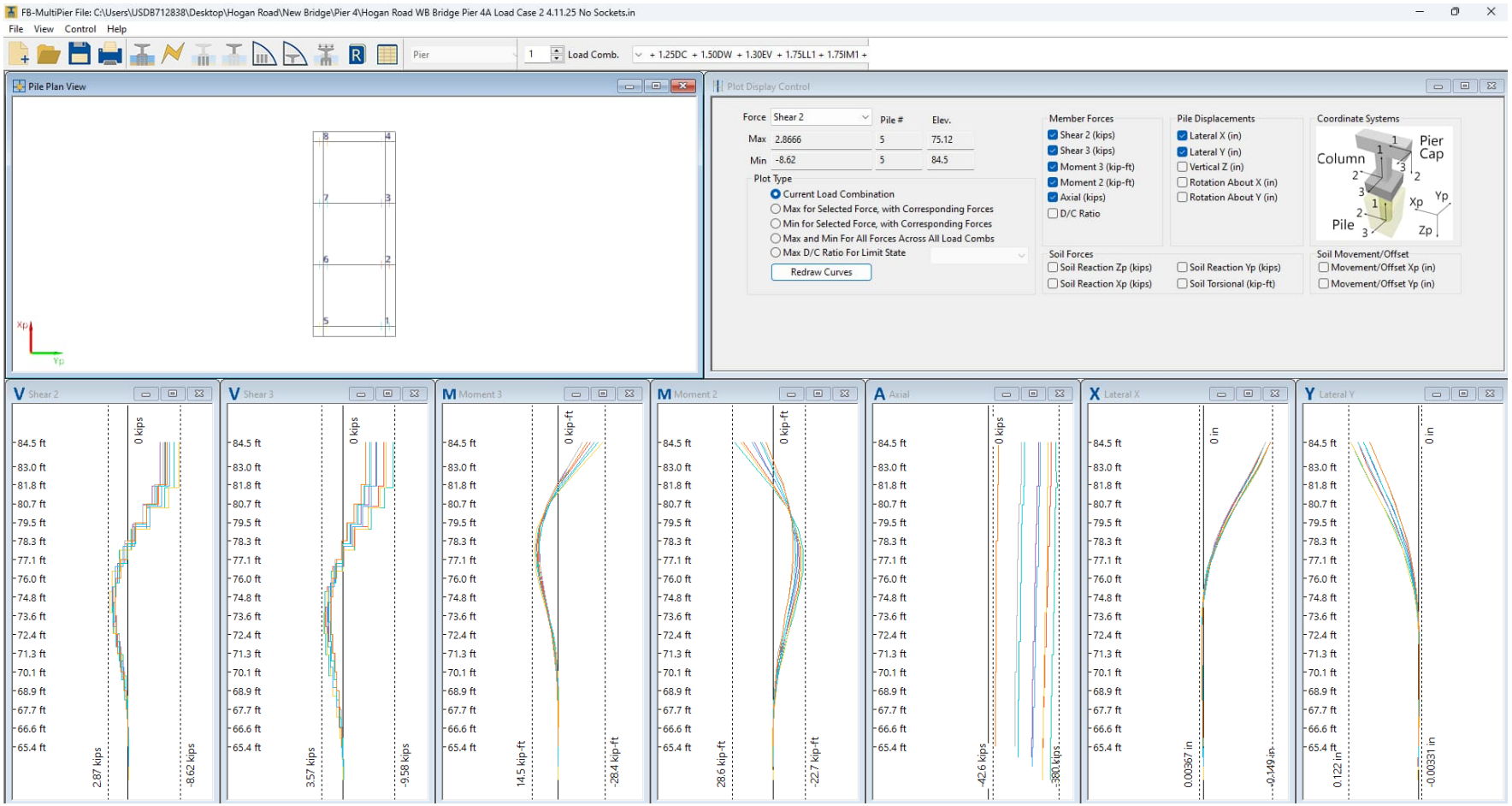
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

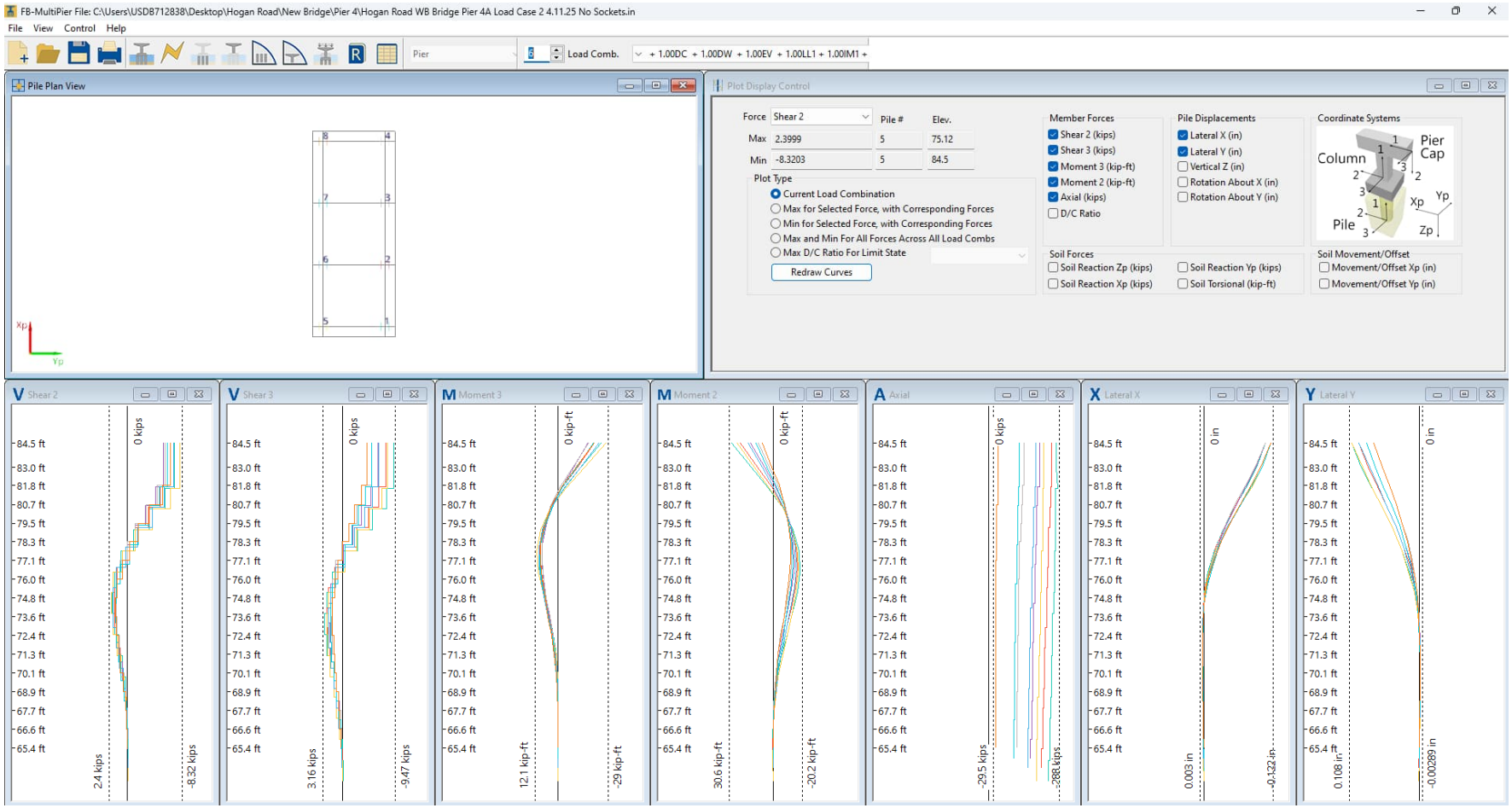
Prepared: DEB
Checked: ATM
Reviewed:

Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I



Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

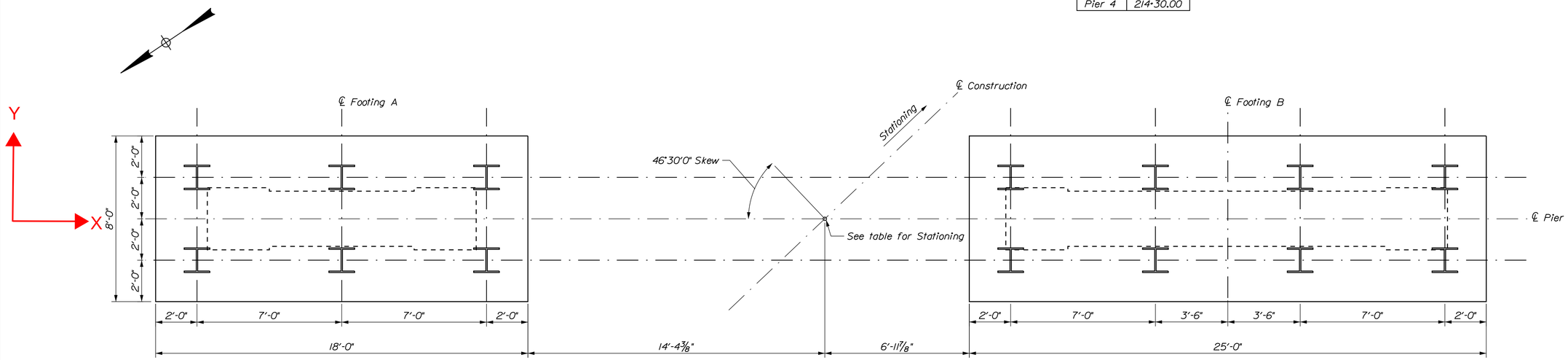
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

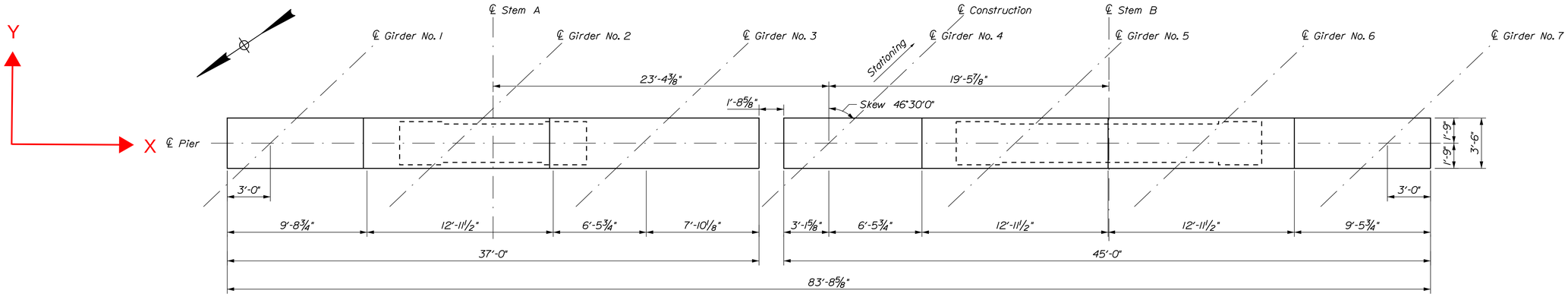
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

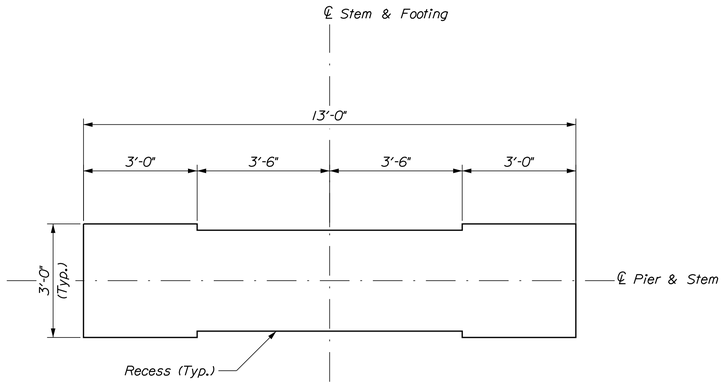
SHEET NUMBER

B25

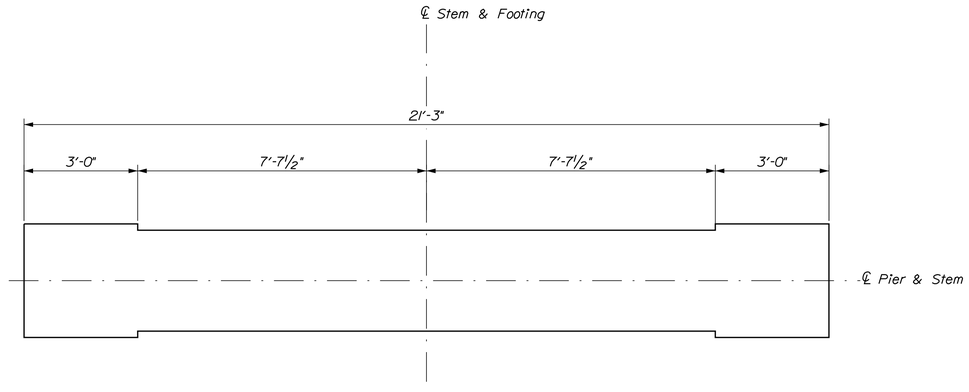
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

