Geotechnical Design Report

Dover Bridge #5118 over Piscataquis River WIN 023120.00

Dover-Foxcroft, Maine

Submitted to:
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Project No. 2305541

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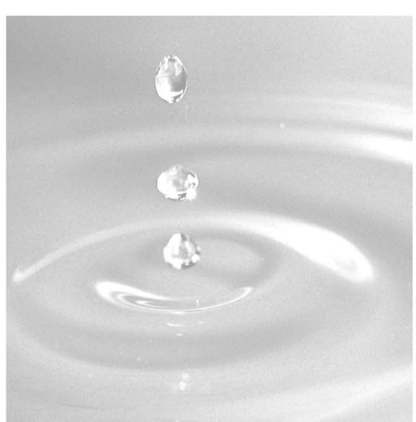

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Executive Summary

This report presents the results of our subsurface explorations and geotechnical recommendations for the proposed replacement of Dover Bridge (#5118), which carries Essex Street over the Piscataquis River in Dover-Foxcroft, Piscataquis County, Maine.

New England Boring Contractors of Hermon, Maine, drilled three Phase 1 preliminary borings (BB-DFPR-101 through -103) from November 2 to 5, 2021, and three Phase 2 final design borings (BB-DFPR-201 through -203) from January 2 to 8, 2024. The drillers performed soil sampling with Standard Penetration Testing (SPT) at approximately 5-foot intervals and cored approximately 10 to 24 feet of bedrock in each boring. A GEI Consultants, Inc., engineer observed and documented the borings.

The borings at the abutments generally encountered 19 to 34 feet of very loose to very dense granular fill and glacial till/weathered bedrock overlying metasiltstone bedrock. The boring performed in the middle of the river encountered 3.5 feet of river sediment overlying metasiltstone bedrock. Bedrock was encountered from approximately El. 324 to El. 309 (about 19 to 34 feet below existing ground surface).

We understand the bridge replacement will be a 2-span bridge supported on semi-integral abutments with a center pier. Proposed Abutment 1 will be a spread footing bearing on bedrock, and proposed Abutment 2 will be supported on rock socketed H-piles. The proposed pier will be a mass concrete wall with a spread footing bearing on bedrock. Geotechnical recommendations for both spread footings bearing on bedrock and rock-socketed H-piles are included in this report.

The hydraulic study for the bridge (GEI 2025) indicates that the potential scour depths at the pier could remove all of the soil down to the bedrock. To protect against scour, we recommend that the foundations be supported on spread footings bearing on bedrock, bearing on concrete fill extending down to bedrock, or rock-socketed H-piles. We understand that the return wingwalls at Abutment 1 will be founded on bedrock due to the short length of the walls from the abutment. The upstream wingwall at Abutment 2 will have two segments separated by a construction joint. The second wingwall segment (farthest from Abutment 2) will likely have the footing step-up away from the river and may be founded on existing fill if scour is not a concern. The upstream wingwall segment 1 and the downstream wingwall will be founded on rock socketed H-piles, similar to Abutment 2. We have included a bearing resistance calculation for the wingwall on existing fill for segment 2 of the upstream wingwall at Abutment 2.

Cofferdams will be required for construction of the abutments and pier based on the Q1.1 water elevations. Geotechnical recommendations for cofferdams are provided in this report.

Our professional services for this project have been performed in accordance with generally accepted engineering practices; no warranty, express or implied, is made.

1. Introduction

1.1. Purpose

This report presents the results of our subsurface explorations and geotechnical recommendations for the proposed replacement of Dover Bridge (#5118), which carries Essex Street over the Piscataquis River in Dover-Foxcroft, Piscataquis County, Maine as shown in Sheet 1.

1.2. Scope

Our scope of work included:

- Reviewing available published geologic data for the project vicinity and the design drawings of the existing bridges.
- Preparing a Health and Safety Plan prior to conducting field activities.
- Preparing a Traffic Control Plan in accordance with Work Zone Traffic Control Guidebook, MaineDOT, March 2015, and the MUTCD (FHWA).
- Engaging a drilling subcontractor to conduct a subsurface exploration program.
- Providing full-time observation during the exploration program and classification of the soil samples in general accordance with Maine Department of Transportation (MaineDOT) guidelines.
- Engaging a third-party laboratory to perform grain size analyses of representative soil and Unconfined Compressive Strength (UCS) tests of rock core samples.
- Evaluating the soil conditions and developing geotechnical design and construction recommendations.
- Preparing this geotechnical design report.

1.3. Authorization

We performed this work in accordance with the Agreement for Subconsulting Services between GEI Consultants, Inc. and Thornton Tomasetti dated November 14, 2023.

1.4. Project Personnel

The following personnel at GEI were involved with the field exploration, evaluations, recommendations, and preparation of this report:

Gillian Williams, P.E.	Senior Project Manager
Nicolas Betancur, P.E.	Senior Geotechnical Engineer
Michael Johnescu, P.E.	Geotechnical Engineer

Shradha Poudyal, E.I.T.	Geotechnical Engineer
Sebastian Carvajal, E.I.T.	Project Professional
Carley Jones	Drafter
Laureen Beintum, P.E. (MA)	In-house Consultant

1.5. Elevation Datum

Elevations in this report are in feet and are referenced to the 1988 North American Vertical Datum (NAVD 1988).

2. Site and Project Description

2.1. Site and Project Description

We understand MaineDOT is considering replacing Dover Bridge (#5118), which carries Essex Street over the Piscataquis River in Dover-Foxcroft, Maine. Dover Bridge was constructed in 1930 and consists of six spans. The bridge deck is currently listed in poor condition with advanced deterioration. The channel bank protection condition is indicated as needing minor repairs and the channel is listed as stable for scour condition. Dover Bridge is located approximately 3,200 feet downstream of Dover Upper Dam (also known as “Moosehead Dam”) and approximately 30 to 90 feet upstream of Brown’s Mill Dam. Dover Upper Dam is owned by Moosehead Energy, Inc., and Brown’s Mill Dam is owned and operated by KEI Power Management, LLC (KEI).

From review of the Maine Highway Commission bridge drawings dated 1929 and 1930, it appears the existing bridge is supported on footings founded on bedrock. The drawings note that Piers 1, 2, 4, and 5 are founded on bedrock approximately 27.25 feet below top of bridge deck. The abutments appear to be founded on bedrock approximately 26 feet below top of bridge deck, and Pier 3 approximately 29 feet below top of bridge deck. The drawing also notes that bedrock drops off sharply between points on the upstream side of the bridge and the dam, which is located approximately 30 to 90 feet downstream of the bridge.

We prepared a Preliminary Geotechnical Design Report (PGDR), dated August 2022, summarizing the results of the Phase 1 borings and our preliminary design and construction recommendations. The recommendations in the PGDR are superseded by this report.

We understand that the bridge replacement will be a 2-span bridge supported on semi-integral abutments with a center pier. Abutment 1 will be a spread footing bearing on bedrock, and Abutment 2 will be supported on rock-socketed H-piles. The proposed pier will be a mass concrete wall with a spread footing bearing on bedrock. The design preference is to keep final grades as close as possible to existing grades, but the approach to Abutment 1 will be raised approximately 2 to 2.5 feet.

2.2. Project Design Basis

Our recommendations are based on the Maine Department of Transportation (MaineDOT) Bridge Design Guide (BDG), dated August 2003 and revised June 2018. Our recommendations conform to the AASHTO 2020 LRFD Bridge Design Specifications, 9th Edition.

3. Subsurface Conditions

3.1. Site Geology

The Reconnaissance Surficial Geology of the Dover-Foxcroft Quadrangle, Maine, prepared by the Maine Geological Survey in 1981, indicates the surficial material on both sides of the river in the area of the bridge is glacial till consisting of a heterogeneous mixture of sand, silt, clay, and stones. Just downstream of the bridge, Presumpscot Formation soils, consisting of mostly silt and clay, are present on the east side of the river. The surficial geology map is shown in Figure A-1 in Appendix A.

The Reconnaissance Bedrock Geology of the Dover-Foxcroft Quadrangle, Maine, prepared by the Maine Geological Survey in 1971, indicates bedrock at the site consists of the limestone member of the Sangerville Formation, described as pelitic limestone and calcareous metasiltstone. Exposed bedrock is present on the downstream side of Brown's Mill Dam. The bedrock geology map is shown in Figure A-2 in Appendix A.

3.2. Phase 1 Subsurface Exploration Program

New England Boring Contractors of Hermon, Maine drilled three borings (BB-DFPR-101 through BB-DFPR-103) as part of the Phase 1 preliminary design between November 2 and November 5, 2021, on Essex Street over the Piscataquis River. The boring locations are shown in Sheet 2. The boring locations were chosen in the field based on access and clearance from existing utilities. A GEI field engineer coordinated the drilling and logged the borings. Boring logs are provided in Appendix B.1. The as-drilled boring locations were surveyed by MaineDOT. The boring locations and elevations are included on the boring logs and summarized in Table 1.

A Mobile B-53 track-mounted drill rig was used to advance the borings. The borings at the abutments, BB-DFPR-101 and BB-DFPR-103, were drilled using a combination of hollow stem augers (HSA) for the first 10 feet of fill material, 4-inch (HW) and 3-inch-inside-diameter (NW) (ID) steel casing with drill and wash methods to top of bedrock, and rock coring (2-inch, NQ-sized). The 4-inch-inside-diameter steel casing was advanced to top of bedrock. Three-inch ID casing was then telescoped to top of bedrock for coring. For BB-DFPR-102, which was located towards the center of the bridge above the river, the boring began by coring through the approximately 1.1-foot-thick concrete bridge deck, followed by 3-inch ID (NW) casing driven to the top of bedrock, and rock coring (2-inch, NQ-sized) thereafter. The casing was driven using a 300-lb hammer and a tri-cone roller bit with water was used to clean the soil cuttings from inside the casing.

Standard Penetration Tests (SPT) were obtained at approximate 5-foot depth intervals. The split spoons were advanced with an automatic hammer consisting of a hydraulically actuated 140-lb weight falling 30 inches in accordance with ASTM D 1586. Approximately 20 to 24 feet of bedrock was cored at each boring location. New England Boring Contractors provided the Standard Penetration Test Energy Measurement Calibration Report prepared by Geosciences Testing and Research, Inc. for the Mobile B-53 drill rig used at the site. The calibration results for the automatic hammer (NEBC D-28) indicate an average energy transfer ratio of 92.4%. Therefore, we used an average hammer energy ratio correction factor of C_E=1.54 to correct SPT N values for hammer energy.

Recovered split-spoon soil samples were placed in jars, and rock core samples were placed in wooden boxes. The soil and rock samples were sent to our Portland, Maine office for verification of field classification. Individual sample descriptions are provided in the boring logs in Appendix B.1. Rock core photographs are provided in Appendix B.2. The Energy Measurement Calibration Report summary table for NEBC D-28 dated September 23, 2021, is provided in Appendix B.3.

BB-DFPR-101 was backfilled with soil cuttings and gravel and patched with asphalt upon completion. BB-DFPR-102 and BB-DFPR-103 were backfilled with bentonite chips in bedrock and then soil cuttings and gravel. The bridge deck at BB-DFPR-102 was repaired with high strength concrete, and BB-DFPR-103 was asphalt patched.

Boring coordinates and depth to bedrock are shown in Table 1.

3.3. Phase 2 Subsurface Exploration Program

New England Boring Contractors of Hermon, Maine drilled three borings (BB-DFPR-201 through BB-DFPR-203) as part of the Phase 2 final design between January 2 and January 8, 2024, on Essex Street over the Piscataquis River. The boring locations are shown in Sheet 2. The boring locations were selected based on the location of the proposed abutments and wingwalls. BB-DFPR-201 and BB-DFPR-203 required offset borings to be drilled due to difficulties advancing the casing. At BB-DFPR-201, the casing broke at a depth of 17 feet, and BB-DFPR-201A was drilled at an offset of approximately 6 feet west. At BB-DFPR-203, the casing tilted out of plumb at a depth of 11 feet due to the presence of probable boulders/cobbles, and BB-DFPR-203A was drilled at an offset of approximately 6 feet east. A GEI field engineer coordinated the drilling and logged the borings. Boring logs are provided in Appendix B.1. The as-drilled boring locations were surveyed by MaineDOT. The boring locations and elevations are included on the boring logs and summarized in Table 1.

A Mobile B-53 track-mounted drill rig was used to advance the borings. The borings were drilled using a combination of solid stem augers (SSA) for the first 4 to 9 feet of fill material, 4-inch (HW) and 3-inch-inside-diameter (NW) (ID) steel casing with spin and wash methods to top of bedrock, and rock coring (2-inch, NQ-sized). The 4-inch-inside-diameter steel casing was advanced to top of bedrock. Three-inch ID casing was then telescoped to top of bedrock for coring. Spin and wash techniques were used instead of drive and wash due to the presence of boulders/cobbles in the granular fill material. The casing was fitted with a cutting shoe and spun to cut through the soil and rock, and a tri-cone roller bit with water was used to clean the soil cuttings from inside the casing.

Standard Penetration Tests (SPT) were obtained at approximate 5-foot depth intervals. The split spoons were advanced with an automatic hammer consisting of a hydraulically actuated 140-lb weight falling 30 inches in accordance with ASTM D 1586. Approximately 10 to 11 feet of bedrock was cored at each boring location. New England Boring Contractors provided the Standard Penetration Test Energy Measurement Calibration Report prepared by GZA GeoEnvironmental, Inc. for the Mobile B-53 drill rig used at the site. The calibration results for the automatic hammer (NEBC D-28) indicate an average energy transfer ratio of 76.5%. Therefore, we used an average hammer energy ratio correction factor of C_E=1.28 to correct SPT N values for hammer energy.

Recovered split-spoon soil samples were placed in jars, and rock core samples were placed in wooden boxes. The soil and rock samples were sent to our Portland, Maine office for verification of field classification. Individual sample descriptions are provided in the boring logs in Appendix B.1. Rock core photographs are provided in Appendix B.2. The Energy Measurement Calibration Report summary table for NEBC D-28 dated April 23, 2023, is provided in Appendix B.3.

All the borings were backfilled with gravel and patched with asphalt upon completion. Boring coordinates and depth to bedrock are shown in Table 1.

3.4. Sample Review

The soil and rock samples from the Phase 1 and Phase 2 explorations were examined at the office by Gillian Williams and Michael Johnescu. Based on our review, it is our opinion that the descriptions in the boring logs in Appendix B.1 are a reasonable characterization of the conditions encountered.

3.5. Laboratory Testing

We engaged GeoTesting Express, Inc. (GTX) of Acton, Massachusetts to perform grain size analyses (ASTM D 6913) on seven soil samples and moisture contents (ASTM D2216) on four soil samples to confirm the sample descriptions. GTX was also engaged to perform unconfined compressive strength (UCS) tests (ASTM D 7012C) on six rock core samples, one from each of the six borings. The results of these analyses are provided in Appendix C.

One grain size analysis with hydrometer (ASTM D 7928) was performed on a grab sample obtained from the riverbank for scour evaluations performed under a separate scope of work. The result for the grain size with hydrometer is also included in Appendix C.

3.6. Subsurface Conditions

The soil layers encountered in the borings are described below in order of increasing depth. Conditions are only known at the boring locations, and conditions between borings may differ from those indicated below and shown in the interpretive subsurface profile in Sheet 3.

The soil descriptions below refer to N_{60}, which is the measured N-value corrected to an equivalent hammer energy of 60 percent efficiency (i.e., the standard energy assumed in many SPT correlations). Field-measured N-values as well as corrected, N_{60} values are reported on the boring logs in Appendix B.1.

- Granular Fill – Granular fill extended to about 16.3 feet to 23.0 feet below ground surface (bgs) in the borings performed at the abutments. Approximately 0.5 to 1.0 foot of asphalt was encountered above the fill.

  The granular fill observed in the borings generally consisted of brown, loose to very dense, fine to coarse-grained sand or silty sand with varying amounts of gravel or gravel with varying amounts of sand and silt. One of the split-spoon samples of the fill in BB-DFPR-103 contained a ¼-inch layer of decomposed wood.

BB-DFPR-201/201A and -202 were located within the existing abutments based on the 1929/1930 Maine Highway Commission drawings, which were likely filled with crushed stone or rockfill. Refusal on probable boulders was encountered for most of the SPTs performed in BB-DFPR-201/201A and -202.

Grain size analyses performed on fill samples indicated the percent fines ranged from about 16 to 37 percent. USCS classifications were SM and GM, and AASHTO classifications were A-1-b, A-2-4, and A-4. Water contents in the fill ranged from 5.6 to 24.8 percent.

Excluding BB-DFPR-201/201A and -202 located within the abutment rockfill, corrected N-values (N_{60}) in the fill ranged from 5 to over 100 blows per foot (bpf), with an average of 30 bpf and a median of 17 bpf, indicating a mostly medium dense soil.

- <u>River Sediment</u> – BB-DFPR-102 was drilled through the concrete bridge deck and encountered river sediment below the water surface. The sediment was about 3.5 feet thick, and one SPT sample was collected in this layer. The river sediment generally consisted of fine to coarse gravel with some fine to coarse sand and some silt.

  The N_{60} value was 5 bpf, indicating a loose soil.

- <u>Glacial Till/Weathered Rock</u> – A layer of glacial till or possible weathered bedrock was encountered below the fill in BB-DFPR-201/201A, -202 and -203A. The thickness of the glacial till/weathered bedrock ranged between 2 to 11.5 feet. The material is variable ranging from brown to grey, medium dense to very dense, clayey/silty sand, with some to little gravel and angular rock fragments; to brown, hard clay and silt, trace sand. Some of the soil samples appeared to have structure, indicating the potential for highly weathered rock. Grain size analyses performed on glacial till samples indicated the percent fines ranged from about 26.4 to 51.9 percent. USCS classifications were SC, SM and ML, and AASHTO classifications were A-2-4(0) and A-4(0).

  Corrected N-values (N_{60}) in the glacial till/weathered rock ranged from 22 to over 100 bpf, with an average of 47 bpf and a median of 33 bpf, indicating a mostly dense or hard soil.

- <u>Bedrock</u> – Bedrock was encountered in the borings at depths of 19 feet to 34 feet bgs (approximately El. 309 to El. 324). Bedrock was deepest along the north side of Abutment 2 (34 and 29 feet bgs at BB-DFPR-202 and BB-DFPR-203A, respectively) and shallower at Abutment 1 (19 and 21 feet bgs at BB-DFPR-101 and BB-DFPR-201, respectively). Based on the three borings drilled behind Abutment 2, the bedrock appears to slope downward from the southeast to northwest, with a high point of 20 feet bgs at boring BB-DFPR-103.

  The bedrock was generally classified as grey to dark grey, moderately hard to hard, fine-grained metasiltstone with calcite intrusions, and ranged from fresh to slightly weathered. The rock quality designation (RQD) in the metasiltstone ranged from 0 to 100 percent, with an average of 43 percent and a weighted average of 54 percent, indicating very poor to excellent rock quality.

  The bedrock had steeply dipping fractures and vertical bedding planes, as shown in the rock core photos in Appendix B.2. The top approximately 5 to 6 feet of rock core in BB-DFPR-103 and -202 at Abutment 2 was more weathered than the bedrock observed in other cores.

  Unconfined compressive strength (UCS) tests were performed in general accordance with AASTM D7012 on one bedrock sample from each boring and the results are provided Table 3.

The unconfined compressive strength ranged from 3,818 psi to 15,195 psi. Due to the prevalent vertical bedding planes observed in the cores, we had a limited selection of rock core samples to choose for testing. As shown in the photographs in Appendix C, the fractures in the UCS test samples were vertical or near vertical.

3.7. Groundwater and Surface Water Levels

Water levels were not measured in BB-DFPR-101 through BB-DFPR-103 because water was introduced during drilling and there was insufficient time for the water to stabilize with the groundwater. River water levels were measured relative to the bridge deck during drilling. River levels measured ranged from 13.5 to 14.2 feet bgs (approximately El. 329.9 to El. 329.2). Groundwater levels were measured in BB-DFPR-201/201A through BB-DFPR-203/203A and ranged from 16.1 to 17.4 feet bgs (approximately El. 326.5 to 325.4). These measurements may not accurately reflect the true groundwater level or average river levels. Significantly different groundwater levels may occur at other times and locations.

4. Engineering Evaluations & Recommendations

4.1. General

We understand the bridge replacement will be a 2-span bridge supported on semi-integral abutments with a center pier. Abutment 1 will be supported on a spread footing bearing on bedrock, and Abutment 2 will be supported on rock socketed H-piles. The proposed pier will be a mass concrete wall with a spread footing bearing on bedrock. The new abutments will be built behind the existing abutments, and the final grades will be kept as close as possible to existing grades, except at Abutment 1 where the grade will be raised approximately 2 to 2.5 feet. The center pier will be located southwest of existing pier 3 and will partially overlap the existing pier 3.

The borings encountered bedrock at El. 324.2 and 322.1 (approximately 4 to 6 feet below bottom of footing) at Abutment 1, and bedrock at Abutment 2 varied from El. 322.9 to 308.8 (2 to 16 to feet below the proposed bottom of pile cap). At both abutments, bedrock was overlain by fill and glacial till. Two of the borings at Abutment 2 (BB-DFPR-202 and -203A) also encountered a layer of possible weathered rock directly above the bedrock. At the center pier, rock was encountered around El. 314.4 (approximately 8 feet below the proposed bottom of footing).

Recommendations for designing foundations for the replacement abutments and the pier are presented below. Calculations supporting these recommendations are included in Appendix D.

4.2. Soil Properties and Lateral Earth Pressures

Recommended soil properties and earth pressure coefficients for design are presented in Table 4. We selected these values based on published correlations to SPT N-values, our review of the soil descriptions, and our engineering judgment.

For the semi-integral abutments, the lateral earth pressures developed against the end diaphragm are a function of the movement of the abutment against the backfill and can range from at-rest pressure to full passive pressure. The end diaphragm reinforcement should be designed for the earth pressure that results when the bridge expands against the backfill. This earth pressure should be calculated using the formula provided in Section 5.4.2.11 of the MaineDOT BDG. The Passive Lateral Earth Pressure Coefficient (Kp) needed for this equation is provided in Table 4 and was evaluated using FHWA NHI-06-089 Figure 10-4. This Kp value was obtained assuming a magnitude of wall rotation equal of 0.02 expressed in terms of the ratio of wall movement to wall height (Y/H). However, the designer should calculate an appropriate earth pressure coefficient based on estimated superstructure thermal movement using both FHWA NHI-06-089 and MassDOT Bridge Design Manual Figure 3.10.8-1 and use the more stringent value. It should also be noted that the design earth pressure coefficient should be no less than Kp calculated using Rankine, regardless of estimated wall rotation.

The earth pressure acting on the portions of the abutments below the end diaphragms and on the wingwalls will be in active condition since these are not integral with the superstructure. Passive pressure resistance in front of spread footings should be ignored when checking for sliding and overturning stability to account for the possibility of these materials being removed or scoured.

4.3. Spread Footing Design (Abutment 1, Wingwalls, and Pier)

Abutment 1 and the center pier can be supported on spread footings bearing on sound bedrock or bearing on concrete or grout fill extending down to bedrock. Based on the June 2025 Final Design Hydrologic and Hydraulic Report prepared by GEI, we understand that pier scour depths associated with the 100-year and 500-year flood events are 8.7 feet and 9.2 feet, respectively, assuming a depth to bedrock in excess of 9.5 feet. However, the river sediment was only 3.5 feet thick in BB-DFPR-102 performed through the riverbed. Consequently, the river sediment would be fully removed by scour during these flood events. Clear water contraction and abutment scour were calculated to be 15.4 feet and 39 feet for the 100-yer flood and 20.6 feet and 41 feet for the 500-year flood, respectively. For the purpose of foundation design, we assume that the scour depths will be limited to the top of bedrock.

If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps, excavated to be completely level, or the concrete foundation anchored or doweled to bedrock. This may be a stability concern at the abutments if the bedrock is sloping down towards the river and in the downstream direction. Footings on bedrock should be at least 3 feet wide for constructability.

For design of foundations bearing directly on the bedrock, or on concrete or grout fill placed over rock, we recommend using a nominal bearing resistance value of 30 kips per square foot (ksf) for the Strength Limit state and 16 ksf for the Service Limit state. Resistance factors for calculation of factored bearing resistance are provided in Table 5. The applied bearing pressure should be limited to the lesser of the estimated rock bearing resistance or the nominal resistance of the concrete or grout taken as 0.3f'c. Loose, highly weathered, and loose, fractured rock should be removed prior to placement of footing concrete and concrete or grout fill.

We understand that the return wingwalls at Abutment 1 will be founded on bedrock due to the short length of the walls from the abutment. The upstream wingwall at Abutment 2 will have two segments separated by a construction joint. The second wingwall segment (farthest from Abutment 2) will likely have the footing step-up away from the river and may be founded on existing fill if scour is not a concern. The upstream wingwall segment 1 and the downstream wingwall will be founded on rock socketed H-piles, similar to Abutment 2. For design of the Abutment 2 upstream wingwall footing bearing on existing fill, we recommend using the factored bearing resistance curves in Sheet 4. Footings on existing fill or granular borrow should be at least 3 feet wide.

Supporting calculations for these recommendations are provided in Appendix D. Table 5 provides recommended resistance factors that should be applied to the recommended bearing resistances.

When evaluating sliding along the base of the abutment, pier, and wingwall footings, we recommend that a nominal coefficient of friction of 0.70 be used for cast-in-place footings on bedrock, 0.55 for existing fill and granular borrow, and 0.60 for crushed stone. Applicable Resistance Factors for evaluating sliding are provided in Table 5.

The analysis of lateral stability (overturning and sliding) should include evaluation of the combined structure, consisting of the substructure and underlying concrete or grout fill, considering scour down to the bedrock surface in front of the abutment and pier. If necessary, the stability under this condition could be improved by anchoring the substructure to the underlying bedrock.

Based on the geotechnical conditions, we expect total and differential settlements of foundations on bedrock, concrete or grout fill placed over bedrock to be negligible. If the wingwall footings are supported on the existing fill or granular borrow as described above, we estimate that total and differential settlements will be less than 1 inch if bearing pressures are below the Service Limit curves in Sheet 4. We anticipate that most of this settlement would occur during construction.

4.4. Rock Socketed H-pile Design (Abutment 2)

We understand that proposed Abutment 2 will consist of a semi-integral substructure where the superstructure end diaphragm overhangs the back of the abutment. Based on the results of our subsurface explorations conducted during the preliminary design phase and supplemental explorations obtained during the final design phase, significant variation in elevation of the top of the bedrock surface is anticipated at Abutment 2. Within the footprint of proposed Abutment 2, the rock slopes approximately 13 feet in a southeast to northwest direction.

Because of the relatively shallow depth to bedrock and scour depth estimates extending to the top of rock, we recommend that Abutment 2 be supported on deep foundations consisting of rock-socketed steel H-piles. Our recommendations are based on design analyses performed using the computer program FB Multipier v6.1.2 (FBMP) by Florida Bridge Software Institute, a program for the soil-structure-interaction analysis of pile group foundations subject to axial and lateral loading.

Based on the results of our group analyses considering scour conditions extending to top of bedrock, we recommend that the deep foundations consist of a total of 24 HP 14X89steel piles. The recommended pile layout consists of two rows of 12 HP 14X89 piles oriented with the strong axis bending (pile flanges perpendicular to the centerline of girders). The piles can be spaced at 7 and 5 feet on-center along the bridge longitudinal and transverse directions, respectively. The piles should be installed in a minimum 30-inch-diameter rock socket extending a minimum of 10 feet into bedrock. The rock socket should be tremie filled with grout with a minimum compressive strength of 4 ksi.

This foundation layout is applicable for a subsurface profile with top of bedrock elevation varying between El. 323 and 309. Given the uncertainty with the top of bedrock elevation, we performed sensitivity analyses to evaluate the deep foundation layout at different top of bedrock elevations. Based on the results of our sensitivity analysis, if bedrock is encountered below El. 307 during the probing program, the pile lengths should be re-evaluated to determine if the Abutment 2 foundation design is still feasible. As indicated in Section 5.5., we recommend that the Contractor be required to perform a probing program to better establish the depth to top of bedrock at Abutment 2 prior to installing the production piles. Alternatively, if the field verification via probing indicates the top of bedrock is shallow, within 5 feet of the proposed footing elevation, and relatively uniform within the footprint of the substructure, the proposed Abutment 2 foundation could potentially be redesigned as a spread footing bearing on rock similar to proposed Abutment 1.

The recommended minimum rock socket embedment was controlled by Service Limit State displacements rather than by axial resistance under Strength or Extreme Limit States. We estimated the recommended minimum rock socket length by limiting the maximum lateral pile displacement under Service I Limit State to approximately 1 inch at the pile head while limiting the pile tip movement to approximately 0.005 inches. Achieving full fixity at the pile tip was not considered necessary based on discussions with Thronton Tomasetti and provisions in the MaineDOT Bridge Design Guide. Guidance contained in Section 5.4.2.5.B of the MaineDOT Bridge Design Guide on the use of short piles for integral abutments indicates that “short steel piles (14 feet or less) may not develop fixity but perform adequately and do not experience stresses larger than those seen by longer piles.” Based on the recommended rock socket lengths, the structural design of the piles should account for a boundary condition at the pile tip which is not fully fixed. Because the piles will not be driven, but rather drilled and placed inside grouted rock sockets, structural resistance factors for combined axial and flexural resistance for undamaged piles can be considered. We considered a corrosion allowance of 1/16 inch all around the perimeter of the HP piles which results in a total section loss of 1/8 inch. A corroded pile section was used in our FBMP analyses for the entire length of the piles to evaluate Service Limit State displacements. An intact pile section was modeled when evaluating the Strength and Extreme Limit States to maximize the moment demand on the piles.

The geotechnical resistance of the piles ignores the contribution from the overburden soils which are assumed to be scoured during the design and check floods. The rock socket is designed as a shear socket that relies on side resistance only between the walls of the rock socket and the concrete surface following ASSHTO procedures for drilled shaft design. End bearing resistance is ignored in the geotechnical resistance of the rock socket. A resistance factor of 0.55 for Strength I Limit State was adopted in accordance with AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.4-1 assuming no load testing is performed. Our recommended factored geotechnical axial resistance for a 30-inch-diameter, 10-foot-long rock socket for Strength I Limit State is 562 kips.

Supporting calculations for these recommendations are provided in Appendix D.

4.5. Seismic Design Parameters

Based on the explorations and our seismic design parameter calculations (Appendix D), we conclude that the site should be classified as Site Class C.

Based on the 2020 AASHTO LRFD seismic hazard maps for the 1,000-year return period, we recommend the following parameters for seismic design:

- Horizontal Peak Ground Coefficient (PGA) = 0.074
- Horizontal Response Spectral Coefficient (period = 0.2 sec) (S_s) = 0.155
- Horizontal Response Spectral Coefficient (period = 1.0 sec) (S_1) = 0.047

The applicable site coefficients for peak ground acceleration ([F_{PGA}], short-period range [F_A], and long-period range [F_v]) at this site are 1.2, 1.2, and 1.7, respectively. Application of these site coefficients results in the following recommended coefficients for development of design response spectra:

- Response Spectral Acceleration, A_s = 0.089
- Design Spectral Acceleration Coefficient at 0.2 second period, S_{DS} = 0.186
- Design Spectral Acceleration Coefficient at 1.0 second period, S_{D1} = 0.080

This site falls into Seismic Zone 1, based on the 1-second-period design spectral acceleration. For multiple span bridges in Seismic Zone 1, there is no detailed seismic analysis required other than connection design and seat bearing length.

Semi-integral abutments, where the superstructure end diaphragm overhangs the back of the abutment, should be checked for resistance to overturning from 100% of the seismic active soil force calculated by the Mononobe-Okabe method. The seismic active coefficient is included in Table 4. This check is required for semi-integral abutments since the superstructure cannot act as a strut because there is no backwall for it to engage, and therefore the abutment must rely on its own stability to prevent it from tipping over and resulting in failure of the bridge structure.

4.6. Settlement and Stability

The proposed bridge design calls for minimal grade raises at the approaches, and the existing fill and glacial till encountered in the borings was generally medium dense to very dense sand and gravel, with lesser amounts of silt and clay. The site also has relatively shallow bedrock at the location of the approaches. Based on the material encountered in the borings, MaineDOT slopes of 2H:1V or 1.75H:1V if riprap protected, are expected to be stable. Furthermore, we do not anticipate settlement related issues based on the subsurface conditions encountered.

4.7. Frost Penetration

Foundations placed on bedrock are not subject to heave by frost, and there are no frost embedment requirements for project footings placed directly on sound bedrock. Based on the MaineDOT Bridge Design Manual, Figure 5-1, the site has approximately 2,000 to 2,100 degree-days. The laboratory testing results indicate water contents of the soil samples range from 5.6 to 24.8 percent, with an average moisture content of about 14 percent. Therefore, the estimated frost depth is approximately 7.5 feet. The bottom of the footings should extend a minimum of 7.5 feet below the lowest final exterior grade for frost protection where the bearing layer is soil. Riprap does not contribute to the frost depth.

5. Construction Recommendations

5.1. Excavation and Dewatering

All excavations should be made in accordance with OSHA standards. For construction of spread footings on bedrock at Abutment 1 and the pier and pile cap at Abutment 2, cofferdams will be required. Cofferdams should be constructed in accordance with Section 511 of the MaineDOT Standard Specifications. Cofferdams will need to be designed to support the unbalanced soil pressure and the hydrostatic pressure. The contractor should provide pressure relief ports located at the design water level to control buoyancy during high water events. Given the relatively shallow depths to rock, the contractor should anticipate having to provide cofferdam toe embedment into rock. When scour is probable, steel sheeting should be left in place and anchored to seal with Z bars.

For construction of substructure foundations on bedrock, we recommend excavating to the top of bedrock in the wet inside the cofferdam, cleaning off the bedrock surface, and placing a tremie cement concrete seal extending from the top of the bedrock to the bottom of the substructure footing or pile cap. Where there is little soil above the bedrock, it may be necessary to seal the bottom edges of the sheet piles with sandbags to prevent concrete from leaking out under the sheet piles. The cofferdam must be designed to resist any unbalanced soil and water pressures between the land side and water side. The tremie seal should be thick enough to resist the hydrostatic uplift force when the cofferdam is dewatered. If necessary, the concrete tremie seal could be anchored to the bedrock to provide additional uplift resistance.

Foundations should be constructed in the dry. The bedrock type, fracturing, and slope on the bearing surfaces will not be known until excavation is complete. Bedrock surfaces should be cleaned with high pressure air or water.

If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps, excavated to be completely level, or the concrete foundation anchored or doweled to bedrock.

A professional engineer registered in the State of Maine and engaged by the contractor should design the cofferdam and tremie seal. The design should be submitted to the engineer for review before installation.

Groundwater will be encountered during excavation of the cofferdams. The contractor should be prepared to manage and control groundwater during excavation and to control surface water from entering excavations to provide a dry and stable subgrade. The contractor should be responsible for selecting the dewatering methods based on their proposed means and methods. Groundwater levels should be maintained at least 2 feet below excavation subgrade levels at all times, or deeper if necessary to maintain stable conditions.

The dewatering plan and systems should be designed by an experienced Professional Engineer registered in the State of Maine and retained by the contractor. The contractor should submit a dewatering plan for review prior to the start of excavation. Dewatering efforts must satisfy requirements of local, state, and federal environmental and conservation authorities.

5.2. Vibration Monitoring

While not expected to be required, blasting should be performed in accordance with Sections 105.2.7 and 203.042 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre- and post-blast surveys, as well as blast vibration monitoring at nearby residences and at the downstream dam structure in accordance with industry standards during all blasting operations and potentially during mechanical rock removal activities.

5.3. Footings on Bedrock

It is anticipated that competent bedrock will be encountered in footing excavations. Concrete for footings on sound bedrock may be placed directly on the prepared bedrock surface. The prepared bedrock surface should be a minimum of 6 inches below top of bedrock. The bedrock below the footing should be relatively level and sound. If the bedrock surface is sloping, the bedrock surface should be cut to an approximately level surface (within 10 degrees of horizontal) in all directions. The bedrock surface can be stepped as necessary to achieve this slope.

For concrete footings on weathered bedrock, if the bedrock is uneven, irregularities in the rock should be filled with crushed stone or lean concrete to provide a level working surface. Loose rock must be removed.

Tremie seals will likely be needed to construct cofferdams for foundation construction, and therefore rock subgrade will be prepared underwater. Tremie seals should be placed directly on bedrock. Prior to placing the tremie concrete, the bedrock surface shall be cleaned using an air-lift and inspected by divers to confirm that any loose material has been removed.

If the subgrade at the proposed bearing elevations is partially rock and partially suitable soil, care must be taken in preparing the bearing surface at the transition between the two conditions. An abrupt transition between stiff rock bearing surfaces and soil bearing surfaces could create a hard spot, allowing unacceptable differential settlement to occur over a short distance. We recommend that where a section of footing is directly on bedrock and an adjacent section is soil supported, the bedrock be excavated to a depth of 18 inches below the bottom of the footing and backfilling with Gravel Borrow for Bridge Foundations or the soil be excavated to bedrock and backfilled with concrete (minimum thickness 6 inches).