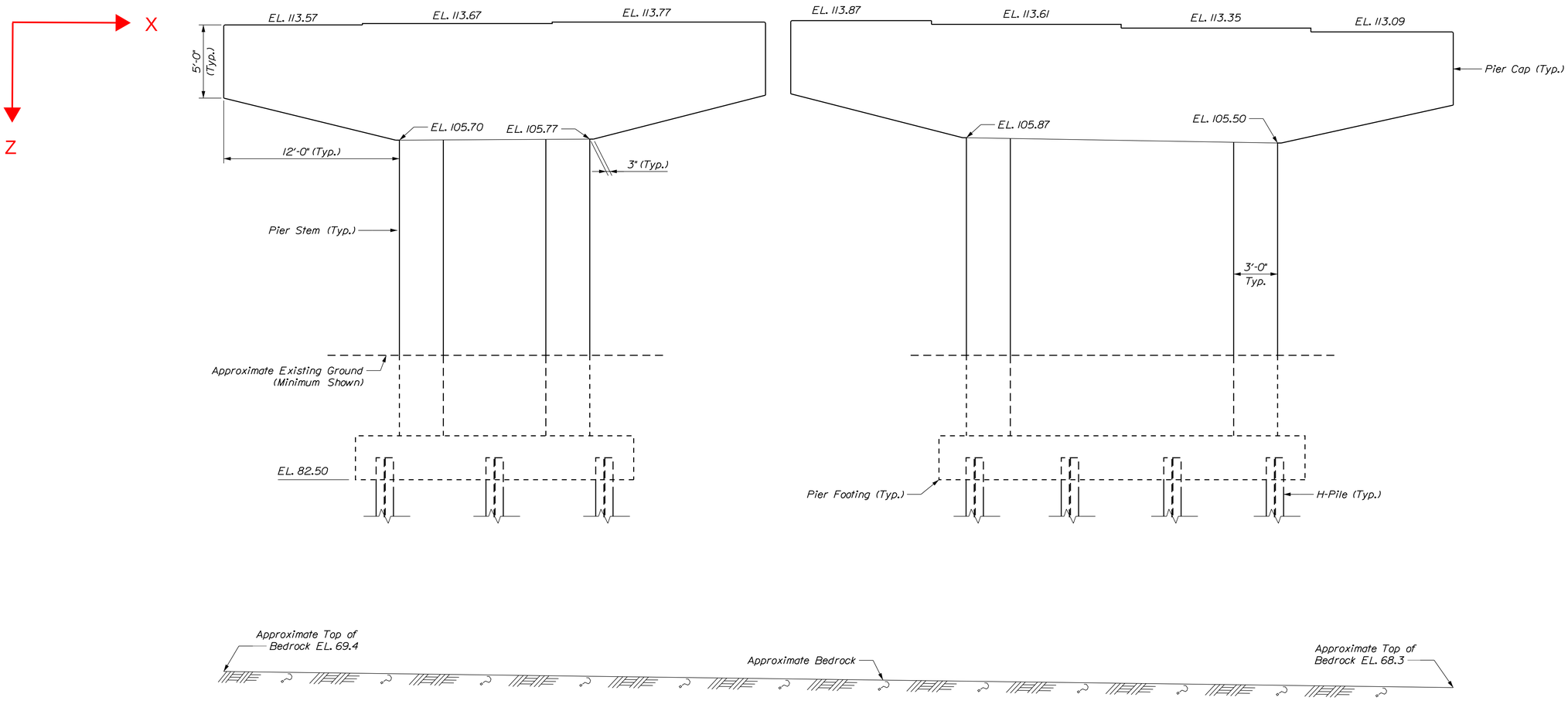
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 4B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Determine if the proposed HP14x89 piles provide adequate support for WB Pier 4B based on the design load Case 2 (Strength I) provided by VHB.

METHOD

Use the FB-MultiPier software package (Ref. 1) to run a pile group analysis to obtain maximum axial force, corresponding bending moment, and depth to fixity for a single pile. This analysis follows the procedures in the FHWA Design and Construction of Driven Pile Foundations manual (Ref. 3) and AASHTO (Ref. 7) to evaluate the geotechnical and structural resistance of the individual piles. This evaluation analyzes the maximum axial load on a single pile at Pier 4B (Pile 2), and the corresponding moment and shear forces on that pile.

REFERENCES

1. Florida Bridge Software Institute. FB-MultiPier software package, version 5.9.0.
2. American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE/SEI 7-16 Standard.
3. Federal Highway Administration (FHWA). 2016. GEC 012: Design and Construction of Driven Pile Foundations, Volume I. Publication No. FHWA-NHI-16-009.
4. American Institute of Steel Construction (AISC). 2005. Steel Construction Manual, 13th Edition.
5. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
6. WSP Lab Field Data Summary Plots V4
7. American Association of State Highway and Transportation Officials (AASHTO). 2020. LRFD Bridge Design Specifications, 9th Ed.
8. Maine Department of Transportation Standard Specifications Highways and Bridges, revision of June 1981.
9. WSP boring logs.
10. GeoTesting Express. Laboratory testing results for rock core samples. Dated November 16, 2023.
11. Loading provided by VHB in Computation package titled "1983 Bridge Pier 5 Bearing Loads 2024-7-22"
12. WSP Hogan Road WB Bridge Pier 1 FB-Multipier Soil Properties
13. WSP calculation package titled Soil Corrosivity Evaluation for Bridge Foundations, dated 2/12/2025.
14. Email from VHB received on Friday October 25th, 2024 with subject line "Hogan Road WB Pier Bearing Loads"
15. Hoek, Evert. Practical Rock Engineering, 2023 ed. https://www.rocscience.com/learning/hoeks-corner.
15. WSP calculation package titled Frost Penetration Depth - Bridge structures in Fill, dated 3/4/2024.

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 4B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

ATTACHMENTS

1. Output image of pile model and pile cap dimensions
2. Table of FB-MultiPier model input properties
3. FB-Multipier Strength I and Service I Output
5. Pier Loads

ASSUMPTIONS

1. The proposed Pier 3B is assumed to be supported by ten (10) plumb HP 14x89 Grade 50 piles with a center to center spacing of 9 feet in the y-direction and 6.75 feet in the x-direction (Attachment 1).
2. It is assumed the piles were installed with a pile cap in firm contact with the ground, a group efficiency of 1.0 will be used, and no reduction in nominal resistance for the pile group will be necessary (Reference 3, Section 7.2.2.2).
3. The provided loads (Ref. 14) are applied at the bearings on the pier cap.

4. All piles are assumed to have reinforced pile tips (Ref. 8).

5. Pile fixity is assumed to occur at the depth where the bending moment is less than 5 percent of the maximum bending moment on the subject pile, and there is a lateral deflection at the toe of the pile of no more then 1/8th of an inch.

6. The pile cap concrete elastic modulus was increased from Ec = 4,000 psi to Ec = 40,000 psi to increase stiffness in the pile cap and better distribute loads to account for the absence of pile cap reinforcement in FB-Multipier.

7. A vertical earth load EV was added based on the difference between the existing ground surface El. of 93.4 feet and the top of Pier 2A footing of El. 86 feet, resulting in 7.4 feet of overburden soil, both pile caps for Pier 2 are located at the same footing elevation of 83 feet which is at the calculated frost depth of 7.3 feet (Ref. 16) below the lowest adjacent ground surface along Pier 2. The resultant earth pressure was distributed evenly as point loads at the center of the pile locations along the pile cap.

8. The upper 1-foot of bedrock is neglected due to poor RQD, piles are assumed to penetrate 1-foot in to bedrock.

9. For slenderness ratio limits and element widths for axial compression (Ref. 7, Table 6.9.4.2.1-1) it is assumed the elements are supported along one longitudinal edge.

10. A φdyn of 0.65 using (Ref. 9 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

CALCULATIONS

Date:	9/25/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	WB Bridge # 6787 Pier 4B H-Pile Analysis Load Case 2	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

A. Select the pile section parameters.

Corrosion loss per year using non-linear methods

Design life =	100	years	(per Maine DOT)
Total corrosion loss =	0.08	inches	(Ref. 13)

Pile size: HP 14x89

	Intact Section			Corroded Section		(ref. 13)
Depth, d =	13.8	in	(Ref. 4, Table 1-4)	Depth, d =	13.64	in
Width, b_f =	14.7	in	(Ref. 4, Table 1-4)	Width, b_f =	14.5	in
Web thickness, t_w =	0.615	in	(Ref. 4, Table 1-4)	Web thickness, t_w =	0.455	in
Flange thickness, t_f =	0.615	in	(Ref. 4, Table 1-4)	Flange thickness, t_f =	0.455	in
Fillet area =	0.3	in^2		Fillet area =	0.3	in^2
Section area =	26.1	in^2	(Ref. 4, Table 1-4)	Section area =	19.3	in^2
Section area =	0.18	ft^2		Section area =	0.13	ft^2
				Moment of inertia about the x-axis, I_x =	653	in^4
				Moment of inertia about the y-axis, I_y =	233	in^4
				Radius of gyration about the x-axis, r_x =	5.82	in
				Radius of gyration about the y-axis, r_y =	3.47	in
				Elastic section modulus about the x-axis, S_x =	95.8	in^3
				Elastic section modulus about the y-axis, S_y =	32.1	in^3
				Plastic section modulus about the x-axis, Z_x =	105.7	in^3
				Plastic section modulus about the y-axis, Z_y =	48.8	in^3

Steel yield stress, F_y =	50	ksi	(Grade 50)
Steel elastic modulus, E_{st} =	29,000	ksi	
Top of pile cap elevation =	86.0	ft	Sheet B15 (Ref. 5)
Pile cap thickness =	3	ft	
Pile cap midplane elevation =	84.5	ft	
As-modeled pile cap midplane elevation =	84.5	ft	Sheet B15 (Ref. 5)
Base of pile cap elevation =	83.0	ft	Sheet B15 (Ref. 5)
Pile length =	17.9	ft, includes upper 1-foot of bedrock (Assumption 8).	

CALCULATIONS

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Soil stratigraphy and selected models:

Soil Layer		Soil Type	Elevation (ft)		Lateral Model	Axial Model	Torsional Model	Tip Model
			Top	Bottom				
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Stiff Clay	2	Cohesive	82.0	77.7	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Medium Clay	3	Cohesive	77.7	71.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Glacial Till	4	Cohesionless	71.8	67.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)
Sandstone Bedrock	5	Rock	67.6	-	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)

Note: Stratigraphy based on Pier 1 (Sheet X Interpretive Subsurface Cross Section X-X').

B. Summarize the results from the FB-MultiPier analysis.

From the FB-MultiPier model:

	(Pile 2)					
Maximum axial force at the top of pile, P_{u_top} =	392.7	kips				
Maximum axial force at the pile tip, P_{u_tip} =	344.1	kips				
Maximum bending moment in the piles, $M_x = M_2$ =	20.7	kip-ft	=	247.9	in-kips	
Maximum bending moment in the piles, $M_y = M_3$ =	27.5	kip-ft	=	330.1	in-kips	
Maximum shear force in the piles, V_u =	8.1	kips				
Depth below pile head to fixity =	13.4	ft	=	Elev.	71.07	ft
Depth below pile cap to fixity =	11.9	ft	=	Elev.	71.07	ft

wsp

CALCULATIONS

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C. Determine the nominal and factored geotechnical resistance of a single pile in axial compression using a static analysis prediction.

Static Analysis

Compute the shaft resistance R_s for all layers through which the pile extends and the toe resistance R_p for the layer at the pile tip

Unit shaft resistance, using the Nordlund method for cohesionless soils:

$$f_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \omega)}{\cos \omega}$$ (Ref. 7, Eq. 10.7.3.8.6f-1)

where:

Pile soil displacement, V =	0.13	ft^3/ft	(from Part A)
δ/ϕ_f =	0.78		(Ref. 7, Fig. 10.7.3.8.6f-6, based on V and type "H-piles"
Angle of pile taper from vertical, ω =	0	degrees	(piles are not tapered)
Coefficient K_δ for ϕ_f of 30° =	0.89		(Ref. 7 Fig. 10.7.3.8.6f-2, based on V and ω)
Coefficient K_δ for ϕ_f of 35° =	1.23		(Ref. 7 Fig. 10.7.3.8.6f-3, based on V and ω)
Coefficient K_δ for ϕ_f of 40° =	1.87		(Ref. 7 Fig. 10.7.3.8.6f-4, based on V and ω)

	Granular Borrow	Glacial Till	
Friction angle, ϕ_f =	32	35	degrees (Ref. 6)
δ =	25	27	degrees (based on δ/ϕ_f from above)
K_δ, interpolated =	1.02	1.23	(interpolation among Ref. 7, Figs. 10.7.3.8.6f-2, -3, & -4)
Correction factor, C_F =	0.91	0.89	(Ref. 7, Fig. 10.7.3.8.6f-5, based on φf and δ/φf)
Vertical effective stress, σ'_v =	0.063	1.119	ksf (at midpoint of soil layer using stratigraphy in Part A)
$f_s = \alpha s_u$			
f_s =	0.024	0.557	ksf

Unit shaft resistance, using the alpha method for cohesive soils:

$$f_s = \alpha s_u$$ (Ref. 7, Eq. 10.7.3.8.6b-1)

where:

	Stiff Clay		Medium Clay		
	Top	Bottom	Top	Bottom	
Undrained shear strength, s_u =	1.340	1.340	0.888	0.888	ksf (Ref. 6)
Adhesion factor, a =	0.88	0.88	0.95	0.95	(Ref. 7 Fig. 10.7.3.8.6b-1, sand
f_s =	1.179	1.179	0.844	0.844	ksf

CALCULATIONS

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Nominal shaft resistance:

$R_s = f_s A_s$ (Ref. 7, Eq. 10.7.3.8.6a-4)

where:

As = pile shaft surface area. As per Ref. 7 Article C10.7.3.8.6f, for the H-piles being analyzed the perimeter or "box" area will be used to compute the surface area of the pile side.

Box perimeter = $(d \times 2) + (b_f \times 2)$ = 56.4 in = 4.7 ft (from Part A)

	Granular Borrow	Stiff Clay	Medium Clay	Glacial Till		
Thickness =	1.0	4.3	5.9	4.2	ft	(from stratigraphy in Part A)
A_s =	4.7	20.2	27.7	19.7	ft^1	
R_s per layer =	0.1	23.8	23.4	11.0	kips	

Total R_s = 58.3

Unit toe resistance, using the Kulhawy and Goodman method:

$q_p = 0.33 q_u$ for RQD ≤ 70%
$q_p = 0.80 q_u$ for RQD = 100%
(Ref. 3, Section 7.2.1.4.1)

where:

Rock quality designation, RQD = 9 % (Ref. 9, minimum RQD in borings with lab testing)
Unconfined compressive strength, q_u = 1,279 ksf (Ref. 10, minimum UCS from lab test data)

q_p = 422.1 ksf

Nominal toe resistance:

$R_p = q_p A_p$ (Ref. 7, Eq. 10.7.3.8.6a-3)

where:

$R_n = R_s + R_p$ Pile toe area, A_p = 0.13 ft^2 (from Part A)

Total R_p = 56.6 kips (bedrock)

Nominal and factored geotechnical resistance:

$R_n = R_s + R_p$ (Ref. 7, Eq. 10.7.3.8.6a-2)

R_n = 115 kips (bedrock)

$R_r = \varphi_{stat} R_n$ (Ref. 7, Eq. 10.7.3.8.6a-1)

CALCULATIONS

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where: φ_{stat} = 0.35 for shaft and toe resistance of cohesive soil by the α-method

(Ref. 7 Tbl. 10.5.5.2.3-1) φ_{stat} = 0.45 for shaft and toe resistance of cohesionless soil by the Nordlund method

φ_{stat} = 0.45 for shaft and toe resistance End Bearing in rock (Canadian Geotech Society)

	Granular Borrow	Stiff Clay	Medium Clay	Glacial Till	Pile Tip (bedrock)	
R_s or R_p =	0.1	23.8	23.4	11.0	56.6	kips
φ_{stat} =	0.45	0.35	0.35	0.45	0.5	(Ref. 7, Table 10.5.5.2.3-1)
R_r per layer =	0.1	8.3	8.2	4.9	25.5	

Total R_r = 47 kips (bedrock)

Wave Equation Analysis

Nominal and factored geotechnical resistance obtained from GRWEAP at 89% of the steel yield strength (Ref. 16):

R_n = 800 kips (Ref. 14)

φ_{dyn} = 0.65 (Assumption 10)

R_f = 520 kips

Check that the geotechnical axial resistance Obtained from GRLWEAP (Ref. 15) is sufficient to support axial load on pile:

P_u = 392.7 kips (from Part B)

R_r = 520 kips > P_u = 392.7 kips OK

E. Determine the nominal and factored structural resistance of a single pile in axial compression.

Limiting slenderness ratio:

$$\frac{Kl}{r_s} \leq 120$$ (Ref. 7, Article 6.9.3)

where:

Effective length factor, K = 1.2 (Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)

Unbraced length, l = 143.10 in (from Part B)

Minimum radius of gyration, r_s = 3.47 in (from Part A)

Check: Kl/r_s = 49 < 120 OK

CALCULATIONS

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Local buckling and equivalent nominal yield resistance :

$\frac{b}{t} \leq \lambda_r$ (Ref. 7, Eq. 6.9.4.2.1-1) $0.56\sqrt{\frac{E_{st}}{F_y}}$ (Ref. 7, Table 6.9.4.2.1-1) (Assumption 9)

where:

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Steel yield stress, F_y =	50	ksi	(from Part A)

Check:

$\frac{b}{t}$ = 15.98 > 13.49 $= 0.56\sqrt{\frac{E_{st}}{F_y}}$

Element does not satisfy the width-to-thickness ratio limits specified in Ref. 7, Article 6.9.4.2.1.

Calculate equivalent nominal yield resistance:

$P_o = F_y A_g$ (Ref. 7, Article 6.9.4.1.1)

Equivalent nominal yield resistance, P_o = 966 kips

Elastic critical buckling resistance:

$P_e = \frac{\pi^2 E_{st}}{\left(\frac{Kl}{r_s}\right)^2} A_g$ for flexural buckling (Ref. 7, 6.9.4.1.2-1)

where:

Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Cross-sectional area, A_g =	19.3	in^2	(from Part A)
Effective length factor, K =	1.2		(Ref. 7, Table C4.6.2.5-1; rotation fixed translation free)
Unbraced length, l =	143.10	in	(from Part B)
Minimum radius of gyration, r_s =	3.47	in	(from Part A)
P_e (flexural) =	2,263	kips	

Elastic and flexural torsional buckling resistance:

$\frac{D}{t_w} \leq 150$ Web proportions (Ref. 7, Eq. 6.10.2.1.1-1)

where:

Web depth, D =	12.730	in	D= d - (2 × tf)
Web thickness, t_w =	0.455	in	(from Part A)

Check: D/t_w = 28 < 150 OK

CALCULATIONS

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Since the pile is a doubly symmetric I-section member satisfying the proportion limits In Ref. 7, Article 6.10.2, torsional buckling will not be evaluated (Ref. 7, Article 6.9.4.1.3)

Nominal and factored structural resistance in axial compression:

$P_n = P_o 0.658^{P_o/P_e}$ if Po/Pe ≤ 2.25 (Ref. 7, Eq. 6.9.4.1.1-1)

otherwise

$P_n = 0.877 P_e$ (Ref. 7, Eq. 6.9.4.1.1-2)

$P_o / P_e =$ 0.43 < 2.25, so use Eq. 6.9.4.1.1-1

$P_n =$ 808 kips

For compression member cross sections containing and slender elements, the nominal compressive resistance, P_n, shall be taken as the smallest value based on the applicable modes of flexural buckling (Ref. 7, Article 6.9.4.2.2).

$$P_n = F_{cr} A_{eff}$$ (Ref. 7, Eq. 6.9.4.2.2a-1)

$$F_{cr} = \frac{P_{cr}}{A_g}$$ (Ref. 7, Eq. 6.9.4.2.2a-2)

where

P_{cr} =	808	kips	Nominal compressive resistance (flexural)
A_g =	19.3	in^2	(from Part A)
F_{cr} =	41.8	ksi	

$A_{eff} = A_g - \sum (b - b_e)t$ For rolled-sections containing slender elements (Ref. 7, Eq. 6.9.4.2.2a-3)

The effective width, b_e, of slender elements shall be determined as follows: (Ref. 7, Article 6.9.4.2.2b)

$$If \ \frac{b}{t} \leq \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b$$ (Ref. 7, Eq. 6.9.4.2.2b-1)

$$If \ \frac{b}{t} > \lambda_r \sqrt{\frac{F_y}{F_{cr}}} \quad \text{then} \quad b_e = b\left[\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} - c_3\right]$$ (Ref. 7, Eq. 6.9.4.2.2b-2)

$$F_{el} = \left(c_2 \frac{\lambda_r}{\left(\frac{b}{t}\right)}\right)^2 F_y$$ (Ref. 7, Eq. 6.9.4.2.2b-4)

CALCULATIONS

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where

Half Flange width, b_f =	7.3	in	(from Part A)
Flange thickness, t_f =	0.455	in	(from Part A)
λ_r =	13.49		(from Part D)
Fy =	50	ksi	(from Part A)
Fcr =	41.8	ksi	

$$\lambda_r \sqrt{\frac{F_y}{F_{cr}}} = 14.7$$

check b/t = 15.98 > , so use Eq. 6.9.4.2.2b-2

Determine effective width b_e :

c_1 =	0.22		(Ref. 7, Table 6.9.4.2.2b-1)
c_2 =	1.49		(Ref. 7, Table 6.9.4.2.2b-1)
c_3 =	0.00		(Ref. 7, Table 6.9.4.2.2b-1)
F_{el} =	79.09	ksi	(Ref. 7, Eq. 6.9.4.2.2b-4)
b_e =	6.97	in	(Ref. 7, Eq. 6.9.4.2.2b-2)

Determine effective area A_{eff}:

A_{eff} =	18.78	in^2	(Ref. 7, Eq. 6.9.4.2.2a-4)

Determine nominal and factored structural resistance in axial compression for a slender element:

P_n =	785.5	kips	(Ref. 7, Eq. 6.9.4.2.2a-1)

$P_{r_{axial}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.50		(Ref. 7, Article 6.5.4.2))
P_r =	393	kips	

$P_{r_{axial/flexural}} = \varphi_c P_n$ (Ref. 7 Eq. 6.9.2.1-1)

φ_c =	0.70		(Ref. 7, Article 6.5.4.2))
$P_{r,\ axial/\ flexural}$ =	550	kips	

Check that the structural axial resistance is sufficient to support axial load on pile:

P_{u_tip} =	344	kips	(from Part B)

$P_{r,\ axial}$ =	393	kips	>	P_{u_tip} =	344	kips	OK

CALCULATIONS

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F. Determine the nominal and factored structural resistance of a single pile in flexure.

Flange slenderness ratio and limiting slenderness:

(Ref. 7, Eqn 6.12.2.2.1-3)

$$\lambda_f = \frac{b_f}{2t_f}$$

λ_f = 15.98

(Ref. 7, Eqn 6.12.2.2.1-4)

$$\lambda_{pf} = 0.38\sqrt{\frac{E_{st}}{F_y}}$$

λ_{pf} = 9.15

(Ref. 7, Eqn 6.12.2.2.1-5)

$$\lambda_{rf} = 0.83\sqrt{\frac{E_{st}}{F_y}}$$

λr_f = 19.99

Nominal and factored structural resistance in flexure:

$$M_n = M_p = 1.5F_yS_y \quad \text{if } \lambda_f \le \lambda_{pf}$$ (Ref. 7, Eqn 6.12.2.2.1-1)

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E_{st}}{F_y}}}\right)\right]F_yZ_y \quad \text{if } \lambda_{pf} < \lambda_f \le \lambda_{rf}$$ (Ref. 7, Eqn 6.12.2.2.1-2)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Steel elastic modulus, E_{st} =	29,000	ksi	(from Part A)
Elastic section modulus about x, S_x =	95.8	in^3	(from Part A)
Elastic section modulus about y, S_y =	32.1	in^3	(from Part A)
Plastic section modulus about x, Z_x =	105.7	in^3	(from Part A)
Plastic section modulus about y, Z_y =	48.8	in^3	(from Part A)

Since λpf < λf <= λrf, use Eq. 6.12.2.2.1-2:

	x-axis	y-axis	
M_n =	4,973	1,913	in-kips
φ_f =	1.00		(Ref. 7, Article 6.5.4.2)
M_{rx} =	4,973	in-kips	
M_{ry} =	1,913	in-kips	

Check that the structural flexural resistance is sufficient to support bending moment in pile:

M_{ux} =	248	in-kips (from Part B)
M_{uy} =	330	in-kips (from Part B)

M_{rx} =	4,973	in-kips	>	M_{ux} =	248	in-kips	OK
M_{ry} =	1,913	in-kips	>	M_{uy} =	330	in-kips	OK

wsp

CALCULATIONS

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G. Check the combined axial compression and flexure interaction.