5.4. Preparation of Subgrade for Footings on Soil

The wingwalls at Abutment 1 will be founded on bedrock, and the downstream wingwall at Abutment 2 will be founded on piles. We understand that a portion of the upstream wingwall at Abutment 2 may be founded on existing fill. Prior to foundation construction, soil foundation subgrade should be compacted with at least four passes of a smooth-wheel vibratory compactor weighing at least 10,000 pounds. In confined areas, compact with a vibratory plate compactor that weighs at least 200 pounds and imparts an impact load of at least 2.5 tons. Where exposed footing subgrades are at or near the groundwater level, static compaction may be recommended by the Geotechnical Engineer in lieu of vibratory compaction. Loose or soft zones of existing fill at subgrade level should be over-excavated and replaced with compacted granular borrow.

If fill is placed below the groundwater level, the fill should be crushed stone. Crushed stone should be wrapped in a nonwoven geotextile with a minimum overlap of 2 feet. The nonwoven geotextile should meet the requirements for subsurface drainage in MaineDOT Standard Specification Section 722.

Bearing surfaces should be free of standing water, frost, and loose soil before placement of reinforcing steel and concrete. Areas of the subgrade disturbed by traffic, frost, or surface water should be re-compacted. We recommend that a qualified Geotechnical Engineer evaluate the soil subgrades of shallow foundations prior to placement of footings and fill.

5.5. Rock Socketed Pile Installation

The depth to top of bedrock should be probed prior to attempting installation of the production piles to minimize the uncertainty with the pile lengths. Probe piles can be used by the Contractor to verify the top of rock elevation at the proposed pile locations. Probe piles can consist of the same or smaller HP sections as the production piles. In lieu of probe piles, the Contractor can probe the depth to rock using a drill rig at the proposed pile locations.

Rock socketed H-piles should be installed in accordance with Section 501 of the MaineDOT Standard Specifications. The piles will need to be installed by means of temporary casing seated into rock to facilitate excavation of the rock sockets and provide a seal to allow for placement of tremie grout within the rock socket. A minimum of 3 inches of 4,000 psi grout cover should be provided between the bottom of the rock socket and the tip of the HP piles. H-Piles should be equipped with a steel bearing shoe plate welded to the toe of the pile. The Contractor should provide means of temporarily supporting the pile between the bottom of the steel plate and the bottom of the rock socket excavation to ensure the minimum grout cover below the pile toe is provided. This temporary support will also facilitate supporting the pile plumb while the grout attains sufficient strength prior to backfilling around and along the remaining pile length above the grout column.

The Contractor needs to be aware of the strength of the rock encountered at the site when selecting tooling for excavation of the rock sockets. Average measured uniaxial compressive strength of intact specimens as high as 15,000 psi should be anticipated. The metasiltstone encountered on site may be susceptible to softening if exposed to air and water. Rock sockets need to be tremie filled with grout immediately after excavation to minimize the risk of side resistance softening due to prolonged exposure to air and water. The Contractor must carefully plan the sequence of pile installation to minimize the amount of time that rock sockets are left open prior to grout placement. Excavated rock sockets should not be allowed to remain open and exposed to air and water overnight. We recommend a maximum waiting period of 4 hours between the end of rock socket excavation and grout placement.

Temporary casing may need to be equipped with carbide teeth to clear obstructions and to be seated into rock. The piles should be backfilled with Granular Borrow prior to removing the temporary casing. The backfill material needs to extend from the top of the rock socket to the bottom of the pile cap.

5.6. Obstructions

The borings indicate the presence of boulders and cobbles in the fill and glacial till. Adequate tooling will be necessary to clear obstructions at the locations of the proposed rock socketed piles. Temporary casing equipped with hardened shoes or carbide teeth and or the use of down-the-hole-hammers may be required to penetrate obstruction during pile installation. Where the obstructions are relatively shallow, the contractor may be able to remove them using an excavator.

5.7. Backfilling

MaineDOT granular borrow for underwater backfill should be used behind the abutments in accordance with MaineDOT BDG, Section 5.4.2.13. Drainage behind the integral abutment should be designed in accordance with MaineDOT BDG, Section 5.4.1.9, to minimize hydrostatic pressure and control erosion of the underside of the abutment embankment riprap.

Fill for the roadway and behind the abutments, backfill of excavations for utilities, and crushed stone for scour protection, if any, should be placed and compacted in accordance with MaineDOT Standard Specifications Section 206 (2020 version). However, we recommend that compaction in areas too small for a smooth wheel vibratory compactor, within 5 feet of walls less than 15 feet high, or within 10 feet of walls greater than 15 feet high, should be performed using a vibratory walk-behind roller or plate compactor (weighing at least 200 pounds imparting an impact load of at least 2.5 tons), with soil placed in maximum 6-inch-loose lifts.

5.8. Re-Use of Existing Materials

Based on the soil descriptions on the boring logs, some of the existing on-site granular soils may meet the requirements for common borrow. Suitability for reuse can be confirmed by testing samples to evaluate if the soil in question meets the MaineDOT requirements for common borrow. The on-site soils may have oversized cobbles and boulders that would need to be removed prior to re-use as common borrow. The Contractor should be aware that materials that are not free draining may be difficult to compact in wet weather.

5.9. Freezing Conditions

If construction is performed during freezing weather, special precautions will be required to prevent the soil subgrades from freezing. Freezing of the soil beneath foundations and pavements during construction may result in heave and subsequent settlement of the structure.

All soil subgrades should be free of frost before foundation construction. Frost-susceptible soils that have frozen should be removed and replaced with compacted gravel borrow. The foundation and the soil adjacent to the foundation should be insulated until they are backfilled.

Soil placed as fill should be free of frost, as should the ground on which it is placed.

6. Limitations

Our recommendations are based on the project information provided to us at the time of this report and may require modification if there are any changes in the nature, design, or location of the proposed construction. We recommend that GEI be engaged to review the final plans and specifications to evaluate whether changes in the project affect the validity of our recommendations and whether our recommendations have been properly implemented in the design.

The recommendations in this report are based in part on the data obtained from the borings. The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. Therefore, we recommend that GEI be engaged to make site visits during construction to: a) check that the subsurface conditions exposed during construction are in general conformance with our design assumptions, and b) ascertain that, in general, the geotechnical aspects of the work are being performed in compliance with the contract documents.

Our professional services for this project have been performed in accordance with generally accepted engineering practices; no warranty, express or implied, is made.

Tables

Table 1. Subsurface Explorations
Geotechnical Design Report
Dover Bridge (#5118) Essex Street over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Exploration Number	Northing (ft)	Easting (ft)	Surface Elevation[1] (ft)	Depth of Exploration (ft)	Depth to Groundwater After Drilling (ft)	Depth to Fill (Asphalt Thickness) (ft)	Depth to River Sediment (ft)	Depth to Glacial Till/Possble Weathered Rock (ft)	Depth to Top of Bedrock (ft)	Notes
November 2021 and January 2024 Exploration Programs										
BB-DFPR-101	613748.7	1615991.3	343.2	39.0	NM	0.6	NE	NE	19.0	River level = 14.2 ft from bridge deck
BB-DFPR-102	613826.5	1616114.1	343.4	49.3	NM	NE	25.5	NE	29.0	River level = 13.5 ft from bridge deck
BB-DFPR-103	613903.5	1616245.8	342.9	43.9	NM	0.5	NE	NE	20.0	River level = 13.8 ft from bridge deck
BB-DFPR-201	613768.1	1615998.1	343.0	18.0	NM	1.0	NE	16.3	NE	Offset required due to broken casing
BB-DFPR-201A	613766.9	1615996.3	343.1	31.9	16.3	1.0	NE	19.0	21.0	
BB-DFPR-202	613912.3	1616225.5	342.8	44.8	17.4	0.6	NE	23.0	34.0	
BB-DFPR-203	613925.4	1616246.0	342.6	11.0	NM	1.0	NE	NE	NE	Offset required due to casing tilt
BB-DFPR-203A	613926.1	1616247.0	342.6	39.0	16.1	1.0	NE	17.5	29.0	

Notes:

1. The boring coordinates and elevations were surveyed by MaineDOT. Elevations are referenced to NAVD88.
2. BB-DFPR-102 elevation and depths are from the bridge deck.
3. All river level measurements taken from approximately the midspan (near Pier 3) of the bridge. Bridge deck elevation assumed at El. 343.4
4. NE = Not Encountered
5. NM = Not Measured

Table 2. Grain Size Analysis Results
Geotechnical Design Report
Dover Bridge (#5118) Essex Street over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Exploration Number	Sample Number	Sample Depth (feet)	Material	Description	Moisture Content %	AASHTO	USCS Classification
BB-DFPR-101	2D	5 to 6.7	Fill	Brown fine to coarse SAND, some silt, little gravel	14.5	A-2-4	SM
BB-DFPR-101	3D	10 to 12	Fill	Brown Silty SAND, trace gravel	24.8	A-4	SM
BB-DFPR-103	2D	5 to 7	Fill	Brown Sandy GRAVEL, little silt	5.6	A-1-b	GM
BB-DFPR-103	4D	15 to 17	Fill	Brown fine to coarse SAND, some gravel, little silt	11.7	A-1-b	SM
BB-DFPR-201A	2D	19 to 19.9	Glacial Till	Brown and Grey Clayey fine to coarse SAND, trace gravel		A-4 (0)	SC
BB-DFPR-202	6D	23 to 25	Glacial Till / Weathered Rock	Brown SAND, some silt, little gravel		A-2-4 (0)	SM
BB-DFPR-203A	3D	24.7 to 25.1	Glacial Till / Weathered Rock	Tan to light brown Sandy SILT, trace gravel		A-4 (0)	ML

Table 3. Bedock Laboratory Test Results
Geotechnical Design Report
Dover Bridge (#5118) Essex Street over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Exploration Number	Ground Surface El. (ft)	Depth to Bedrock (ft)	Run Number	Sample Depth (ft)			Sample Depth into Bedrock (ft)			Sample El.	Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Rock Classification
BB-DFPR-101	343.2	19	R5	31.3	-	31.7	12.3	-	12.7	311.9	177	9,615	Metasiltstone
BB-DFPR-102	343.4	29	R4	40.3	-	40.7	11.3	-	11.7	303.1	175	7,128	Metasiltstone
BB-DFPR-103	342.9	20	R7	41.5	-	41.9	21.5	-	21.9	301.4	174	3,818	Metasiltstone
BB-DFPR-201A	343.1	21	R4	22.4	-	22.8	1.4	-	1.8	320.7	173	15,195	Metasiltstone
BB-DFPR-202	342.8	34	R5	44.3	-	44.6	10.3	-	10.6	298.6	175	6,632	Metasiltstone
BB-DFPR-203A	342.6	29	R1	33.3	-	33.7	4.3	-	4.7	309.3	173	9,671	Metasiltstone

Table 4. Soil Properties
Geotechnical Design Report
Dover Bridge (#5118) Essex Street over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Layer/Soil Type	Unit Weight, γ (pcf)	Friction Angle, ϕ (deg)	Earth Pressure Coefficients[1]				
			Active, $K_{a\_Rankine}$[3]	Active, $K_{a\_Coulomb}$[3]	Seismic Active, K_{ae}[2]	At Rest, K_0	Passive, K_p[4]
Existing Fill	125	32	0.31	0.28	0.33	0.47	5.8
River Sediment	115	30	0.33	0.30	0.36	0.50	3.0
Glacial Till	135	38	0.24	0.22	0.27	0.38	5.8
Granular Borrow	125	32	0.31	0.27	0.33	0.47	5.8
Gravel Borrow	135	36	0.26	0.24	0.29	0.41	5.8

Notes:

1. Recommended earth pressure coefficients are associated with vertical wall face and horizontal ground both in front and behind the wall, and are in accordance with the recommendations of AASHTO LRFD 3.11.5.3 and 3.11.5.4. Supporting calculations are included in Appendix D. For sloping wall face, calculate using log spiral method and actual wall slope angle, with the interface angle assumed to be half the angle of internal friction of the soil.
2. The bridge is classified under Seismic Zone 1. Semi-integral abutments where the superstructure end diaphragm overhangs the back of the abutment should be checked for overturning with 100% of the seismic active force.
3. Active earth pressure using Coulomb's Theory should be used for gravity and short-heel cantilever walls. Use Rankine's Theory for long-heel cantilever walls.
4. Passive earth pressure for walls should be neglected for cases outlined in MaineDOT BDG 3.6.9. MaineDOT BDG 5.4.2.11 recommends abutment and wingwall reinforcement be sized assuming passive earth pressure on the backface of the wall. Design passive earth pressure coefficient should be calculated using MassDOT BDM Figure 3.10.8-1 and NHI-06-089 Figure 10-4, and the more stringent value should apply. However, passive earth pressure should be no less than Rankine passive earth pressure, regardless of wall rotation.

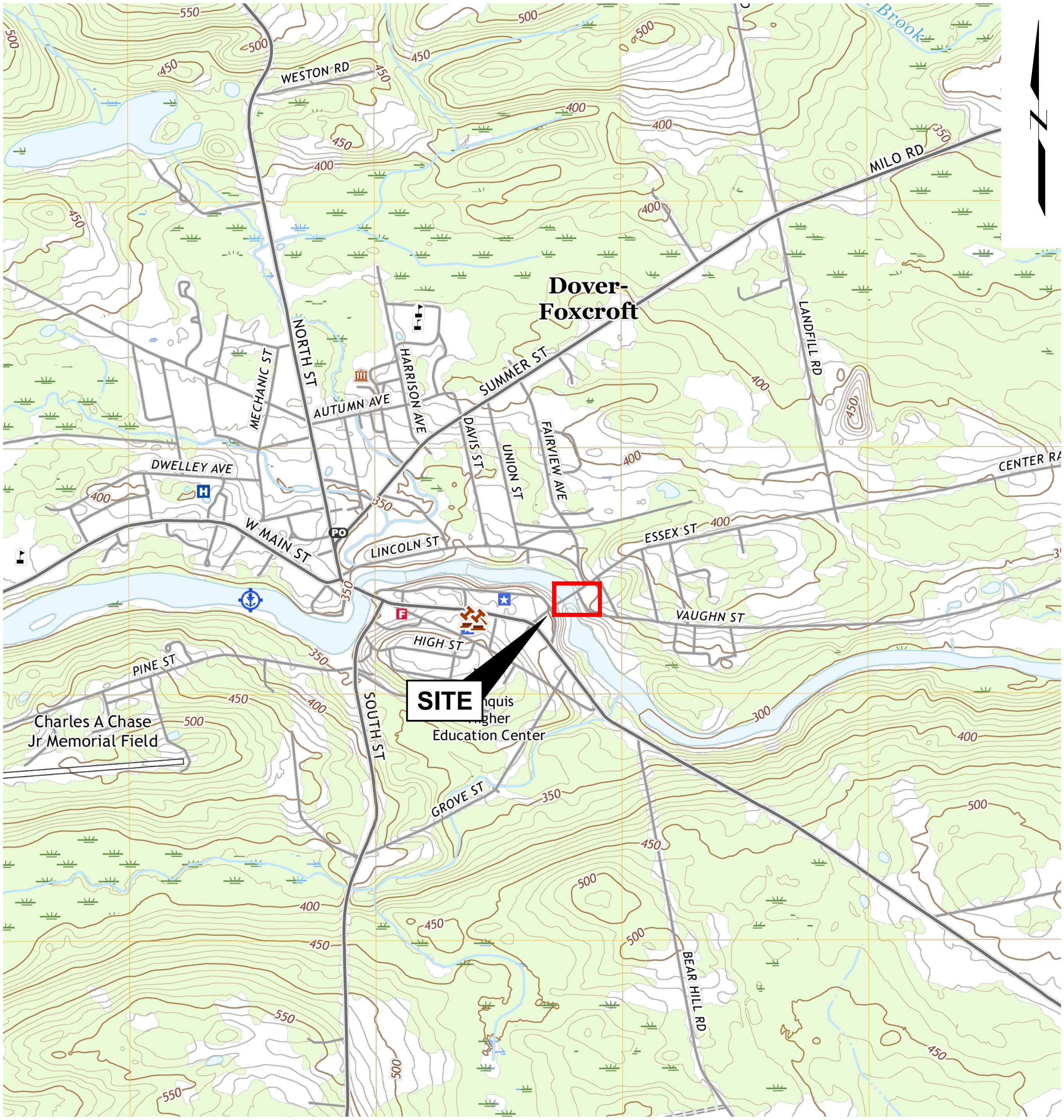
Table 5. Resistance Factors for Spread Footings
Geotechnical Design Report
Dover Bridge (#5118) Essex Street over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

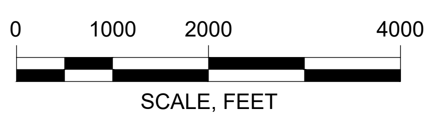
Load Case	Strength Limit State(2)	Service Limit State(3)	Extreme Limit State(4)
Cast-in-Place Cantilever Abutments			
Bearing resistance of shallow foundations	0.45	1.0	1.0
Sliding (Cast-in-place concrete)	0.8	1.0	1.0
Global Stability(5)	0.65	NA	NA
Cast-in-place Cantilever Walls			
Bearing resistance	0.55	1.0	0.8
Sliding	1.0	1.0	1.0
Global Stability(5)	0.75	NA	NA

General Notes:

1. Resistance factors above were obtained from the AASHTO LRFD Bridge Design Specifications (AASHTO).
2. The strength limit state resistance factors for bearing and sliding of shallow foundations were obtained from AASHTO Table 10.5.5.2.2-1 and Table 11.5.7-1.
3. Both AASHTO Sections 10.5.5.1 and 11.5.7-1 indicate that a resistance factor of 1.0 should be used for bearing resistance and sliding at the Service Limit State.
4. AASHTO Sections 10.5.5.3 and 11.5.8 provide resistance factors for the Extreme Limit State.
5. Per AASHTO Articles 10.5.5.2.1 and 11.6.3.7, global (overall) stability analysis is required using Strength I load combination with a Load Factor of 1.0 on vertical earth loading and Load Factors from Table 3.4.1-1 for other loads. Global stability analysis is not required for the Extreme Event Limit State, because seismic analysis of abutments and walls is not necessary, except for semi-integral abutments and MSE walls supporting stub abutments.

Sheets



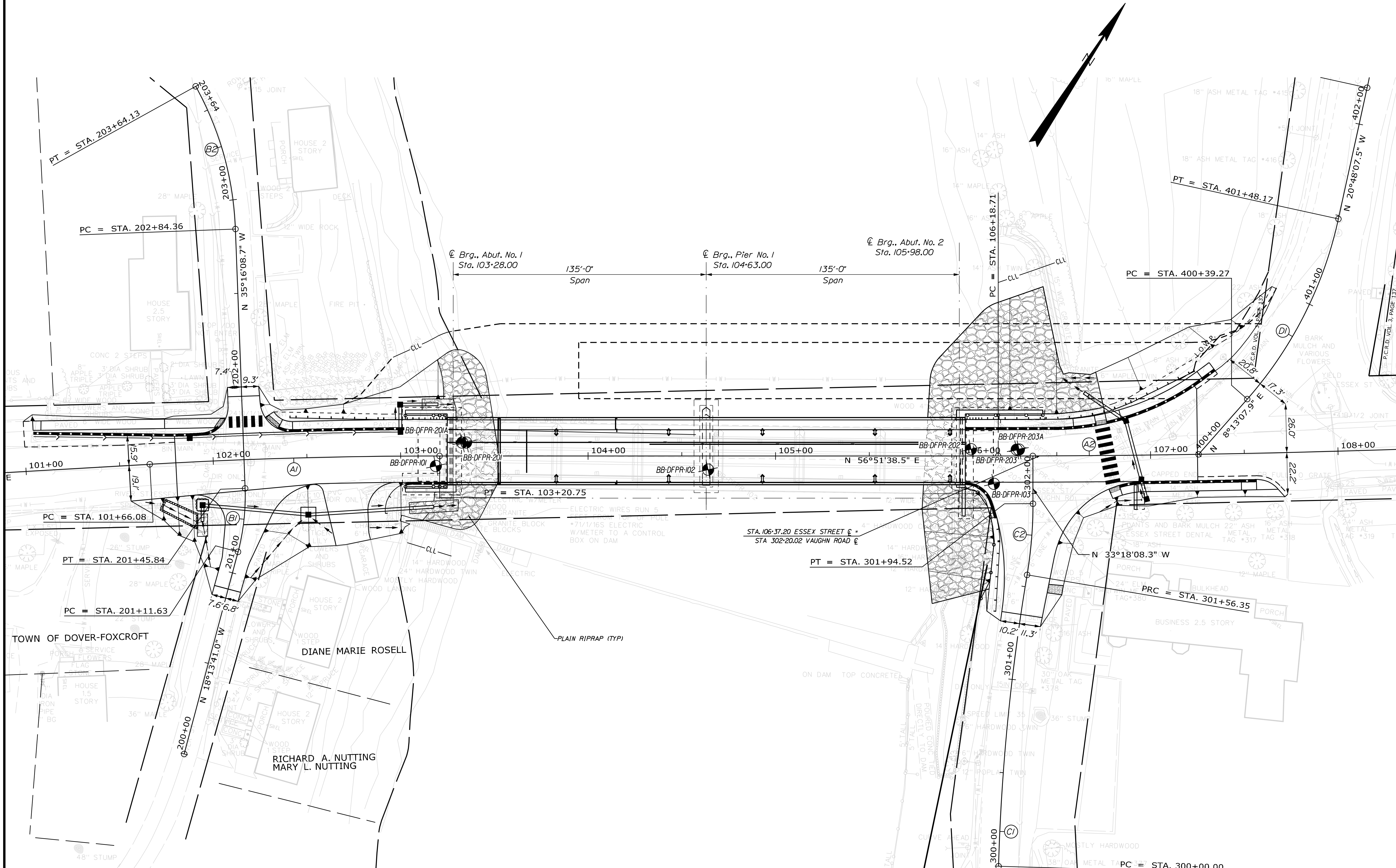


SOURCE:

USGS TOPOGRAPHIC QUADRANGLE, 7.5 MINUTE SERIES: DOVER-FOXCROFT QUADRANGLE, MAINE, 2021
NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88)
20-FOOT CONTOUR INTERVAL

Dover Bridge Replacement Project Dover-Foxcroft, Maine WIN 23120.00	GEI Consultants	SITE LOCATION MAP
Thornton Tomasetti Portland, Maine	Project 2305541	August 2025 Sheet 1

AHMED, MUSTAKI B:\Working\THORNTON TOMASETTI\2305541 MaineDOT Dover Bridge Final Design\00_CAD\Figures\Site Location\Site Location Map Dover.dwg - 8/26/2025



LEGEND

CASED WASHED BORINGS

SOURCE

Base plan is developed from electronic files (borings.dgn, rwplan.dgn, text.dgn, topo.dgn) provided to GEI by Thornton Tomasetti on February 16, 2024 and from electronic files (z_pier.dgn, z_abutments.dgn, z_superstructure.dgn, Bridge.dgn, alignments.dgn) provided to GEI by Thornton Tomasetti on April 30, 2025.

NOTES

1. Borings BB-DFPR-101 through BB-DFPR-103 were drilled by New England Boring Contractors of Hermon, Maine between November 2 and November 5, 2021 and observed by GEI personnel.
2. Borings BB-DFPR-201 through BB-DFPR-203A were drilled by New England Boring Contractors of Hermon, Maine between January 2 and January 8, 2024 and observed by GEI personnel.
3. As-drilled boring locations were surveyed by MaineDOT and provided to GEI.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	
CHIEF ENGINEER:	

		BY	DATE
PROJ. MANAGER	G. WILLIAMS		
DESIGN-DETAILED	M. JOHNESCU	Y. SOJO	AUG 2025
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			

SIGNATURE

P.E. NUMBER

DATE

BRIDGE NO. 5118
PISCATAQUIS RIVER
DOVER

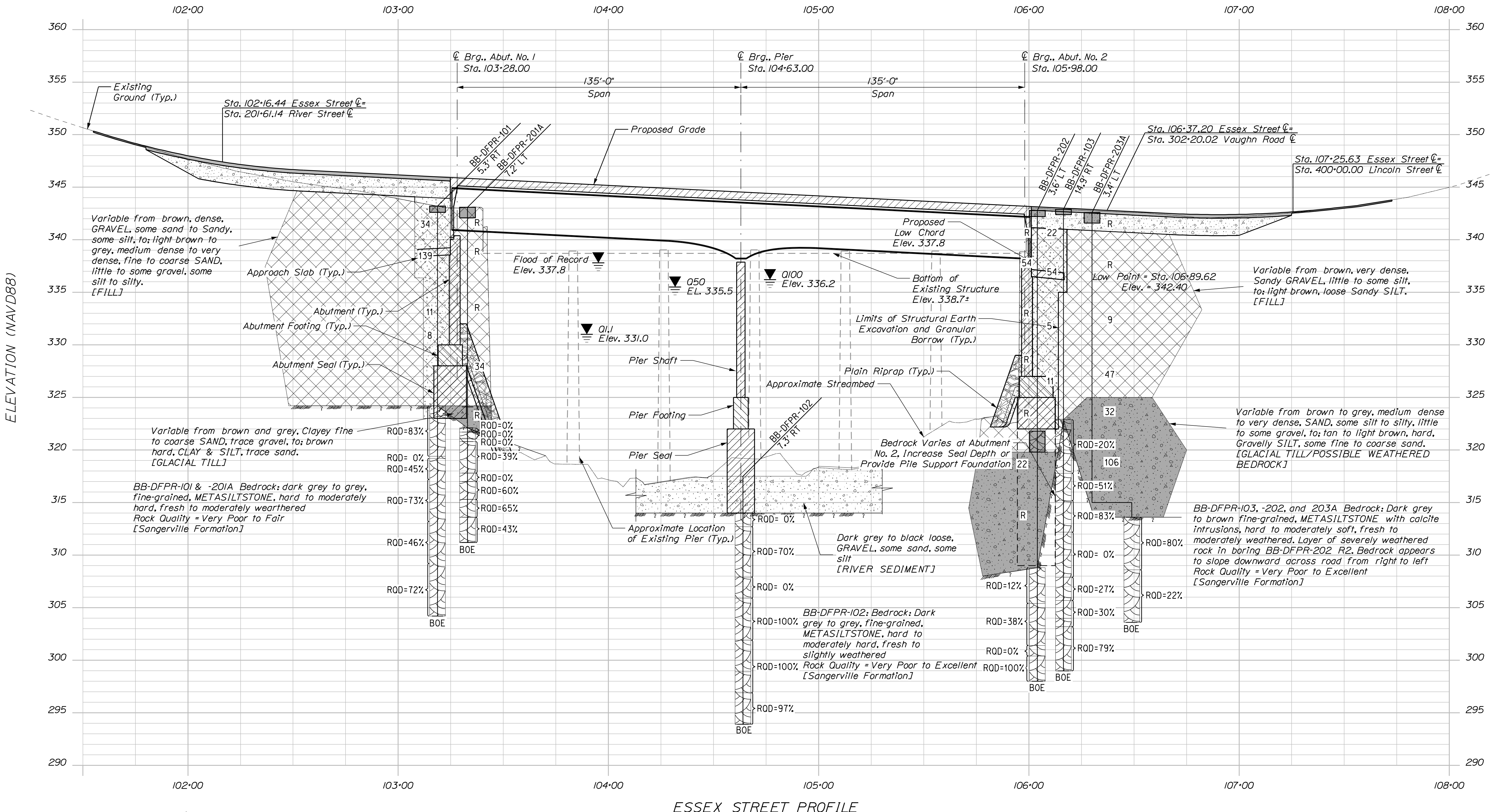
BORING LOCATION PLAN

SHEET NUMBER

2

OF 4

Filename: ...\DGN\003_GeoProfile_ 1.dgn Division: SURVEY Username: common Date:8/19/2025



LEGEND

Boring No. Offset, if shown

Pavement Thickness if applicable

6 Energy-Corrected SPT N60 Value (blows/foot)

R Split Spoon Refusal (>50 blows for 1' penetration)

Strata Interface

Advanced core barrel through possible boulder/rock

RQD= Rock Quality Designation for Rock Core Sample

BOE= Bottom of Exploration

Top of Intact Bedrock

Weathered and/or Fractured Bedrock

PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

NOTES

1) Profile developed from electronic file (Profile.dgn) provided to GEI by Thornton Tomasetti on April 30, 2025.

2) As-drilled boring locations were surveyed by MaineDOT and provided to GEI.

3) Borings BB-DFPR-201 and BB-DFPR-203 are not shown for clarity, and the original and offset borings are shown as one boring in the profile. Refer to the boring logs for more specific information at individual locations.

4) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more information refer to the boring logs.

PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	
CHIEF ENGINEER:	

	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
PROJ. MANAGER	G. WILLIAMS				
DESIGN-DETAILED	M. JOHNESCU Y. SOJO	AUG 2025			
CHECKED-REVIEWED					
DESIGN2-DETAILED2					
DESIGN3-DETAILED3					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					

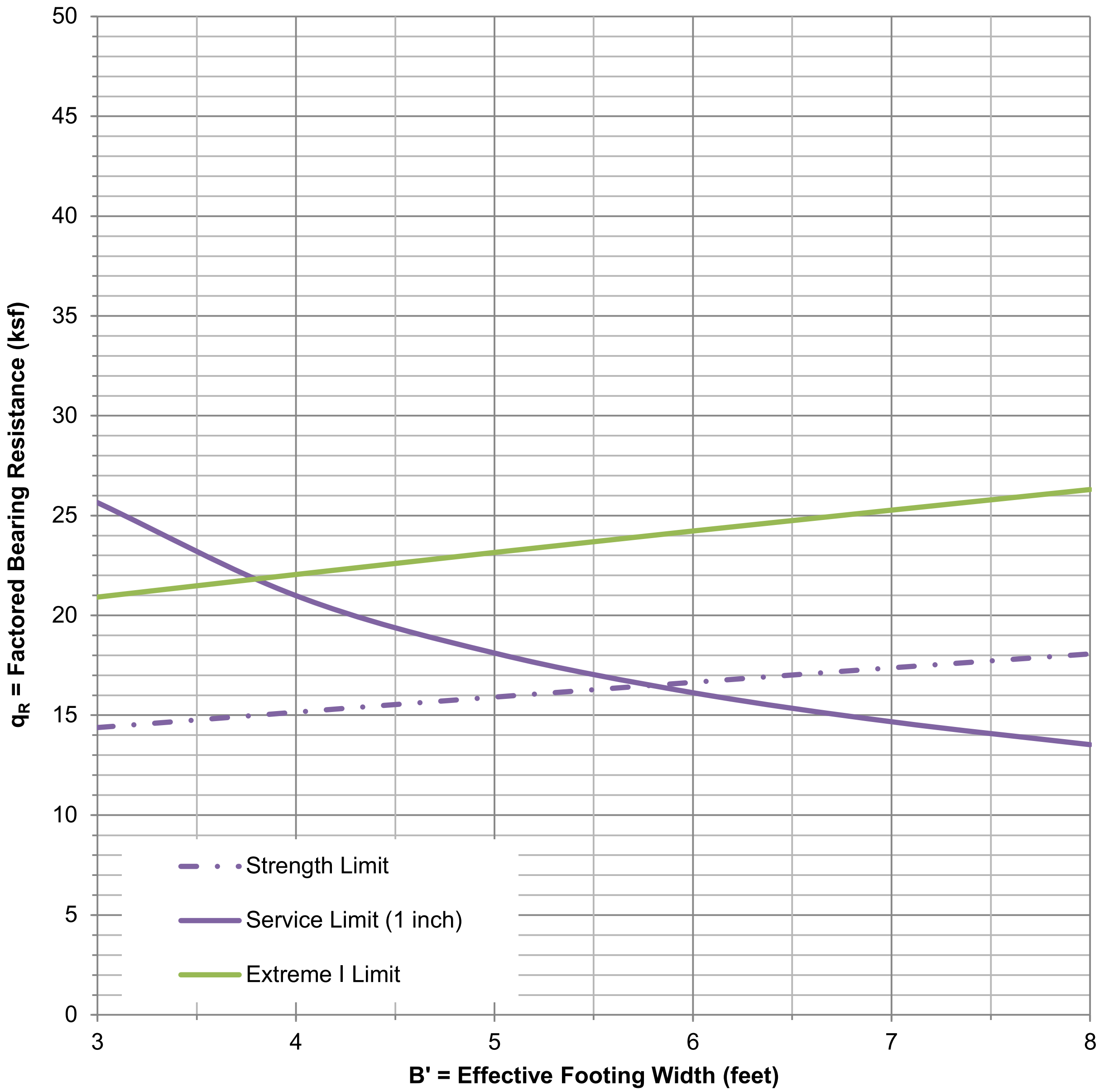
BRIDGE NO. 5118
PISCATAQUIS RIVER
DOVER

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4



Notes:

1. B' represents the smallest dimension (i.e. effective footing width). Length of footing assumed to be 23.5 ft.
2. Groundwater was assumed to be 12 ft below the ground surface.
3. The strength values are based on a resistance factor of 0.55 for gravity and cantilever retaining walls, and the extreme limit values are based on a resistance factor of 1.0.
4. An embedment depth of 7.5 ft. was assumed based on local frost depth.
5. Level ground in front and behind the wingwalls was assumed (i.e., no sloping ground).

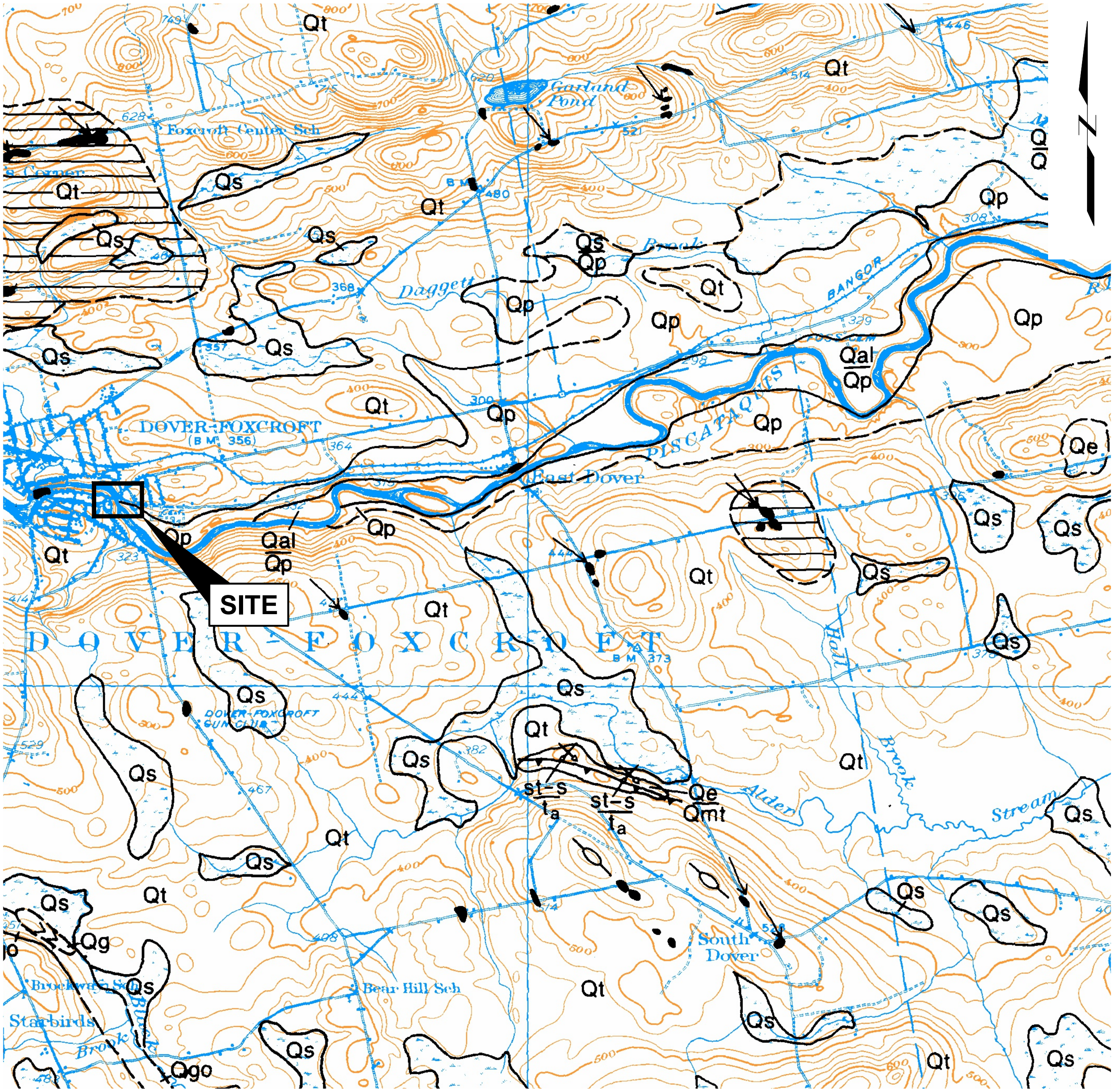
Dover Bridge Replacement Project Dover-Foxcroft, Maine WIN 23120.0	GEI Consultants	FACTORED BEARING RESISTANCE VERSUS EFFECTIVE FOOTING WIDTH - WINGWALLS ON FILL	
Thornton Tomasetti Portland, Maine	2305541	August 2025	Sheet 4

Appendix A Geology

A.1. Surficial Geology Map

A.2. Bedrock Geology

A.1. Surficial Geology Map



1:62,500

SCALE: AS NOTED

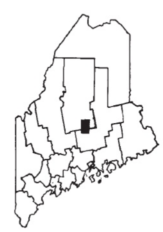
LEGEND:

Qt - Till: Heterogeneous mixture of sand, silt, clay, and stones.