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} < 0.2$ (Ref. 7, Eq. 6.9.2.2.1-1)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0$$ if $P_u/P_{r,\ axial/\ flexural} \ge 0.2$ (Ref. 7, Eq. 6.9.2.2.1-2)

$P_u/P_{r,\ axial/\ flexural}$ = 0.6 >= 0.2, so use Eq. 6.9.2.2.1-2

Check: 0.82 < 1.0 OK

H. Determine the nominal and factored structural resistance of a single pile in shear.

Check that the structural shear resistance is sufficient to support shear load on pile:

$$V_p = 0.58F_y d_w t_w$$ (Ref. 7, 6.10.9.2-2))

$$V_n = CV_p = V_{cr}$$ (Ref. 7, 6.10.9.2-1)

where:

Steel yield stress, F_y =	50	ksi	(from Part A)
Web depth, d_w =	12.730	in	$d_w = d - (2 \times t_f)$
Web thickness, t_w =	0.455	in	(from Part A)

$$\frac{D}{t_w} \le 1.12\sqrt{\frac{Ek}{F_y}}$$ then C = 1.0 (Ref. 7, Eq. 6.10.9.3.2-4)

where:

k =	5.0		(Ref. 7, Article 6.10.9.2)
D =	12.730	in	D= d - (2 × tf)
t_w =	0.455	in	(from Part A)
$1.12\sqrt{\frac{Ek}{F_y}}$ =	60.3	ksi	(Ref. 7, Eq. 6.10.9.3.2-4)

Check: D/t_w = 28 < 60.3 C = 1.0

V_p =	168	kips
V_n =	168	kips

$V_u \le \varphi_v V_n$

φ_v =	1.00		(Ref. 7, Article 6.5.4.2)
V_r =	168	kips	

CALCULATIONS

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Check that the structural shear resistance is sufficient to support shear load on pile:

$V_u \leq \varphi_v V_n$ (Ref. 7, Eq. 6.10.9.1-1)

$V_u =$ 8.1 kips (from Part B)

$V_r =$	168	kips	>	$V_u =$	8.1	kips	OK

H. Determine nominal pile driving resistance.

$P_u =$ 393 kips (from Part B)

$\varphi_{dyn} =$ 0.65 (Ref. 3, Table 7-2)

for driving criteria established by dynamic testing of at least two piles per site condition and no less than 2% of production piles

Nominal pile driving resistance, $R_{ndr} =$ 604 kips

Confirm R_{ndr} does not exceed P_n:

R_{ndr}			P_n	(from Part D)	
604	kips	<	785	kips	OK

CALCULATIONS

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CONCLUSIONS

The results of the analysis indicate that under the Strength I load combination, load case 2, and an estimated pile length of 17.9 feet, a maximum axial force of 392.7 kips at the top of the pile, the corresponding strong axis (y-axis) moment of 330.1 in-kips (27.5 ft-kips) occurs in the HP14x89 piles at Pier 4B in Pile 2 of the proposed westbound Hogan Road Bridge. The estimated pile length is sufficient for the piles to achieve fixity. The analysis indicates that the maximum top of pile deflection at the service limit state is less than 0.15 inches in the X and Y directions. The WB Pier 4 piles should be driven to a nominal resistance of 604 kips. This evaluation considers section loss for a 100 year design life, this evaluation does not consider downdrag as none is anticipated. A summary table is presented below.

Pier 4B Summary Table		
Pile Length (ft)		17.9
Depth below Pile Head to Fixity (ft)		13.4
Geotechnical Axial Resistance (kips)	Nominal	800
	Factored	520
Structural Axial Resistance (kips)	Nominal	808
	Factored	393
Structural Flexural Resistance around X-axis (kip-ft)	Nominal	414
	Factored	414
Structural Flexural Resistance around Y-axis (kip-ft)	Nominal	159
	Factored	159
Combined Axial and Flexural Interaction Ratio		0.82
Shear Resistance (kips)	Nominal	168
	Factored	168

Attachment 1 - Pile Model Output Images and Pile Cap Dimensions

Figure 1a: Plan View of Pile Cap

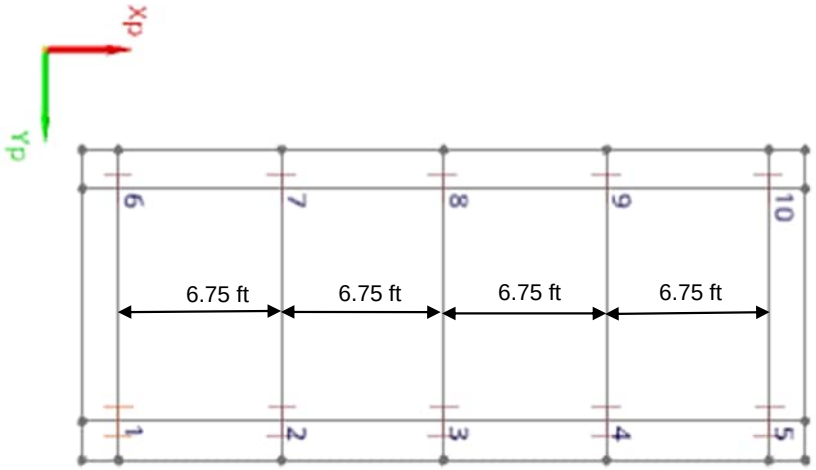


Figure 1b: Profile View of Pile Cap (Transverse)

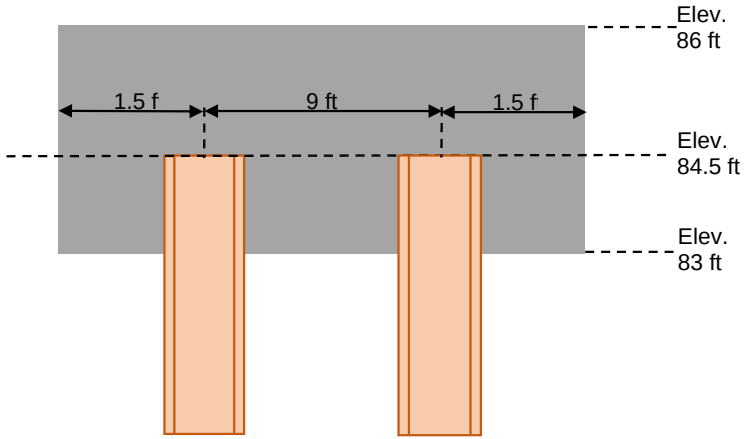


Figure 1c: 3D View of Pile Model

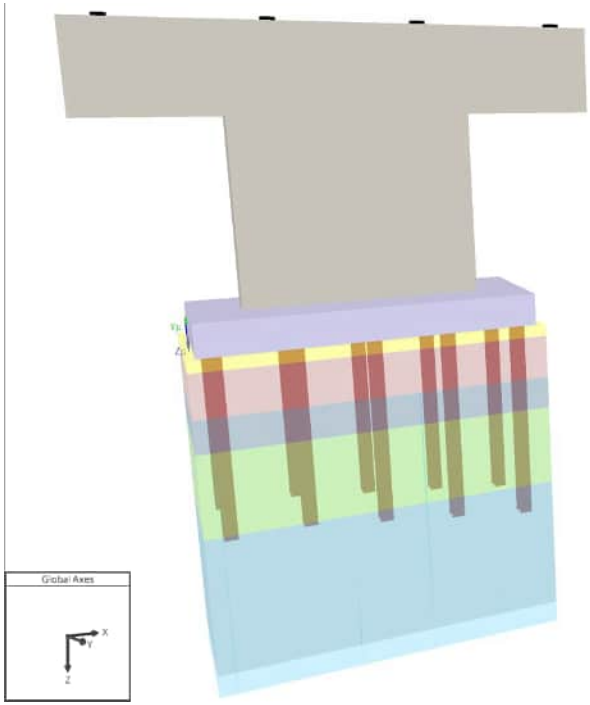
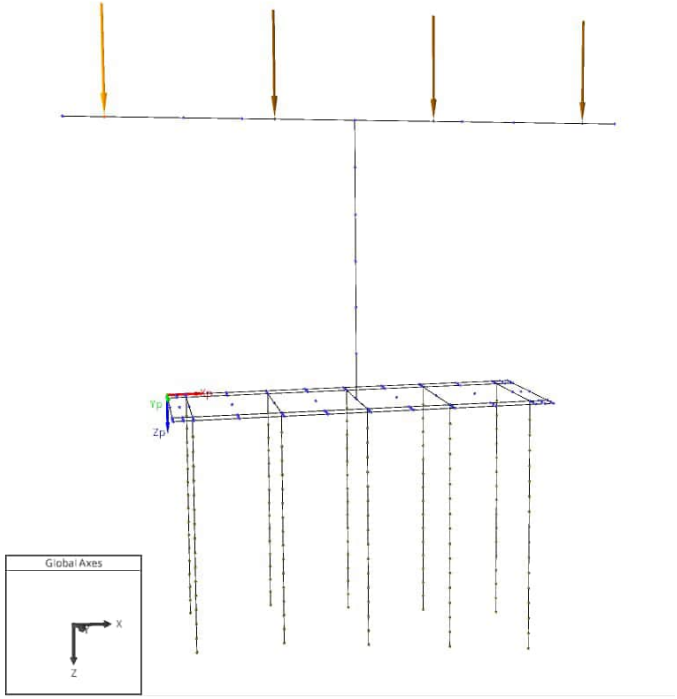


Figure 1d: 3D View of Model Loading



Attachment 2 - Table of FB-MultiPier model input parameters

Soil Layer		Soil Type	Elevation (ft) [1]		Lateral Model	Axial Model	H	Tip Model	Unit Weight (pcf) [2]	Friction Angle (°) [2]	Subgrade Modulus (pci) [3]	Undrained Shear Strength (psf) [2]		Major Principal Strain @ 50% [4]	Major Principal Strain @ 100% [4]	Shear Modulus (ksi) [3]		Poisson's Ratio [3]	Nominal Unit Skin Friction and Torsional Shear Stress (psf) [5,6,7]		Nominal Unit Tip Resistance (ksf) [6,7]	Nominal Tip Resistance (kips)	UCS (psf) [8]
			Top	Bottom								Top	Bottom			Top	Bottom		Top	Bottom			
Granular Borrow	1	Cohesionless	83.0	82.0	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	125	32	88	-		-	-	0.46		0.30	24		-	-	-
Stiff Clay	2	Cohesive	82.0	77.7	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	122	-	-	1340	1340	0.005	0.015	2.64	2.64	0.50	1179	1179	-	-	-
Medium Clay	3	Cohesive	77.7	71.8	Clay (O'Neill)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	121	-	-	888	888	0.010	0.030	1.44	1.44	0.45	844	844	-	-	-
Glacial Till	4	Cohesionless	71.8	67.6	Sand (Reese)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	130	35	88	-		-	-	0.74		0.38	557		-	-	-
Bedrock	5	Rock	67.6	60.0	Limestone (McVay)	Driven Pile (McVay)	Hyperbolic	Driven Pile (McVay)	158	-	-	-		-	-	47.33		0.20	20000		4412	800	1,278,785

Notes:

1. The in-situ soil stratigraphy is based on Pier 4 profile (Sheet B15 Pier 4 Elevation).

2. The unit weight, friction angle, and undrained shear strength values are determined from results of local engineering experience, field vane tests, and standard penetrating testing (SPT) at site. Details are provided in the Lab Field Data Summary Plots.

3. The subgrade modulus, shear modulus, and Poisson's ratio values are interpolated based on friction angle (for cohesionless soils), undrained shear strength (for cohesive soils), or RQD (for bedrock) using the Bridge Software Institute FB-MultiPier Soil Parameter Table (*https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf)* . For the cohesionless shear modulus, it is assumed that α = 5 (sands with fines).

4. Strain values for the stiff and medium clay layers at 50%, and the medium clay at 100% are taken from the Bridge Software Institute FB-MultiPier Soil Parameter Table (https://bsi.ce.ufl.edu/downloads/files/MultiPier_Soil_Table.pdf). Strain values for the Stiff clay layer at 100% are taken from consolidated undrained triaxial testing of sample U1 in BB-BHR-201.

5. Unit skin friction values for cohesionless soil are calculated from the friction angle using the Nordlund method (FHWA GEC-12, Section 7.2.1.3.1).

6. Unit skin friction and unit tip resistance values for cohesive soil are calculated from the undrained shear strength using the α-method (FHWA GEC-12, Section 7.2.1.3.2).

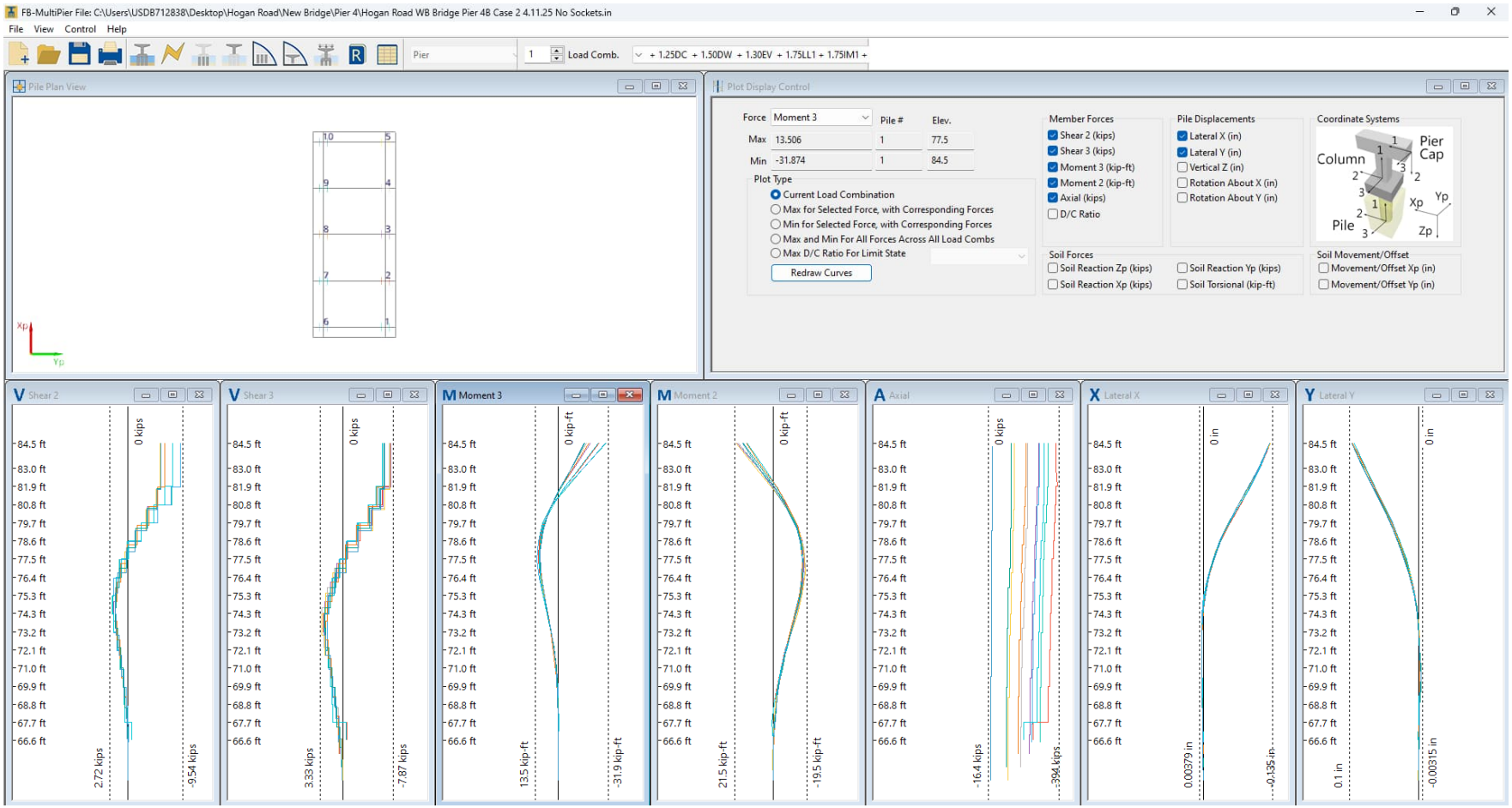
7. Unit skin friction and unit tip resistance values for rock are calculated from UCS, RQD, and rock type using Table 7-13 (FHWA GEC-12) for shaft resistance and the Kulhawy and Goodman method (FHWA GEC-12, Section 7.2.1.4.2) for tip resistance.

8. Uniaxial compressive strength of rock is taken from lab testing results for BB-BHR-201 Run 4, BB-BHR-206 Run 1, BRP-BHR-202 Run 1, and BRP-BHR-203 Run 1. The minimum UCS from testing is used.

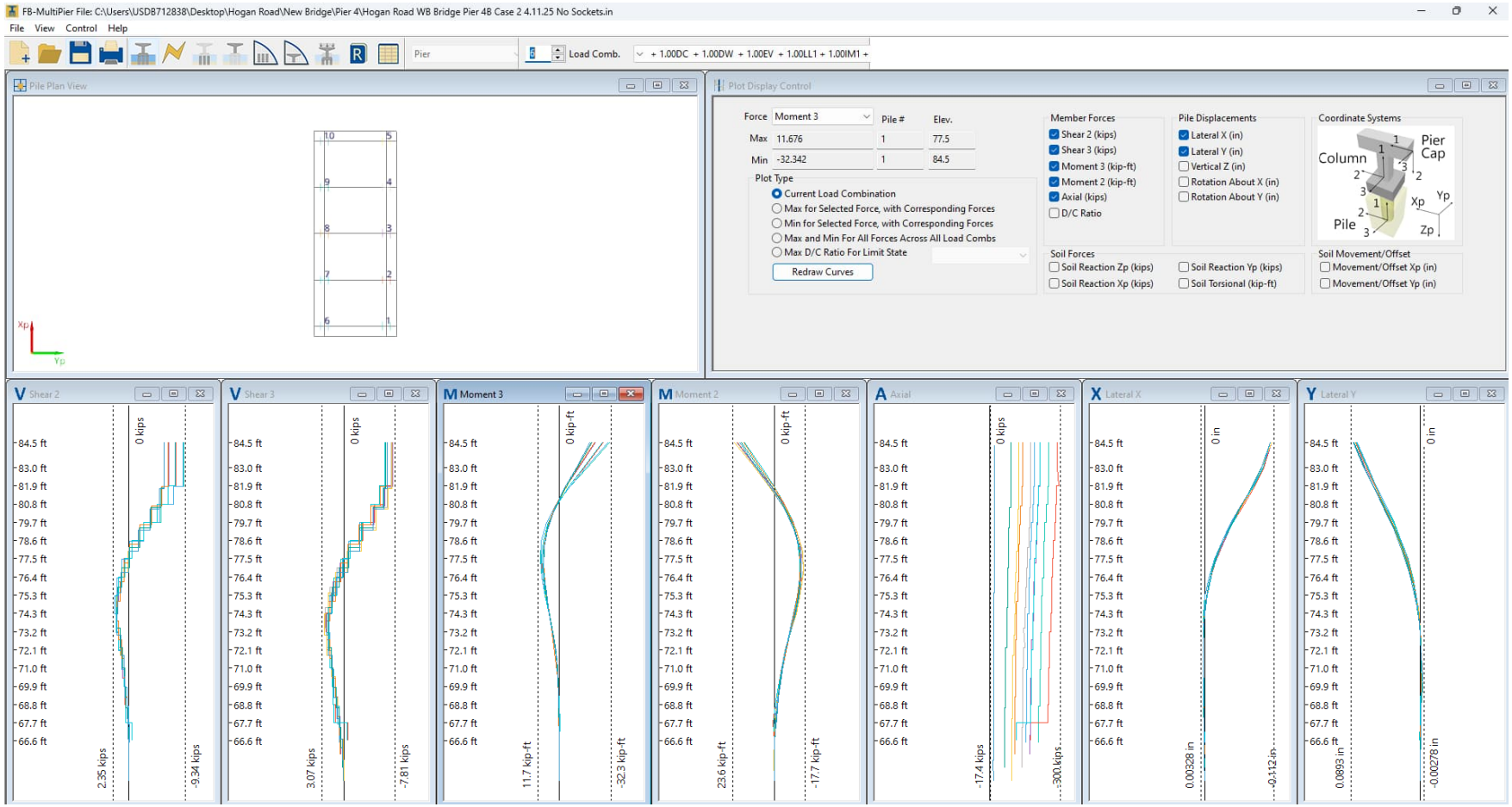
Prepared: DEB
Checked: ATM
Reviewed:

Attachment 3 - FB-Multipier Strength I and Service I Output

Case 2 Strength I



Case 2 Service I



Project	Bangor DDI Hogan Rd WB	Project #	55758.00
Location	Bangor, ME	Sheet #	1 of 1
Calculated By	YP	Date	10/4/2024
Checked By	SMH	Date	10/23/2024
Title	Bangor DDI Hogan Rd WB Bearing Loads		

Computation

Unfactored Bearing Loads, Pier

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							6	135	128
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	18	18	18	18	18	18			
	WS	0	0	5	0	0	5			
	WL	0	0	3	0	0	3			

	Case	Pier Bearing Reactions								
		Fx (kip), Longitudinal			Fy (kip), Transverse			Fz (kip), Vertical		
		G1	G2	G3	G1	G2	G3	G1	G2	G3
Stem A	LL1							128	135	6
	LL2							100	141	100
	DC							148	148	148
	DW							32	32	32
	FR	-18	-18	-18	-18	-18	-18			
	WS	0	0	-5	0	0	-5			
	WL	0	0	-3	0	0	-3			

	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									0	14	116	169
	LL4									56	91	121	82
	DC									148	148	148	148
	DW									32	32	32	32
	FR	18	18	18	18	18	18	18	18				
	WS	5	5	0	0	5	5	0	0				
	WL	3	3	0	0	3	3	0	0				

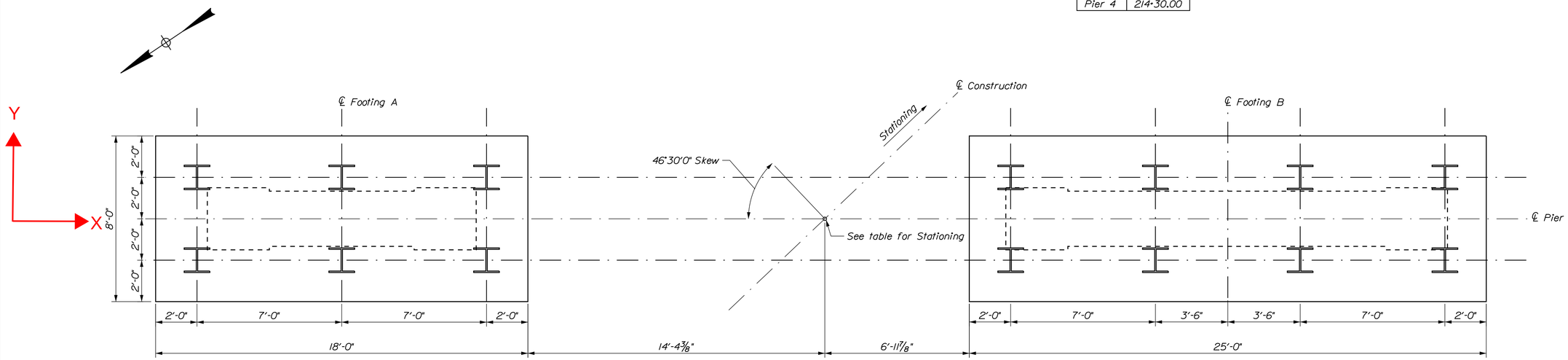
	Case	Pier Bearing Reactions											
		F_x (kip), Longitudinal				F_y (kip), Transverse				F_z (kip), Vertical			
		G4	G5	G6	G7	G4	G5	G6	G7	G4	G5	G6	G7
Stem B	LL3									169	116	14	0
	LL4									82	121	91	56
	DC									148	148	148	148
	DW									32	32	32	32
	FR	-18	-18	-18	-18	-18	-18	-18	-18				
	WS	-5	-5	0	0	-5	-5	0	0				
	WL	-3	-3	0	0	-3	-3	0	0				

Filename: ...\MSTA\B25_WB_Pier_Footing.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Location	Stationing
Pier 1	211+27.00
Pier 2	212+28.00
Pier 3	213+29.00
Pier 4	214+30.00

PIER NOTES

1. Reinforcing steel shall have a minimum concrete cover of 3 inches unless otherwise noted.



FOOTING PLAN & PILE LAYOUT

PIER FOOTING REINFORCING PLAN

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON	
DESIGN-DETAILED	JAH	8/31/24
CHECKED-REVIEWED	YP	8/31/24
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

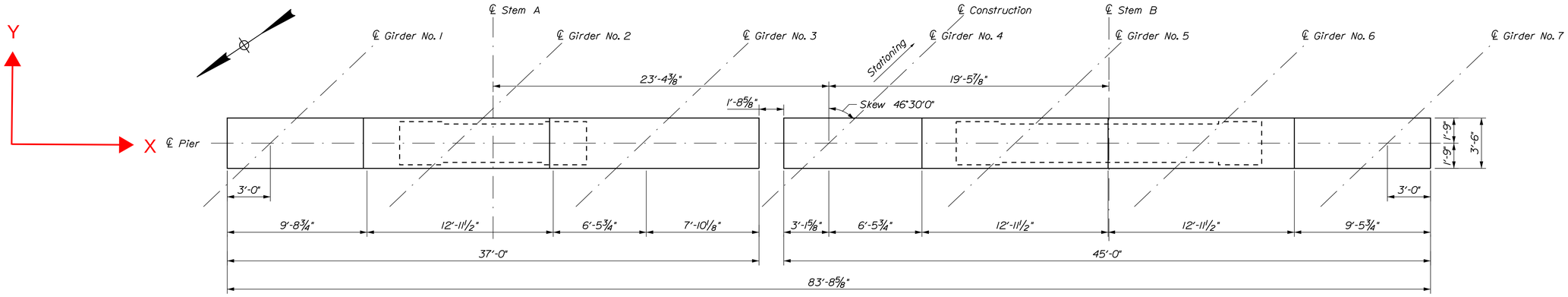
HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER TYPICAL
FOOTING PLAN

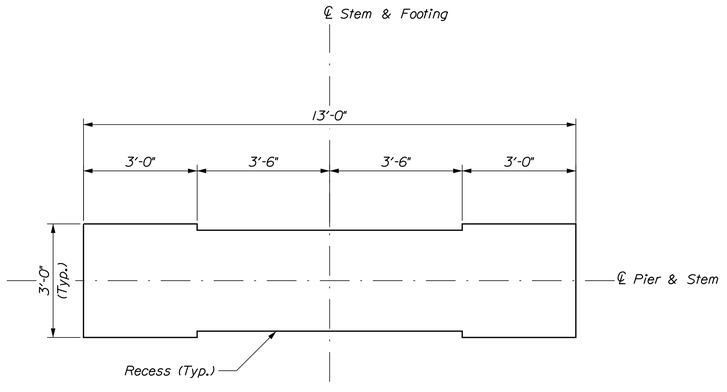
SHEET NUMBER

B25

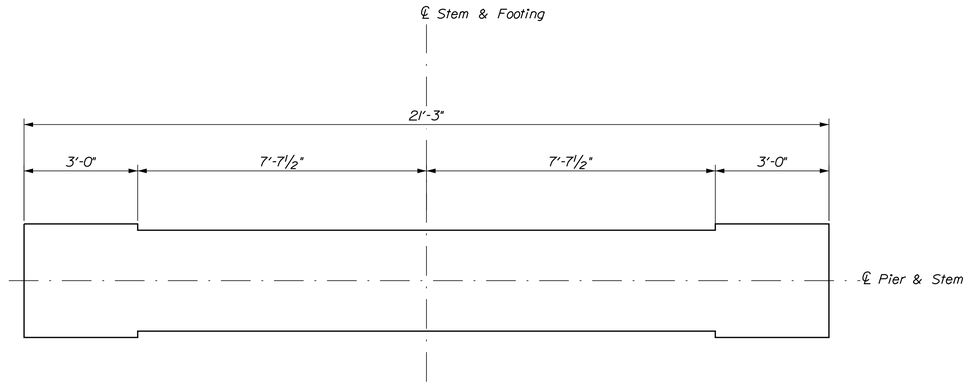
OF B49



PIER CAP PLAN
(Pier Footing Not Shown)