Qp - Glacial-marine deposits (Presumpscot Formation): Mostly silt and clay. Low permeability. Poor drainage.

SOURCE:

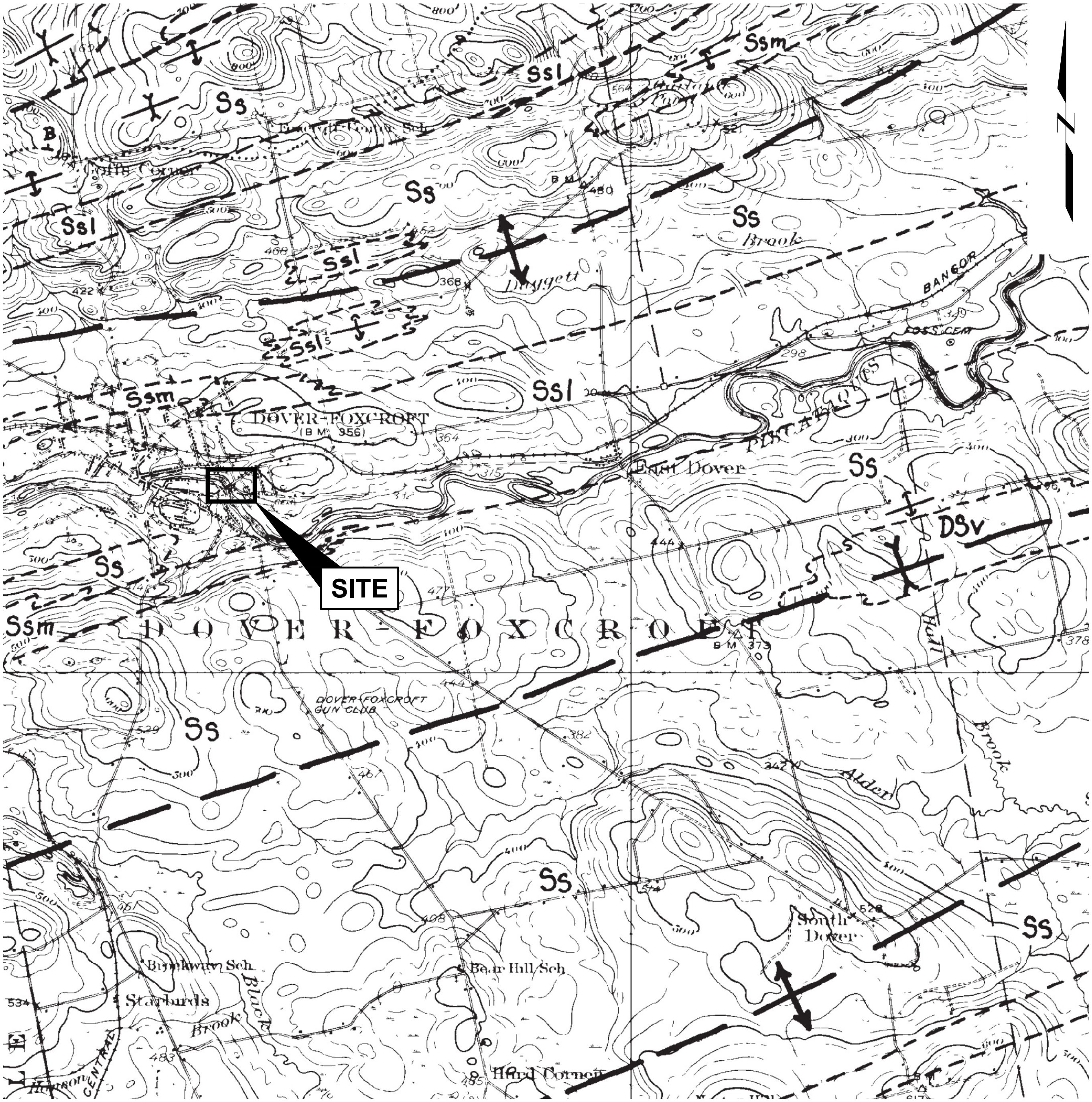
Map created with Reconnaissance Surficial Geology of the Dover-Foxcroft Quadrangle, Maine, prepared by the Maine Geological Survey in 1981.



Dover Bridge (#5118) over Piscataquis River WIN 023120.00 Dover-Foxcroft, Maine	GEI Consultants	SURFICIAL GEOLOGY MAP
Thornton Tomasetti Portland, Maine	Project 2305541	August 2025 Fig. A-1

AHMED, MUSTAKI B:\Working\THORNTON TOMASETTI\2305541 MaineDOT Dover Bridge Final Design\00_CAD\Figures\Geology\A-1 Surficial Geology - Dover Bridge.dwg - 8/26/2025

A.2. Bedrock Geology Map



1:62,500

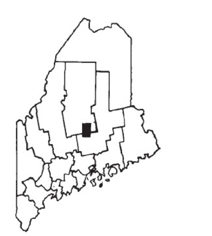
SCALE: AS NOTED

LEGEND:

Ssl - Sangerville Formation: Limestone member, pelitic limestone and calcareous metasiltstone.

SOURCE:

Map created with Maine Reconnaissance Bedrock Geology of the Dover-Foxcroft Quadrangle, Maine, prepared by the Maine Geological Survey in 1971.



Quadrangle Location

Dover Bridge (#5118) over Piscataquis River WIN 023120.00 Dover-Foxcroft, Maine	GEI Consultants	BEDROCK GEOLOGY MAP
Thornton Tomasetti Portland, Maine	Project 2305541	August 2025 Fig. A-2

AHMED, MUSTAKI B:\Working\THORNTON TOMASETTI\2305541 MaineDOT Dover Bridge Final Design\00_CAD\Figures\Geology\A-2 Bedrock Geology - Dover Bridge.dwg - 8/26/2025

Appendix B Boring Logs and Rock Core Photographs

B.1 Boring Logs

B.2 Rock Core Photographs

B.3 Hammer Calibration Summary Tables

B.1 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):

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

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

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-101

WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.2	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-2-2021; 10:05-15:26	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613748.67 E 1615991.32	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**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							HSA	342.6		-ASPHALT- 0.6	
	1D	24/12	1.00 - 3.00	12/12/10/9	22	34				Brown, moist, dense, fine to coarse SAND, some gravel, some fines (Fill).	
5	2D	20/10	5.00 - 6.67	WOH/5/85/10(.2")	90	139				(0-8"): Brown, very dense, fine to coarse SAND, some fines, little gravel (Fill). (8"-10"): Rock fragments, conglomerate, coarse-grained.	G#643272 A-2-4, SM WC=14.5%
10	3D	24/18	10.00 - 12.00	3/3/4/5	7	11	34			Brown, dry, medium dense, Silty fine to coarse SAND, nonplastic, trace gravel. 1/4" layer of white, angular rock at 9". Slightly more fines and moist from 9"-18" (Fill).	G#643273 A-4, SM WC=24.8%
							32				
							22				
	4D	24/0	13.00 - 15.00	3/3/2/4	5	8	27			No recovery.	
							37				
15							16				
							26				
							61				
							120				
	R1	57.6/54	19.00 - 23.80	RQD = 83%			NQ	324.2		19.0 Top of Bedrock at Elev. 324.2 R1: Bedrock: Dark grey to grey, fine-grained, METASILTSTONE, hard, slightly weathered. Horizontal to steep, closely-spaced joints from 8" to 52", slightly weathered with iron staining, tight. Bottom 6" is crushed, angular pieces. [Sangerville Formation] Rock Quality = Good 94% Recovery R1: Core Times (min:sec): 19.0-20.0' (2:57) 20.0-21.0' (2:40) 21.0-22.0' (1:59)	
20											
	R2	0.2/0.2	23.80 - 23.82	RQD = 0%							
	R3	24/18	24.00 - 26.00	RQD = 45%							
25											

Remarks:

Apparent cobble/boulder at 6 ft. Augered through cobble/boulder. Switch to drive and wash at 10 ft. Advanced 4" casing. Wash return transitions from red, angular rock pieces to grey, hard, angular rock pieces at 17 ft. Telescoped 3" casing for coring. Roller bit from 18.8 ft to 19 ft to begin coring. Bottom of boring at 39 ft. Borehole backfilled with soil cuttings and gravel and patched with asphalt. Water level in borehole not measured; river level measured at 14.2 ft below bridge deck.
Autohammer Serial No. NEBC-28

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-101
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.2	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-2-2021; 10:05-15:26	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613748.67 E 1615991.32	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MU = Unsuccessful Thin Wall Tube Sample Attempt
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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	Laboratory Testing Results/ AASHTO and Unified Class.
25										
	R4	48/43	26.00 - 30.00	RQD = 73%						
30	R5	48/40	30.00 - 34.00	RQD = 46%						qp=1385 ksf
	R6	60/60	34.00 - 39.00	RQD = 72%						
35										
								304.2		
40										
45										
50										

Visual Description and Remarks

22.0-23.0' (2:38)
23.0-23.8' (3:10)
R2: Bedrock: Dark grey to grey, fine-grained, METASILTSTONE, moderately hard, moderately weathered, crushed throughout.
[Sangerville Formation]
Rock Quality = Very Poor
100% Recovery
R2: Core times (min:sec):
23.8-24.0' (3:38)
R3: Bedrock: Dark grey to grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered, limestone veins throughout. Steep, closely-spaced joints from 5" to 14". Iron staining on joints.
[Sangerville Formation]
Rock Quality = Poor
75% Recovery
R3: Core Times (min:sec)
24.0-25.0' (3:40)
25.0-26.0' (2:23)
R4: Bedrock: Grey, fine-grained, METASILTSTONE, hard, slightly weathered, limestone veins throughout. Horizontal, mechanical break at 2" and 25". Steep joint at 41" with iron staining. Crushed rock pieces at bottom 6".
[Sangerville Formation]
Rock Quality = Fair
90% Recovery
R4: Core Times (min:sec):
26.0-27.0' (1:34)
27.0-28.0' (2:36)
28.0-29.0' (2:32)
29.0-30.0' (2:16)
R5: Bedrock: Dark grey to grey, banded, fine-grained, METASILTSTONE, moderately hard to hard, slightly to moderately weathered. Limestone veins throughout. Horizontal, tight joint with iron staining at 11". Steep, very closely-spaced joints from 4" to 6", slightly weathered. Crushed rock with iron staining and moderate weathering at the bottom 7".
[Sangerville Formation]
Rock Quality = Poor
83% Recovery
R5: Core Times (min:sec):
30.0-31.0' (2:09)
31.0-32.0' (2:38)
32.0-33.0' (3:07)
33.0-34.0' (2:31)
R6: Bedrock: Dark grey to grey, fine-grained, METASILTSTONE, hard, slightly weathered. Limestone veins throughout. Horizontal, closely spaced, tight joints from 5" to 53". Iron staining on horizontal joints at 5" and 43". Steeply dipping joint at 47".
[Sangerville Formation]
Rock Quality = Fair
100% Recovery
R6: Core Times (min:sec):
34.0-35.0' (3:12)

Remarks:

Apparent cobble/boulder at 6 ft. Augered through cobble/boulder. Switch to drive and wash at 10 ft. Advanced 4" casing. Wash return transitions from red, angular rock pieces to grey, hard, angular rock pieces at 17 ft. Telescoped 3" casing for coring. Roller bit from 18.8 ft to 19 ft to begin coring. Bottom of boring at 39 ft. Borehole backfilled with soil cuttings and gravel and patched with asphalt. Water level in borehole not measured; river level measured at 14.2 ft below bridge deck.
Autohammer Serial No. NEBC-28

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-101
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.2	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-2-2021; 10:05-15:26	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613748.67 E 1615991.32	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										35.0-36.0' (3:06) 36.0-37.0' (1:06) 37.0-38.0' (2:00) 38.0-39.0' (2:05)	
										39.0	
										Bottom of Exploration at 39.0 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

Apparent cobble/boulder at 6 ft. Augered through cobble/boulder. Switch to drive and wash at 10 ft. Advanced 4" casing. Wash return transitions from red, angular rock pieces to grey, hard, angular rock pieces at 17 ft. Telescoped 3" casing for coring. Roller bit from 18.8 ft to 19 ft to begin coring. Bottom of boring at 39 ft. Borehole backfilled with soil cuttings and gravel and patched with asphalt. Water level in borehole not measured; river level measured at 14.2 ft below bridge deck.
Autohammer Serial No. NEBC-28

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:	Dover Bridge #5118 carries Essex St over Piscataquis River	**Boring No.:**	BB-DFPR-102
Location:	Dover-Foxcroft, Maine	**WIN:**	23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.4	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-4-2021; 09:08-15:00	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613826.47 E 1616114.06	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	13.5 ft
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
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MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 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										-CONCRETE- (Bridge deck)	
								342.3		1.1	
5											
10											
15											
20											
25											

Remarks:

Advanced 4-inch casing through 1.1 ft of concrete bridge deck. Telescoped 3" casing through 4" casing. Bottom of boring at 49.3 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel to top of river sediments and patched with high strength concrete.
Autohammer Serial No. NEBC-28

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-102
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.4	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-4-2021; 09:08-15:00	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613826.47 E 1616114.06	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	13.5 ft
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
25	1D	24/2.5	25.50 - 27.50	1/2/1/2	3	5	1	317.9	
							11		
							8		
							32/.5	314.4	
	R1	15.6/12	29.00 - 30.30	RQD = 0%			NQ		
30	R2	60/60	30.30 - 35.30	RQD = 70%					
35	R3	24/20.4	35.30 - 37.30	RQD = 0%					
	R4	48/48	37.30 - 41.30	RQD = 100%					
40									qp=1026 ksf
	R5	60/60	41.30 - 46.30	RQD = 100%					
45									
	R6	36/36	46.30 - 49.30	RQD = 97%				294.1	
50									

Visual Description and Remarks

25.5
Dark grey to black, wet, loose, fine to coarse GRAVEL, some fine to coarse sand, some silt (River sediments).

29.0
Top of Bedrock at Elev. 314.4
R1: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, moderately hard, slightly weathered. Horizontal, slightly open joint at 5" with slight silt and sand infilling. Vertical, closely spaced joints throughout. Crushed rock bottom 3".
[Sangerville Formation]
Rock Quality = Very Poor
77% Recovery
R1: Core Times (min:sec)
29.0-30.0' (4:08)
30.0-30.3' (0:37)
R2: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, moderately hard to hard, slightly weathered to fresh. Vertical, tight to moderately tight joint with iron staining from 0" to 29". Horizontal, close-spaced joints from 2.4" to 55". Moderately dipping joint at 19" with iron staining. Steep, slightly weathered joint at 30".
[Sangerville Formation]
Rock Quality = Fair
100% Recovery
R2: Core Times (min:sec)
30.3-31.3' (2:20)
31.3-32.3' (2:41)
32.3-33.3' (3:04)
33.3-34.3' (1:59)
34.3-35.3' (2:36)
R3: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, moderately hard to hard, slightly weathered. Vertical and horizontal, closely spaced, tight to open joints throughout. Slight brown silt and iron staining on joints.
[Sangerville Formation]
Rock Quality = Very Poor
85% Recovery
R4: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, hard, slightly weathered to fresh. Horizontal, slightly weathered joint at 34". Moderately dipping joint at 41" with clay infilling. Limestone veins throughout.
[Sangerville Formation]
Rock Quality = Excellent
100% Recovery
R4: Core Times (min:sec)
37.3-38.3' (2:11)
38.3-39.3' (3:43)
39.3-40.3' (1:57)
40.3-41.3' (2:07)

Remarks:

Advanced 4-inch casing through 1.1 ft of concrete bridge deck. Telescoped 3" casing through 4" casing. Bottom of boring at 49.3 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel to top of river sediments and patched with high strength concrete.
Autohammer Serial No. NEBC-28

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Dover Bridge #5118 carries Essex St over Piscataquis River Location: Dover-Foxcroft, Maine	Boring No.: BB-DFPR-102 WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.4	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-4-2021; 09:08-15:00	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613826.47 E 1616114.06	**Casing ID/OD:**	3.00/3.50 (NW), 4.00/4.50 (HW)	**Water Level*:**	13.5 ft
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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V = Field Vane Shear Test, PP = Pocket Penetrometer
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SSA = Solid Stem Auger
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WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R5: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, hard, fresh. Moderately dipping joint at 2", slightly weathered. Limestone veins throughout. Steep joint at 54". [Sangerville Formation] Rock Quality = Excellent 100% Recovery R5: Core Times (min:sec) 41.3-42.3' (2:22) 42.3-43.3' (1:56) 43.3-44.3' (2:06) 44.3-45.3' (2:43) 45.3-46.3' (1:07)	
55										R6: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, hard, fresh. Horizontal, slightly weathered joint at 1". Partial horizontal joint at 34". Vertical joint from 34" to 36". Limestone veins throughout. [Sangerville Formation] Rock Quality = Excellent 97% Recovery R6: Core Times (min:sec) 46.3-47.3' (4:21) 47.3-48.3' (2:17) 48.3-49.3' (2:23)	
60										49.3 **Bottom of Exploration at 49.3 feet below ground surface.**	
65											
70											
75											

Remarks:

Advanced 4-inch casing through 1.1 ft of concrete bridge deck. Telescoped 3" casing through 4" casing. Bottom of boring at 49.3 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel to top of river sediments and patched with high strength concrete.
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Dover Bridge #5118 carries Essex St over Piscataquis River Location: Dover-Foxcroft, Maine	Boring No.: BB-DFPR-103 WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.9	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-3-2021; 08:54-15:30	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613903.51 E 1616245.85	**Casing ID/OD:**	4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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							HSA	342.4		-ASPHALT- 0.5	
	1D	24/16	1.00 - 3.00	5/6/8/10	14	22				Brown, dry, medium dense, Silty SAND, some gravel (Fill).	
5	2D	24/7	5.00 - 7.00	6/28/7/6	35	54				Brown, dry, very dense, Sandy GRAVEL, little silt (Fill).	G#643274 A-1-b, GM WC=5.6%
10	3D	24/14	10.00 - 12.00	2/1/2/2	3	5	20			Light brown, dry, loose, Sandy SILT. Red, 1/4" layer of decomposed wood and wood chip at 4" from bottom (Fill).	
							16				
							18				
							20				
							46				
15	4D	24/0	15.00 - 17.00	5/3/4/6	7	11	32			No recovery. Redrive: Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt (Fill).	G#643275 A-1-b, SM WC=11.7%
							36				
							44				
							95				
20	R1	60/48	20.00 - 25.00	RQD = 20%			NQ	322.9		20.0 Top of Bedrock at Elev. 322.9 R1: Bedrock: Dark grey and brown, fine-grained, METASILTSTONE, moderately hard to moderately soft, moderately weathered. Iron staining throughout. Vertical, closely spaced joints from 17" to 48". Moderately dipping bedding joint with limestone infill at 14". Low angle, closely spaced joints from 18" to 38". Crushed rock with significant weathering from 18" to 22", 32" to 35" and along vertical joints. [Sangerville Formation] Rock Quality = Very poor	
25											

Remarks:

Musty odor from hole at 5 ft. Switched to drive and wash at 10 ft. Advanced 4" casing. Weathered rock encountered at 19.3 ft. Solid rock encountered at 19.7 ft. Roller cone to 20 ft. Odor from 5 ft still present at the start of coring. Bottom of boring at 43.9 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel and patched with asphalt. Water level in borehole not measured. River level measured at 13.8 ft below bridge deck.
Autohammer Serial No. NEBC-28

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Boring No.: BB-DFPR-103

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.9	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-3-2021; 08:54-15:30	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613903.51 E 1616245.85	**Casing ID/OD:**	4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Laboratory Testing Results/ AASHTO and Unified Class.
25	R2	31.2/28.8	25.10 - 27.70	RQD = 51%						
	R3	36/36	27.70 - 30.70	RQD = 83%						
30	R4	51.6/42	30.70 - 35.00	RQD = 0%						
35	R5	26.4/24	35.00 - 37.20	RQD = 27%						
	R6	30/15.6	37.20 - 39.70	RQD = 30%						
	R7	50.4/46	39.70 - 43.90	RQD = 79%						
40										qp=550 ksf
								299.0		
45										
50										

Visual Description and Remarks

79% Recovery
R1: Core Times (min:sec)
20.0-21.0' (3:39)
21.0-22.0' (4:17)
22.0-23.0' (2:28)
23.0-24.0' (2.50)
24.0-25.1' (3.23)
R2: Bedrock: Dark grey, fine-grained, METASILTSTONE, moderately hard to hard, moderately to sligtly weathered. Iron staining throughout. Vertical joint from 0" to 24". Horizontal joints at 17" and 22". Many vertical joints from 18" to 25". Crushed rock from 26" to 29".
[Sangerville Formation]
Rock Quality = Fair
92% Recovery
R2: Core Times (min:sec)
25.1-26.1' (3:26)
26.1-27.1' (3:39)
27.1-27.7' (2:19)
R3: Bedrock: Dark grey, fine-grained, METASILTSTONE, hard, slightly weathered to fresh. Stepped, moderately dipping joint from 0" to 6". Horizontal joints at 18" and 35", slightly weathered. Crushed rock from 35" to 36". Vertical fractures from 18" to 35", fresh.
[Sangerville Formation]
Rock Quality = Good
100% Recovery
R3: Core Times (min:sec)
27.7-28.7' (1:18)
28.7-29.7' (1:30)
29.7-30.7' (1:43)
R4: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, moderately hard to moderately soft, slightly weathered. Vertical, closely spaced, tight joints throughout. Horizontal fracture at 20". Very close vertical and horizontal fracturing in bottom 1.5".
[Sangerville Formation]
Rock Quality = Very Poor
81% Recovery
R4: Core Times (min:sec)
30.7-31.7' (1:40)
31.7-32.7' (1:28)
33.7-34.7' (1:17)
34.7-35.0' (0:36)
R5: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, moderately hard to hard. Vertical, closely spaced, tight joints throughout. Horizontal, closely spaced joints from 11" to 20".
[Sangervile Formation]
Rock Quality = Poor
91% Recovery
R5: Core Times (min:sec)
35.0-36.0' (2:21)
36.0-37.0' (2:09)
37.0-37.2' (0:20)

Remarks:

Musty odor from hole at 5 ft. Switched to drive and wash at 10 ft. Advanced 4" casing. Weathered rock encountered at 19.3 ft. Solid rock encountered at 19.7 ft. Roller cone to 20 ft. Odor from 5 ft still present at the start of coring. Bottom of boring at 43.9 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel and patched with asphalt. Water level in borehole not measured. River level measured at 13.8 ft below bridge deck.
Autohammer Serial No. NEBC-28

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-103
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.9	**Auger ID/OD:**	ID 4.25"/OD 7.625"
Operator:	M. Porter	**Datum:**	NAVD88	**Sampler:**	Split Spoon
Logged By:	D. Pelletier	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	11-3-2021; 08:54-15:30	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N 613903.51 E 1616245.85	**Casing ID/OD:**	4.00/4.50 (HW)	**Water Level*:**	Not Measured
Hammer Efficiency Factor:	0.924	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R6: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, hard, slightly weathered. Tight, closely spaced joints. Steep joints from 4" to 12". Horizontal, very close joints from 12" to 17". Joint at 14" has iron staining. [Sangerville Formation] Rock Quality = Poor 52% Recovery R6: Core Times (min:sec) 37.2.38.3' (1:40) 38.2-39.2' (2:06) 39.2-39.7' (0:59) R7: Bedrock: Grey to dark grey, fine-grained, METASILTSTONE, hard, slightly weathered. Steep, tight joint from 8" to 29". Steep joint, slightly weathered with iron staining from 37" to 42". Horizontal joints with iron staining at 29", 35" and 42". Banded from 21" to 40". Thin limestone band at 27". [Sangerville Formation] Rock Quality = Good 91% Recovery R7: Core Times (min:sec) 39.7-40.7' (2:24) 40.7-41.7' (3:50) 41.7-42.7' (3:28) 42.7-43.7' (1:30) 43.7-43.9' (0:27) 43.9 **Bottom of Exploration at 43.9 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Musty odor from hole at 5 ft. Switched to drive and wash at 10 ft. Advanced 4" casing. Weathered rock encountered at 19.3 ft. Solid rock encountered at 19.7 ft. Roller cone to 20 ft. Odor from 5 ft still present at the start of coring. Bottom of boring at 43.9 ft. Borehole backfilled with bentonite chips in rock, then soil cuttings and gravel and patched with asphalt. Water level in borehole not measured. River level measured at 13.8 ft below bridge deck.
Autohammer Serial No. NEBC-28

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Boring No.: BB-DFPR-103

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Dover Bridge #5118 carries Essex St over Piscataquis River **Location:** Dover-Foxcroft, Maine	**Boring No.:** BB-DFPR-201 **WIN:** 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.0	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/2/2024; 09:00-13:00	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613768.05, E:1615998.08	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	NM
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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			12" ASPHALT.	
								342.0		1.0	
	1D	10/5	1.0 - 1.8	50/50(4")	--	--				Dark grey, moist, fine to coarse SAND, some silt, little gravel, (Fill).	
5	2D	16/6	4.0 - 5.3	9/6/13(4")	--	--	SPIN			Light brown to grey, dry, fine to coarse SAND, some gravel, some silt, (Fill).	
10	3D	0/0	10.0 - 10.0	5(0")	--	--	NQ			No Recovery. NQ core barrel used to advance 6' through existing abutment.	
15											
	4D	24/5	16.0 - 18.0	3/12/15/1	27	34	SPIN	326.7		(0-4"): Brown, wet, dense, Sandy GRAVEL, some silt, (Fill). 16.3	
										(4"-5"): Brown, wet, hard, CLAY & SILT, trace sand, (Glacial Till). Casing broke at 17 ft. Boring abandoned.	
								325.0		18.0	
										Bottom of Exploration at 18.0 feet below ground surface.	
20											
25											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 4.0 ft, switch to Spin & Wash using 4" casing. 3" Casing broke at 17 ft.
3. Water level not measured.
4. Borehole backfilled with gravel and patched with cold patch asphalt.
5. Offset approximately 2.1 feet west to drill boring BB-DFPR-201A.

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: Dover Bridge #5118 carries Essex St over Piscataquis River Location: Dover-Foxcroft, Maine	Boring No.: BB-DFPR-201A WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.1	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/2/24 13:00 - 1/3/24 15:00	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613766.88, E:1615996.29	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	16.3 ft bgs.
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			12" ASPHALT.	
								342.1		1.0	
5							26				
							50				
							SPIN				
	1D	7/2	9.0 - 9.6	13/9(1")	--	--				Grey, wet, coarse GRAVEL, some fine to medium sand, little silt, (Fill).	
10											
15											
								324.1		19.0	G#756478 A-4 (0), SC
20	2D	11/5	19.0 - 19.9	17/50(5")	--	--				Brown and grey, wet, Clayey fine to coarse SAND, trace gravel, moderately plastic fines, (Glacial Till).	
								322.1		21.0	
	R1	5/3	21.0 - 21.4	RQD = 0%			NQ			Approximate Top of Bedrock at Elev. 322.1 ft. R1: Grey, fine-grained, METASILTSTONE, hard, slightly weathered, crushed throughout. [Sangerville Formation] Rock Quality = Very Poor 60% Recovery R1: Core Times (min:sec) 21.0-21.4 ft (3:18)	qp= 2188 ksf
	R2	7/7	21.4 - 22.0	RQD = 0%							
	R3	3/3	22.0 - 22.3	RQD = 0%							
25	R4	36/30	22.3 - 25.3	RQD = 39%							

Remarks:
1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 5 ft, switch to Drive & Wash using 4" casing at 5 ft. Switch to Spin & Wash at 7 ft. Telescoped 3" casing for coring.
3. Water levels measured at end of drilling. River level at 16.9 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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: Dover Bridge #5118 carries Essex St over Piscataquis River

Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-201A

WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.1	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/2/24 13:00 - 1/3/24 15:00	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613766.88, E:1615996.29	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	16.3 ft bgs.
Hammer Efficiency Factor: 0.765		**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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	R5	10/10	25.3 - 26.1	RQD = 0%							
	R6	20/20	26.1 - 27.8	RQD = 60%							
	R7	20/16	27.8 - 29.5	RQD = 65%							
	R8	30/29	29.4 - 31.9	RQD = 43%							
30											
								311.2			
35											
40											
45											
50											

R2: Dark grey to grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered, vertical, tight joints with iron staining on joints.
[Sangerville Formation]
Rock Quality = Very Poor
100% Recovery
R2: Core Times (min:sec)
21.4-22.0 ft (3:37)
R3: Grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered, steep to vertical, tight joints with fine light grey infilling.
[Sangerville Formation]
Rock Quality = Very Poor
100% Recovery
R3: Core Times (min:sec)
22.0-22.3 ft (4:42)
R4: Dark grey to grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered. Steep to vertical, tight joints from 18" to 27" with fine brown infilling. Horizontal to low angle, mod. close, tight to open joints, with iron staining at 18" and 27".
[Sangerville Formation]
Rock Quality = Poor
83% Recovery
R4: Core Times (min:sec)
22.3-23.3 ft (2:44)
23.4-24.3 ft (2:22)
24.3-25.3 ft (1:47)
R5: Dark grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered, steep to vertical, tight joints with fine light grey infilling.
[Sangerville Formation]
Rock Quality = Very Poor
100% Recovery
R5: Core Times (min:sec)
25.3-26.1 ft (5:02)
R6: Grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered. Vertical, tight joints from 0 to 3", with fine light grey infilling. Steep, very close to close, tight joints at 3", and 10" to 20" with fine light grey infilling.
[Sangerville Formation]
Rock Quality = Fair
100% Recovery
R6: Core Times (min:sec)
26.1-27.1 ft (7:34)
27.1-27.8 ft (4:06)
R7: Grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered. Vertical, tight joints from 3" to 16" with fine light grey infilling. Horizontal, close, tight joints at 3", 6", and 13" with fine light grey infilling.
[Sangerville Formation]
Rock Quality = Fair
80% Recovery
R7: Core Times (min:sec)
27.8-28.8 ft (4:30)
28.8-29.4 ft (2:30)
R8: Grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered. Steep, tight joints from 10" to 18" with fine light grey infilling.

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 5 ft, switch to Drive & Wash using 4" casing at 5 ft. Switch to Spin & Wash at 7 ft. Telescoped 3" casing for coring.
3. Water levels measured at end of drilling. River level at 16.9 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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: Dover Bridge #5118 carries Essex St over Piscataquis River

Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-201A

WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	343.1	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/2/24 13:00 - 1/3/24 15:00	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613766.88, E:1615996.29	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	16.3 ft bgs.
Hammer Efficiency Factor:	0.765	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample 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										Low angle, very close, tight to open joints at 5" with fine light brown infilling. [Sangerville Formation] Rock Quality = Poor 97% Recovery R8: Core Times (min:sec) 29.4-30.4 ft (3:15) 30.4-31.4 ft (3:33) 31.4-31.9 ft (1:52)	
										31.9	
										Bottom of Exploration at 31.9 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 5 ft, switch to Drive & Wash using 4" casing at 5 ft. Switch to Spin & Wash at 7 ft. Telescoped 3" casing for coring.
3. Water levels measured at end of drilling. River level at 16.9 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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.

Boring No.: BB-DFPR-201A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-202
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.8	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/4/24 8:37 - 1/5/24 12:40	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613912.29, E:1616225.51	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	17.4 ft bgs.
Hammer Efficiency Factor:	0.765	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	342.2		7" ASPHALT (0.6)	
	1D	9/9	1.5 - 2.3	30/50(3")	--	--				Dark grey, dry, Gravelly fine to coarse SAND, little silt, (Fill).	
5	2D	24/7	4.0 - 6.0	19/20/22/18	42	54				(0-3"): Dark grey, dry, very dense, Gravelly fine to medium SAND, little silt, (Fill). (3"-7"): Light grey, dry, very dense, Sandy GRAVEL, little silt, (Fill).	
10	3D	13/4	9.0 - 10.1	15/11/9(1")	--	--	SPIN			Brown, wet, Sandy GRAVEL, some silt, (Fill).	
15	4D	4/1	14.0 - 14.3	30(4")	--	--				Light brown, wet, Gravelly fine to coarse SAND, some silt, (Fill). 1" angular rock.	
20	5D	1/0	19.0 - 19.1	5(1")	--	--				No recovery.	
	R1	24/0	21.0 - 23.0	RQD = 0%			NQ			No recovery.	
								319.8		(23.0)	
	6D	24/9	23.0 - 25.0	12/11/6/10	17	22	SPIN			Brown, wet, medium dense, SAND, some silt, little gravel. Angular rock fragments, (Glacial Till/Possible Weathered Rock).	G#756479 A-2-4 (0), SM
25											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 9 ft. Switch to Spin & Wash using 4" casing. Telescoped 3" at 19 ft.
3. Water levels measured at end of drilling. River level at 18.0 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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:** Dover Bridge #5118 carries Essex St over Piscataquis River **Location:** Dover-Foxcroft, Maine	**Boring No.:** BB-DFPR-202 **WIN:** 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.8	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/4/24 8:37 - 1/5/24 12:40	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613912.29, E:1616225.51	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	17.4 ft bgs.
Hammer Efficiency Factor:	0.765	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
	7D	3/2	29.0 - 29.3	50(3")	--	--				Brown, wet, Silty SAND, some gravel. Angular rock fragements, (Glacial Till/Possible Weathered Rock).	
30											
								308.8		No recovery. 34.0	
	8D	0/0	34.0 - 34.0	5(0")	--	--	NQ				
35	R2	41/27	34.0 - 37.4	RQD = 12%						Approximate Top of Bedrock at Elev. 308.8 ft. R2: Grey, fine-grained, METASILTSTONE, hard, severely to slightly weathered, crushed rock from 0 to 14", and 19" to 21". [Sangerville Formation] Rock Quality = Very Poor 60% Recovery R2: Core Times (min:sec) 34.0-35.0 ft (5:04) 35.0-36.0 ft (3:33) 36.0-37.0 ft (4:32) 37.0-37.4 ft (3:30)	
	R3	48/48	37.4 - 41.4	RQD = 38%						R3: Grey, fine-grained, METASILTSTONE with calcite intrusions from 0 to 24", hard, fresh to slightly weathered. Horizontal to low angle, very close to close, tight joints at 11", 13", 19", 30", 34", 40", and 42" with fine to medium grey and brown infilling. Many steep to vertical, tight joints from 30" to 48" with fine to medium grey infilling. [Sangerville Formation] Rock Quality = Poor 100% Recovery R3: Core Times (min:sec) 37.4-38.4 ft (3:56) 38.4-39.4 ft (3:15) 39.4-40.4 ft (5:23) 40.4-41.4 ft (4:50)	
40											
	R4	12/10	41.4 - 42.4	RQD = 0%						R4: Grey, fine-grained, METASILTSTONE, hard, fresh to slightly weathered, steep, tight joints with fine grey infilling. [Sangerville Formation] Rock Quality = Very Poor 83% Recovery R4: Core Times (min:sec) 41.4-42.4 ft (2:16)	
	R5	28/28	42.4 - 44.8	RQD = 100%				298.0		R5: Grey, fine-grained, METASILTSTONE with calcite intrusions, hard,	qp= 955 ksf
45											
50											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 9 ft. Switch to Spin & Wash using 4" casing. Telescoped 3" at 19 ft.
3. Water levels measured at end of drilling. River level at 18.0 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-202
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.8	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/4/24 8:37 - 1/5/24 12:40	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613912.29, E:1616225.51	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	17.4 ft bgs.
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60}=(Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										fresh. Low angle, close, tight joints at 8". Moderately dipping, close, tight joint at 15" with iron staining and grey infilling. [Sangerville Formation] Rock Quality = Excellent 100% Recovery R5: Core Times (min:sec) 42.4-43.4 ft (5:41) 43.4-44.4 ft (4:17) 44.4-44.8 ft (2:50) 44.8 **Bottom of Exploration at 44.8 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 9 ft. Switch to Spin & Wash using 4" casing. Telescoped 3" at 19 ft.
3. Water levels measured at end of drilling. River level at 18.0 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-203
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.6	**Auger ID/OD:**	4.5" SSA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/5/2024; 13:40-14:38	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NA
Boring Location:	N:613925.43, E:1616246.01	**Casing ID/OD:**	HW-4"	**Water Level*:**	NM
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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			12" ASPHALT.	
								341.6		1.0	
	1D	10/6	1.0 - 1.8	30/50(4")	--	--				Grey, dry, Gravelly fine to medium SAND, little silt, (Fill).	
5	2D	16/7	4.0 - 5.3	20/23/11(4")	--	--				Grey, dry, very dense, Gravelly fine to coarse SAND, little silt, (Fill). Rock fragments last 2".	
10	3D	24/8	9.0 - 11.0	2/4/3/3	7	9				Brown, moist, loose, Silty fine to coarse SAND, little gravel, (Fill).	
								331.6		Casing tilt due to probable cobbles/boulder. Boring abandoned. 11.0	
										Bottom of Exploration at 11.0 feet below ground surface.	
15											
20											
25											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Advance SSA to 9.0 ft, switch to Spin & Wash using 4" casing. 4" casing tilt. Boring abandoned.
3. Water level not measured.
4. Borehole backfilled with gravel and patched with cold patch asphalt.
5. Offset approximately 1.2 feet east to drill boring BB-DFPR-203A.