STEM A SECTION



STEM B SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN
18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER CAP TYPICAL PLAN
& COLUMN DETAILS

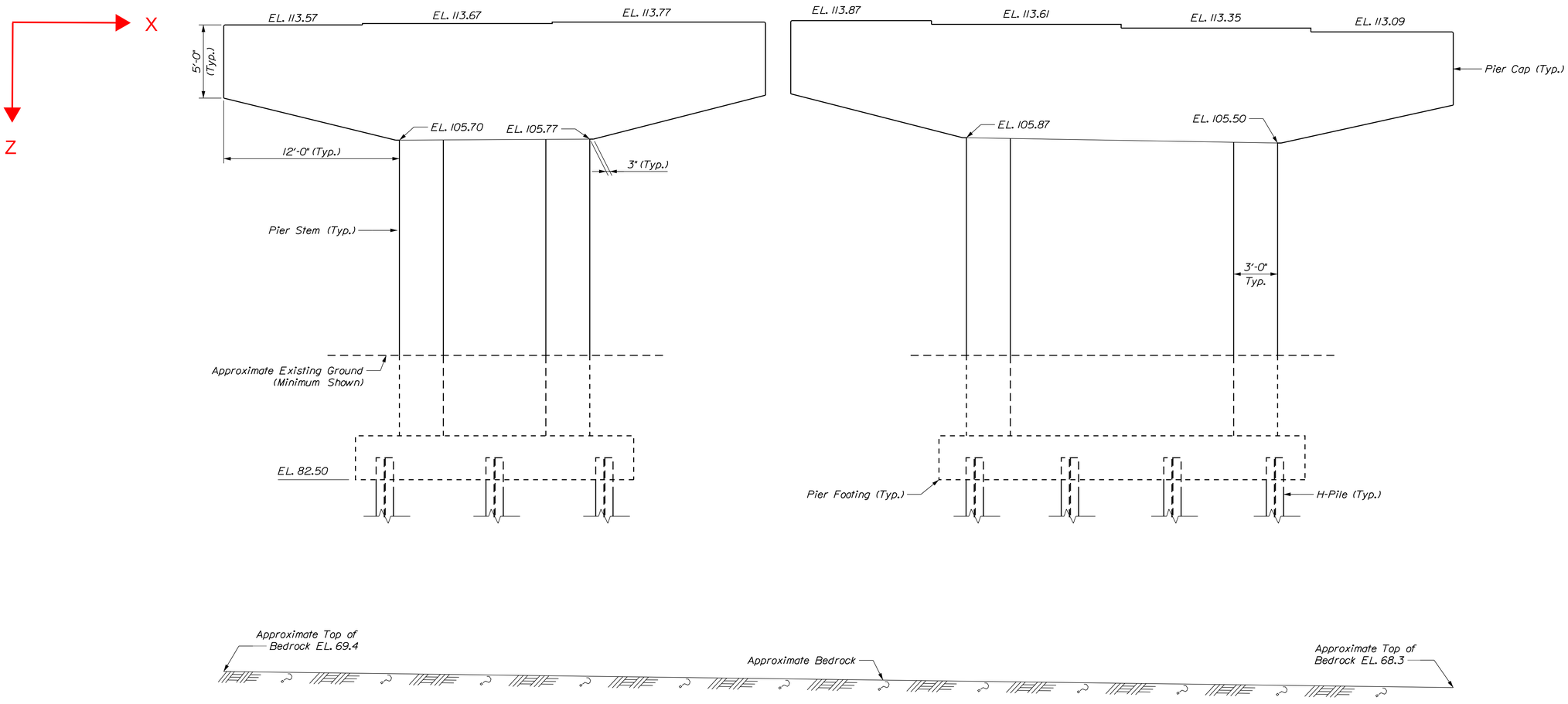
SHEET NUMBER

B26

OF B49

Filename: ...\MSTA\B26_WB_Pier_Cap.dgn Division: HIGHWAY Username: jharris Date:8/30/2024

Filename: ...\BRIDGE\MSTA\B28_WB_Pier_1.dgn Division: HIGHWAY Username: jharris Date:8/30/2024



ELEVATION

Note To Reviewer:

Estimated Bedrock shown for informational purposes and will be removed from final plans typ.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18595.10

WIN 18595.10

BRIDGE PLANS

60% BRIDGE PLANS
NOT FOR CONSTRUCTION
8/31/2024

vhb

PROJ. MANAGER	R. MOULTON		
DESIGN-DETAILED	JAH	BY	DATE
CHECKED-REVIEWED	YP	JAH	8/31/24
DESIGN2-DETAILED2		SMH	8/31/24
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

HOGAN ROAD WESTBOUND BRIDGE
I-95
BANGOR PENOBSCOT

PIER NO. 1 ELEVATION

SHEET NUMBER

B28

OF B49

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Pier 4 - HP14x89 - Resistance	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine the factored pile resistance of the proposed HP 14x89 piles at WB Pier 4 based on the driving criteria outline in the MaineDOT Standard Specification March 2020.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Pier 4 H-Pile Analysis, dated 4/11/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.
9. Federal Highway Administartion (FHWA), FHWA-NHI-126-009 - Geotechnical Engneering Circular 12, Design and Construction of Driven Piles Vol II, July 2016
10. WSP boring logs.

ASSUMPTIONS

1. Pile tip elevations are based on an interpretation of the as-drilled top of bedrock elevations (Ref. 10 and Ref. 3 Sheet B15).

2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 1 (soil set 1).

3. As per FHWA GEC 12 Section 14.2.6.C-8 (Ref. 9), absolute refusal during pile driving is defined as 20 blows for one inch or less or 10 blows for a half inch can be used to reduce the risk of pile damage. Driving stresses shall not exceed 90% of the pile yield strength per Maine Department of Transportation 2020 Standard Specifications Section 501 Foundation (Ref. 4).

4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap and strike plate is assumed for the analysis (Ref. 8).

5. This analysis looks at resistance upon reaching the tip elevation in bedrock and does not consider the resistance during driving. A resistance distribution of 10% shaft resistance and 90% toe resistance is assumed upon reaching the tip elevation.

6. A φdyn of 0.65 using (Ref. 9 Table 10.5.5.2.3-1).is assumed based on the use of dynamic testing on at least two piles per site condition but no less than 2% of production piles.

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Pier 4 - HP14x89 - Resistance	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using monocast cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in	(Ref. 2)
Section width, b_f =	14.7	in	(Ref. 2)
Web thickness, t_w =	0.615	in	(Ref. 2)
Flange thickness, t_f =	0.615	in	(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2 = 0.18 ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi	(Grade 36, Ref 3. Sheet 3 of 53)
90% of steel yield stress =	45	ksi	
Steel elastic modulus, E_{st} =	29,000	ksi	(typical value for steel)
Steel unit weight =	491	pcf	
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in = 4.8 ft	
Pile head elevation =	84.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)
Pile tip elevation =	66.6	ft	(Ref. 3, Sheet 13 of 53)
Pile length =	17.9	ft	(Ref. 3, Sheet 13 of 53)

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.13	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation =	84.5	ft	
Groundwater elevation =	78.0	ft = 6.5	ft below pile head

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Granular Borrow	83.0	82.0	1.5	2.5	125	32	-	-	-	Sand
Stiff Clay	82.0	77.7	2.5	6.8	122	-	1340	-	-	Clay
Medium Clay	77.7	71.8	6.8	12.7	121	-	888	-	-	Clay
Glacial Till	71.8	67.6	12.7	16.9	130	35	-	-	-	Sand
Competent Bedrock	67.6	-	16.9	-	158	-	-	20.0	422.0	Rock Other

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Pier 4 - HP14x89 - Resistance	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

2. Analyze an open ended diesel (OED) hammer to determine the R_{ndr} for the abutment based on the stress and blow counts outline in Ref. 4.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fule Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
Reaching tip elevation	10%	90%	APE D25-42	45,892	Fuel Setting 2 (74%)	800	20.0	44.74	89%

3. Determine nominal and factored toe resistance.

Nominal toe resistance = 800 kips (Intact section from GRLWEAP)

φ_{dyn} = 0.65 (Ref. 9, Table 10.5.5.2.3-1)

Factored toe resistance = 520.0 kips

CONCLUSIONS

This analysis indicates that an HP14x89 pile driven with an APE D25-42 hammer operated at 74% of the combustion pressure is expected to develop a nominal bearing resistance of 808 kips at WB pier 4 while limiting driving stresses to a maximum of 45 ksi. Blow counts were limited to 10 blows per 1/2" of penetration, in accordance with FHWA GEC 12 (Ref. 9) and Section 501 of the Maine Department of Transportation 2020 Standard Specifications for Construction (Ref. 4). The resulting factored toe resistance from this WEAP analysis is 520 kips.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

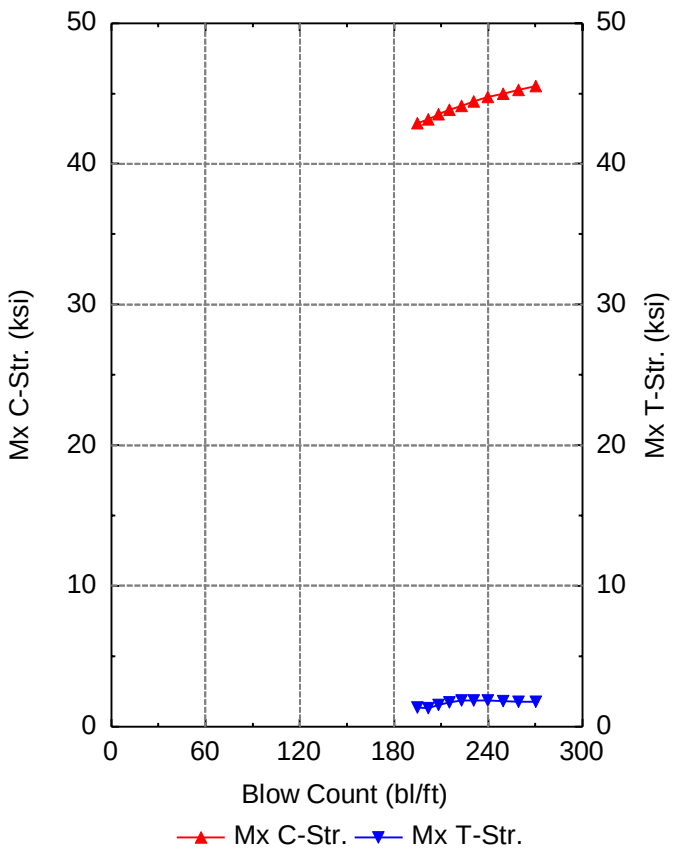
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

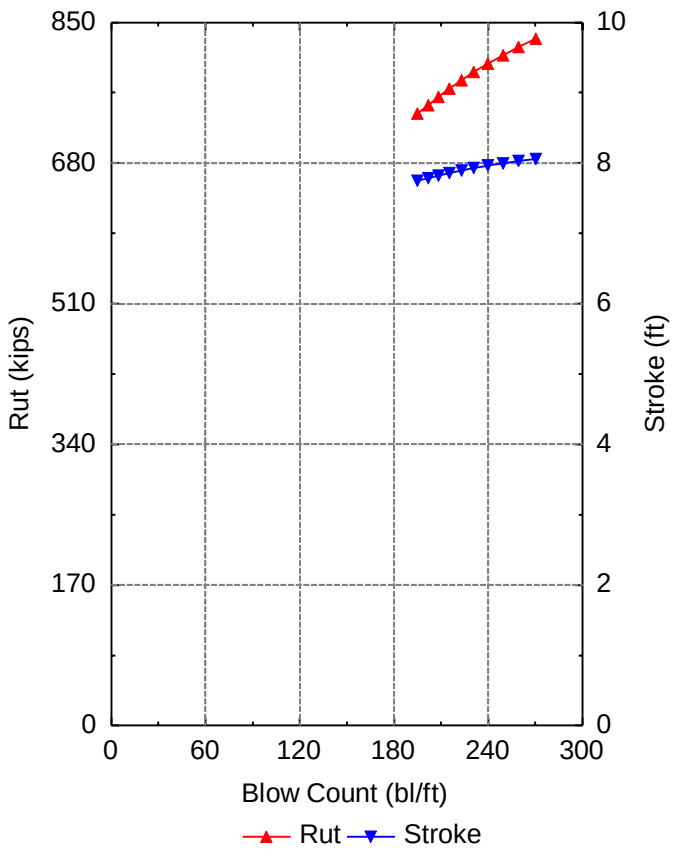
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

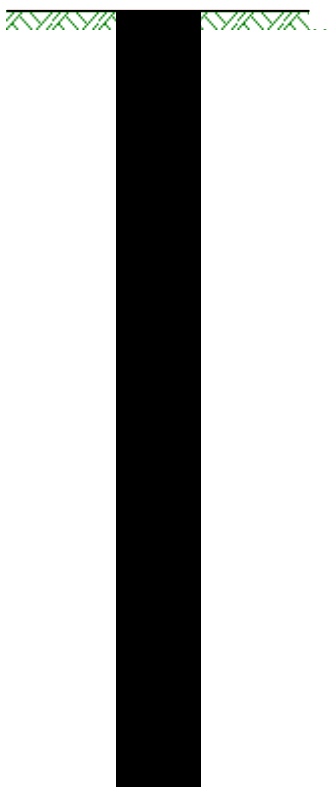
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



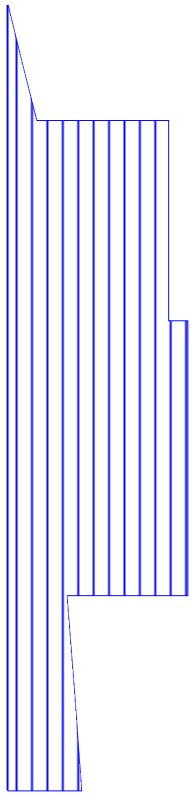
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1054.5 (74%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.130	s/ft
Toe Damping	0.150	s/ft
Pile Length	16.90	ft
Pile Penetration	16.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=10%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham.	Pow. Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
740.0	42.87	42.87	1.33	194.9	7.75	13.96	D 25-42	22.5
750.0	43.17	43.17	1.32	201.8	7.78	14.02	D 25-42	22.6
760.0	43.50	43.50	1.53	208.3	7.82	14.11	D 25-42	22.8
770.0	43.82	43.82	1.73	215.2	7.86	14.20	D 25-42	22.9
780.0	44.14	44.14	1.85	223.1	7.89	14.26	D 25-42	23.0
790.0	44.45	44.45	1.85	230.8	7.93	14.35	D 25-42	23.1
800.0	44.74	44.74	1.82	239.6	7.96	14.41	D 25-42	23.2
810.0	45.00	45.00	1.78	249.6	7.99	14.46	D 25-42	23.3
820.0	45.27	45.27	1.75	259.2	8.03	14.53	D 25-42	23.4
830.0	45.51	45.51	1.76	270.4	8.06	14.57	D 25-42	23.5

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Pier 4 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

OBJECTIVE

Perform a drivability analysis using GRLWEAP to determine if the proposed H-piles at WB Pier 4 can be driven to the nominal pile driving resistance while maintaining blow counts and pile stresses within the specified limits.

REFERENCES

1. GRLWEAP 14 Software Package Version 14.1.20.1, Built December 1, 2022.
2. AISC Shapes Database v15.0. November 2017. https://www.aisc.org/globalassets/aisc/manual/v15.0-shapes-database/aisc-shapes-database-v15.0.xlsx
3. Bangor Hogan Road DDI 60% Westbound Bridge Plans dated 8/31/2024.
4. State of Maine, Department of Transportation. March 2020. Standard Specifications.
5. Guertin Elkerton & Associates for Maine Department of Transportation. Bridge Design Guide. Dated August 2003 with 2018 updates.
6. WSP calculation package titled Hogan Road WB Bridge Pier 4 H-Pile Analysis, dated 4/11/2025.
7. APE Model D25-42 Single Acting Diesel Impact Hammer, https://www.americanpiledriving.com/pdfs/diesels/specs/d25-42.pdf, Accessed April 2025.
8. AASHTO LRFD Bridge Design Specifications, 9th Ed. 2020.

ASSUMPTIONS

1. The proposed Pier 4A is assumed to be supported on eight (8) plumb HP14x89 Grade 50 piles with a center-to-center pile spacing of 9 feet in the x-direction and 9 feet in the y-direction, and the proposed Pier 4B is assumed to be supported on ten (10) plumb HP 14x89 Grade 50 piles with a center-to-center spacing of 9 feet in the y-direction and 6.75 feet in the x-direction (Ref. 3). The piles are to be driven to bedrock.
2. The soil profile used in the analysis is based on the profile used in the H-pile analysis (Ref. 6) and shortest installed pile at Abutment 1 (soil set 1).
3. As per the MaineDOT Standard Specifications Section 501.042 (Ref. 4), the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material.
4. An APE D 25-42 single acting diesel pile driving hammer with a box lead size of 26 inches, and a DB 26 drive cap, DB-26 Micarta cushion, and DB-26 strike plate is assumed for the analysis (Ref. 7).

ATTACHMENTS

1. GRLWEAP tables of recommended quake and damping values for impact driven piles.
2. GRLWEAP output for a variable resistance analysis using the APE D25-42 hammer.

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
Project No.:	31404817.004	**Checked by:**	ATM
Subject:	Pile Drivability at WB Bridge # 6787 Pier 4 - HP14x89 - Driving	**Reviewed by:**	CCB
Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

CALCULATION

1. Determine the input parameters.

Cushion Information:

Using micarta cushion and DB 26 Striker Plate and drive cap (Assumption 4).

Cushion area =	398.0	in^2	(Ref. 8)
Elastic modulus =	285.0	ksi	(Ref. 8)
Cushion thickness =	3.5	in	(Ref. 8)
Coefficient of restitution =	0.80		(Ref. 8)
Helmet weight =	2.7	kips	(Ref. 8)

Pile Information:

For an HP14x89 pile section:

Intact Section

Section depth, d =	13.8	in				(Ref. 2)
Section width, b_f =	14.7	in				(Ref. 2)
Web thickness, t_w =	0.615	in				(Ref. 2)
Flange thickness, t_f =	0.615	in				(Ref. 2)
Cross-sectional area, A_s =	26.1	in^2	=	0.18	ft^2	(Ref. 2)
Steel yield stress, F_y =	50	ksi				(Grade 50)
90% of steel yield stress =	45	ksi				
Steel elastic modulus, E_{st} =	29,000	ksi				(typical value for steel)
Steel unit weight =	491	pcf				
Box perimeter = (d × 2) + (b_f × 2) =	57.0	in	=	4.8	ft	
Pile head elevation =	84.5	ft	(Ref. 3, Sheet 10 of 53, with 1.5-foot pile cap embedment)			
Pile tip elevation =	66.6	ft	(Ref. 3, Sheet 13 of 53)			
Pile length =	17.9	ft	(Ref. 3, Sheet 13 of 53)			

Ultimate Capacities:

Axial load on pile, P_u =	394	kips	(Ref. 6)
φ_{dyn} =	0.65		(Ref. 8, Table 10.5.5.2.3-1)
Pile driving resistance, R_{ndr} =	607	kips	

For soil above bedrock while driving pile to bedrock:

Nominal shaft resistance, R_s =	58	kips	(Ref. 6)
Maximum nominal toe resistance of soil, $R_{p\text{-}soil}$ =	57	kips	(Ref. 6)
Total soil resistance =	115	kips	

Quake and Damping Soil Parameters:

Shaft quake =	0.10	in	(Ref. 1, included as Attachment 1)
Toe quake during driving =	0.123	in	D/120 for very dense soils (Ref. 1, included as Attach. 1)
Toe quake reaching tip elev. =	0.04	in	for hard rock (Ref. 1, included as Attachment 1)
Shaft damping =	0.13	s/ft	for mixed soils (Ref. 1, using Variable Resistance Distr.)
Toe damping =	0.15	s/ft	(Ref. 1, included as Attachment 1)

Shaft and Toe Resistance (during driving):

Nominal shaft resistance, R_s =	58	kips	=	51	% of R_n	(Ref. 6)
Nominal toe resistance, $R_{p\text{-}rock}$ =	57	kips	=	49	% of R_n	(Ref. 6)
Nominal resistance, R_n =	115	kips				

Assume that once the pile has encountered bedrock, the resistance distribution will switch to 49% shaft resistance and 51% toe resistance. A second analysis will be performed to model these conditions.

Soil Profile Input, to calculate Variable Resistance Distribution:

(Ref. 8)

Pile head elevation =	84.5	ft			
Groundwater elevation =	78.0	ft	=	6.5	ft below pile head

CALCULATIONS

Date:	5/13/2025	**Made by:**	DEB
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Project Title:	I-95 Hogan Road Bridge Replacement (Exit 187) Bangor, ME (WIN 18595.10)		

Soil Layer (input into GRLWEAP)	Elevation (ft)		Depth below pile head (ft)		Unit Weight (pcf)	Friction Angle (°)	Average Undrained Shear Strength (psf)	Skin Friction	End Bearing	Soil Type
	Top	Bottom	Top	Bottom				(ksf)	(ksf)	
Granular Borrow	83.0	82.0	1.5	2.5	125	32	-	-	-	Sand
Stiff Clay	82.0	77.7	2.5	6.8	122	-	1340	-	-	Clay
Medium Clay	77.7	71.8	6.8	12.7	121	-	888	-	-	Clay
Glacial Till	71.8	67.6	12.7	16.9	130	35	-	-	-	Sand
Competent Bedrock	67.6	-	16.9	-	158	-	-	20.0	422.0	Rock Other

2. Analyze an open ended diesel (OED) hammer to determine the blow count and stress at Rndr for the abutment.

Analysis Case	Shaft Resistance	Toe Resistance	Hammer	Energy / Power (lb-ft)	Fuel Setting (%)	R_{ndr} (kips)	Blows/in	Stress (ksi)	Percent of Steel F_y (ksi)
During Driving	51%	49%	APE D25-42	55,814	Fuel Setting 3 (90%)	607	9.4	34.7	69%
Reaching tip elevation	49%	51%	APE D25-42	55,814	Fuel Setting 2 (74%)	607	7.9	38.4	77%

CONCLUSIONS

The analysis indicates the APE D25-42 hammer operated at fuel setting 3 (90% of the combustion pressure) while driving through soil and upon reaching bedrock should be able to overcome a soil resistance of 115 kips in order to drive the piles at WB Pier 4 to bedrock and a nominal resistance of 607 kips while staying at a blow count between 3 and 15 blows per inch and limiting the driving stresses to below 45 kips per square inch (ksi), in accordance with Section 501.042 of the MaineDOT Standard Specifications.

Reference Tables

Table of Recommended Quake Values

Recommended Quake Values for Impact Driven Piles*

	Soil Type	Pile Type or Size	Quake (in)	Quake (mm)
Shaft Quake	All soil types	All Types	0.10	2.5
Toe Quake	All soil types, soft Rock	Non-displacement piles** i.e. driving unplugged	0.10	2.5
	Very dense or hard soils	Displacement Piles*** of diameter or width D	D/120	D/120
	Loose or soft soils	Displacement Piles*** of diameter or width D	D/60	D/60
	Hard Rock	All Types	0.04	1.0

*For vibratory driven piles in cohesive soils, quakes should be doubled.
** Non-displacement piles are sheet pile, H-Piles, or open-ended pipe piles which are not plugging during driving. Normally it can be assumed that pipe piles with diameters of 30 inches (900 mm) or more will not plug during driving while H-Piles and pipe piles of diameter 20 inches (500 mm) or less will plug during driving into a bearing layer. Between 20 and 30 inches (500 and 750 mm), pipe piles may or may not plug.
*** Displacement piles are closed-ended pipe piles, pipe piles, or H-Piles that are plugged during driving and solid concrete piles. Normally, we would analyze H-Piles and pipe piles with diameters 20 inches (500 mm) or less as displacement piles.

Recommended quake values, both shaft and toe, for vibratory driven piles are somewhat speculative recommendations as not much experience exists.

Reference Tables

Table of Recommended Damping Values

Recommended Damping Values for Impact Driven Piles:

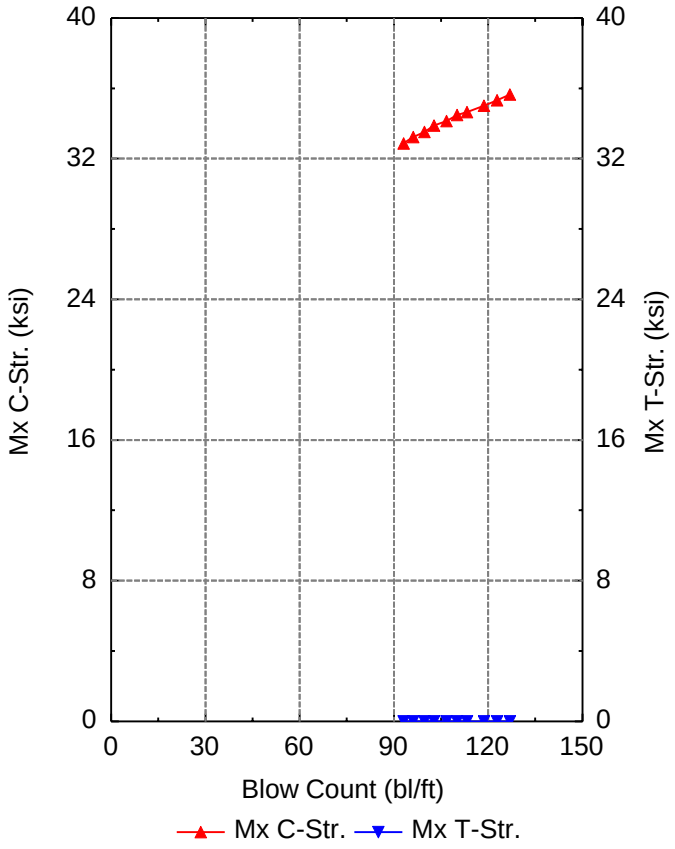
	Soil Type	Damping Factor s/ft	Damping Factor s/m
Shaft Damping	Non-cohesive soils**	0.05	.16
	Cohesive soils**	0.20	.65
Toe damping	In all soil types	0.15	0.50

* For vibratory driven piles, use double values (Smith-viscous).
** For mixed soils, intermediate values may be appropriate; for example, a sandy silt or clayey sand may be modeled with 0.10 s/ft (0.33 s/m), a cohesive silt or a sandy clay with 0.15 s/ft (0.50 s/m).