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: Dover Bridge #5118 carries Essex St over Piscataquis River Location: Dover-Foxcroft, Maine	Boring No.: BB-DFPR-203A WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.6	**Auger ID/OD:**	NA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/5/24 14:45 - 1/8/24 14:45	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613926.13, E:1616247.03	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	16.1 ft bgs.
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN			12" ASPHALT.	
								341.6		1.0	
5											
10											
	1D	24/0	14.0 - 16.0	19/21/16/11	37	47				No recovery.	
15											
								325.1		17.5	
	2D	24/11	19.0 - 21.0	8/9/14/16	25	32				Brown, wet, dense, Silty SAND, little gravel, slightly plastic fines. Angular rock fragments, (Glacial Till/Possible Weathered Rock).	
20											
	3D	24/14	24.0 - 26.0	25/40/43/26	83	106				(0-8"): Brown and grey, wet, very dense, Clayey SAND, little gravel, moderately plastic fines. Angular rock fragments, (Glacial Till/Possible	
25											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Spin & Wash using 4" casing to 10 ft. Telescoped 3" casing to sampling and coring.
3. Water levels measured at end of drilling. River level at 17.0 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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: Dover Bridge #5118 carries Essex St over Piscataquis River
Location: Dover-Foxcroft, Maine

Boring No.: BB-DFPR-203A
WIN: 23120.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	342.6	**Auger ID/OD:**	NA
Operator:	G. McDougal	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	S. Carvajal	**Rig Type:**	Mobile B-53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	1/5/24 14:45 - 1/8/24 14:45	**Drilling Method:**	Spin & Wash	**Core Barrel:**	NQ-2"
Boring Location:	N:613926.13, E:1616247.03	**Casing ID/OD:**	HW-4" & NW-3"	**Water Level*:**	16.1 ft bgs.
Hammer Efficiency Factor:	0.765	**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
$S_{u/r}$ = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane 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										Weathered Rock). (8"-14"): Tan to light brown, wet, hard, Gravelly SILT, some fine to coarse sand. Angular rock fragments, (Glacial Till/Possible Weathered Rock).	G#756480 A-4(0), ML
	4D	0/0	29.0 - 29.0	5(0")	--	--	NQ	313.6		No recovery. 29.0	
30	R1	60/58	29.0 - 34.0	RQD = 80%						Approximate Top of Bedrock at Elev. 313.6 ft. R1: Light grey, fine-grained, METASILTSTONE with calcite intrusions throughout, hard, fresh to moderately weathered, low angle, close to mod. close, tight joints with fine grey infilling and iron staining. Crushed rock from 0 to 7". [Sangerville Formation] Rock Quality = Good 97% Recovery R1: Core Times (min:sec) 29.0-30.0 ft (3:57) 30.0-31.0 ft (2:41) 31.0-32.0 ft (3:00) 32.0-33.0 ft (4:10) 33.0-34.0 ft (3:07)	qp= 1393 ksf
35	R2	60/58	34.0 - 39.0	RQD = 22%				303.6		R2: Light grey to grey, fine grained, METASILSTONE with calcite intrusions from 36" to 58", hard, fresh to slightly weathered. Steep, tight joints with iron staining from 0" to 26", and 44" to 54". Vertical, tight joints with iron staining 26" to 33", and 38" to 44". Crushed rock from 33" to 38", and 54" to 58". [Sangerville Formation] Rock Quality = Very Poor 97% Recovery R2: Core Times (min:sec) 34.0-35.0 ft (4:50) 35.0-36.0 ft (4:34) 36.0-37.0 ft (4:48) 37.0-38.0 ft (4:40) 38.0-39.0 ft (3:49) 39.0 **Bottom of Exploration at 39.0 feet below ground surface.**	
40											
45											
50											

Remarks:

1. Automatic hammer NEBC D-28. Energy Transfer Ratio = 0.765.
2. Spin & Wash using 4" casing to 10 ft. Telescoped 3" casing to sampling and coring.
3. Water levels measured at end of drilling. River level at 17.0 ft from top of bridge deck.
4. Borehole backfilled with gravel and patched with cold patch asphalt.

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.

B.2 Rock Core Photographs

MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-101	R1	19.0-23.8	57.6	54	48	83	Metasiltstone	1
BB-DFPR-101	R2	23.8-23.8	0.2	0.2	0	0	Metasiltstone	2
BB-DFPR-101	R3	24.0-26.0	24	18	11	45	Metasiltstone	2
BB-DFPR-101	R4	26.0-30.0	48	43	35	73	Metasiltstone	2,3
BB-DFPR-101	R5	30.0-34.0	48	40	22	46	Metasiltstone	3,4
BB-DFPR-101	R6	34.0-39.0	60	60	43	72	Metasiltstone	4

Notes:

1. "Box Row" indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.
3. BB-DFPR-101 R5 depth 31.4 feet selected for lab testing.

MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-101	R6	34.0-39.0	60	60	43	72	Metasiltstone	1
BB-DFPR-103	R1	20.0-25.0	60	48	12	20	Metasiltstone	1,2
BB-DFPR-103	R2	25.1-27.7	31.2	28.8	16	51	Metasiltstone	2
BB-DFPR-103	R3	27.7-30.7	36	36	30	83	Metasiltstone	3
BB-DFPR-103	R4	30.7-35.0	51.6	42	0	0	Metasiltstone	4

Notes:

1. “Box Row” indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.

MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-103	R5	35.0-37.2	26.4	24	7	27	Metasiltstone	1
BB-DFPR-103	R6	37.2-39.7	30	15.6	9	30	Metasiltstone	1
BB-DFPR-103	R7	39.7-43.9	50.4	46	40	79	Metasiltstone	2
BB-DFPR-102	R1	29.0-30.3	15.6	12	0	0	Metasiltstone	3
BB-DFPR-102	R2	30.3-35.3	60	60	42	70	Metasiltstone	3,4

Notes:

1. "Box Row" indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.
3. Boring BB-DFPR-103 R7 depth 41.2 selected for lab testing.

MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-102	R3	35.3-37.3	24	20.4	0	0	Metasiltstone	1
BB-DFPR-102	R4	37.3-41.3	48	48	48	100	Metasiltstone	2
BB-DFPR-102	R5	41.3-46.3	60	60	48	100	Metasiltstone	2,3
BB-DFPR-102	R6	46.3-49.3	36	36	35	97	Metasiltstone	4

Notes:

1. "Box Row" indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.
3. Boring BB-DFPR-102 R4 depth 40.2 selected for lab testing.

MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-201A	R1	21.0-21.4	5	3	0	0	Metasiltstone	1
BB-DFPR-201A	R2	21.4-22.0	7	7	0	0	Metasiltstone	1
BB-DFPR-201A	R3	22.0-22.3	3	3	0	0	Metasiltstone	1
BB-DFPR-201A	R4	22.3-25.3	36	30	14	39	Metasiltstone	1
BB-DFPR-201A	R5	25.3-26.1	10	10	0	0	Metasiltstone	1-2
BB-DFPR-201A	R6	26.1-27.8	20	20	12	60	Metasiltstone	2
BB-DFPR-201A	R7	27.8-29.4	20	16	13	65	Metasiltstone	2
BB-DFPR-201A	R8	29.4-31.9	30	29	13	43	Metasiltstone	2-3
BB-DFPR-202	R2	34.0-37.4	41	27	5	12	Metasiltstone	3
BB-DFPR-202	R3	37.4-41.4	48	48	18	38	Metasiltstone	4

Notes:

1. “Box Row” indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.

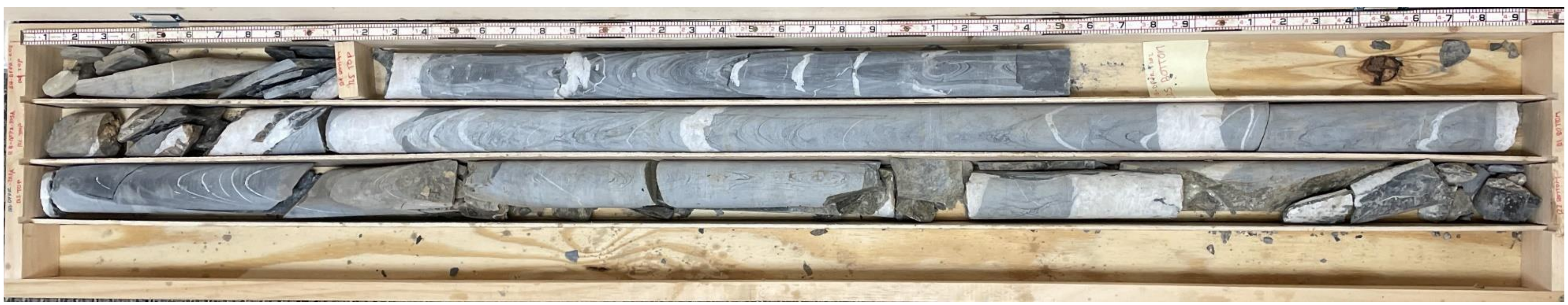
MaineDOT

Dover Bridge #5118 carrying Essex Street over Piscataquis River

Dover-Foxcroft, ME

Rock Core Photographs

Boring No.	Run	Depth (ft)	Penetration (in)	Recovery (in)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-DFPR-202	R4	41.4-42.4	12	10	0	0	Metasiltstone	1
BB-DFPR-202	R5	42.4-44.8	28	28	28	100	Metasiltstone	1
BB-DFPR-203A	R1	29.0-34.0	60	58	48	80	Metasiltstone	2
BB-DFPR-203A	R2	34.0-39.0	60	58	13	22	Metasiltstone	3



Notes:

1. “Box Row” indicates the section of the box where core run is contained: 1 = top, 4 = bottom.
2. Top of core at left. Increasing depth left to right.

B.3 Hammer Calibration Summary Tables

From the report titled "Standard Penetration Test Energy Measurement Calibration, Mobile Drill B-53 with Automatic Hammer (S/N NEBC-28)," by Geosciences Testing and Research, Inc, dated September 23, 2021.

TABLE 1
SPT ROD[1] CALIBRATION
MOBILE B-53 (NEBC-28) WITH AUTOMATIC HAMMER
SUMMARY OF RESULTS

RIG TYPE	HAMMER TYPE	BORING	DATE	Test Number	OPERATOR	DEPTH	SAMPLE [2] DESCRIPTION	BLOW [2] COUNT	BLOWS [3] ANALYZED		EMX[4] (k-ft)	ER[5] (k-ft)	ETR[6] (%)	FMX (kips)	BPM (bpm)	Cn[7]
Mobile B-53	Auto Hammer	B-2	9/20/21	#1	M.P	20-22	f.c SAND, some Gravel, Trace Silt	9,8,5,7	13	Average	0.321	0.350	91.8	40.0	58.2	1.53
										Std.Dev.	0.008	0.000	2.4	1.0	0.1	
										Maximum	0.333	0.350	95.1	41.0	58.5	
										Minimum	0.309	0.350	88.2	38.0	58.0	
				#2	M.P	22-24	f.c SAND, some Gravel, Trace Silt	7,9,8,13	17	Average	0.334	0.350	95.4	38.0	59.0	1.59
										Std.Dev.	0.005	0.000	97.9	1.0	0.1	
										Maximum	0.343	0.350	92.8	40.0	59.3	
										Minimum	0.325	0.350	82.9	36.0	58.7	
				#3	M.P	24-26	f.c SAND, some Gravel, Trace Silt	16,10,9,9	19	Average	0.315	0.350	90.0	36.0	56.3	1.50
										Std.Dev.	0.008	0.000	2.2	1.0	0.3	
										Maximum	0.333	0.350	95.2	36.0	57.1	
										Minimum	0.305	0.350	87.0	34.0	55.8	
				#4	M.P	26.5-28.5	f.c SAND, some Gravel, Trace Silt	8,7,8,7	15	Average	0.325	0.350	92.8	39.0	57.9	1.55
										Std.Dev.	0.009	0.000	2.6	1.0	0.3	
										Maximum	0.338	0.350	96.6	40.0	58.4	
										Minimum	0.309	0.350	88.3	37.0	57.2	
				#5	M.P	30-32	f.c SAND, some Gravel, Trace Silt	8,11,14,7	25	Average	0.321	0.350	91.6	40.0	57.8	1.53
										Std.Dev.	0.007	0.000	2.0	1.0	0.2	
										Maximum	0.332	0.350	94.7	42.0	58.4	
										Minimum	0.306	0.350	87.5	38.0	57.4	
				Average[8]		-	-	-	89	Average	0.323	0.350	92.2	38.6	57.8	1.54
										Maximum	0.343	0.350	96.6	42.0	59.3	
										Minimum	0.305	0.350	82.9	34.0	55.8	

Phase 1 Borings

Notes:
1. NWJ rods used with NWJ instrumented rod.
2. The soil description and SPT N-value were recorded by GTR. The SPT N-value is the sum of the middle 2 numbers when the sampler s driven for 4 - six inch intervals
3. Blows analyzed correspond to SPT N-value and may not match up exactly with the N-value due to differences in blow count logging between PDA and inspector or poor data quality.
4. EMX is the integration of F and V obtained from the PDA.
5. ER is the rated energy of 0.35 kip-ft based on 140 pound hammer and 2.5 feet drop height.
6. ETR is the energy transfer ratio based on (EMX/ER)*100%.
7. Cn is the energy correction factor which is equal to ETR/60% and is used to convert the measured SPT N-value to the corrected equivalent value representing 60% energy transfer.

TABLE 3 - SUMMARY OF SPT TEST RESULTS
MOBIL B53 - NEBC DRILL RIG #28 (SERIAL NUMBER D28-2/21)

SPT Analyzer Results PDA-S Ver. 2022.35.2 - Printed: 4/23/2023

Summary of SPT Test Results

Project: Mobil B53 D-28, Test Date: 4/21/2023

BPM: Blows/Minute
FMX: Maximum Force
AMX: Maximum Acceleration
VMX: Maximum Velocity

DMX: Maximum Displacement
DFN: Final Displacement
EMX: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average BPM bpm	Average FMX kips	Average AMX g's	Average VMX ft/s	Average DMX in	Average DFN in	Average EMX ft-lb	Average ETR %
19.00	**12-19-20-25**	**39**	**49**	50.0	39	3725	14.2	0.42	0.31	252	72.0
24.00	**8-39-26-26**	**65**	**82**	52.7	37	4030	15.1	0.33	0.18	268	76.6
29.00	**5-8-11-13**	**19**	**24**	54.3	40	4426	15.5	0.67	0.63	277	79.2
34.00	**8-7-8-6**	**15**	**19**	54.3	39	3041	14.4	0.83	0.80	270	77.1
39.00	**3-4-6-5**	**10**	**12**	54.2	39	2906	14.4	1.22	1.20	279	79.7
44.00	**11-14-23-15**	**37**	**47**	54.2	40	2694	12.9	0.41	0.32	275	78.7
			Overall Average Values:	52.8	39	3598	14.4	0.49	0.39	268	$\boxed{76.5}$
			Standard Deviation:	1.6	1	700	1.1	0.26	0.28	11	3.1
			Overall Maximum Value:	55.1	40	5470	17.0	1.50	1.50	288	82.3
			Overall Minimum Value:	49.7	36	2058	12.2	0.25	0.15	240	68.7

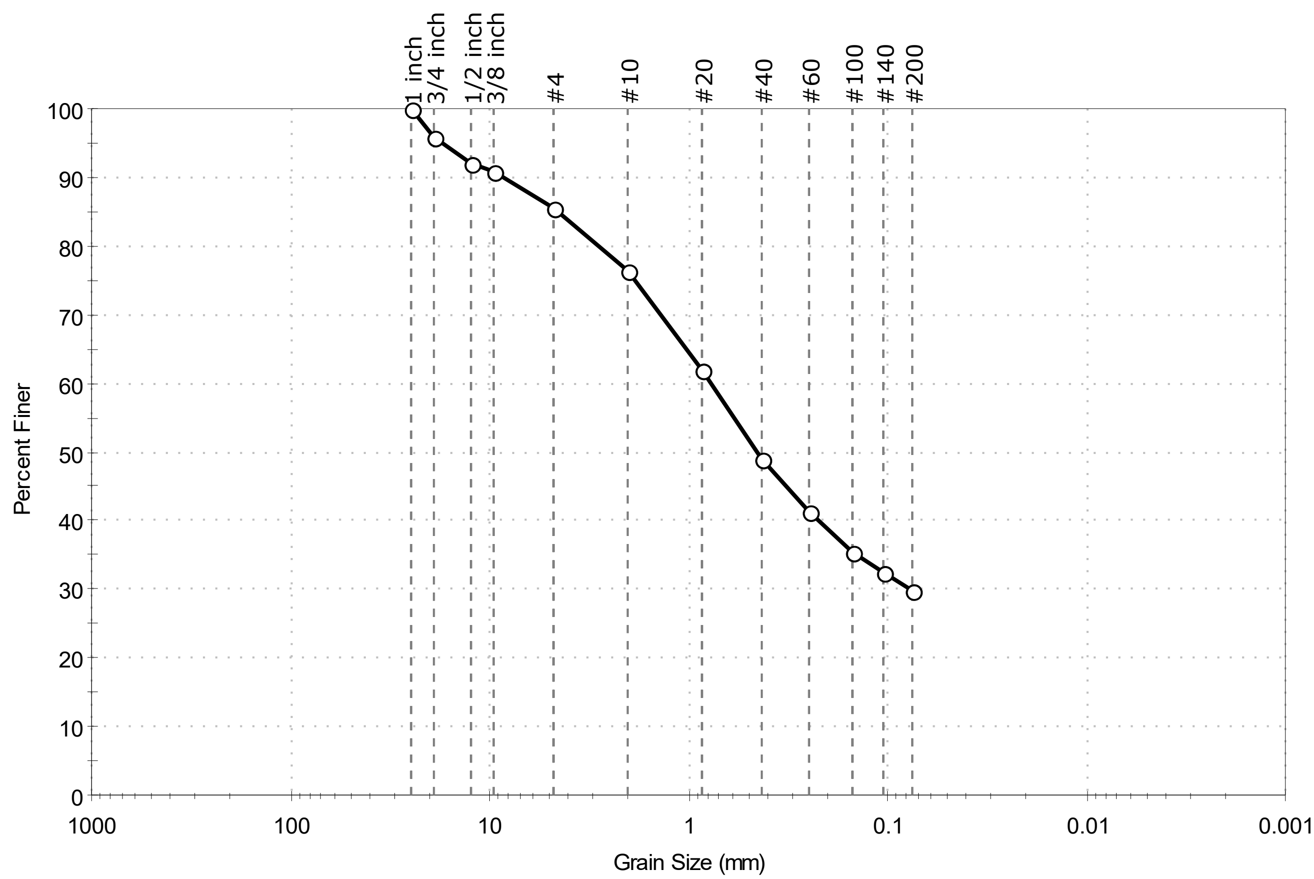
Phase 2 Borings

Appendix C Laboratory Testing

GeoTesting EXPRESS

Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Bridge Replace				
Location:	Dover-Foxcroft, ME			Project No:	GTX-314652
Boring ID:	BB-DFPR-101	Sample Type:	cylinder	Tested By:	ckg
Sample ID:	2D(0"-8")	Test Date:	12/06/21	Checked By:	bfs
Depth :	5-6.7 ft	Test Id:	643272		
Test Comment:	---				
Visual Description:	Moist, olive brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	14.5	55.6	29.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 inch	25.00	100		
3/4 inch	19.00	96		
1/2 inch	12.50	92		
3/8 inch	9.50	91		
#4	4.75	86		
#10	2.00	76		
#20	0.85	62		
#40	0.42	49		
#60	0.25	41		
#100	0.15	35		
#140	0.11	33		
#200	0.075	30		

Coefficients

D_{85} =4.5224 mm D_{30} =0.0762 mm
D_{60} =0.7694 mm D_{15} =N/A
D_{50} =0.4516 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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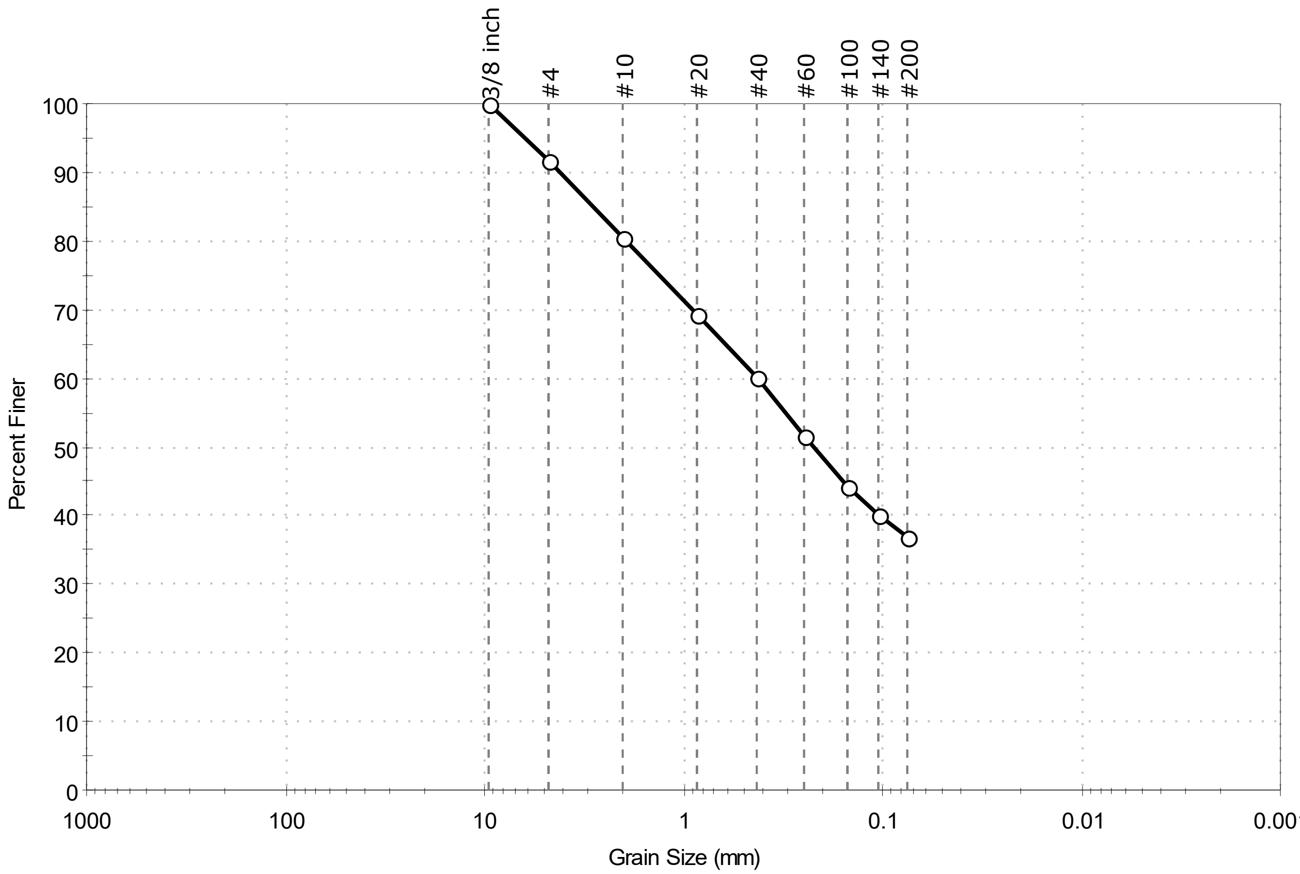
Client:	GEI Consultants, Inc.			
Project:	WIN 23120 Dover Bridge Replace			
Location:	Dover-Foxcroft, ME		Project No:	GTX-314652
Boring ID:	BB-DFPR-101	Sample Type: cylinder	Tested By:	ckg
Sample ID:	3D(9"18")	Test Date: 12/06/21	Checked By:	bfs
Depth :	10-12 ft	Test Id: 643273		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	8.2	54.9	36.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
3/8 inch	9.50	100		
#4	4.75	92		
#10	2.00	80		
#20	0.85	69		
#40	0.42	60		
#60	0.25	52		
#100	0.15	44		
#140	0.11	40		
#200	0.075	37		

Coefficients

D_{85} =2.8263 mm D_{30} =N/A
D_{60} =0.4223 mm D_{15} =N/A
D_{50} =0.2228 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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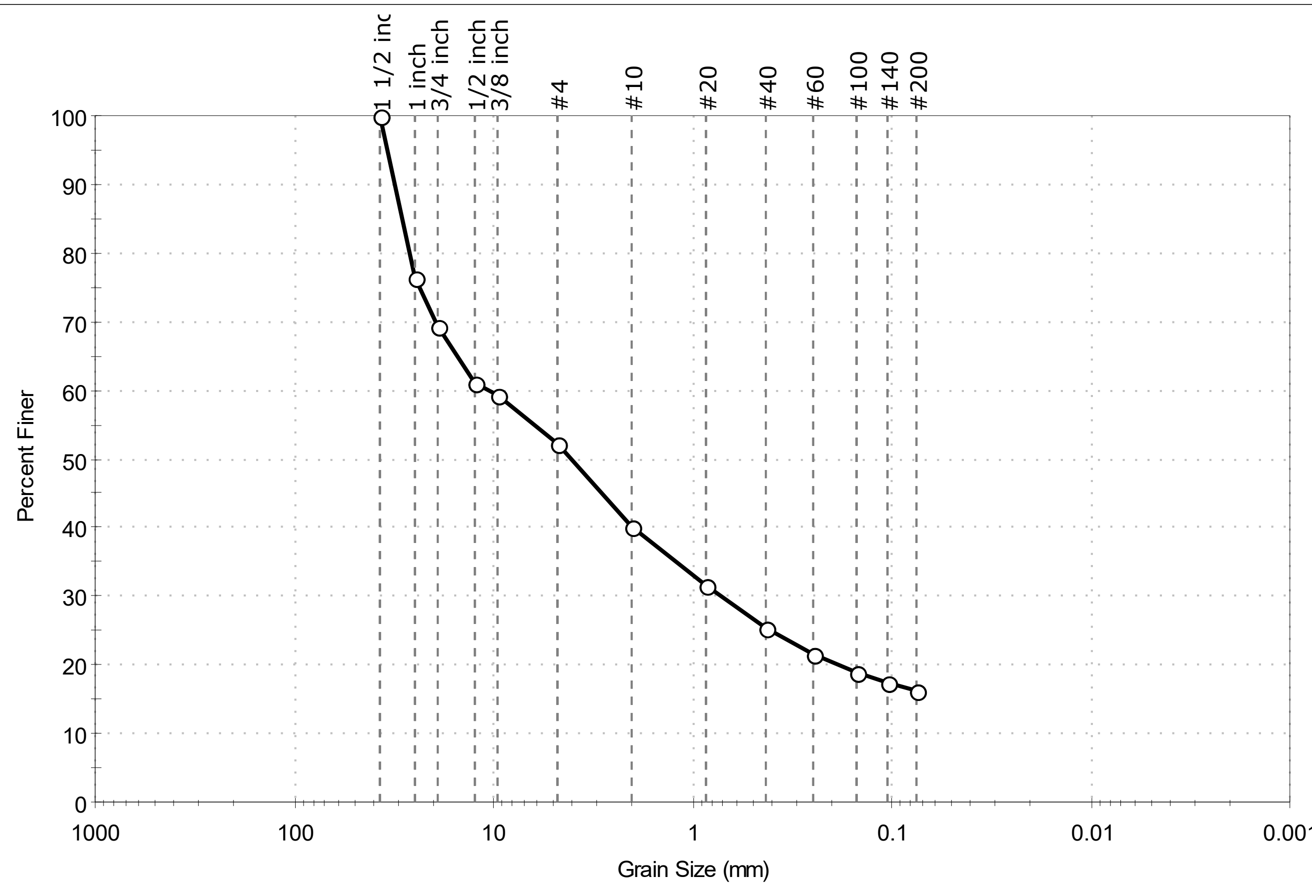
Client:	GEI Consultants, Inc.		
Project:	WIN 23120 Dover Bridge Replace		
Location:	Dover-Foxcroft, ME	Project No:	GTX-314652
Boring ID:	BB-DFPR-103	Sample Type: cylinder	Tested By: ckg
Sample ID:	2D(0"-7")	Test Date: 12/06/21	Checked By: bfs
Depth :	5-7 ft	Test Id: 643274	

Test Comment: ---

Visual Description: Moist, olive brown silty gravel with sand

Sample Comment: ---

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	47.9	36.0	16.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	76		
3/4 inch	19.00	69		
1/2 inch	12.50	61		
3/8 inch	9.50	59		
#4	4.75	52		
#10	2.00	40		
#20	0.85	32		
#40	0.42	25		
#60	0.25	21		
#100	0.15	19		
#140	0.11	17		
#200	0.075	16		

Coefficients

D_{85} =29.0038 mm D_{30} =0.7055 mm

D_{60} =10.5542 mm D_{15} =N/A

D_{50} =4.0804 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

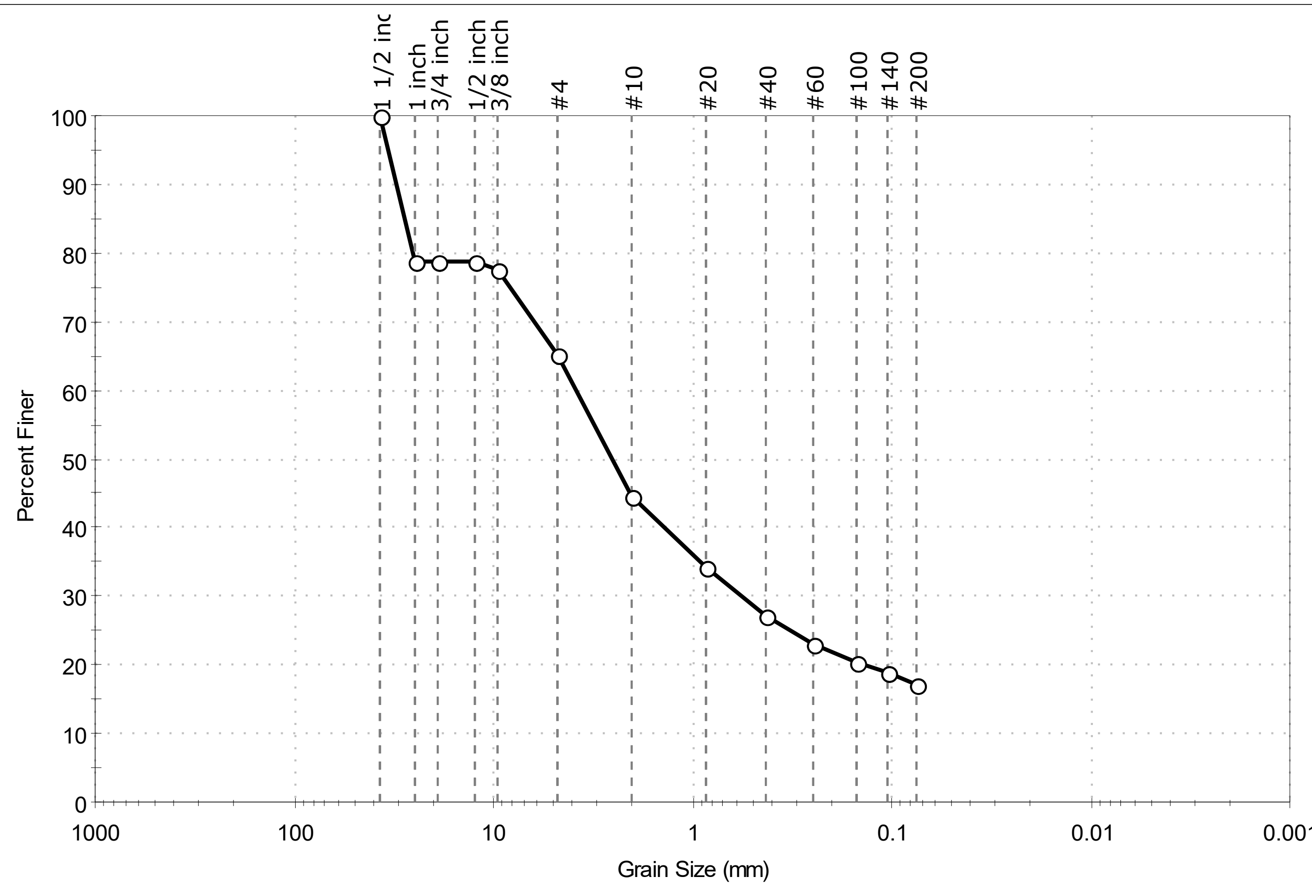
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Bridge Replace				
Location:	Dover-Foxcroft, ME			Project No:	GTX-314652
Boring ID:	BB-DFPR-103	Sample Type:	cylinder	Tested By:	ckg
Sample ID:	4D(0"-5")	Test Date:	12/06/21	Checked By:	bfs
Depth :	15-17 ft	Test Id:	643275		
Test Comment:	---				
Visual Description:	Moist, dark olive brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	34.7	48.1	17.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 1/2 inch	37.50	100		
1 inch	25.00	79		
3/4 inch	19.00	79		
1/2 inch	12.50	79		
3/8 inch	9.50	77		
#4	4.75	65		
#10	2.00	45		
#20	0.85	34		
#40	0.42	27		
#60	0.25	23		
#100	0.15	20		
#140	0.11	19		
#200	0.075	17		

Coefficients

D_{85} =28.1444 mm D_{30} =0.5637 mm

D_{60} =3.8089 mm D_{15} =N/A

D_{50} =2.5112 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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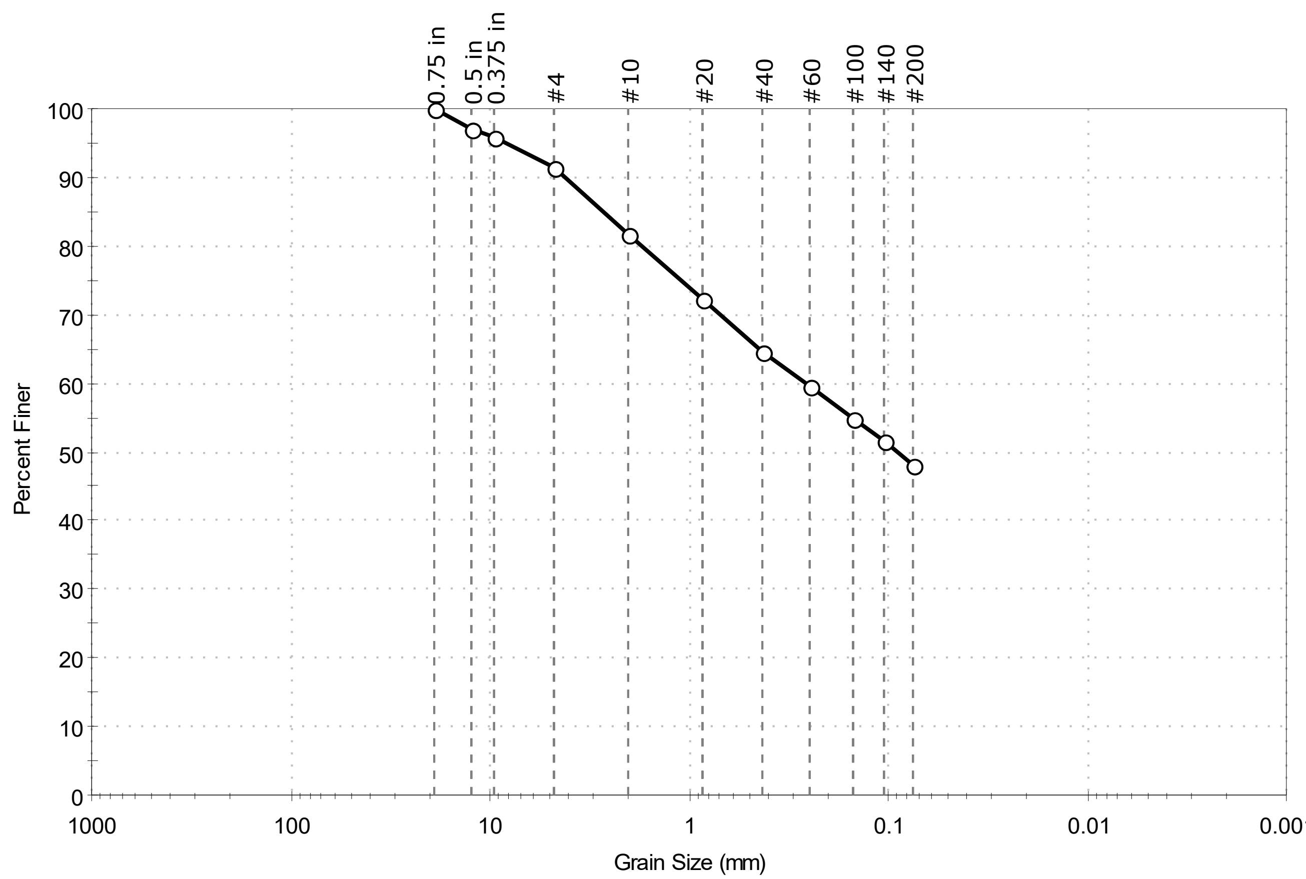
Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Brdg Replacement				
Location:	Dover-Foxcroft, ME			Project No:	GTX-318514
Boring ID:	BB-DFPR-201A	Sample Type:	tube	Tested By:	ckg
Sample ID:	2D	Test Date:	01/29/24	Checked By:	ank
Depth :	19'-19.9'	Test Id:	756478		