In general the Smith Soil Damping Option should be used. An exception would be a Residual Stress Analysis for which the Smith-Viscous Soil Damping Option would be more appropriate and Vibratory Hammer Analysis. All other Soil Damping Options are experimental. Smith damping factors and Smith-Viscous damping are both considered independent of pile type or pile size and have the same dimensions.

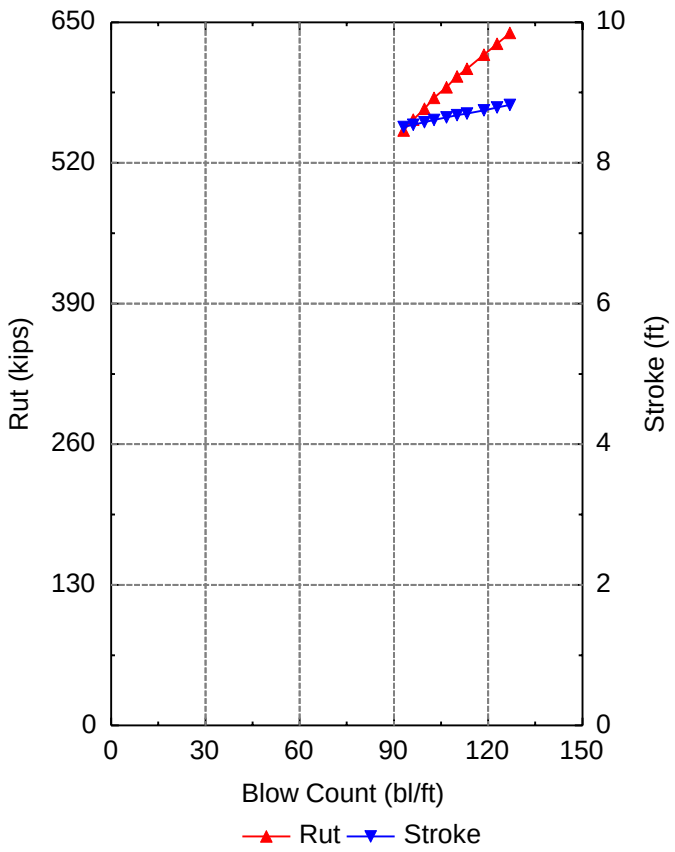
It is recommended that the user perform sensitivity analyses with reasonable ranges of quake and damping factors whenever a situation is ambiguous. If either quake or damping values are known from measurements such values also should be tried. Note that small quake values tend to make capacity values higher (non-conservative) for the same blow count. However, stresses would be predicted conservatively higher, particularly in cases of concentrated end bearing. On the other hand, choosing damping values too low would make capacity results non-conservative.

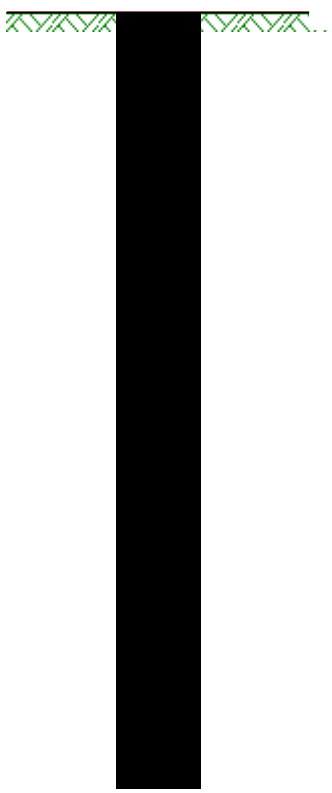
Note that the different soil damping types may be chosen under **Options/General Options/Damping**. The above recommendations are only available for Smith or Smith-viscous models; no recommedations can be made for the other damping options.



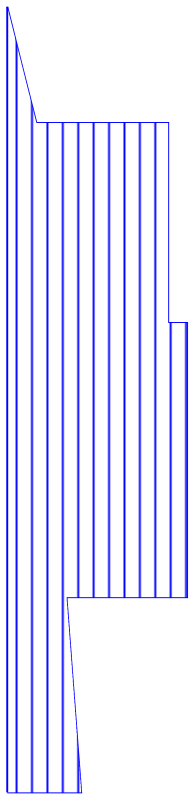
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.123	in
Skin Damping	0.130	s/ft
Toe Damping	0.150	s/ft
Pile Length	16.90	ft
Pile Penetration	16.90	ft
Pile Top Area	26.10	in²
RSA	No	





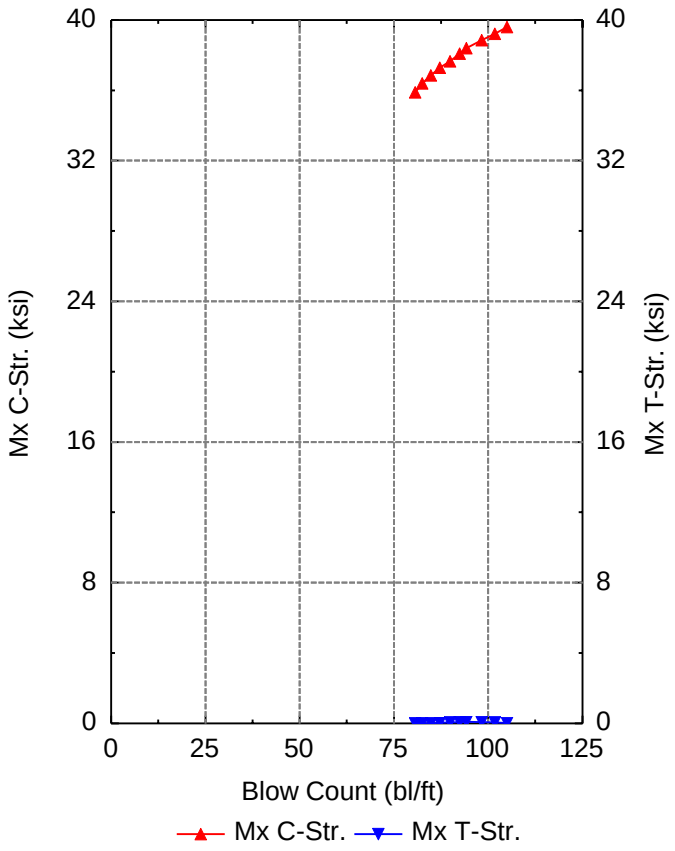
Pile Model



Shaft=51%
(Prop., SA)

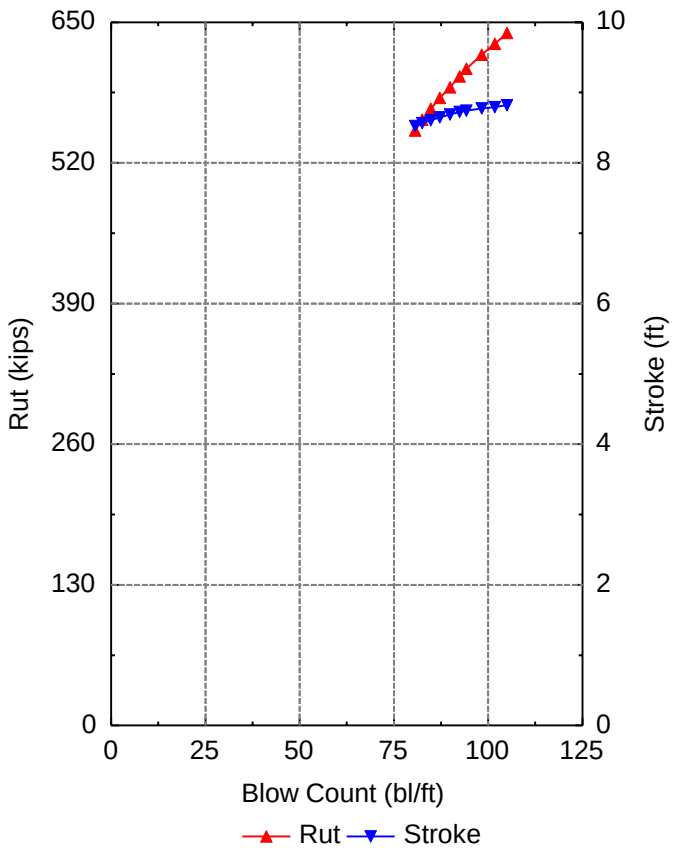
Bearing Graph Summary — APE D 25-42

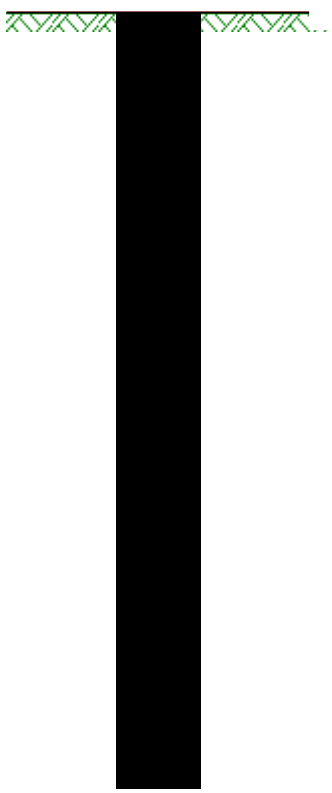
Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
550.0	32.87	32.87	0.00	93.1	8.51	17.53	D 25-42	28.3
560.0	33.22	33.22	0.00	96.1	8.54	17.57	D 25-42	28.3
570.0	33.52	33.52	0.00	99.7	8.58	17.57	D 25-42	28.3
580.0	33.88	33.88	0.00	102.8	8.61	17.63	D 25-42	28.4
590.0	34.13	34.13	0.00	106.7	8.65	17.62	D 25-42	28.4
600.0	34.48	34.48	0.00	110.1	8.68	17.69	D 25-42	28.5
607.0	34.65	34.65	0.00	113.3	8.70	17.66	D 25-42	28.5
620.0	35.02	35.02	0.00	118.7	8.75	17.70	D 25-42	28.5
630.0	35.33	35.33	0.00	122.8	8.79	17.75	D 25-42	28.6
640.0	35.65	35.65	0.00	126.9	8.82	17.82	D 25-42	28.7



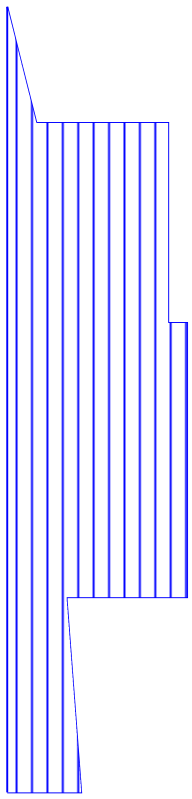
APE D 25-42

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1282.5 (90%)	psi
Helmet Weight	2.700	kips
Hammer Cushion	32408.9	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.130	s/ft
Toe Damping	0.150	s/ft
Pile Length	16.90	ft
Pile Penetration	16.90	ft
Pile Top Area	26.10	in²
RSA	No	





Pile Model



Shaft=49%
(Prop., SA)

Bearing Graph Summary — APE D 25-42

Rut	Mx C-Str.	Top Str.	Mx T-Str.	Blow Ct	Stroke	ENTHRU	Ham. Pow.	Trfr. R.
kips	ksi	ksi	ksi	bl/ft	ft	kip-ft	APE	%
550.0	35.88	35.88	0.00	80.6	8.52	17.34	D 25-42	28.0
560.0	36.39	36.39	0.00	82.5	8.57	17.43	D 25-42	28.1
570.0	36.84	36.84	0.00	84.8	8.61	17.47	D 25-42	28.2
580.0	37.28	37.28	0.02	87.2	8.65	17.51	D 25-42	28.2
590.0	37.64	37.64	0.05	89.9	8.69	17.49	D 25-42	28.2
600.0	38.09	38.09	0.07	92.4	8.72	17.55	D 25-42	28.3
607.0	38.40	38.40	0.08	94.2	8.74	17.57	D 25-42	28.3
620.0	38.86	38.86	0.07	98.3	8.77	17.55	D 25-42	28.3
630.0	39.21	39.21	0.05	101.8	8.80	17.49	D 25-42	28.2
640.0	39.59	39.59	0.02	105.0	8.82	17.51	D 25-42	28.2

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