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

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	8.4	43.4	48.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	96		
#4	4.75	92		
#10	2.00	82		
#20	0.85	72		
#40	0.42	65		
#60	0.25	60		
#100	0.15	55		
#140	0.11	52		
#200	0.075	48		

Coefficients

D_{85} =2.6577 mm D_{30} =N/A
D_{60} =0.2625 mm D_{15} =N/A
D_{50} =0.0899 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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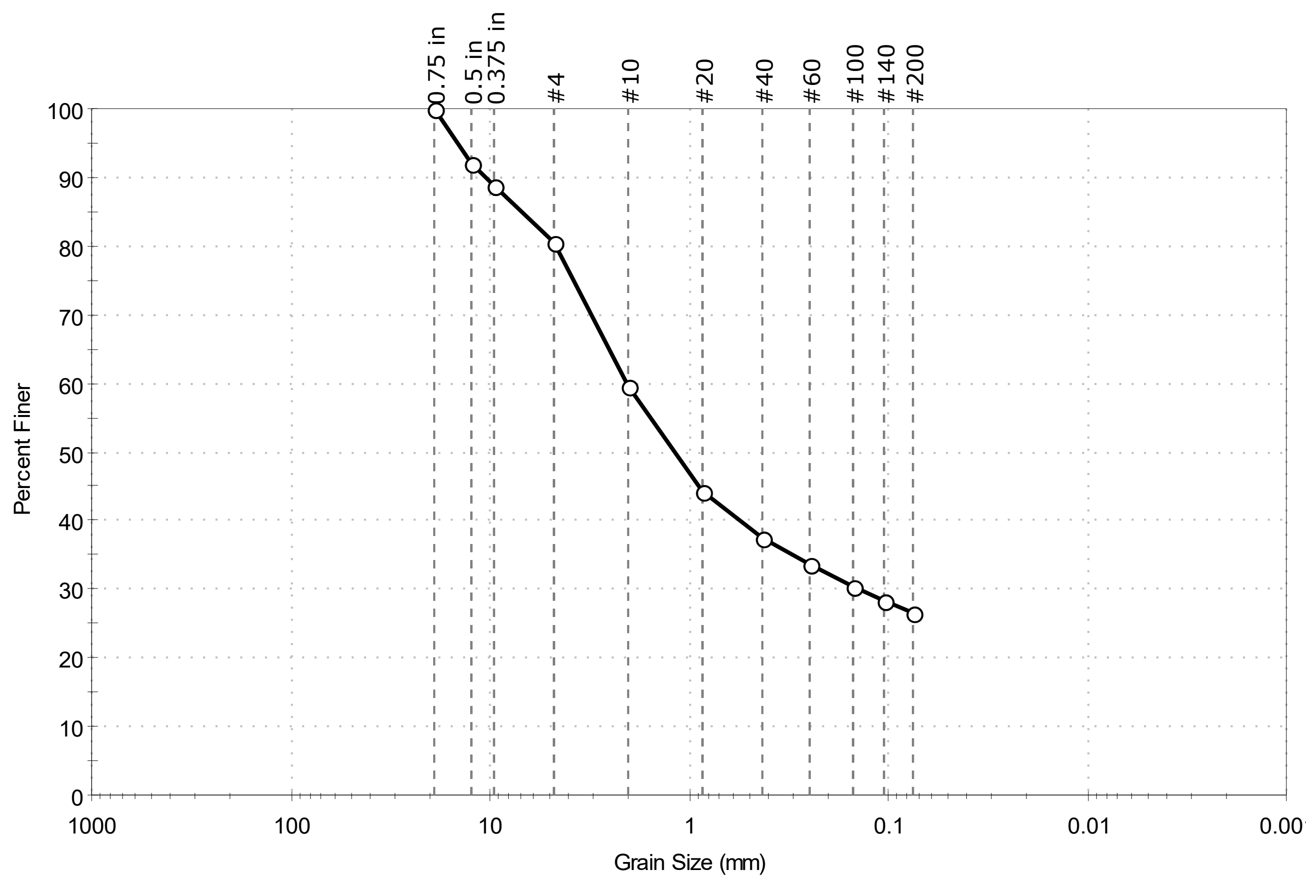
Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Brdg Replacement				
Location:	Dover-Foxcroft, ME			Project No:	GTX-318514
Boring ID:	BB-DFPR-202	Sample Type:	tube	Tested By:	ckg
Sample ID:	6D	Test Date:	01/29/24	Checked By:	ank
Depth :	23'-25'	Test Id:	756479		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	19.4	54.2	26.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	92		
0.375 in	9.50	89		
#4	4.75	81		
#10	2.00	60		
#20	0.85	44		
#40	0.42	37		
#60	0.25	34		
#100	0.15	30		
#140	0.11	28		
#200	0.075	26		

Coefficients

D_{85} =6.8501 mm
D_{30} =0.1416 mm
D_{60} =2.0382 mm
D_{15} =N/A
D_{50} =1.1711 mm
D_{10} =N/A
C_u =N/A
C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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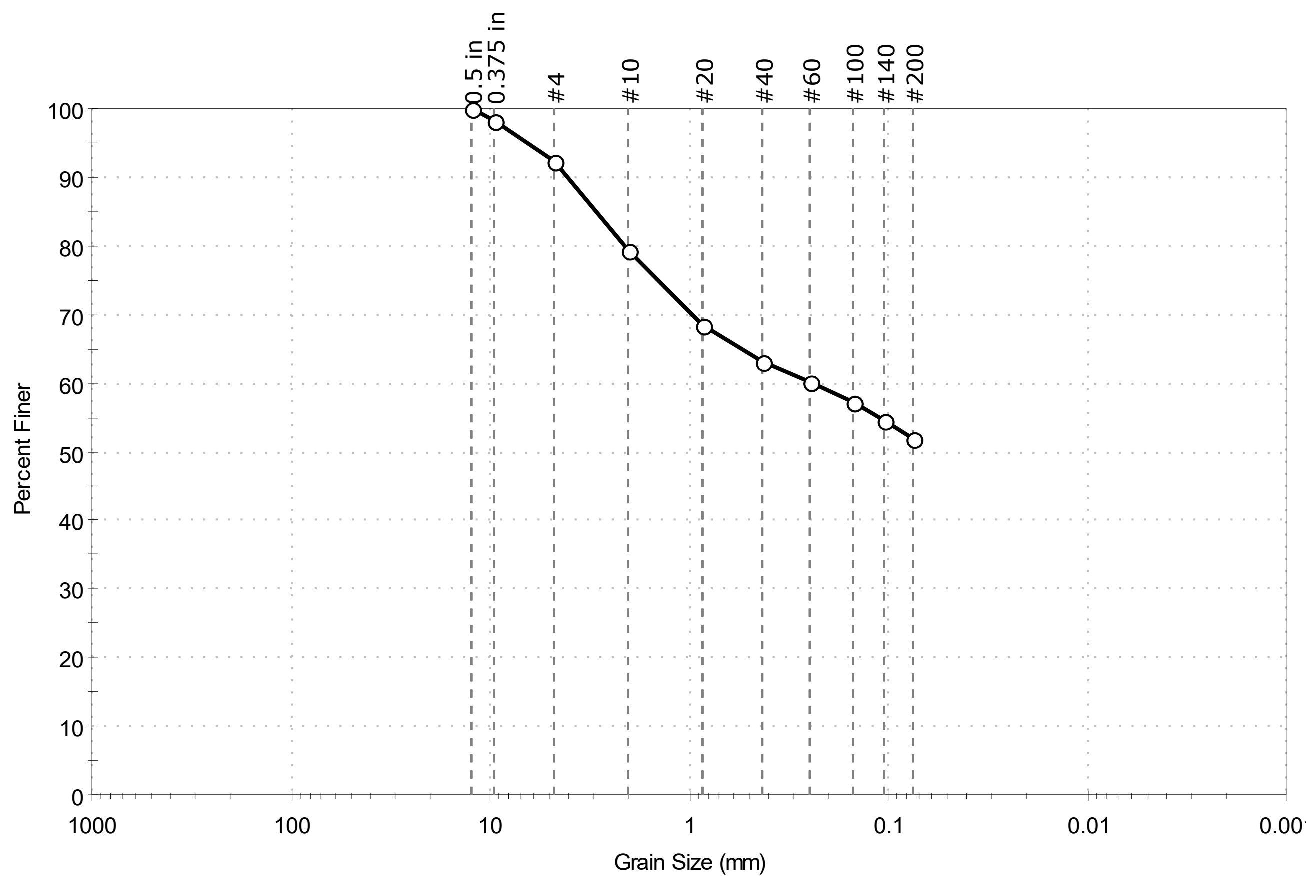
GeoTesting EXPRESS

Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Brdg Replacement				
Location:	Dover-Foxcroft, ME			Project No:	GTX-318514
Boring ID:	BB-DFPR-203A	Sample Type:	tube	Tested By:	ckg
Sample ID:	3D	Test Date:	01/29/24	Checked By:	ank
Depth :	24-26'	Test Id:	756480		
Test Comment:	---				
Visual Description:	Moist, yellowish brown sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	7.6	40.5	51.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	92		
#10	2.00	79		
#20	0.85	69		
#40	0.42	63		
#60	0.25	60		
#100	0.15	57		
#140	0.11	55		
#200	0.075	52		

Coefficients

D_{85} =2.9075 mm　　D_{30} =N/A

D_{60} =0.2449 mm　　D_{15} =N/A

D_{50} =N/A　　D_{10} =N/A

C_u =N/A　　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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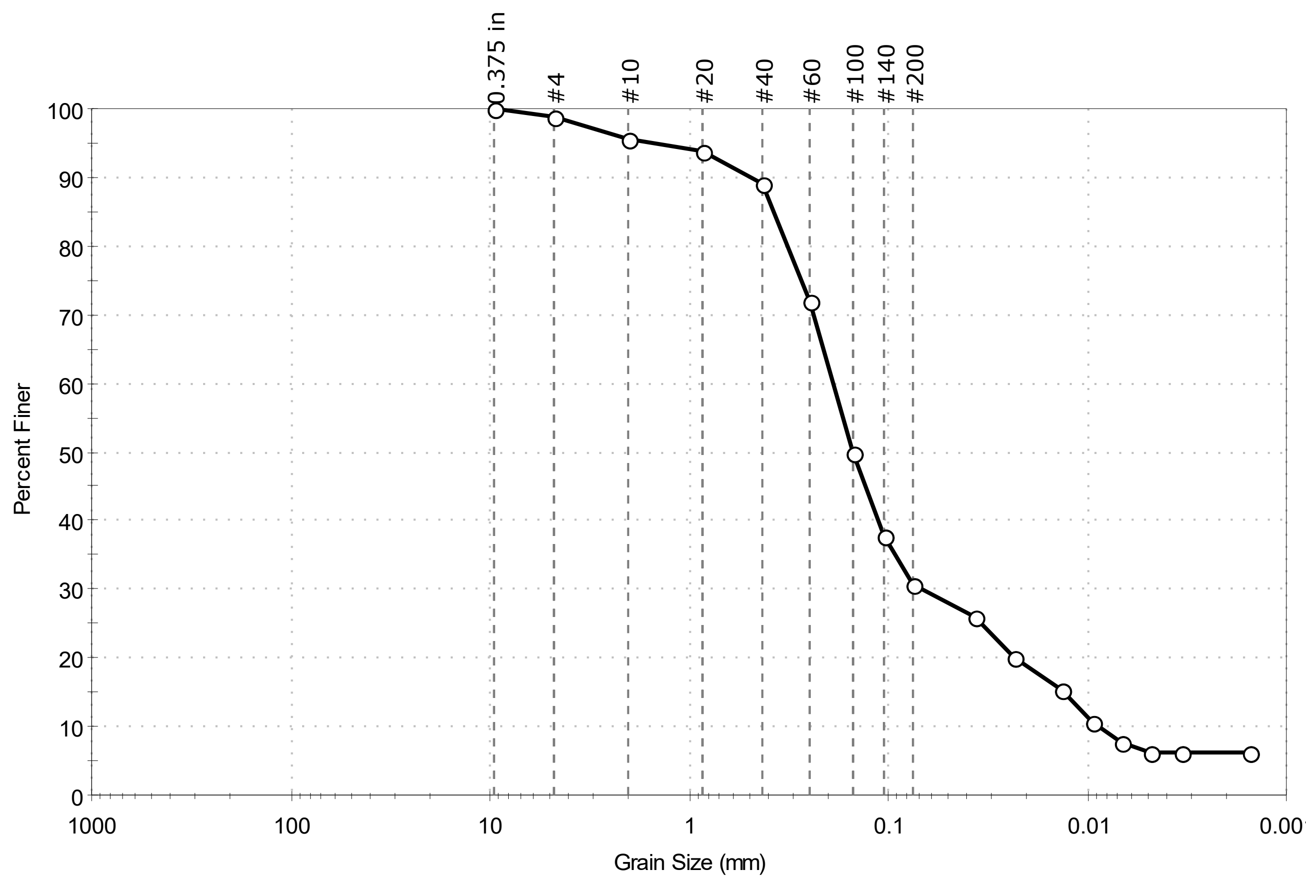
Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Brdg Replacement				
Location:	Dover-Foxcroft, ME			Project No:	GTX-318514
Boring ID:	GRAB	Sample Type:	tube	Tested By:	ckg
Sample ID:	G1	Test Date:	01/30/24	Checked By:	ank
Depth :	0'	Test Id:	756481		

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

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.2	68.0	30.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	96		
#20	0.85	94		
#40	0.42	89		
#60	0.25	72		
#100	0.15	50		
#140	0.11	38		
#200	0.075	31		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0366	26		
---	0.0231	20		
---	0.0134	15		
---	0.0095	11		
---	0.0068	8		
---	0.0048	6		
---	0.0034	6		
---	0.0015	6		

Coefficients

D_{85} =0.3739 mm
D_{30} =0.0664 mm
D_{60} =0.1899 mm
D_{15} =0.0131 mm
D_{50} =0.1507 mm
D_{10} =0.0088 mm
C_u =21.580
C_c =2.638

Classification

ASTM N/A

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

Sample/Test Description

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

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GeoTesting EXPRESS

Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Bridge Replace				
Location:	Dover-Foxcroft, ME			Project No:	GTX-314652
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	01/19/22	Checked By:	bfs
Depth :	---	Test Id:	652035		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-DFPR-101	2D(0"- 8")	5-6.7 ft	Moist, olive brown silty sand	14.5
BB-DFPR-101	3D(9"18")	10-12 ft	Moist, olive brown silty sand	24.8
BB-DFPR-103	2D(0"- 7")	5-7 ft	Moist, olive brown silty gravel with sand	5.6
BB-DFPR-103	4D(0"- 5")	15-17 ft	Moist, dark olive brown silty sand with gravel	11.7

Notes: Temperature of Drying : 110º Celsius

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Client:	GEI Consultants, Inc.		
Project:	WIN 23120 Dover Bridge Replace		
Location:	Dover-Foxcroft, ME	Project No:	GTX-314652
Boring ID: ---	Sample Type: ---	Tested By:	tlm
Sample ID: ---	Test Date: 11/30/21	Checked By:	smd
Depth : ---	Test Id: 643005		

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-DFPR-101	R5	31.32-31.69 ft	177	9615	3	Yes	---
BB-DFPR-102	R4	40.28-40.65 ft	175	7128	3	Yes	---
BB-DFPR-103	R7	41.51-41.88 ft	174	3818	2	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens 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.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure (See attached photographs)

GeoTesting EXPRESS

Client:	GEI Consultans, Inc.	Test Date:	11/24/2021
Project Name:	WIN 23120 Dover Bridge Replace	Tested By:	kdp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	314652		
Boring ID:	BB-DFPR-101		
Sample ID:	R5		
Depth:	31.32-31.69 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:	4.45	4.45	4.45		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	637.28				
Bulk Density, lb/ft^3	177			**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2			**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020
Diameter 2, in (rotated 90°)	-0.00060	-0.00060	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00020	0.00020	0.00030	0.00030	0.00040	0.00040	0.00040

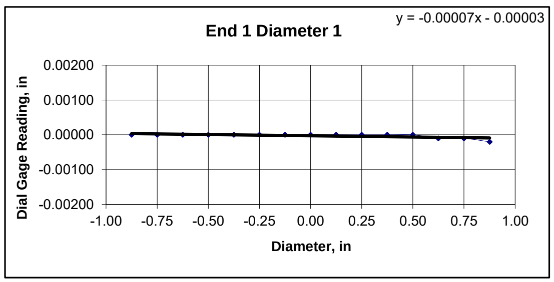
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00100

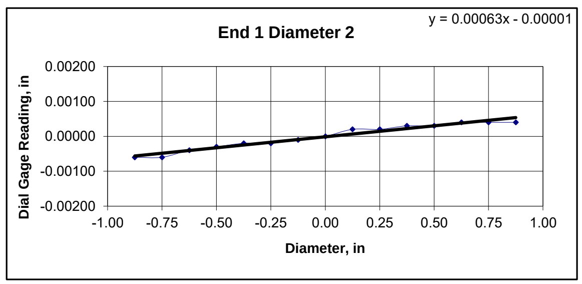
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	0.00060	0.00050	0.00040	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050

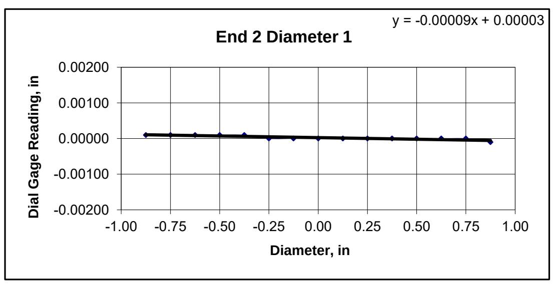
Difference between max and min readings, in:
0° = 0.0002 90° = 0.0011

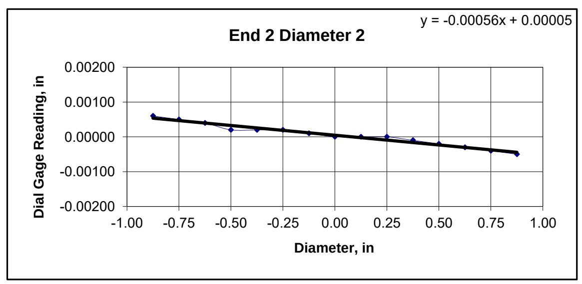
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00055

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00007
Angle of Best Fit Line: 0.00409

End 2:
Slope of Best Fit Line 0.00009
Angle of Best Fit Line: 0.00524

Maximum Angular Difference: 0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00063
Angle of Best Fit Line: 0.03601

End 2:
Slope of Best Fit Line 0.00056
Angle of Best Fit Line: 0.03209

Maximum Angular Difference: 0.00393

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	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00100	1.980	0.00051	0.029	YES
END 2					
Diameter 1, in	0.00020	1.980	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00110	1.980	0.00056	0.032	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Bridge Replace
Project Location:	Dover-Foxcroft, ME
GTX #:	314652
Test Date:	11/30/2021
Tested By:	kdp
Checked By:	smd
Boring ID:	BB-DFPR-101
Sample ID:	R5
Depth, ft:	31.32-31.69

After cutting and grinding

After break

Client:	GEI Consultants, Inc.	Test Date:	11/24/2021
Project Name:	WIN 23120 Dover Bridge Replace	Tested By:	kdp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	314652		
Boring ID:	BB-DFPR-102		
Sample ID:	R4		
Depth:	40.28-40.65 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:	4.49	4.49	4.49
Specimen Diameter, in:	1.97	1.97	1.97
Specimen Mass, g:	630.91		
Bulk Density, lb/ft^3	175	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

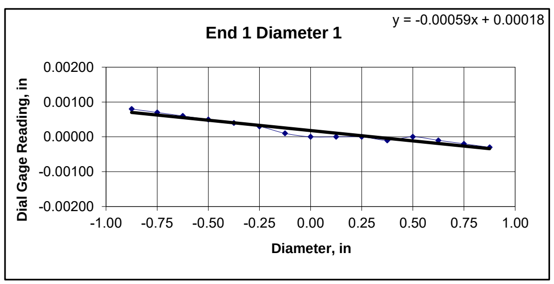
END FLATNESS AND PARALLELISM (Procedure FP1)

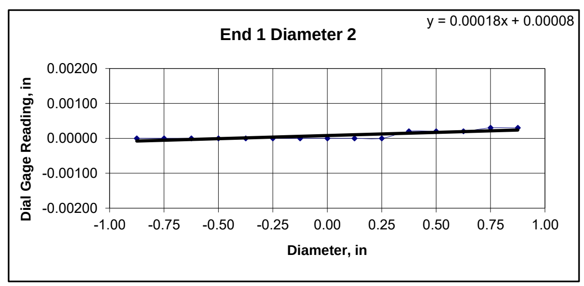
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00080	0.00070	0.00060	0.00050	0.00040	0.00030	0.00010	0.00000	0.00000	0.00000	-0.00010	0.00000	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00020	0.00020	0.00020	0.00030	0.00030

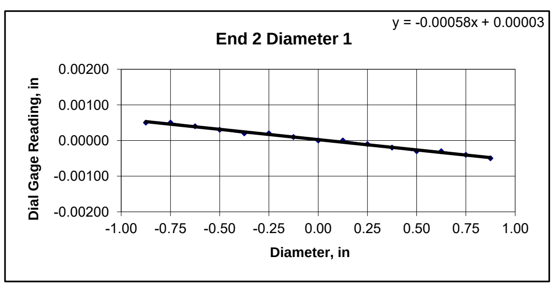
Difference between max and min readings, in:
0° = 0.00110 90° = 0.00030

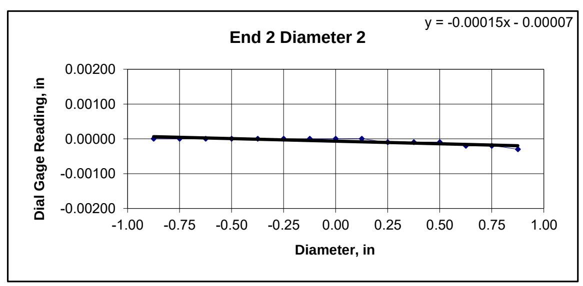
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00050	0.00050	0.00040	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0.001 90° = 0.0003
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00055
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00059
Angle of Best Fit Line: 0.03405

End 2:
Slope of Best Fit Line 0.00058
Angle of Best Fit Line: 0.03307

Maximum Angular Difference: 0.00098

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00018
Angle of Best Fit Line: 0.01031

End 2:
Slope of Best Fit Line 0.00015
Angle of Best Fit Line: 0.00851

Maximum Angular Difference: 0.00180

Parallelism Tolerance Met? **YES**
Spherically Seated

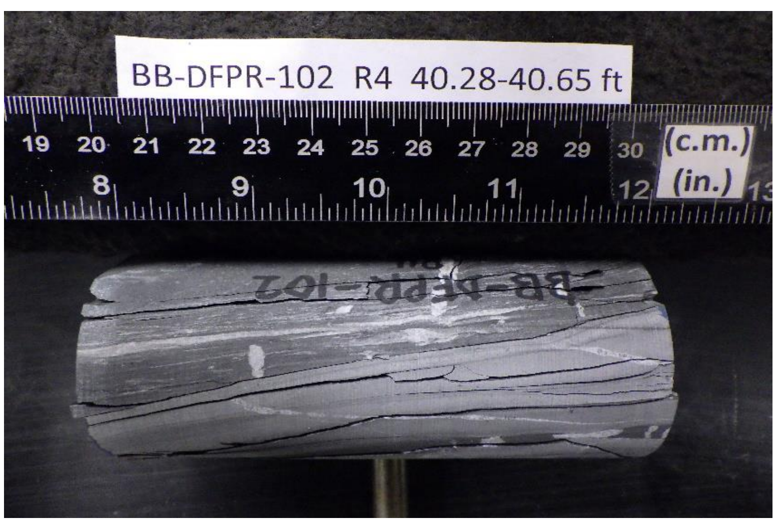
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00110	1.970	0.00056	0.032	YES
Diameter 2, in (rotated 90°)	0.00030	1.970	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00100	1.970	0.00051	0.029	YES
Diameter 2, in (rotated 90°)	0.00030	1.970	0.00015	0.009	YES

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

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Bridge Replace
Project Location:	Dover-Foxcroft, ME
GTX #:	314652
Test Date:	11/30/2021
Tested By:	kdp
Checked By:	smd
Boring ID:	BB-DFPR-101
Sample ID:	R4
Depth, ft:	40.28-40.65

After cutting and grinding



After break

Client:	GEI Consultants, Inc.	Test Date:	11/24/2021
Project Name:	WIN 23120 Dover Bridge Replace	Tested By:	kdp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	314652		
Boring ID:	BB-DFPR-103		
Sample ID:	R7		
Depth:	41.51-41.88 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:	4.51	4.51	4.51		
Specimen Diameter, in:	1.98	1.98	1.98		
Specimen Mass, g:	635.5				
Bulk Density, lb/ft^3	174			**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3			**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? YES

END FLATNESS AND PARALLELISM (Procedure FP1)

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

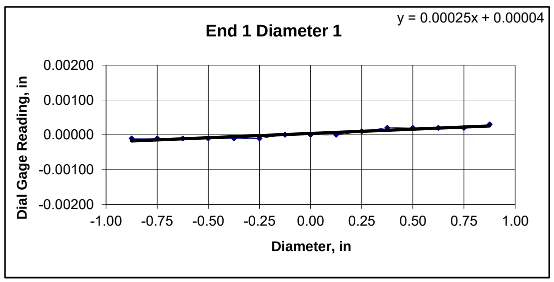
Difference between max and min readings, in:
0° = 0.00040 90° = 0.00030

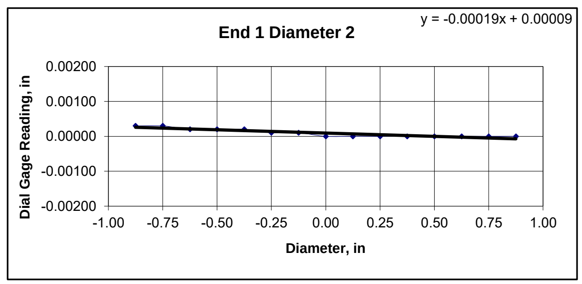
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00030	-0.00020	-0.00020	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00020	0.00020	0.00020
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00010	-0.00010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

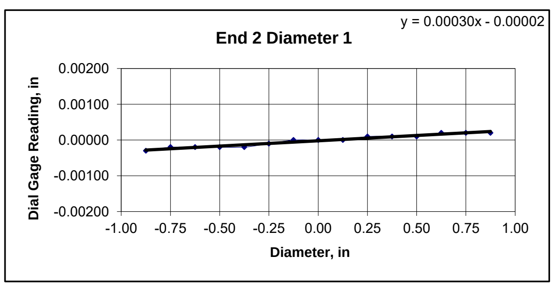
Difference between max and min readings, in:
0° = 0.0005 90° = 0.0003

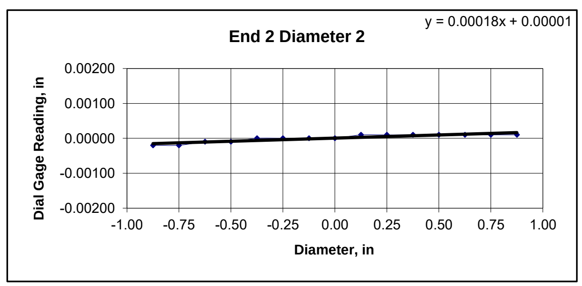
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00025

Flatness Tolerance Met? YES









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00025
Angle of Best Fit Line:	0.01408
End 2:	
Slope of Best Fit Line	0.00030
Angle of Best Fit Line:	0.01703
Maximum Angular Difference:	0.00295

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00019
Angle of Best Fit Line:	0.01080
End 2:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01031
Maximum Angular Difference:	0.00049

Parallelism Tolerance Met? YES
Spherically Seated

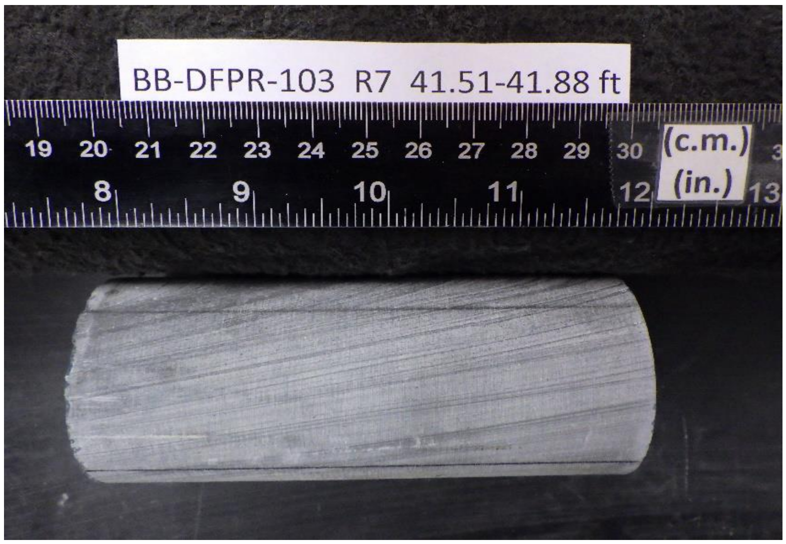
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00040	1.980	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00050	1.980	0.00025	0.014	YES
Diameter 2, in (rotated 90°)	0.00030	1.980	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? YES

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Bridge Replace
Project Location:	Dover-Foxcroft, ME
GTX #:	314652
Test Date:	11/30/2021
Tested By:	kdp
Checked By:	smd
Boring ID:	BB-DFPR-103
Sample ID:	R7
Depth, ft:	41.51-41.88



After cutting and grinding

After break

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-DFPR-201A	R4	22.40-22.77 ft	173	15195	3	No	1,*
BB-DFPR-203A	R1	33.31-33.68 ft	173	9671	3	No	1,*

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens 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.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure (See attached photographs)

1: Best effort end preparation. See Tolerance report for details.
2: The as-received core did not meet the ASTM side straightness tolerance due to irregularities in the sample as cored.
3: Specimen L/D < 2.
4: The as-received core did not meet the ASTM minimum diameter tolerance of 1.875 inches.
5: Specimen diameter is less than 10 times maximum particle size.
6: Specimen diameter is less than 6 times maximum particle size.

*Because the indicated tested specimens did not meet the ASTM D4543 standard tolerances, the results reported here may differ from those for a test specimen within tolerances.

GeoTesting EXPRESS

Client:	GEI Consultants, Inc.	Test Date:	2/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-201A		
Sample ID:	R4		
Depth:	22.40-22.77 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:	4.24	4.24	4.24
Specimen Diameter, in:	1.97	1.97	1.97
Specimen Mass, g:	588.94		
Bulk Density, lb/ft^3	173	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	0.00030	0.00030	0.00030	0.00030	0.00020	0.00020	0.00010	0.00000	0.00000	-0.00020	-0.00030	-0.00050	-0.00080	-0.00100	-0.00120

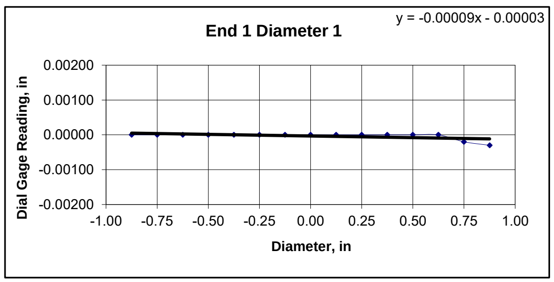
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00150

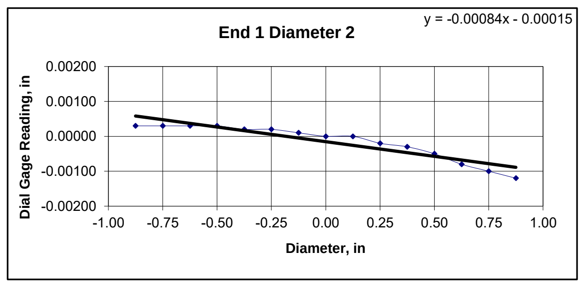
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90°)	-0.00100	-0.00090	-0.00070	-0.00050	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00020	0.00020

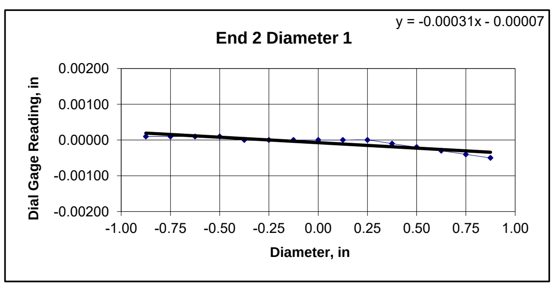
Difference between max and min readings, in:
0° = 0.0006 90° = 0.0012

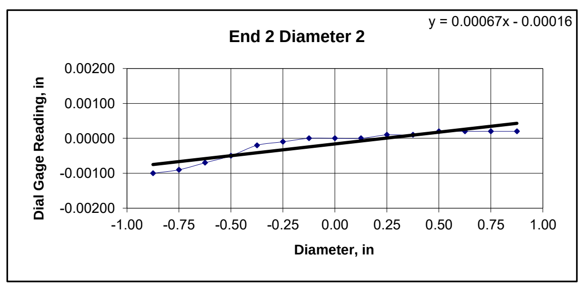
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00075

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00009
Angle of Best Fit Line: 0.00540

End 2:
Slope of Best Fit Line 0.00031
Angle of Best Fit Line: 0.01752

Maximum Angular Difference: 0.01211

Parallelism Tolerance Met? **NO**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00084
Angle of Best Fit Line: 0.04813

End 2:
Slope of Best Fit Line 0.00067
Angle of Best Fit Line: 0.03863

Maximum Angular Difference: 0.00949

Parallelism Tolerance Met? **NO**
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.970	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00150	1.970	0.00076	0.044	YES
END 2					
Diameter 1, in	0.00060	1.970	0.00030	0.017	YES
Diameter 2, in (rotated 90°)	0.00120	1.970	0.00061	0.035	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	GEI Consultants, Inc.	Test Date:	2/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-201A	Reliable dial gauge measurements could not be performed on this rock type. Tolerance measurements were performed using a machinist straightedge and feeler gauges to ASTM specifications.	
Sample ID:	R4		
Depth (ft):	22.40-22.77		
Visual Description:	See photographs		

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

END FLATNESS

END 1

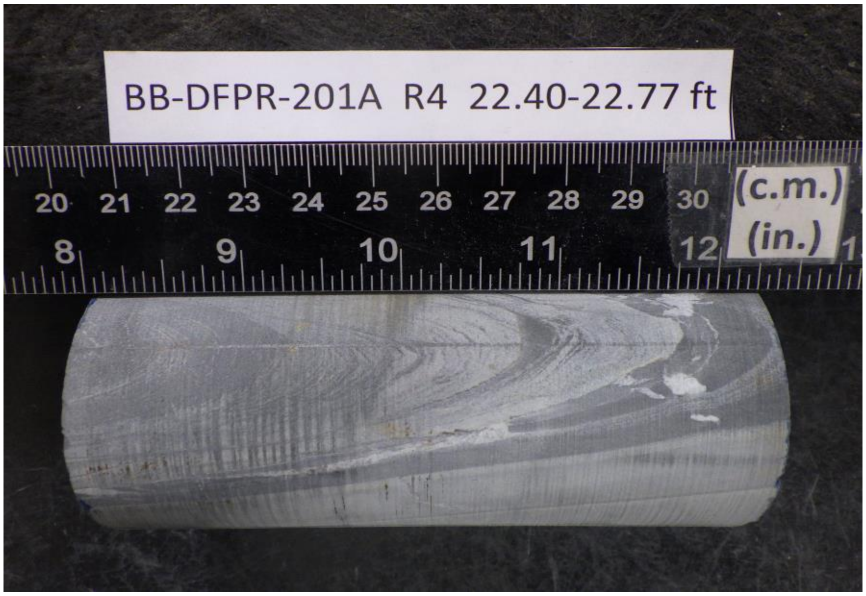
Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90^o)	Is the maximum gap $\leq \pm 0.001$ in.?	YES

END 2

Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	NO
Diameter 2 (rotated 90^o)	Is the maximum gap $\leq \pm 0.001$ in.?	YES

End Flatness Tolerance Met? NO

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Brdg Replacement
Project Location:	Dover-Foxcroft, ME
GTX #:	318514
Test Date:	2/12/2024
Tested By:	te
Checked By:	smd
Boring ID:	BB-DFPR-201A
Sample ID:	R4
Depth, ft:	22.40-22.77



After cutting and grinding

After break

GeoTesting EXPRESS

Client:	GEI Consultants, Inc.	Test Date:	2/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-203A		
Sample ID:	R1		
Depth:	33.31-33.68 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:	1.97	1.97	1.97
Specimen Mass, g:	599.04		
Bulk Density, lb/ft^3	173	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

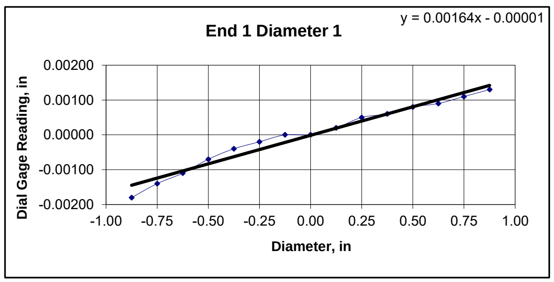
END FLATNESS AND PARALLELISM (Procedure FP1)

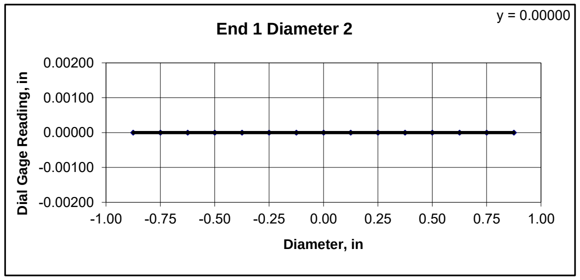
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00180	-0.00140	-0.00110	-0.00070	-0.00040	-0.00020	0.00000	0.00000	0.00020	0.00050	0.00060	0.00080	0.00090	0.00110	0.00130
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

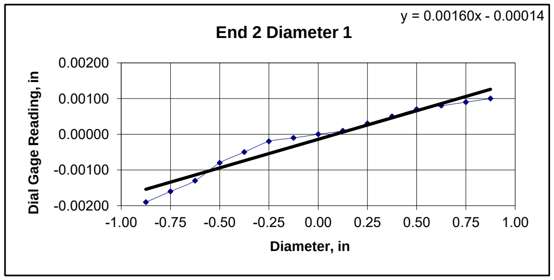
Difference between max and min readings, in:
0° = 0.00310 90° = 0.00000

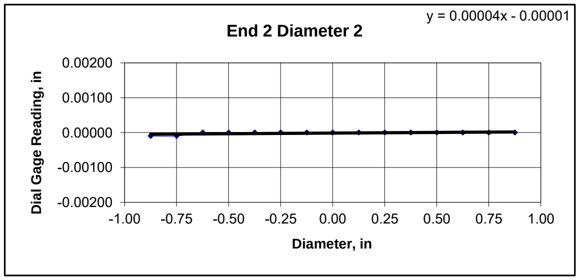
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00190	-0.00160	-0.00130	-0.00080	-0.00050	-0.00020	-0.00010	0.00000	0.00010	0.00030	0.00050	0.00070	0.00080	0.00090	0.00100
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000

Difference between max and min readings, in:
0° = 0.0029 90° = 0.0001
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00155
Flatness Tolerance Met? **NO**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00164
Angle of Best Fit Line:	0.09380
End 2:	
Slope of Best Fit Line	0.00160
Angle of Best Fit Line:	0.09167
Maximum Angular Difference:	0.00213

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

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

Parallelism Tolerance Met? **YES**
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00310	1.970	0.00157	0.090	YES
Diameter 2, in (rotated 90°)	0.00000	1.970	0.00000	0.000	YES
END 2					
Diameter 1, in	0.00290	1.970	0.00147	0.084	YES
Diameter 2, in (rotated 90°)	0.00010	1.970	0.00005	0.003	YES

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

Client:	GEI Consultants, Inc.	Test Date:	2/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-203A	Reliable dial gauge measurements could not be performed on this rock type. Tolerance measurements were performed using a machinist straightedge and feeler gauges to ASTM specifications.	
Sample ID:	R1		
Depth (ft):	33.31-33.68		
Visual Description:	See photographs		

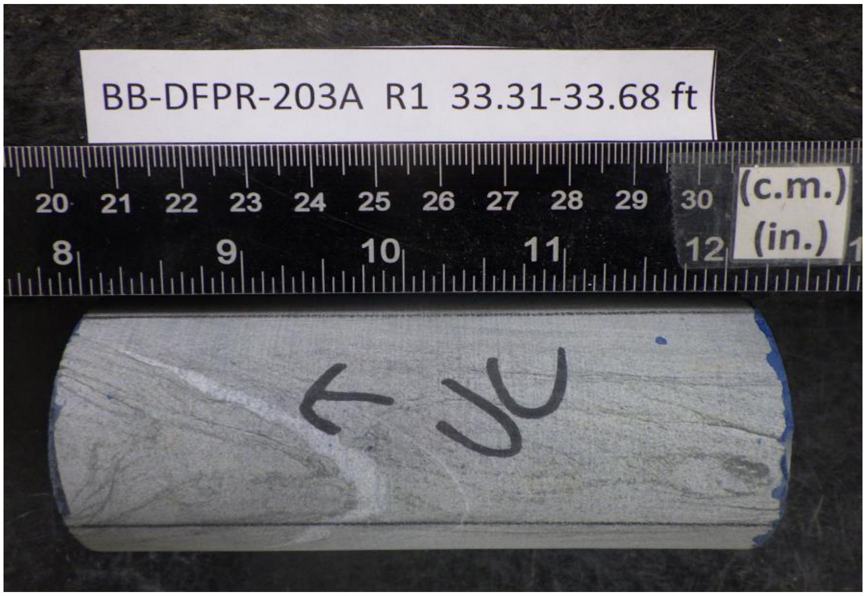
BEST EFFORT END FLATNESS TOLERANCES OF ROCK CORE SPECIMENS TO ASTM D4543

END FLATNESS

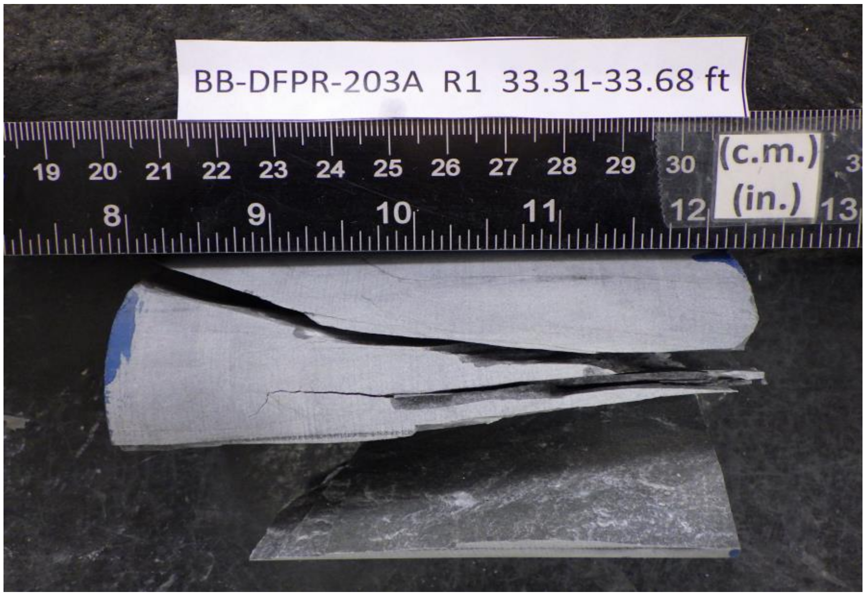
END 1		
Diameter 1	Is the maximum gap $\leq \pm$0.001 in.?	YES
Diameter 2 (rotated 90^{o})	Is the maximum gap $\leq \pm$0.001 in.?	YES
END 2		
Diameter 1	Is the maximum gap $\leq \pm$0.001 in.?	YES
Diameter 2 (rotated 90^{o})	Is the maximum gap $\leq \pm$0.001 in.?	YES

End Flatness Tolerance Met? YES

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Brdg Replacement
Project Location:	Dover-Foxcroft, ME
GTX #:	318514
Test Date:	2/12/2024
Tested By:	te
Checked By:	smd
Boring ID:	BB-DFPR-203A
Sample ID:	R1
Depth, ft:	33.31-33.68



After cutting and grinding



After break

Client:	GEI Consultants, Inc.				
Project:	WIN 23120 Dover Brdg Replacement				
Location:	Dover-Foxcroft, ME			Project No:	GTX-318514
Boring ID:	BB-DFPR-202	Sample Type:	Cylinder	Tested By:	te
Sample ID:	R5	Test Date:	03/12/24	Checked By:	smd
Depth :	43.7-44.8	Test Id:	760888		
Test Comment:	---				
Visual Description:	See photograph(s)				
Sample Comment:	---				

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-DFPR-202	R5	44.25-44.63 ft	175	6632	3	No	1,*

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens 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.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure (See attached photographs)

1: Best effort end preparation. See Tolerance report for details.
2: The as-received core did not meet the ASTM side straightness tolerance due to irregularities in the sample as cored.
3: Specimen L/D < 2.
4: The as-received core did not meet the ASTM minimum diameter tolerance of 1.875 inches.
5: Specimen diameter is less than 10 times maximum particle size.
6: Specimen diameter is less than 6 times maximum particle size.

*Because the indicated tested specimens did not meet the ASTM D4543 standard tolerances, the results reported here may differ from those for a test specimen within tolerances.

GeoTesting EXPRESS

Client:	GEI Consultants, Inc.	Test Date:	3/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-202		
Sample ID:	R5		
Depth:	44.25-44.63 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:	4.46	4.46	4.46
Specimen Diameter, in:	1.98	1.98	1.98
Specimen Mass, g:	633.2		
Bulk Density, lb/ft^3	175	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

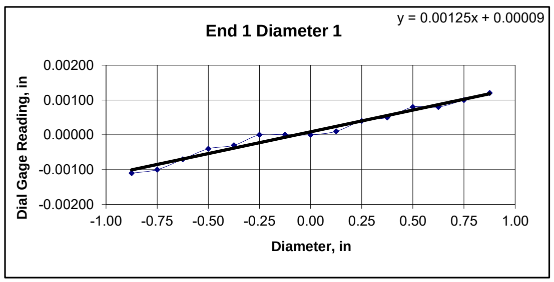
END FLATNESS AND PARALLELISM (Procedure FP1)

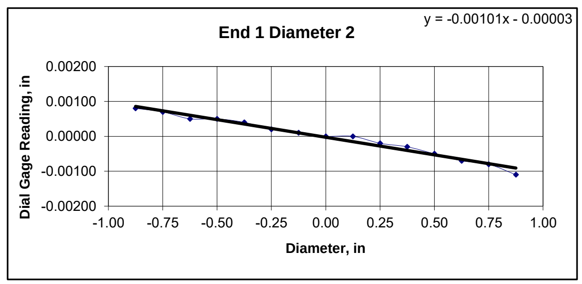
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00110	-0.00100	-0.00070	-0.00040	-0.00030	0.00000	0.00000	0.00000	0.00010	0.00040	0.00050	0.00080	0.00080	0.00100	0.00120
Diameter 2, in (rotated 90°)	0.00080	0.00070	0.00050	0.00050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

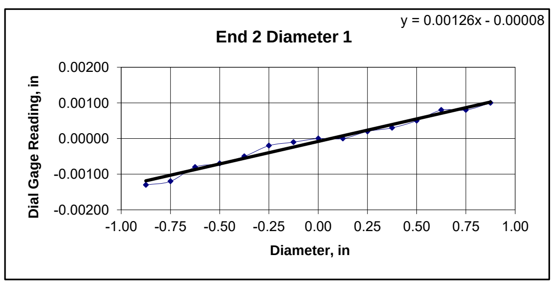
Difference between max and min readings, in:
0° = 0.00230 90° = 0.00190

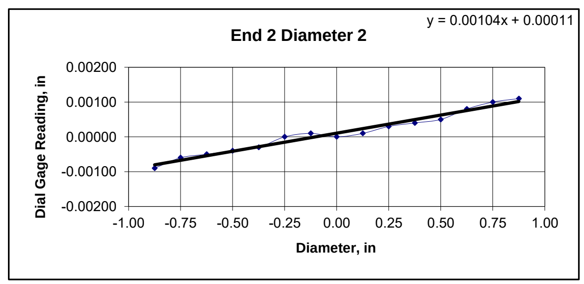
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00130	-0.00120	-0.00080	-0.00070	-0.00050	-0.00020	-0.00010	0.00000	0.00000	0.00020	0.00030	0.00050	0.00080	0.00080	0.00100
Diameter 2, in (rotated 90°)	-0.00090	-0.00060	-0.00050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

Difference between max and min readings, in:
0° = 0.0023 90° = 0.002
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00115
Flatness Tolerance Met? **NO**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00125
Angle of Best Fit Line: 0.07154

End 2:
Slope of Best Fit Line 0.00126
Angle of Best Fit Line: 0.07236

Maximum Angular Difference: 0.00082

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00101
Angle of Best Fit Line: 0.05779

End 2:
Slope of Best Fit Line 0.00104
Angle of Best Fit Line: 0.05959

Maximum Angular Difference: 0.00180

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	0.00230	1.980	0.00116	0.067	YES
Diameter 2, in (rotated 90°)	0.00190	1.980	0.00096	0.055	YES
END 2					
Diameter 1, in	0.00230	1.980	0.00116	0.067	YES
Diameter 2, in (rotated 90°)	0.00200	1.980	0.00101	0.058	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	GEI Consultants, Inc.	Test Date:	3/12/2024
Project Name:	WIN 23120 Dover Brdg Replacement	Tested By:	gp
Project Location:	Dover-Foxcroft, ME	Checked By:	smd
GTX #:	318514		
Boring ID:	BB-DFPR-202	Reliable dial gauge measurements could not be performed on this rock type. Tolerance measurements were performed using a machinist straightedge and feeler gauges to ASTM specifications.	
Sample ID:	R5		
Depth (ft):	44.25-44.63		
Visual Description:	See photographs		

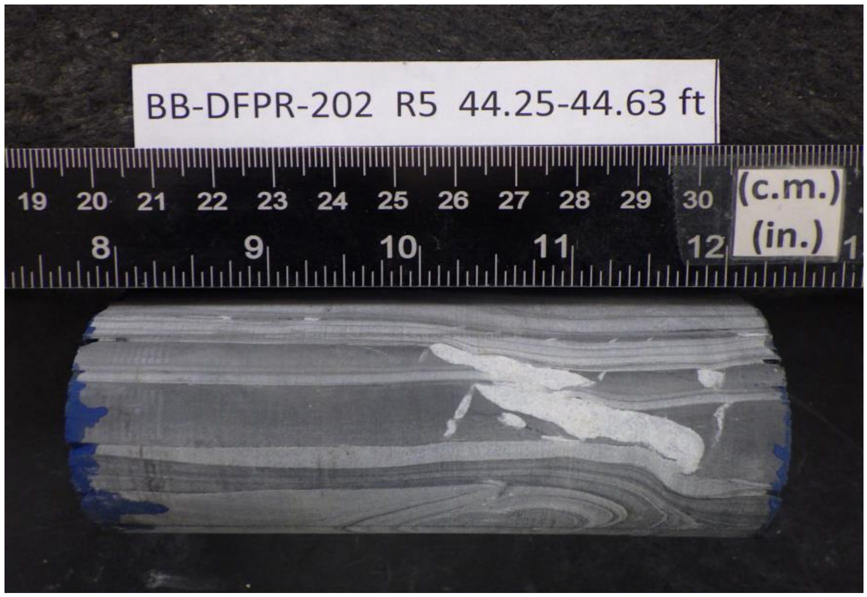
BEST EFFORT END FLATNESS TOLERANCES OF ROCK CORE SPECIMENS TO ASTM D4543

END FLATNESS

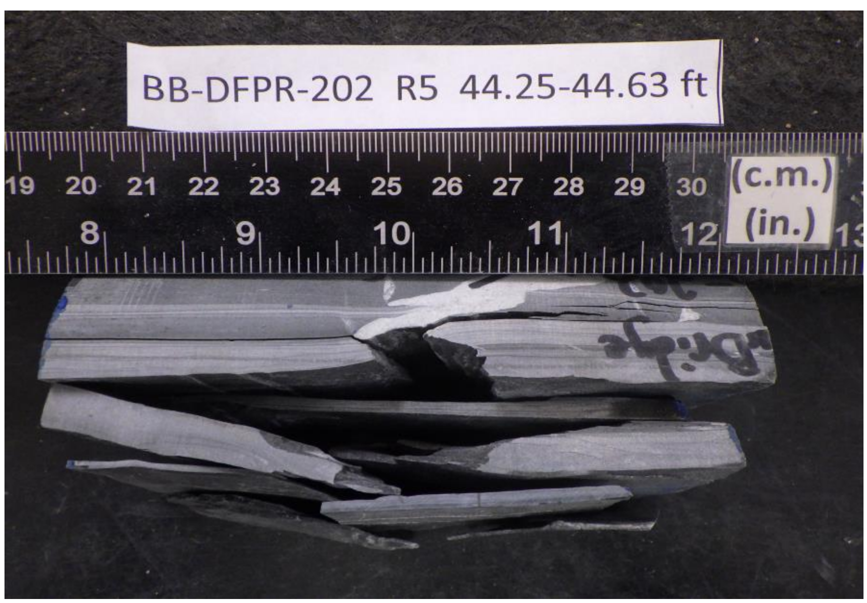
END 1		
Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap $\leq \pm 0.001$ in.?	YES
END 2		
Diameter 1	Is the maximum gap $\leq \pm 0.001$ in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap $\leq \pm 0.001$ in.?	YES

End Flatness Tolerance Met? YES

Client:	GEI Consultants, Inc.
Project Name:	WIN 23120 Dover Brdg Replacement
Project Location:	Dover-Foxcroft, ME
GTX #:	318514
Test Date:	3/12/2024
Tested By:	te
Checked By:	smd
Boring ID:	BB-DFPR-202
Sample ID:	R5
Depth, ft:	44.25-44.63



After cutting and grinding



After break

Appendix D Geotechnical Calculations

D.1 Recommended Soil Properties

D.2 Earth Pressure Coefficients

D.3 Site Class Evaluation

D.4 Frost Depth Calculation

D.5 Bearing Resistance - Spread Footings on Bedrock

D.6 Bearing Resistance - Spread Footings on Fill

D.7 Rock Socketed Piles – Abutment 2

D.1 Recommended Soil Properties

Soil Properties Selection

Purpose:
The purpose of this evaluation is to select representative soil properties for the design of the proposed bridge replacement project. The soil properties will be used in our engineering analyses.

Approach:
We selected values for the engineering properties of soils. Values were selected for the general soil layers observed in the borings.

Unit Weight
We selected a saturated (total) unit weight in pounds per cubic foot (pcf). The buoyant unit weight can then be determined by subtracting the unit weight of fresh water (approximately 62.4 pcf).

Angle of Internal Friction
We selected an angle of internal friction (ϕ) in degrees. We used Mohr-Coulomb drained properties for each soil.

Subsurface Investigation and SPT Correlations for Observed Soil Layers:
We reviewed Standard Penetration Test (SPT) N-Values collected during our subsurface investigation. We estimated angles of internal friction for the soils below based on N-Values corrected for overburden and hammer efficiency ($N1_{60}$). SPTs were performed with an automatic hammer. The automatic hammer for the -100 series borings had an efficiency of 92.4 percent, and the automatic hammer for the -200 series borings had an efficiency of 76.5 percent.

A summary of corrected N-Values based on general soil type is shown below. We did not include refusals due to cobbles or boulders, and we limited the corrected N_{60} and $N1_{60}$ values to a maximum of 100 blows per foot.

Results:
We selected the following soil properties for each layer/soil type based on the references provided in the following pages and our engineering judgment. We did not include N-values for BB-DFPR-201 and -202 for consideration of the fill properties because these borings were performed within the existing abutments, which were likely filled with crushed stone or rockfill. A friction angle lower than suggested by the calculated $N1_{60}$ value was selected for the existing granular fill due to the variability of the layer encountered in the borings.

Soil Type	Average $N1_{60}$ (Blows/ft)	Bulk Unit Weight (γ) (pcf)	Cohesion (c) (lb/ft^2)	Friction Angle (ϕ) (deg)
Fill	36	125	0	32
River Sediment	9	115	0	30
Glacial Till	46	135	0	38

References:

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Terzaghi, K., Peck, R.B., 1968. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley & Sons, New York.
3. Caltrans Geotechnical Manual, March 2014.
4. NAVFAC Design Manual 7.01 Soil Mechanics, Naval Facilities Engineering Command, September 1986.

AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
Table 10.4.6.2.4-1 recommends using the following correlation to select friction angles of granular soils:

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Soil Mechanics in Engineering Practice

Karl Terzaghi and Ralph Peck compiled various parameters of soils into the tables below:

Table 6.3
Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

Description	Porosity, n (%)	Void ratio, e	Water content, w (%)	Unit weight grams/cm³ γ_d	Unit weight grams/cm³ γ	Unit weight lb/ft³ γ_d	Unit weight lb/ft³ γ
1. Uniform sand, loose	46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	34	0.51	19	1.75	2.09	109	130
3. Mixed-grained sand, loose	40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	30	0.43	16	1.86	2.16	116	135
5. Glacial till, very mixed-grained	20	0.25	9	2.12	2.32	132	145
6. Soft glacial clay	55	1.2	45	–	1.77	–	110
7. Stiff glacial clay	37	0.6	22	–	2.07	–	129
8. Soft slightly organic clay	66	1.9	70	–	1.58	–	98
9. Soft very organic clay	75	3.0	110	–	1.43	–	89
10. Soft bentonite	84	5.2	194	–	1.27	–	80

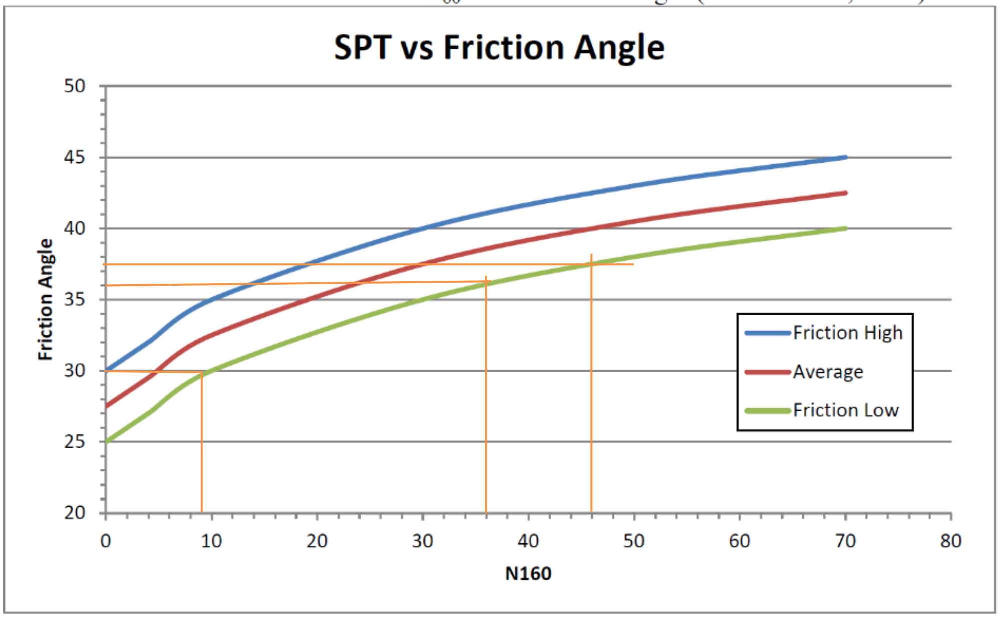
w = water content when saturated, in per cent of dry weight.
γ_d = unit weight in dry state.
γ = unit weight in saturated state.

Table 17.1
Representative Values of ϕ for Sands and Silts

Material	Degrees *Loose*	Degrees *Dense*
Sand, round grains, uniform	27.5	34
Sand, angular grains, well graded	33	45
Sandy gravels	35	50
Silty sand	27–33	30–34
Inorganic silt	27–30	30–35

Caltrans Geotechnical Manual (March 2014)

Chart 1: Correlation of SPT $N1_{60}$ with Friction Angle (after Bowles, 1977)



Choose the friction angle (expressed to the nearest degree) based upon the soil type, particle size(s), and rounding or angularity. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant (about 30+ %) silt-sized material will fall in the lower portion of the range. Coarser materials with less than 5% fines will fall in the upper portion of the range. The extreme range of phi angles for any $N1_{60}$ is five degrees, so the adjustment factors for particle size and roundness should be only a degree or two. The following bullets provide help in determining which value to select for a given $N1_{60}$ and soil type:

- Use the maximum value for GW
- Use the average for GM and SP
- Use the minimum for SC
- Use the minimum + 0.5 for ML
- Use the average +1 for SW
- Use the average -1 for GC
- Use the Maximum -1 for GP

Values may also be increased with increasing grain size and/or particle angularity, and decreased with decreasing grain size and/or increasing roundness. For example, an SP with $N1_{60}$ = 30 could be assigned phi angles of 37, 38 or 39 degrees for fine, medium and coarse grain sizes respectively.

NAVFAC Design Manual 7.01 Soil Mechanics

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	–	–	–	1.0	0.92	–	0.35	47.6	26	–	–	–	–	–	–	–
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	–	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	–	–	–	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	–	0.40	52	29	80	–	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	–	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	–	–	–	–	1.2	–	0.40	55	29	76	–	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	–	0.14	46	12	89	–	$146^{(3)}$	90	$155^{(3)}$	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	–	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	–	–	1.0	–	0.20	50	17	84	–	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	–	0.13	41	11	100	140	$148^{(4)}$	125	$156^{(4)}$	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	–	2.4	–	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	–	–	12	–	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	–	–	–	–	3.0	–	0.55	75	35	40	–	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	–	–	–	–	4.4	–	0.70	81	41	30		100	81	125	18	62

N Value (blows/ft or 305 mm)	Relative Density	Approximate $\bar{\phi}_{tc}$ (degrees) (a)	(b)
0 to 4	very loose	< 28	< 30
4 to 10	loose	28 to 30	30 to 35
10 to 30	medium	30 to 36	35 to 40
30 to 50	dense	36 to 41	40 to 45
> 50	very dense	> 41	> 45

a - Source: Peck, Hanson, and Thornburn (12), p. 310.
b - Source: Meyerhof (13), p. 17.

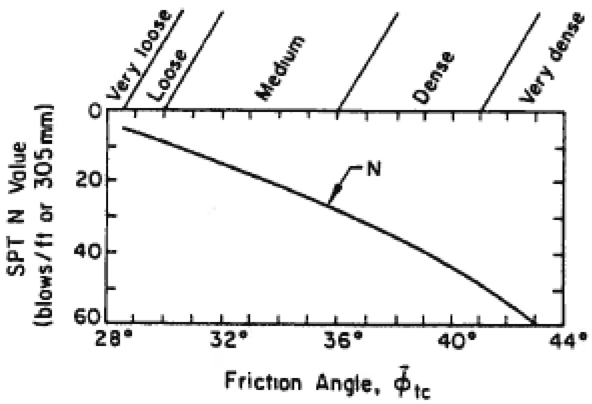


Figure 4-12. N versus $\bar{\phi}_{tc}$

Source: Peck, Hanson, and Thornburn (12), p. 310.

GEI Consultants	**Client:** Thornton Tomasetti - WIN 23120 **Project:** Dover Bridge #5118, Essex Street over Piscataquis River **Project No.:** 2305541 **Subject:** Corrected Blow Counts	**Prepared By:** M. Johnescu **Date:** 1/26/2024 **Checked By:** G. Williams **Date:** 6/17/2025

Summary of Corrected Blow Counts by Layer

Fill

Boring	No. Values	N_{60} Avg.	N_{60} Max.	N_{60} Min.	$N1_{60}$ Avg.	$N1_{60}$ Max.	$N1_{60}$ Min.
BB-DFPR-101	4	38	100	8	45	100	8
BB-DFPR-103	4	23	54	5	31	72	5
BB-DFPR-202	1	54	54	54	75	75	75
BB-DFPR-203	2	28	47	9	30	49	10

Average N_{60}: 30 **Average $N1_{60}$: 36**
Median N_{60}: 17 **Median $N1_{60}$: 25**

*Averages/median not including -202 in rockfill.

River Sediment

Boring	No. Values	N_{60} Avg.	N_{60} Max.	N_{60} Min.	$N1_{60}$ Avg.	$N1_{60}$ Max.	$N1_{60}$ Min.
BB-DFPR-102	1	5	5	5	9	9	9

Average N_{60}: 5 **Average $N1_{60}$: 9**

Glacial Till

Boring	No. Values	N_{60} Avg.	N_{60} Max.	N_{60} Min.	$N1_{60}$ Avg.	$N1_{60}$ Max.	$N1_{60}$ Min.
BB-DFPR-201	1	34	34	34	35	35	35
BB-DFPR-202	1	22	22	22	20	20	20
BB-DFPR-203	2	66	100	32	65	99	31

Average N_{60}: 47 **Average $N1_{60}$: 46**
Median N_{60}: 33

GEI Consultants	**Client:** Thornton Tomasetti - WIN 23120	**Prepared By:** M. Johnescu
	Project: Dover Bridge #5118, Essex Street over Piscataquis River	**Date:** 1/26/2024
	Project No.: 2302742	**Checked By:** G. Williams
	Subject: Corrected Blow Counts	**Date:** 6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where:	N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
			ER = hammer efficiency expressed as percent of theoretical free fall energy
			N = Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where:	$N1_{60}$ = SPT blow count corrected for overburden and hammer efficiency (blows/ft)
			$C_N = 0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
			σ'_v = vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	343.2	ft
Groundwater El.:	331.2	ft
Depth to Groundwater:	12.0	ft
Average Total Unit Weight of Soil:	120	pcf

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	92.4	1.54

Boring:		***BB-DFPR-101***		**Corrected Blow Counts**				**Overburden Correction**					**Hammer Efficiency Correction**		
Depth (ft)	**El. (ft)**	**Layer Name**	**N**	**N_{60}**	**$N1_{60}$**	**Avg. N_{60}**	**Avg. $N1_{60}$**	**σ_v (psf)**	**u (psf)**	**σ'_v (psf)**	**σ'_v (ksf)**	**C_N**	**Hammer Type**	**ER (%)**	**C_E**
2.0	341.2	FILL	22	34	58	38	45	240	0	240	0.240	1.71	Automatic	92.4	1.54
6.0	337.2	FILL	90	100	100			720	0	720	0.720	1.34	Automatic	92.4	1.54
11.0	332.2	FILL	7	11	12			1,320	0	1,320	1.320	1.14	Automatic	92.4	1.54
14.0	329.2	FILL	5	8	8			1,680	125	1,555	1.555	1.09	Automatic	92.4	1.54

Notes:

1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

GEI Consultants	**Client:** Thornton Tomasetti - WIN 23120 **Project:** Dover Bridge #5118, Essex Street over Piscataquis River **Project No.:** 2302742 **Subject:** Corrected Blow Counts	**Prepared By:** M. Johnescu **Date:** 1/26/2024 **Checked By:** G. Williams **Date:** 6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where:	N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
			ER = hammer efficiency expressed as percent of theoretical free fall energy
			N = Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where:	$N1_{60}$ = SPT blow count corrected for overburden and hammer efficiency (blows/ft)
			$C_N = 0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
			σ'_v = vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	317.9	ft	*Top of Riverbed
Groundwater El.:	317.9	ft	
Depth to Groundwater:	0.0	ft	
Average Total Unit Weight of Soil:	120	pcf	

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	92.4	1.54

Boring:	***BB-DFPR-102***			**Corrected Blow Counts**				**Overburden Correction**					**Hammer Efficiency Correction**		
Depth (ft)	**El. (ft)**	**Layer Name**	**N**	**N_{60}**	**$N1_{60}$**	**Avg. N_{60}**	**Avg. $N1_{60}$**	**σ_v (psf)**	**u (psf)**	**σ'_v (psf)**	**σ'_v (ksf)**	**C_N**	**Hammer Type**	**ER (%)**	**C_E**
1	316.9	River Sediment	3	5	9	5	9	120	62	58	0.058	2.00	Automatic	92.4	1.54

Notes:
1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

GEI Consultants

Client: Thornton Tomasetti - WIN 23120
Project: Dover Bridge #5118, Essex Street over Piscataquis River
Project No.: 2302742
Subject: Corrected Blow Counts

Prepared By: M. Johnescu
Date: 1/26/2024
Checked By: G. Williams
Date: 6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where:	N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
			ER = hammer efficiency expressed as percent of theoretical free fall energy
			N = Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where:	$N1_{60}$ = SPT blow count corrected for overburden and hammer efficiency (blows/ft)
			$C_N = 0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
			σ'_v = vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	342.9	ft
Groundwater El.:	330.9	ft
Depth to Groundwater:	12.0	ft
Average Total Unit Weight of Soil:	120	pcf

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	92.4	1.54

Boring: ***BB-DFPR-103***

Depth (ft)	El. (ft)	Layer Name	N	Corrected Blow Counts: N_{60}	$N1_{60}$	Avg. N_{60}	Avg. $N1_{60}$	Overburden Correction: σ_v (psf)	u (psf)	σ'_v (psf)	σ'_v (ksf)	C_N	Hammer Efficiency Correction: Hammer Type	ER (%)	C_E
2	340.9	FILL	14	22	37	23	31	240	0	240	0.240	1.71	Automatic	92.4	1.54
6	336.9	FILL	35	54	72			720	0	720	0.720	1.34	Automatic	92.4	1.54
11	331.9	FILL	3	5	5			1,320	0	1,320	1.320	1.14	Automatic	92.4	1.54
16	326.9	FILL	7	11	11			1,920	250	1,670	1.670	1.06	Automatic	92.4	1.54

Notes:
1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

GEI Consultants	**Client:** Thornton Tomasetti - WIN 23120	**Prepared By:** M. Johnescu
	Project: Dover Bridge #5118, Essex Street over Piscataquis River	**Date:** 1/26/2024
	Project No.: 2302742	**Checked By:** G. Williams
	Subject: Corrected Blow Counts	**Date:** 6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where:	N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
			ER = hammer efficiency expressed as percent of theoretical free fall energy
			N = Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where:	$N1_{60}$ = SPT blow count corrected for overburden and hammer efficiency (blows/ft)
			$C_N = 0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
			σ'_v = vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	343.0	ft
Groundwater El.:	326.7	ft
Depth to Groundwater:	16.3	ft
Average Total Unit Weight of Soil:	120	pcf

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	76.5	1.28

Boring:	*BB-DFPR-201*			Corrected Blow Counts				Overburden Correction					Hammer Efficiency Correction		
Depth (ft)	El. (ft)	Layer Name	N	N_{60}	$N1_{60}$	Avg. N_{60}	Avg. $N1_{60}$	σ_v (psf)	u (psf)	σ'_v (psf)	σ'_v (ksf)	C_N	Hammer Type	ER (%)	C_E
17	326.0	GLACIAL TILL	27	34	35	34	35	2,040	44	1,996	1.996	1.00	Automatic	76.5	1.28

Notes:
1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

GEI Consultants	**Client:** Thornton Tomasetti - WIN 23120	**Prepared By:**	M. Johnescu
	Project: Dover Bridge #5118, Essex Street over Piscataquis River	**Date:**	1/26/2024
	Project No.: 2302742	**Checked By:**	G. Williams
	Subject: Corrected Blow Counts	**Date:**	6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where: N_{60} =	SPT blow count corrected for hammer efficiency (blows/ft)
		ER =	hammer efficiency expressed as percent of theoretical free fall energy
		N =	Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where: $N1_{60}$ =	SPT blow count corrected for overburden and hammer efficiency (blows/ft)
		C_N =	$0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
		σ'_v =	vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	342.8	ft
Groundwater El.:	325.4	ft
Depth to Groundwater:	17.4	ft
Average Total Unit Weight of Soil:	120	pcf

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	76.5	1.28

Boring:	***BB-DFPR-202***			Corrected Blow Counts				Overburden Correction					Hammer Efficiency Correction		
Depth (ft)	El. (ft)	Layer Name	N	N_{60}	$N1_{60}$	Avg. N_{60}	Avg. $N1_{60}$	σ_v (psf)	u (psf)	σ'_v (psf)	σ'_v (ksf)	C_N	Hammer Type	ER (%)	C_E
5	337.8	FILL	42	54	75	54	75	600	0	600	0.600	1.40	Automatic	76.5	1.28
24	318.8	GLACIAL TILL	17	22	20	22	20	2,880	412	2,468	2.468	0.93	Automatic	76.5	1.28

Notes:

1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

GEI Consultants

Client:	Thornton Tomasetti - WIN 23120	**Prepared By:**	M. Johnescu
Project:	Dover Bridge #5118, Essex Street over Piscataquis River	**Date:**	1/26/2024
Project No.:	2302742	**Checked By:**	G. Williams
Subject:	Corrected Blow Counts	**Date:**	6/17/2025

References: 1) American Association of State Highway and Transportation Officials (AASHTO) "LRFD Bridge Design Specifications, 9th Edition, 2020"

Equations:

Ref. 1 Eqn. No.	Equation		
10.4.6.2.4-2	$N_{60} = (ER / 60\%) * N$	where:	N_{60} = SPT blow count corrected for hammer efficiency (blows/ft)
			ER = hammer efficiency expressed as percent of theoretical free fall energy
			N = Uncorrected SPT blow count (blows/ft)
10.4.6.2.4-3	$N1_{60} = C_N * N_{60}$	where:	$N1_{60}$ = SPT blow count corrected for overburden and hammer efficiency (blows/ft)
			$C_N = 0.77 * \log_{10}(40/\sigma'_v)$ $[C_N < 2.0]$
			σ'_v = vertical effective stress (ksf)

Assumptions:

Ground Surface El.:	342.6	ft
Groundwater El.:	326.5	ft
Depth to Groundwater:	16.1	ft
Average Total Unit Weight of Soil:	120	pcf

Hammer Type	ER (%)	C_E = ER / 60%
Donut	45	0.75
Safety	60	1.00
Automatic	76.5	1.28

Boring:	***BB-DFPR-203***			Corrected Blow Counts				Overburden Correction					Hammer Efficiency Correction		
Depth (ft)	**El. (ft)**	**Layer Name**	**N**	**N_{60}**	**$N1_{60}$**	**Avg. N_{60}**	**Avg. $N1_{60}$**	**σ_v (psf)**	**u (psf)**	**σ'_v (psf)**	**σ'_v (ksf)**	**C_N**	**Hammer Type**	**ER (%)**	**C_E**
10	332.6	FILL	7	9	10	28	30	1,200	0	1,200	1.200	1.17	Automatic	76.5	1.28
15	327.6	FILL	37	47	49			1,800	0	1,800	1.800	1.04	Automatic	76.5	1.28
20	322.6	GLACIAL TILL	25	32	31	66	65	2,400	243	2,157	2.157	0.98	Automatic	76.5	1.28
25	317.6	GLACIAL TILL	83	100	99			3,000	555	2,445	2.445	0.93	Automatic	76.5	1.28

Notes:
1. For N_{60} and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.

D.2 Earth Pressure Coefficients

Project: Dover Bridge (#5118) Replacement Project
WIN 023120.00
GEI Project No.: 2305541

By: M. Johnescu
Date: 5/06/2025
Checked By: G. Williams
Date: 06/19/2025

CALCULATE EARTH PRESSURE COEFFICIENTS

Calculations of earth pressure coefficients assigned to soils listed in Soil Properties table of the report are provided in this packet. Active, at-rest, and passive pressures were determined for different soils.

Equations/references utilized for these calculations are provided at the back of this calculation.

	Existing Fill	River Sediment	Glacial Till	Granular Borrow	Gravel Borrow
Friction angle, ϕ (deg)	32	30	38	32	36
Angle of friction between soil and wall, δ (deg)	21	20	25	24	27
Slope of backfill behind wall, β (deg)	0	0	0	0	0
Slope of backfill in front of wall, α (deg) (for passive - enter as neg)	0	0	0	0	0
Angle of back face of wall to horz, θ (deg)	90	90	90	90	90
δ/ϕ	0.7	0.7	0.7	0.8	0.8
β/ϕ	0.0	0.0	0.0	0.0	0.0
Γ	2.81	2.68	3.17	2.87	3.12
Active earth pressure coefficient (Rankine method, MaineDOT BDG 3.6.5.2 and AASHTO C3.11.5.3-1), Ka[1]	0.31	0.33	0.24	0.31	0.26
Active earth pressure coefficient (Coloumb method, AASHTO LRFD 3.11.5.3-1), Ka[1]	0.28	0.30	0.22	0.27	0.24
At-rest earth pressure coefficient (AASHTO LRFD 3.11.5.2-1), Ko	0.47	0.50	0.38	0.47	0.41
Passive earth pressure coefficient[2] (FHWA NHI-06-089 Figure 10-4 Assuming wall rotation of 0.02 for dense and 0.06 for loose)	5.8	3.0	5.8	5.8	5.8

1. For long-heel cantilever walls, use Rankine active earth pressure in accordance with MaineDOT BDG 3.6.5.2 and AASHTO LRFD Figure C3.11.5.3-1.
2. Passive earth pressure for walls should be neglected for cases outlined in MaineDOT BDG 3.6.9. MaineDOT BDG 5.4.2.9 recommends abutment and wingwall reinforcement be sized assuming passive earth pressure on the backface of the wall. Design passive earth pressure coefficient should be calculated using MassDOT BDM Figure 3.10.8-1 and NHI-06-089 Figure 10-4, and the more stringent value should apply. However, passive earth pressure should be no less than Rankine passive earth pressure, regardless of wall rotation.

From AASHTO LRFD 2021:

3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, k_o

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin \phi'_f \qquad (3.11.5.2\text{-}1)$$

where:

ϕ'_f = effective friction angle of soil
k_o = coefficient of at-rest lateral earth pressure

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma\,[\sin^2\theta \sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

δ = friction angle between fill and wall (degrees)
β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
ϕ'_f = effective angle of internal friction (degrees)

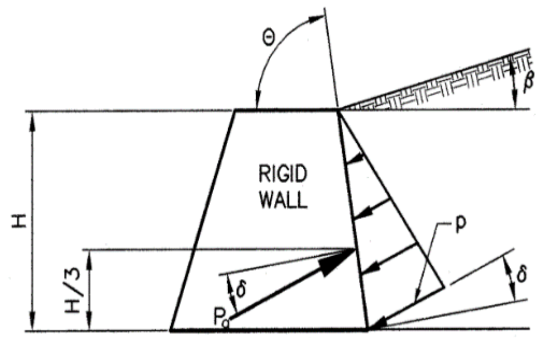


Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure

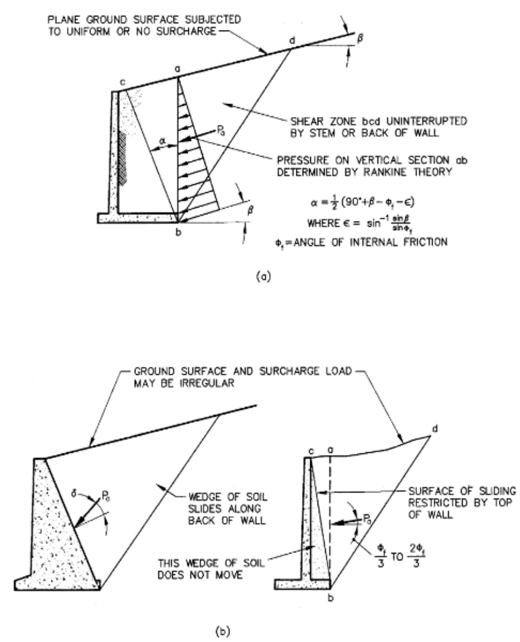
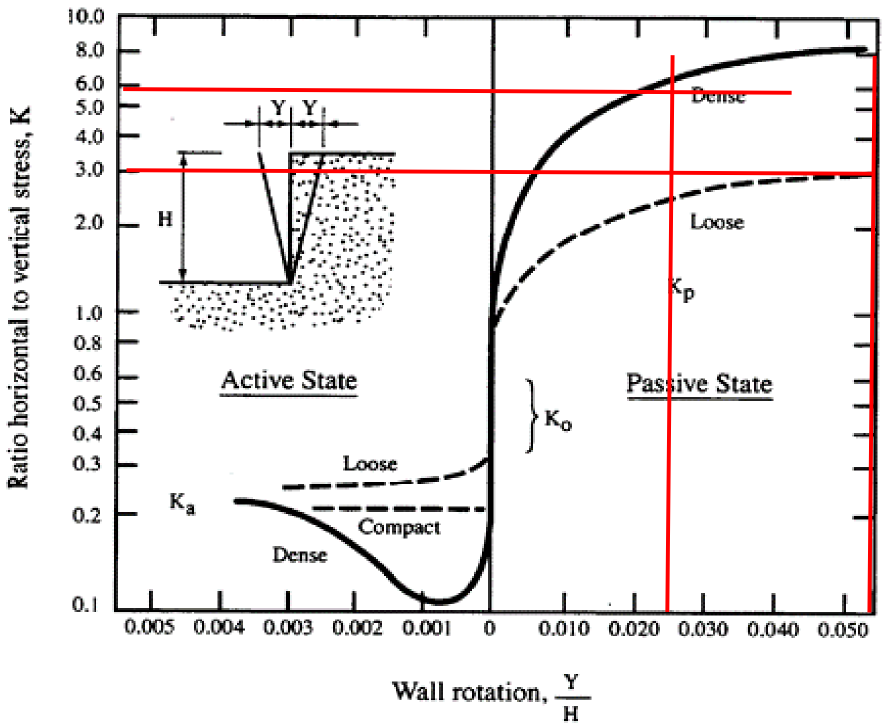


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

From FHWA NHI-06-089:



Magnitude of Wall Rotation to Reach Failure

Soil type and condition	Rotation, Y/H	
	Active	Passive
Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Figure 10-4. Effect of wall movement on wall pressures (after Canadian Geotechnical Society, 1992).

FHWA NHI-06-089
Soils and Foundations – Volume II 10 - 9 10 – Earth Retaining Structures
December 2006

From MaineDOT BDG 2003:

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

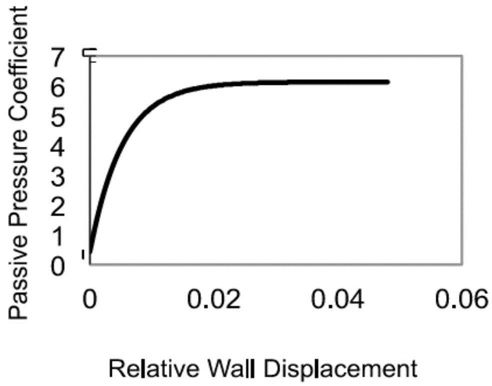
* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

For a sloped backfill surface where $\beta > 0°$, the coefficient of active earth pressure (Rankine), K_a, may be taken as:

$$K_a = \cos\beta \cdot \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

From MassDOT BDM:

$$K = 0.43 + 5.7[1 - e^{-190(\delta_T/H)}]$$



$\delta_T/H =$ 0.02

Kp= 6.0

Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

GEI Consultants

Client: Thornton Tomasetti
Project: Dover Bridge Final Design
Project No.: 2305541
Subject: Lateral Earth Pressures

Prepared By: M. Johnescu
Date: 5/14/2025
Checked By: G. Williams
Date: 6/19/2025

Purpose: Calculate the Seismic Active Earth Pressure Coefficient.

Reference: AASHTO (2017). "AASHTO LRFD Bridge Design Specifications,"

Calculations: **Lateral Earth Pressure Coefficients K_{AE} & K_{PE}**

	Granular Borrow Granular Underwater Backfill 0° foreslope	**Existing Fill 0° foreslope**	**River Sediment 0° foreslope**	**Glacial Till 0° foreslope**	**Gravel Borrow 0° foreslope**
Unit Weight (pcf)	**125**	**125**	**115**	**135**	**135**
Friction Angle of Soil, ϕ_f (deg)	32.0	32.0	30.0	38.0	36.0
Wall Backfill Interface Friction Angle, δ (deg)	24.0	21.4	20.1	25.5	27.0
Backfill Slope Angle, i (deg)	0.0	0.0	0.0	0.0	0.0
Slope of Wall to the Vertical, β (deg)	0.0	0.0	0.0	0.0	0.0
Peak Ground Acceleration, PGA	0.074	0.074	0.074	0.074	0.074
Horizontal Seismic Acceleration Coefficient at Zero Displacement, k_{h0}	0.089	0.089	0.089	0.089	0.089
Horizontal Seismic Acceleration Coefficient, k_h	0.089	0.089	0.089	0.089	0.089
Vertical Seismic Acceleration Coefficient, k_v	0.0	0.0	0.0	0.0	0.0
θ_{MO} (deg)	5.1	5.1	5.1	5.1	5.1
Seismic Active Earth Pressure Coefficient, K_{AE}	**0.33**	**0.33**	**0.36**	**0.27**	**0.29**

Notes:
1. Please see notes on page 2.
2. Semi-integral abutments where the superstructure end diaphragm overhangs the back of the abutment should be checked for overturning with 100% of the seismic active force. Apply 125 psf traffic surcharge as applicable.

D.3 Site Class Evaluation

Seismic Site Class Evaluation – Essex Street over Piscataquis River– Bridge #5118

Purpose: Evaluate seismic design criteria in accordance with AASHTO 9th Ed, 2020. Evaluate borings BB-DFPR-101 through -203A using N60 values and correlating to shear wave velocity using DeJong 2012. N60 values were limited to 100 bpf. Bedrock shear wave velocity assumed to be uniform for metasiltstone and an N60 value of 100 bpf.

BB-DFPR-101

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	240	11.5	34	118.4	388.3	5	0.15	0.0129
2	720	34.5	100	195.3	640.6	5	0.05	0.0078
3	1320	63.2	11	135.1	443.3	3	0.27	0.0068
4	1555	74.5	8	130.4	427.8	6	0.75	0.0140
5	--	--	100	--	3040.0	81	0.81	0.0266
					Σ =	100.00	2.03	0.07
						$\bar{N}$	**49.3**	
						V_{ch}	**1468**	

BB-DFPR-102

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	58	2.8	5	54.9	180.2	4	0.70	0.0194
2	--	--	100	--	3040.0	97	0.97	0.0317
					Σ =	100.00	1.67	0.05
						$\bar{N}$	**60.1**	
						V_{ch}	**1954**	

BB-DFPR-103

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	240	11.5	22	107.1	351.3	5	0.23	0.0142
2	720	34.5	54	169.5	556.0	5	0.09	0.0090
3	1320	63.2	5	112.7	369.8	5	1.00	0.0135
4	1670	80.0	11	142.7	468.0	5	0.45	0.0107
5	--	--	100	--	3040.0	80	0.80	0.0263
					Σ =	100.00	2.57	0.07
						$\bar{N}$	**38.8**	
						V_{ch}	**1356**	

BB-DFPR-201/201A

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	1996	95.6	34	192.7	632.0	21	0.62	0.0332
2	--	--	100	--	3040.0	79	0.79	0.0260
					Σ =	100.00	1.41	0.06
						$\overline{N}$	**71.0**	
						V_{ch}	**1689**	

BB-DFPR-202

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	600	28.7	54	162.5	533.2	23	0.43	0.0431
2	2468	118.2	22	183.0	600.4	11	0.50	0.0183
5	--	--	100	--	3040.0	66	0.66	0.0217
					Σ =	100.00	1.59	0.08
						$\overline{N}$	**63.1**	
						V_{ch}	**1202**	

BB-DFPR-203/203A

Layer	Effective Stress		N_{60}	Shear Wave Velocity		Layer (D_i)	D_i/N_i	Di/V_{si}
	psf	kPa		m/s	ft/s			
1	1200	57.5	9	126.3	414.1	14	1.56	0.0338
2	1800	86.2	47	202.7	664.9	5	0.11	0.0075
3	2157	103.3	32	193.4	634.4	5	0.16	0.0079
4	2445	117.0	100	258.7	848.6	5	0.05	0.0059
5	--	--	100	--	3040.0	71	0.71	0.0234
					Σ =	100.00	2.58	0.08
						$\overline{N}$	**38.8**	
						V_{ch}	**1275**	

Site Average Shear Wave Velocity in the Upper 100':	**1366**	*not including -201 and -202
Site Average $\overline{N}$ in the upper 100':	**47**	

*BB-DFPR-201 and -202 were ommitted from the seismic site class calculation because of shallow bedrock at the bottom of the riverbed and crushed stone/rockfill within the existing abutments.

Notes

a. Borings were terminated within the bedrock. Therefore, soil beneath bottom of boring to a depth of 100 feet is assumed to be bedrock. We input N_{60} = 100 for rock based on AASHTO and ASCE 7 references, and calcaulted a Vsi = 3040 ft/sec based on lab data results and AASHTO LRFD 2020 tables.

b. Vs = Shear Wave Velocity from DeJong 2012 Correlation using N values corrected for hammer energy for calibrated auto hammers (i.e., N60).

$$\overline{V}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{Vs_i}}$$

$$N_{60} = N * C_E$$
$$where\ C_E = 1.33\ (from\ automatic\ hammer)$$

$$\overline{N} = \frac{\sum d_i}{\sum {d_i}/{N_i}}$$

From AASHTO Table 3.10.3.1-1 where 1200 < vs < 2500 or N>50
Site Class C (Very Dense Soil and Soil Rock)

Site Seismic Coefficients

Horizontal Peak Ground Acceleration,	PGA =	0.074	AASHTO - USGS Seismic Hazard Contour Maps for the 1,000-yr return period (7% probability of exceedance in 75 yrs).
Horizontal Response Spectral Acceleration (0.2 sec),	S_s =	0.155	
Horizontal Response Spectral Acceleration (1 sec),	S_1 =	0.047	
	F_{PGA} =	1.2	AASHTO Table 3.4.2.3-1
	F_A =	1.2	AASHTO Table 3.4.2.3-1
	F_V =	1.7	AASHTO Table 3.4.2.3-2

Design Response Spectra

Acceleration Coefficient,	$A_s = PGA \times F_{PGA}$	A_s=	0.089	AASHTO Eq. 3.10.4.2-2
Design Spectral Acceleration (0.2 sec),	$S_{DS} = S_S \times F_A$	S_{DS}=	0.186	AASHTO Eq. 3.10.4.2-3
Design Spectral Acceleration (1 sec),	$S_{D1} = S_1 \times F_V$	S_{D1}=	0.080	AASHTO Eq. 3.10.4.2-6

From AASHTO Table 3.10.6-1
Seismic Zone 1

Objective: Calculate Shear Wave Velocity from Shear Modulus for Metasiltstone
Sources: AASHTO LRFD 10th Ed., 2024

Variable	Quantity	Unit	Notes
q_u (Avg. UCS)	8262	psi	* Excluded the highest and lowest breaks
q_u (Avg. UCS)	57	MPa	* Excluded the highest and lowest breaks
GSI	45		AASHTO Figure 10.4.6.4-1
Em (Modulus of Intact Rock)	6	GPa	AASHTO Table 10.4.6.5-1, qu<100MPa
Em (Modulus of Intact Rock)	820886	lb/in^2	AASHTO Table 10.4.6.5-1, qu<100MPa
v (Poisson's Ratio)	0.18	--	AASHTO Table C10.4.6.5-2
G (Shear Modulus)	347833	lb/in^2	
G (Shear Modulus)	50087989	lb/ft^2	
γ (unit weight)	174.50	lb/ft^3	Average from the 6 rock samples tested
Gravity	32.20	ft/sec^2	
p (density)	5.42	$(lbsec^2/ft^4)$	
Vs (Shear Wave Velocity)	**3040**	**ft/s**	**Use this value for all bedrock below soil**

AASHTO LRFD 9th Ed., 2020:

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s < 5{,}000$ ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s < 2{,}500$ ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s < 1{,}200$ ft/s, or with either $15 < \bar{N} < 50$ blows/ft, or $1.0 < \bar{s}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s < 600$ ft/s or with either $\bar{N} < 15$ blows/ft or $\bar{s}_u < 1.0$ ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u < 0.5$ ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25.0$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile
$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile
$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile
PI = plasticity index (ASTM D4318)
w = moisture content (ASTM D2216)

Table 3.10.6-1—Seismic Zones

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \leq 0.15$	1
$0.15 < S_{D1} \leq 0.30$	2
$0.30 < S_{D1} \leq 0.50$	3
$0.50 < S_{D1}$	4

Shear Wave Velocity in Soil: Guideline for Estimation of Shear Wave Velocity Profiles, DeJong 2012

Table 4.11 Recommended SPT–stress–V_S correlation equations.

Soil Type	Shear Wave Velocity for Quaternary Soils (m/s)	(Eq #)	Age Scaling Factors Holocene	Pleistocene
All Soils	$30\ N_{60}^{0.215}\ \sigma'_v{}^{0.275}$	(4.17)	0.87	1.13
Clays & Silts	$26\ N_{60}^{0.17}\ \sigma'_v{}^{0.32}$	(4.40)	0.88	1.12
Sands	$30\ N_{60}^{0.23}\ \sigma'_v{}^{0.23}$	(4.77)	0.90	1.17
Gravels - Holocene	$53\ N_{60}^{0.19}\ \sigma'_v{}^{0.18}$	(4.98)	----	----
Gravels - Pleistocene	$115\ N_{60}^{0.17}\ \sigma'_v{}^{0.12}$	(4.102)	----	----

σ'_v measured in kPa

AASHTO LRFD 9th Ed., 2020

Table 3.10.3.2-1—Values of Site Factor, F_{pga}, at Zero-Period on Acceleration Spectrum

Site Class	Peak Ground Acceleration Coefficient (*PGA*)[1] *PGA* < 0.10	*PGA* = 0.20	*PGA* = 0.30	*PGA* = 0.40	*PGA* > 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of *PGA*.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-2—Values of Site Factor, F_a, for Short-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 0.2 sec (S_S)[1] S_S < 0.25	S_S = 0.50	S_S = 0.75	S_S = 1.00	S_S > 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_s.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Table 3.10.3.2-3—Values of Site Factor, F_v, for Long-Period Range of Acceleration Spectrum

Site Class	Spectral Acceleration Coefficient at Period 1.0 sec (S_1)[1] S_1 < 0.1	S_1 = 0.2	S_1 = 0.3	S_1 = 0.4	S_1 > 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F[2]	*	*	*	*	*

Notes:

[1]Use straight-line interpolation for intermediate values of S_1.

[2]Site-specific geotechnical investigation and dynamic site response analysis should be performed for all sites in Site Class F.

Shear Wave Velocity of Bedrock:

Calculate Shear Modulus from Young's Modulus

$$G = \frac{E}{2(1+v)} \quad (1)$$

Calculate Shear Wave Velocity from Shear Modulus for Metasiltstone

$$V_s = \sqrt{G/\rho}$$

AASHTO LRFD 9th Ed. 2020:

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m\left(GPa\right)=\sqrt{\frac{q_u}{100}}10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ MPa	Accounts for rocks with q_u < 100 Mpa; notes q_u in Mpa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m\left(GPa\right)=10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa		
$E_m = \frac{E_R}{100} e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)
Notes: E_r = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength, and 1 Mpa = 20.9 ksf.		

Table C10.4.6.5-1—Summary of Elastic Moduli for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_R (ksi × 10^3) Maximum	Minimum	Mean	Standard Deviation (ksi × 10^3)
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Table C10.4.6.5-2—Summary of Poisson's Ratio for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

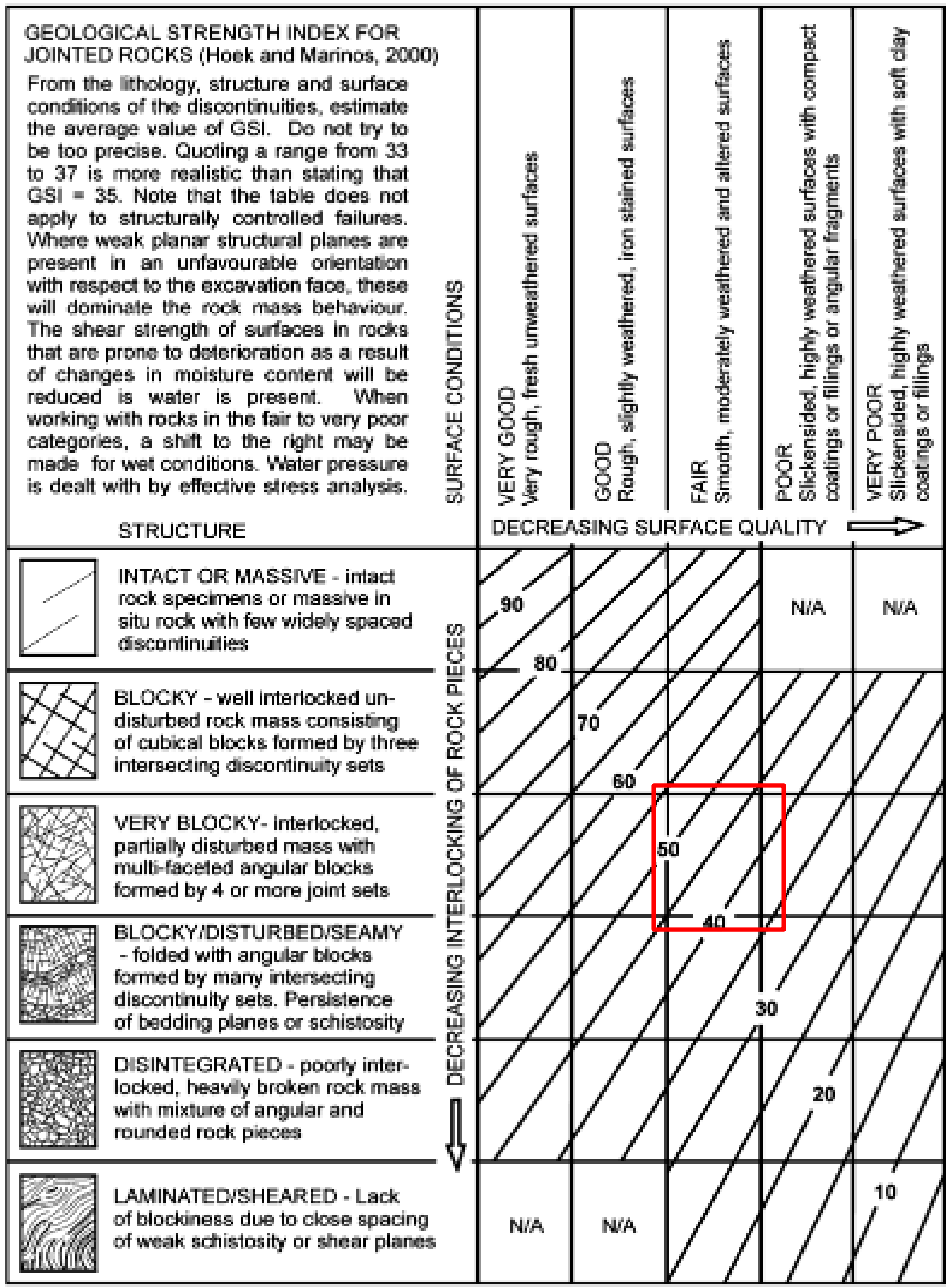


Figure 10.4.6.4-1—Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000)

Figure 3.10.2.1-1 (continued)—Horizontal Peak Ground Acceleration Coefficient for the Conterminous United States (*PGA*) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period)

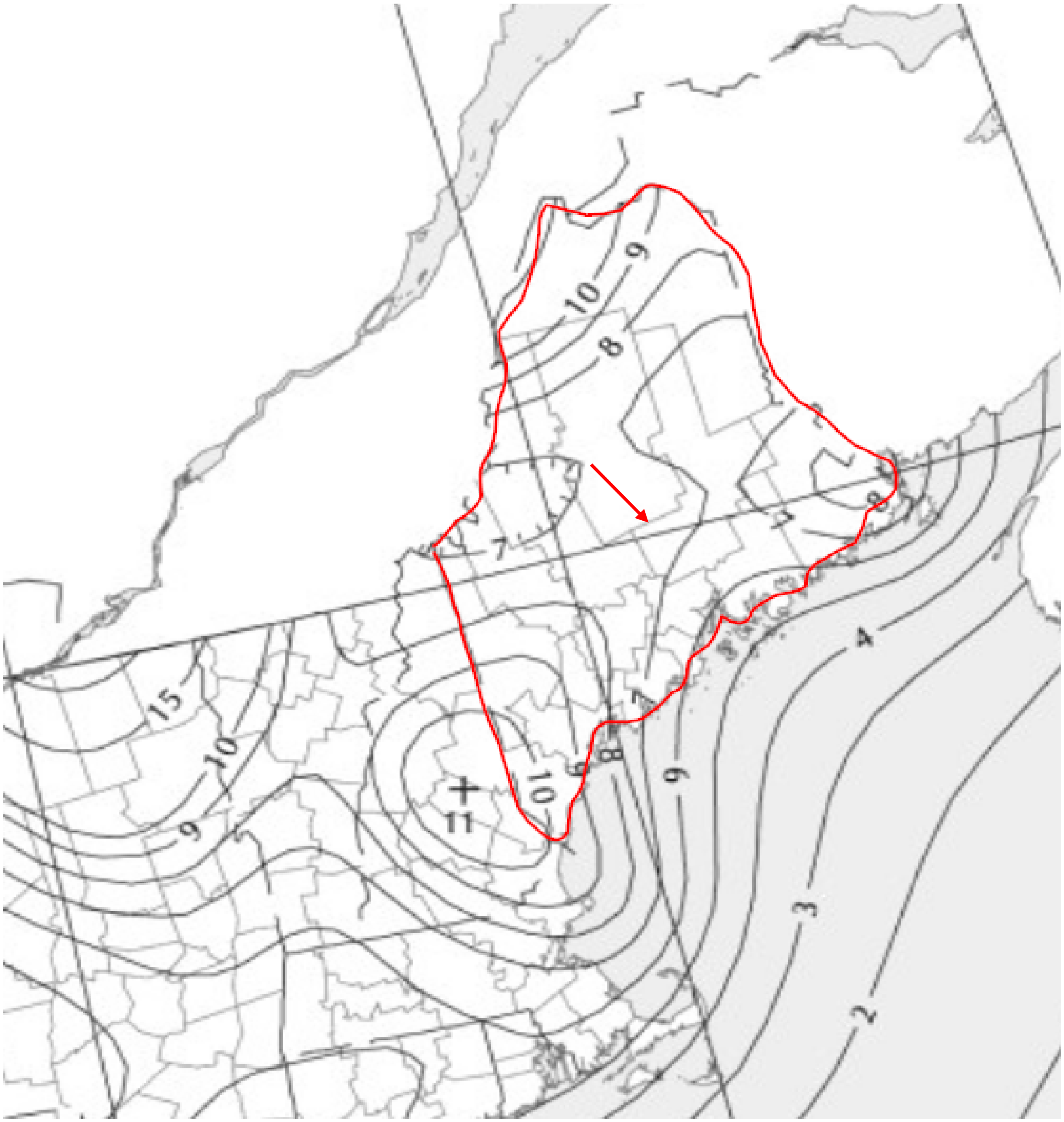


Figure 3.10.2.1-2 (continued)—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 0.2 s (S_S) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping

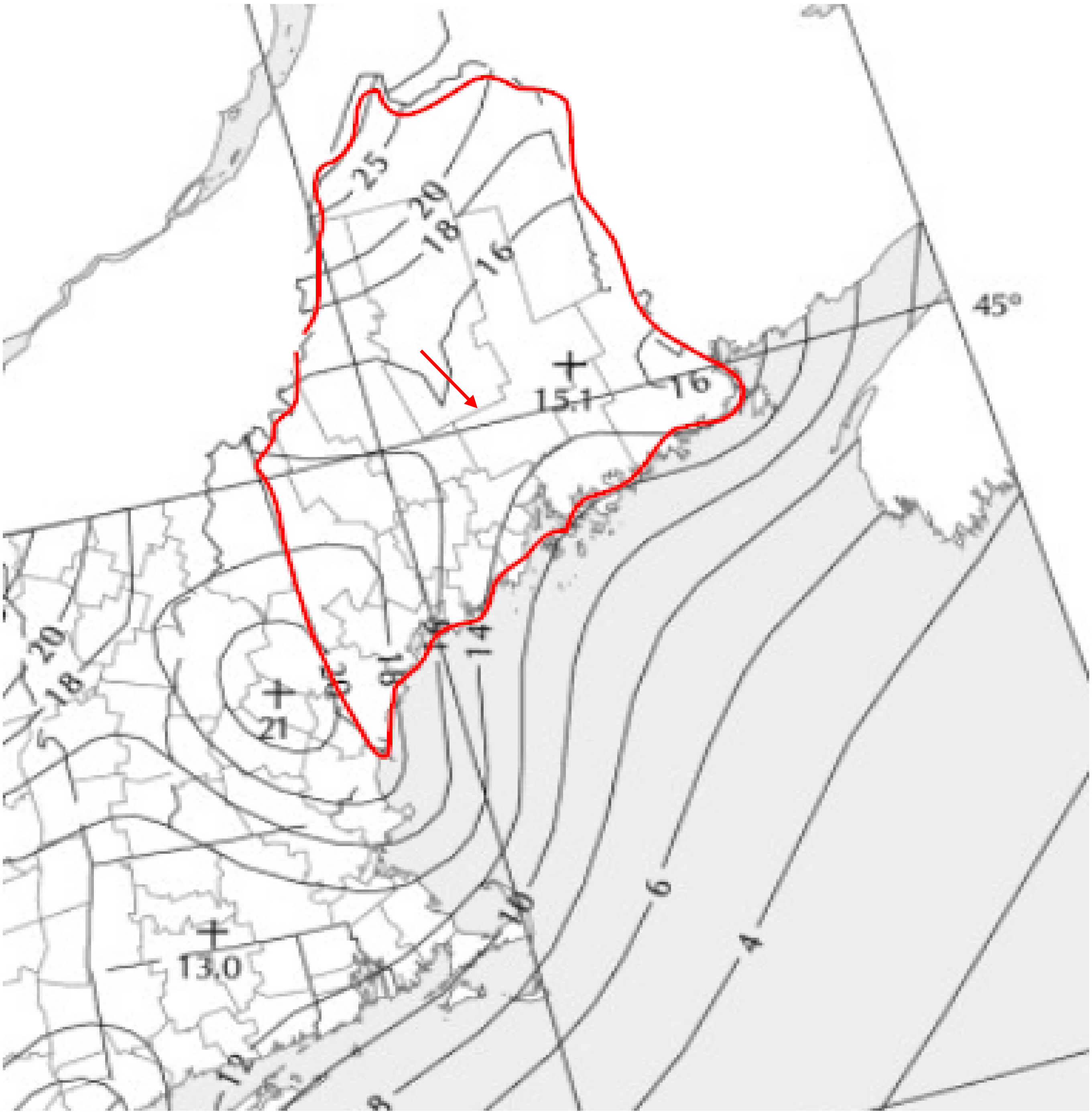
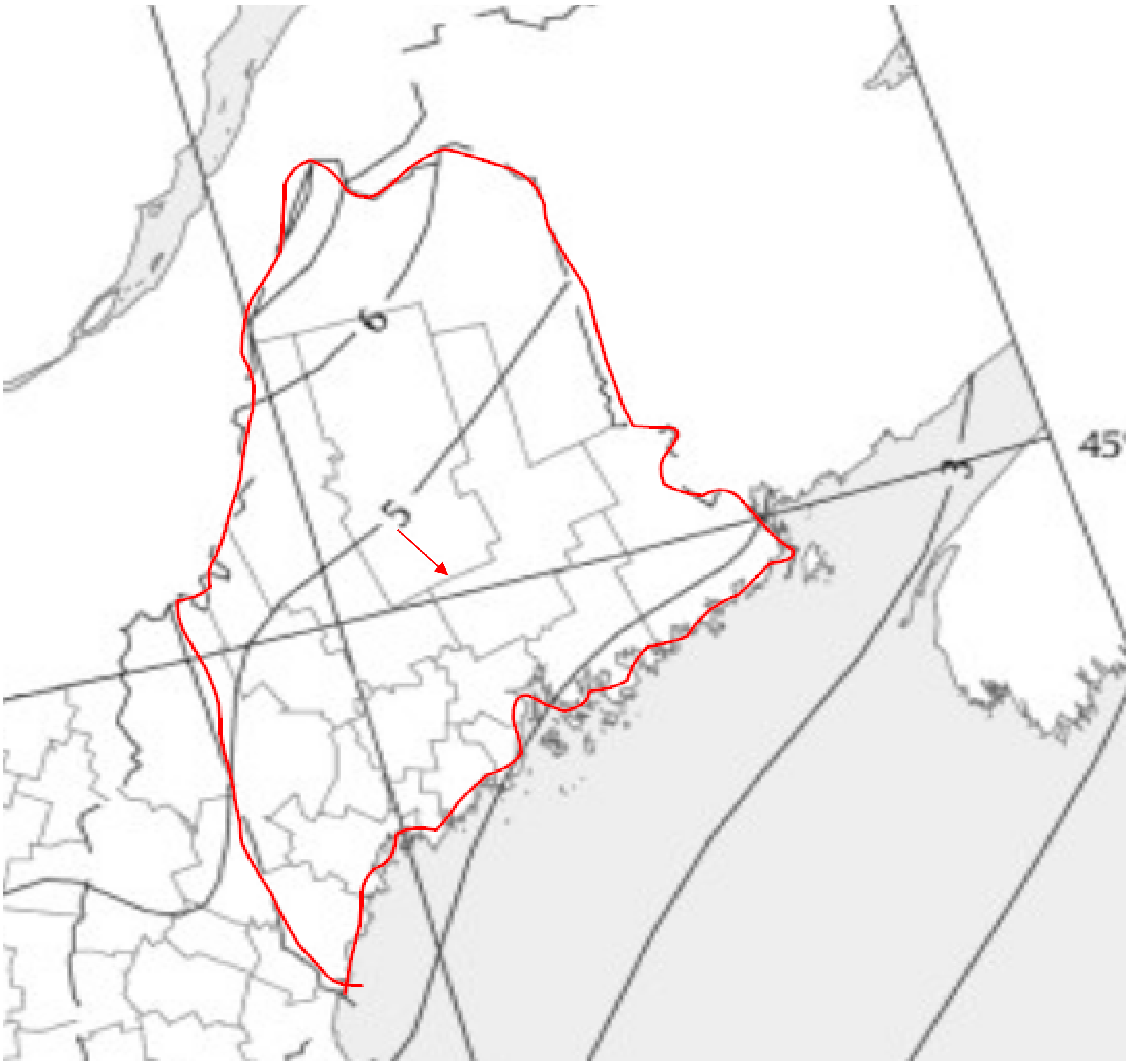


Figure 3.10.2.1-3 (continued)—Horizontal Response Spectral Acceleration Coefficient for the Conterminous United States at Period of 1.0 s (S_1) with Seven Percent Probability of Exceedance in 75 Years (Approx. 1,000-year Return Period) and Five Percent Critical Damping



D.4 Frost Depth Calculation

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Frost Penetration (2050 & w=15%) = (95.1+78.7+97.6+80.7)/4 = 88" = 7.3', say 7.5'

Notes: 1. w = water content

2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

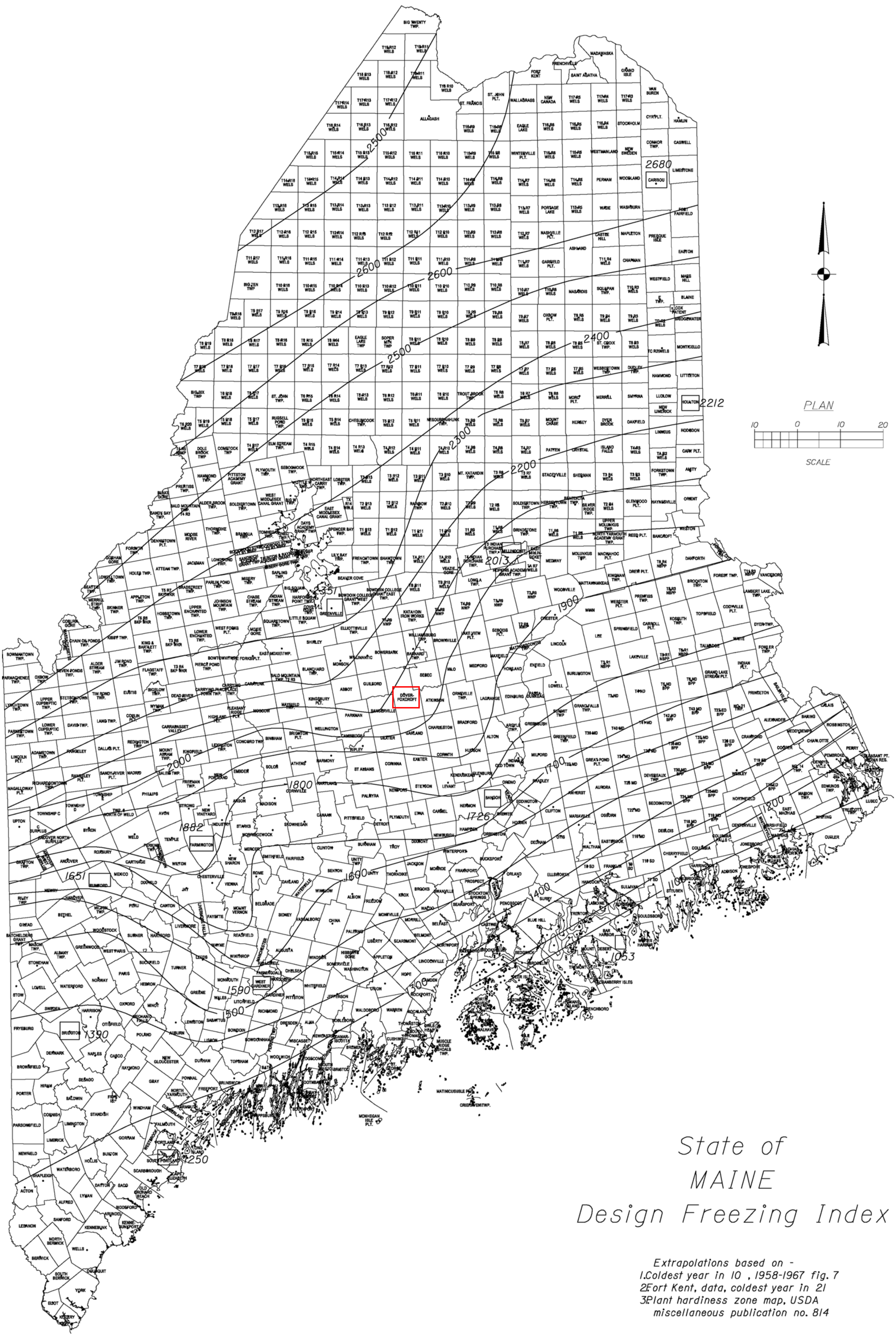


Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given:	Site location is Freeport, Maine
Soil conditions:	Silty fine to coarse Sand

Step 1. From Figure 5-1 Design Freezing Index = 1300 degree-days
Step 2. From laboratory results: soil water content = 28% and major constituent Sand
Step 3. From Table 5-1: Depth of frost penetration = 56 inches = 4.7 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Pile supported integral abutments will be embedded no less than 4.0 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$145 \cdot y = 62.4 \cdot z$$

where:

145 lb/ft^3 =	unit weight of concrete
62.4 lb/ft^3 =	unit weight of water
y =	the depth of seal from top of seal to bottom of seal
z =	the depth of water from water surface to bottom of seal

D.5 Bearing Resistance - Spread Footings on Bedrock

Client:	Thornton Tomasetti	**Prepared By:**	M. Johnescu
Project:	Dover Bridge #5118 WIN 023120.00 Dover, Maine	**Date:**	5/14/2025
Project No.:	2305541	**Checked By:**	G. Williams
		Date:	6/19/2025

Bearing Resistance on Rock

Purpose:
The purpose of this evaluation is to estimate the bearing resistance for the proposed Abutment 1 and Pier bearing on bedrock at Dover Bridge #5118, which carries Essex Street over Piscataquis River in Dover-Foxcroft, Maine.

References:
AASHTO LRFD Bridge Design Specification, 9th Edition, 2020
FHWA NHI-16-072 GEC No. 5 – Geotechnical Site Characterization (Loehr et. al, 2016).
Carter and Kulhawy, 1988. Analysis and Design of Drilled Shaft Foundations Socketed into Rock.
AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002.
Bowles, 1996. Foundation Analysis and Design, Fifth Edition.

Summary:
Rock core samples were collected in the six borings performed at the site. Approximately 10 to 24 feet of bedrock was cored in each boring. We evaluated the rock samples to classify the rock type and estimate rock quality.

The rock observed in the borings consisted of Metasiltstone, generally characterized as a moderately hard to hard, fine grained, metamorphic rock that was typically fresh to moderately weathered. Joint spacing varied between core samples, ranging from <1 inch to about 54 inches. The RQD in the borings ranged from 0 to 100 percent, with a weighted average of 54%.

Approach:
Per AASHTO C10.4.6.4, the design of foundations in rock is according to the Rock Mass Rating (RMR) system. The Rock Mass Strength was estimated using the RMR system described in FHWA NHI-16-072 GEC No. 5 Table 9-5.

1. Strength of Intact Rock = 7
2. RQD = 13
3. Spacing of Joints = 8
4. Condition of Joints = 20
5. Groundwater Conditions = 4

The sum of the relative ratings minus the adjustment for joint orientation is the RMR. The adjustment for joint orientation is shown in Table 9-6 and is equal to 15 due to steep joints which are unfavorable for bearing.

RMR = 7 + 13 + 8 + 20 + 4 – 15 = 37

Bearing Resistance at Strength Limit State
For bearing resistance calculations at the strength limit, AASHTO C10.6.3.2.2 indicates that a semi-empirical procedure by Carter and Kulhawy (1988) can be used for jointed rock. The following equation was used to evaluate the bearing resistance of rock:

$$q_{ult} = \left[\sqrt{s} + \sqrt{m\sqrt{s} + s}\right] q_u$$ Carter and Kulhawy (1988) Equation 3-6

Material constants mi was selected from FHWA NHI-16-072 GEC No. 5 Table 9-10. Based on the rock type and the estimated RMR above, values for ‘s’ and ‘m’ were calculated for Siltstone (closest match to Metasiltstone).

m/m_i = exp((RMR-100)/14) Eqn 18 from Hoek & Brown 1988

m_i = 7 for intact, Siltstone (closest match to Metasiltstone)

m = 0.078

s = exp((RMR-100)/6) Eqn 19 from Hoek & Brown 1988

s = 2.8E-5

AASHTO Standard Specifications for Highway Bridges 17th Ed. Table 4.4.8.1.2B indicates a typical compressive strength for siltstone of approximately 200 to 2,500 ksf. Bowles, Foundation Analysis and Design 5th Edition, 1996, Table 4-11 indicates a typical compressive strength for shale of 146 to 835 ksf. We used an average value of q_u = 1,190 ksf (8,262 psi) taken from the average of the unconfined compressive laboratory test results with the highest and lowest break values removed, and the constants m and s above, to calculate:

q_{ult} =31.3 ksf

The bearing pressure should be limited to the lesser of the estimated rock bearing resistance or the nominal resistance of the concrete taken as 0.3f'c.

<u>Bearing Resistance at Service Limit State</u>
Table C10.6.2.5.1-1 in AASHTO indicates that the normal range of presumptive bearing resistance for spread footing foundations at the service limit state (for 1 inch of settlement) can be between 16 ksf and 24 ksf, with a recommended value of 20 ksf. We recommend using 16 ksf due to the quality of the bedrock samples collected from the borings.

Completed six UCS tests, removed the highest and lowest break values for an average of 1,190 ksf (57 MPa)

Weighted average RQD of 54%

very close to close joints (<2" to 12")

Table 9-5 Rock Mass Rating (RMR) system of rock mass classification (from ASTM D5878, 2008).

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 -10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range – uniaxial compressive test is preferred		
		Uniaxial compressive strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5-25 MPa	1-5 MPa	<1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		>2 m	0,6 - 2 m	200 - 600 mm	60 - 200 mm	<60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities		Very rough surfaces. Not continuous No separation Unweathered wall rock.	Slightly rough surfaces. Separation < 1 mm Slightly weathered walls	Slightly rough surfaces. Separation < 1 mm Highly weathered walls	Slickensided surfaces OR Gouge < 5 mm thick OR Separation 1-5 mm. Continuous	Soft gouge > 5 mm thick OR Separation > 5 mm. Continous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length	None	<10 litres/min	10-25 litres/min	25 - 125 litres/min	>125		
		Ratio joint water pressure / major principal stress	OR 0	OR 0,0-0,1	OR 0,1-0,2	OR 0,2-0,5	OR >0,5		
		General conditions	OR Completely dry	OR Damp	OR Wet	OR Dripping	OR Flowing		
	Rating		15	10	7	4	0		

Table 9-6 RMR System parameter R_6 (from ASTM D5878, 2008).

Horizontal to Vertical

Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	−2	−5	−10	−12
	Foundations	0	−2	−7	−15	−25
	Slopes	0	−5	−25	−50	−60

Several methods are available for establishing appropriate values of GSI for specific rock masses. The system was initially developed to be used based on qualitative descriptions of a rock mass, as illustrated in Figure 9-23. Use of the qualitative descriptions and diagrams for important characteristics of rock masses is generally straightforward to apply when observations of rock mass exposures are available and consistent with the precision with which rock masses can be practically classified. However, use of Figure 9-23 can be challenging when only borehole measurements are available.

Table 9-10 Values of material constant, m_i (from Marinos and Hoek, 2001)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerates (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
IGNEOUS	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
IGNEOUS	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

NOTE: numbers in parentheses are estimates.

* Values for foliated metamorphic rock are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

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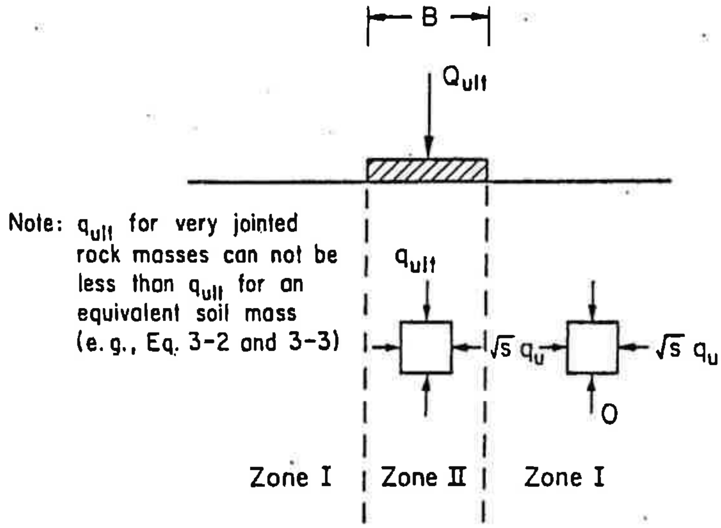

NOTE + VERT. STRESS IN II

Rock Mass Failure Criterion: $\sigma_1 = \sigma_3 + \sqrt{(m q_u \sigma_3 + s q_u^2)}$

Figure 3-5. Lower Bound Solution for Bearing Capacity

the failure criterion. The rock mass beneath a strip footing may be divided into two zones, with homogeneous stress conditions at failure throughout each, as shown in Figure 3-5. The vertical stress in zone I is assumed to be zero, while the horizontal stress is equal to the uniaxial compressive strength of the rock mass, given by Equation 3-5 as $s^{1/2}q_u$. For equilibrium, continuity of the horizontal stress across the interface must be maintained, and therefore the bearing capacity of the strip footing may be evaluated from Equation 3-5 (with $\sigma_3 = s^{1/2}q_u$) as:

$$q_{ult} = [\underbrace{s^{1/2} + (ms^{1/2} + s)^{1/2}}_{N_{ms}}]q_u \qquad (3\text{-}6)$$

EQN.

For a circular foundation, a similar approach may be used, with the interface between the two zones being a cylindrical surface of the same diameter as the foundation. In this axisymmetric case, the radial stress transmitted across the cylindrical surface, at the point of collapse of the foundation, may be greater than $s^{1/2}q_u$, without necessarily violating either radial equilibrium or the failure criterion. However, because of the uncertainty of this value, the radial stress at the interface also is assumed to be $s^{1/2}q_u$ for the case of a circular foundation. Therefore, the predicted (lower bound) bearing capacity is given by Equation 3-6.

Guidelines for selecting s and m for jointed rock masses are given in Table 3-1. The categories in this table are determined by the rock type and the conditions of

Carter & Kulhawy 1988
(Analysis and Design of Drilled Shaft Foundations Socketed into Rock)

order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s.

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s. Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{14}\right) \tag{18}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{6}\right) \tag{19}$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\mathrm{RMR} - 100}{28}\right) \tag{20}$$

$$s = \exp\left(\frac{\mathrm{RMR} - 100}{9}\right) \tag{21}$$

where

m and s are the rock mass constants and
m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\mathrm{RMR} = 9 \,\mathrm{Log}_e\, \mathrm{Q} + 44 \tag{22}$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

Table C10.6.2.5.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.5.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

Bowles, 1996
Fifth Edition

Closest match to Metasiltstone

TABLE 4-11
Range of properties for selected rock groups; data from several sources

Type of rock	Typical unit wt., kN/m³	Modulus of elasticity E, MPa $\times 10^3$	Poisson's ratio, μ	Compressive strength, MPa
Basalt	28	17–103	0.27–0.32	170–415
Granite	26.4	14–83	0.26–0.30	70–276
Schist	26	7–83	0.18–0.22	35–105
Limestone	26	21–103	0.24–0.45	35–170
Porous limestone		3–83	0.35–0.45	7–35
Sandstone	22.8–23.6	3–42	0.20–0.45	28–138
Shale	15.7–22	3–21	0.25–0.45	7–40
Concrete	15.7–23.6	Variable	0.15	15–40

*Depends heavily on confining pressure and how determined; E = tangent modulus at approximately 50 percent of ultimate compression strength.

146 to 835 ksf

the bearing-capacity factors for sound rock are approximately

$$N_q = \tan^6\left(45^\circ + \frac{\phi}{2}\right) \qquad N_c = 5\tan^4\left(45^\circ + \frac{\phi}{2}\right) \qquad N_\gamma = N_q + 1 \tag{4-27}$$

Use the Terzaghi shape factors of Table 4-1 with these bearing-capacity factors. The rock angle of internal friction is seldom less than 40° (often 45° to 55°) and rock cohesion ranges from about 3.5 to 17.5 MPa (500 to 2500 psi). It is evident from Eq. (4-27) that very high values of ultimate bearing capacity can be computed. The upper limit on allowable bearing capacity is, as previously stated, taken as f'_c of the base concrete or not more than the allowable bearing pressure of metal piles.

The angle of internal friction of rock is pressure-dependent, similar to soil. Also, inspection of rock parameters from a number of sources indicates that, similar to sand, we could estimate $\phi = 45^\circ$ for most rock except limestone or shale where values between 38° and 45° should be used. Similarly we could in most cases estimate $s_u = 5$ MPa as a conservative value. Finally we may reduce the ultimate bearing capacity based on RQD as

$$q'_{\text{ult}} = q_{\text{ult}}(\text{RQD})^2$$

In many cases the allowable rock-bearing pressure is taken in the range of one-third to one-tenth the unconfined compression strength obtained from intact rock samples and using RQD as a guide, for example, as one-tenth for a small RQD. Others simply use an allowable bearing pressure from the local building code (as in Table 4-8) based on rock type from a visual inspection of the rock cores.

Few building foundations such as mats or spread bases are placed directly on rock. Most situations involving rock-bearing capacity require large-diameter drilled shafts (termed drilled piers as in Chap. 19), which are socketed 2 to 3 shaft diameters into the rock. Recent load tests on this type of foundation [see Rowe and Armitage (1987)] indicate the allowable bearing pressure is on the order of

$$q_a = q_u \text{ to } 2.5q_u$$

where q_u = unconfined compression strength of intact rock core samples. This value is substantially larger than the values of one-third and one-tenth previously cited. The large increase

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of pressure (R) on the base of footings shall be maintained within B/4 of the center of the footing.

The bearing capacity and settlement of footings on rock is influenced by the presence, orientation and condition of discontinuities, weathering profiles, and other similar features. The methods used for design of footings on rock should consider these factors as they apply at a particular site, and the degree to which they should be incorporated in the design.

For footings on competent rock, reliance on simple and direct analyses based on uniaxial compressive rock strengths and RQD may be applicable. Competent rock is defined as a rock mass with discontinuities that are tight or open not wider than ⅛ inch. For footings on less competent rock, more detailed investigations and analyses should be used to account for the effects of weathering, the presence and condition of discontinuities, and other geologic factors.

4.4.8.1 Bearing Capacity

4.4.8.1.1 Footings on Competent Rock

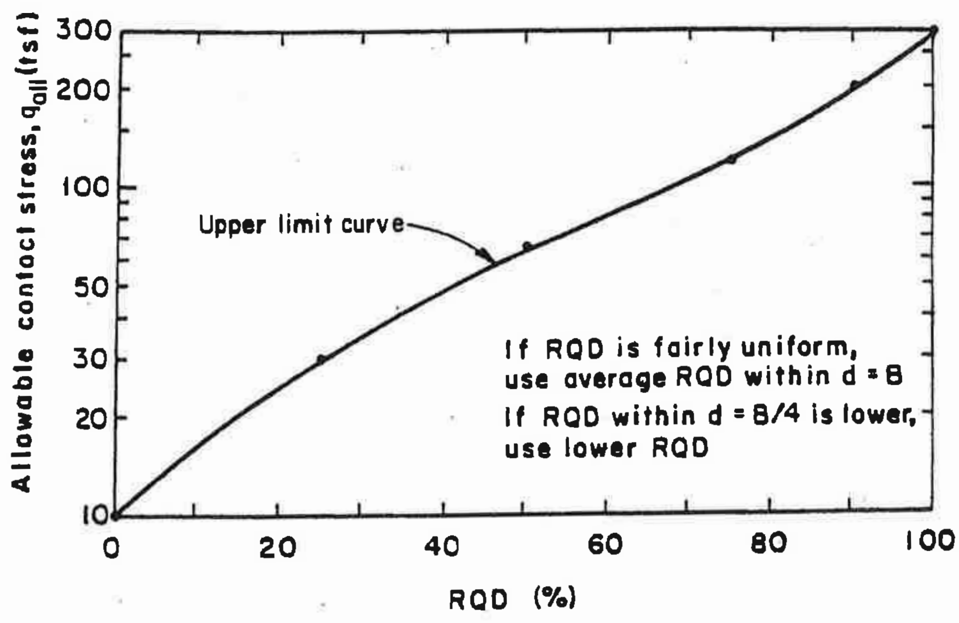
The allowable contact stress for footings supported on level surfaces in competent rock may be determined using Figure 4.4.8.1.1A (Peck, et al. 1974). In no instance shall the maximum allowable contact stress exceed the allowable bearing stress in the concrete. The RQD used in Figure 4.4.8.1.1A shall be the average RQD for the rock within a depth of B below the base of the footing, where the RQD values are relatively uniform within that interval. If rock within a depth of 0.5B below the base of the footing is of poorer quality, the RQD of the poorer rock shall be used to determine q_{all}.

4.4.8.1.2 Footings on Broken or Jointed Rock

The design of footings on broken or jointed rock must account for the condition and spacing of joints and other discontinuities. The ultimate bearing capacity of footings on broken or jointed rock may be estimated using the following relationship:

$$q_{ult} = N_{ms}C_o \qquad (4.4.8.1.2\text{-}1)$$

Refer to Table 4.4.8.1.2A for values of N_{ms}. Values of C_o should preferably be determined from the results of laboratory testing of rock cores obtained within 2B of the base of the footing. Where rock strata within this interval are variable in strength, the rock with the lowest capacity



Note:
q_{all} shall not exceed the unconfined compressive strength of the rock or 0.595 f'c of the concrete.

FIGURE 4.4.8.1.1A Allowable Contact Stress for Footings on Rock with Tight Discontinuities Peck, et al. (1974)

2002 AASHTO

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[(1)] (ksf)	C_o[(1)] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[(2)]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

(1)Range of Uniaxial Compressive Strength values reported by various investigations.
(2)Not including oil shale.

$$\rho = q_o (1 - \nu^2)BI_\rho/E_m, \text{ with } I_\rho = (L/B)^{1/2}/\beta_z \quad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \quad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \quad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

D.6 Bearing Resistance – Spread Footing on Fill

FACTORED BEARING RESISTANCE FOR FOOTINGS ON EXISTING FILL

The following calculation provides bearing resistance calculations for the proposed wingwalls if placed on gravel borrow.

References utilized for these calculations (including those pertaining to resistance factors) are provided at the back of this calculation. Cross sections are attached for reference.

Bearing resistances were calculated with the following formula:

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \quad BN_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

c = cohesion, taken as undrained shear strength (ksf)

N_c = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_q = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

N_γ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

γ = total (moist) unit weight of soil above or below the bearing depth of the footing (kcf)

D_f = footing embedment depth (ft)

B = footing width (ft)

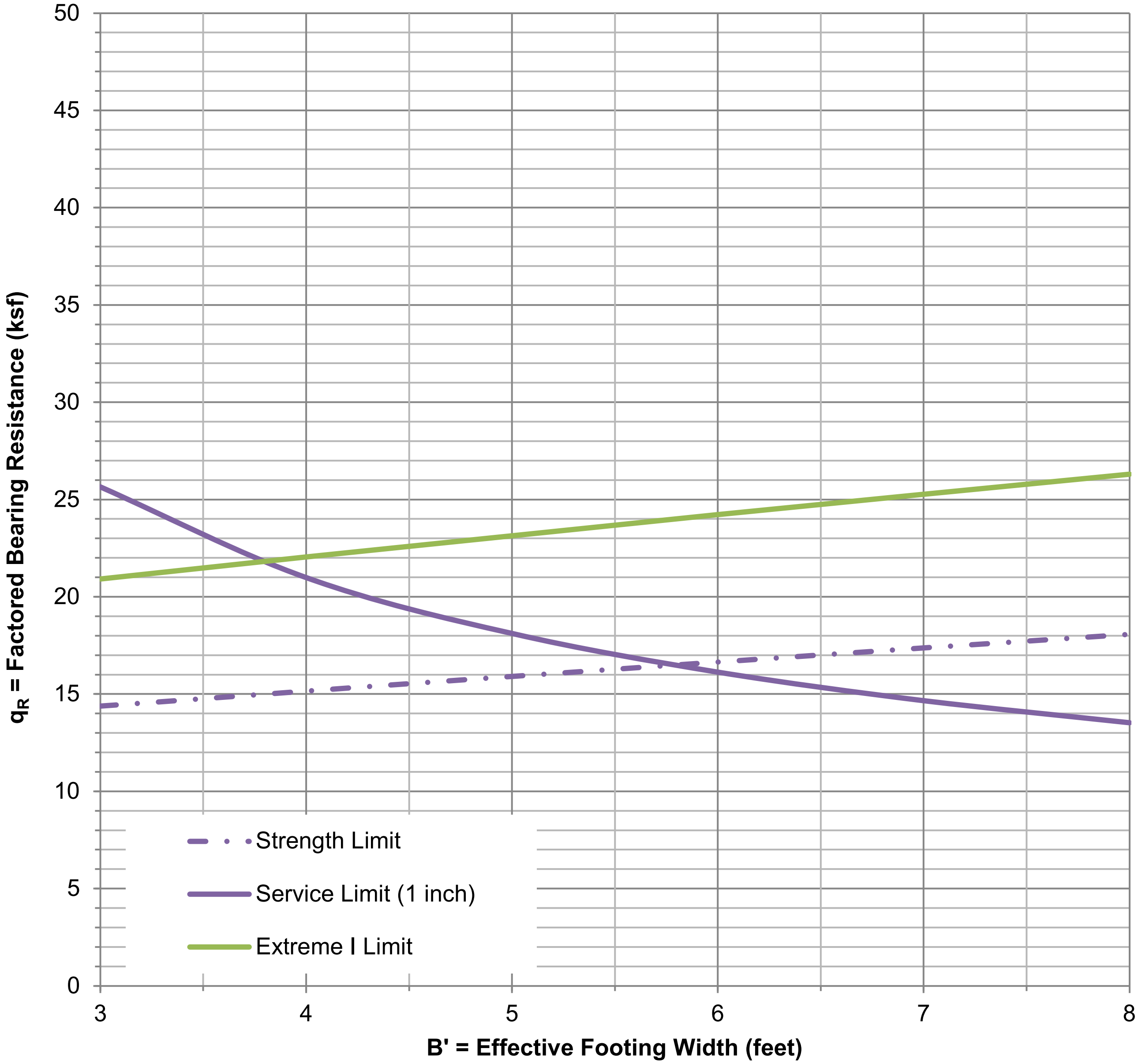
$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

s_c, s_γ, s_q = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)

d_q = correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4 (dim)

i_c, i_γ, i_q = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

Additional formulas for correction factors are provided at the back of this calculation packet.
We assumed all load inclination factors to be 1.0, rather than use the provided equations.



Notes:

1. B' represents the smallest dimension (i.e. effective footing width). Length of footing assumed to be 23.5 ft.
2. Groundwater was assumed to be 12 ft below the ground surface.
3. The strength values are based on a resistance factor of 0.55 for gravity and cantilever retaining walls, and the extreme limit values are based on a resistance factor of 1.0.
4. An embedment depth of 7.5 ft. was assumed based on local frost depth.
5. Level ground in front and behind the wingwalls was assumed (i.e., no sloping ground).

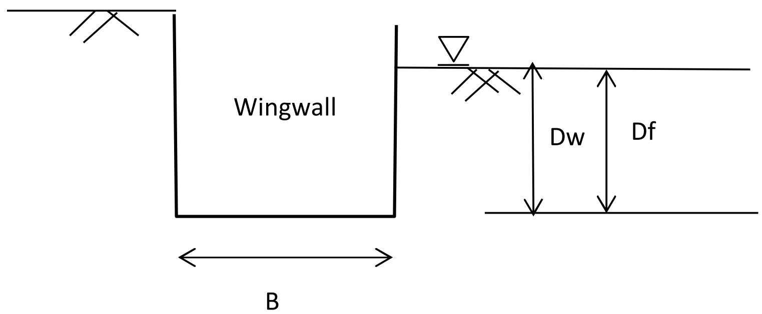
Dover Bridge Replacement Project Dover-Foxcroft, Maine WIN 23120.0	GEI Consultants	FACTORED BEARING RESISTANCE VERSUS EFFECTIVE FOOTING WIDTH - WINGWALLS ON EXISTING FILL
Thornton Tomasetti Portland, Maine	2305541	June 2025 Sheet 4

Project: Dover Bridge Replacement
Dover-Foxcroft, Maine
WIN #23120.00
Project No.: 2305541

Prepared: G. Williams
Date: 6/19/2025
Checked: N. Betancur
Date: 6/24/2025

FACTORED BEARING RESISTANCE FOR FOOTINGS ON EXISTING FILL

Note: All references are to AASHTO LRFD Bridge Design Specifications, unless otherwise noted. See attached sheets with applicable table and equation references.



RESISTANCE FACTORS

Strength Limit	0.55
Extreme I Limit	0.8
Service Limit	1.0

BEARING SOIL PROPERTIES/SUBSURFACE INFORMATION

Bearing Soil Type		Ex Fill
Unit Weight of Bearing Soil (γ)	pcf	125
Cohesion of Bearing Soil (c)	psf	0
Friction Angle of bearing Soil (ϕ')	°	32
Es, Modulus of Elasticity	ksi	12
ν, poissons ratio		0.33
Depth to Groundwater, Dw	ft	12.0
Bearing Capacity Factor (N_c)		35.5
Bearing Capacity Factor (N_q)		23.2
Bearing Capacity Factor (N_γ)		30.2

FOOTING GEOMETRY

Bottom of Footing Elevation (NAVD 88)	ft	330.0	approximate
Minimum Footing Depth (D_f)	ft	7.5	
Footing Length (L)	ft	23.5	*Per 2025-02-25 Progress Set

Effective Width, B' (B' = B - 2e)	ft	3.0	4.0	5.0	6.0	7.0	8.0
Effective Length, L' =L	ft	23.5	23.5	23.5	23.5	23.5	23.5
L'/B'		7.8	5.9	4.7	3.9	3.4	2.9
Df/B'		2.5	1.9	1.5	1.3	1.1	0.9
A'	sf	70.5	94.0	117.5	141.0	164.5	188.0
βz		1.33	1.26	1.21	1.19	1.16	1.15

BEARING RESISTANCE EQUATION FACTORS/COEFFICIENTS

Effective Width, B' (B' = B - 2e)	ft	3.0	4.0	5.0	6.0	7.0	8.0
N_{cm}		38.4	39.4	40.4	41.4	42.4	43.4
Shape Correction Factor (s_c)		1.08	1.11	1.14	1.17	1.19	1.22
Load Inclination Factor (i_c)		1.0	1.0	1.0	1.0	1.0	1.0
N_{qm}		25.0	25.6	26.3	26.9	27.5	28.1
Shape Correction Factor (s_q)		1.08	1.11	1.13	1.16	1.19	1.21
Load Inclination Factor (i_q)		1.0	1.0	1.0	1.0	1.0	1.0
Depth Correction Factor (d_q)		1.0	1.0	1.0	1.0	1.0	1.0
$N_{\gamma m}$		28.7	28.2	27.6	27.1	26.6	26.1
Shape Correction Factor (s_γ)		0.95	0.93	0.91	0.90	0.88	0.86
Load Inclination Factor (i_γ)		1.0	1.0	1.0	1.0	1.0	1.0
Groundwater Coefficient, C_{wq}		1.0	1.0	1.0	1.0	1.0	1.0
Groundwater Coefficient, $C_{w\gamma}$		0.5	0.5	0.5	0.5	0.5	0.5

CALCULATED BEARING RESISTANCES

Nominal Bearing Resistance (q_n, ksf)	26.1	27.6	28.9	30.3	31.6	32.9
Strength Limit Factored Bearing Resistance (CIP): q_R (ksf)	**14.4**	**15.2**	**15.9**	**16.7**	**17.4**	**18.1**
Extreme I Limit Factored Bearing Resistance (CIP): q_R (ksf)	**20.9**	**22.0**	**23.1**	**24.2**	**25.3**	**26.3**
Service Limit Bearing, qo, for 1 inch (Factored) (ksf)	**25.7**	**21.0**	**18.1**	**16.1**	**14.7**	**13.5**

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)

Soil Type	Typical Range of Young's Modulus Values, E_s (ksi)	Poisson's Ratio, ν (dim)
Clay:		
Soft sensitive		0.4–0.5 (undrained)
Medium stiff to stiff	0.347–2.08 2.08–6.94	
Very stiff	6.94–13.89	
Loess	2.08–8.33	0.1–0.3
Silt	0.278–2.78	0.3–0.35
Fine Sand:		
Loose	1.11–1.67	0.25
Medium dense	1.67–2.78	
Dense	2.78–4.17	
Sand:		
Loose	1.39–4.17	0.20–0.36
Medium dense	4.17–6.94	
Dense	6.94–11.11	0.30–0.40
Gravel:		
Loose	4.17–11.11	0.20–0.35
Medium dense	11.11–13.89	
Dense	13.89–27.78	0.30–0.40

Estimating E_s from *SPT N* Value	
Soil Type	E_s (ksi)
Silts, sandy silts, slightly cohesive mixtures	0.056 $N1_{60}$
Clean fine to medium sands and slightly silty sands	0.097 $N1_{60}$
Coarse sands and sands with little gravel	0.139 $N1_{60}$
Sandy gravel and gravels	0.167 $N1_{60}$
Estimating E_s from q_c (static cone resistance)	
Sandy soils	0.028q_c

Table 10.6.3.1.2a-2—Coefficients C_{wq} and $C_{w\gamma}$ for Various Groundwater Depths

D_w	C_{wq}	$C_{w\gamma}$
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

Table 10.6.3.1.2a-3—Shape Correction Factors s_c, s_γ, s_q

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c, s_γ, s_q	$\phi_f = 0$	$1+\left(\frac{B}{5L}\right)$	1.0	1.0
	$\phi_f > 0$	$1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	$1-0.4\left(\frac{B}{L}\right)$	$1+\left(\frac{B}{L}\tan\phi_f\right)$

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

Friction Angle, ϕ_f (degrees)	D_f/B	d_q
32	1	1.20
	2	1.30
	4	1.35
	8	1.40
37	1	1.20
	2	1.25
	4	1.30
	8	1.35
42	1	1.15
	2	1.20
	4	1.25
	8	1.30

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, ϕ_f. Information beyond the range indicated is not available at this time.

$$S_e = \frac{\left[q_o\left(1-\nu^2\right)\sqrt{A'}\right]}{144\ E_s\beta_z} \qquad (10.6.2.4.2\text{-}1)$$

where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of E_s are not available from the results of in situ or laboratory tests (ksi)

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	φ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *CPT*	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using *SPT*	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	φ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	φ_{ep}	Passive earth pressure component of sliding resistance	0.50

10.5.5—Resistance Factors

10.5.5.1—Service Limit States

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in Article 11.6.2.3.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

10.5.5.3—Extreme Limit States

10.5.5.3.1—General

Design of foundations at extreme limit states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

10.5.5.3.2—Scour

The provisions of Articles 2.6.4.4.2 and 3.7.5 shall apply to the changed foundation conditions resulting from scour. Resistance factors at the strength limit state shall be taken as specified herein. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

C10.5.5.3.2

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2005).

10.5.5.3.3—Other Extreme Limit States

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

C10.5.5.3.3

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article 10.5.5.2.3).

Table 11.5.6-1—Resistance Factors for Permanent Retaining Walls

Wall-Type and Condition		Resistance Factor
Nongravity Cantilevered and Anchored Walls		
Axial compressive resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		0.75
Pullout resistance of anchors (1)	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 (1) 0.70 (1) 0.50 (1)
Pullout resistance of anchors (2)	• Where proof tests are conducted	1.0 (2)
Tensile resistance of anchor tendon	• Mild steel (e.g., ASTM A615 bars) • High strength steel (e.g., ASTM A722 bars)	0.90 (3) 0.80 (3)
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls, Gravity Walls, and Semi-Gravity Walls		
Bearing resistance	• Gravity and semi-gravity walls • MSE walls	0.55 0.65
Sliding		1.0
Tensile resistance of metallic reinforcement and connectors	Strip reinforcements (4) • Static loading • Combined static/earthquake loading Grid reinforcements (4) (5) • Static loading • Combined static/earthquake loading	 0.75 1.00 0.65 0.85
Tensile resistance of geosynthetic reinforcement and connectors	• Static loading • Combined static/earthquake loading	0.90 1.20
Pullout resistance of tensile reinforcement	• Static loading • Combined static/earthquake loading	0.90 1.20
Prefabricated Modular Walls		
Bearing		Article 10.5 applies
Sliding		Article 10.5 applies
Passive resistance		Article 10.5 applies

11.5.7—Resistance Factors—Service and Strength

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in Article 11.6.2.3.

For the strength limit state, the resistance factors provided in Table 11.5.7-1 shall be used for wall design, unless region specific values or substantial successful experience is available to justify higher values.

11.5.8—Resistance Factors—Extreme Event Limit State

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state.

For overall stability of the retaining wall when earthquake loading is included, a resistance factor, ϕ, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE walls.

For tensile resistance of metallic reinforcement and connectors, when earthquake loading is included, the following resistance factors shall be used:

- Strip reinforcements, $\phi = 1.0$
- Grid reinforcement, $\phi = 0.85$

Table 11.5.7-1 Notes 4 and 5 also apply to these resistance factors for metallic reinforcements.

For tensile resistance of geosynthetic reinforcement and connectors, a resistance factor, ϕ, of 1.20 shall be used.

For pullout resistance of metallic and geosynthetic reinforcement, a resistance factor, ϕ, of 1.20 shall be used.

D.7 Rock Socketed Piles – Abutment 2

1.0 PURPOSE:

The purpose of this calculation is to analyze the soil-structure-interaction behavior of the group foundation subject to lateral and axial loads and to estimate the demands on the HP pile foundations element for abutment no. 2 of the proposed Dover Bridge Replacement in Foxcroft, Maine. We used the computer program FB-MultiPier v6.1.2 by Florida Bridge Software Institute for the modeling of the substructure pile cap, foundations, sub-surface profile, and applied loading. The input from the superstructure and approach slab was developed by Thorton Tomasetti's structural team and provided to GEI on 4/29/2025. GEI developed the self-weight of the abutment and earth loads as well as earthquake loads for extreme event.

2.0 ELEVATION DATUM

Elevations used in this document are in feet and are referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29).

3.0 ASSUMPTIONS AND INPUT:

- Assumed 3 ft thick concrete cap with f'c = 5 ksi and E_c = 4030 ksi.
- HP14X89 steel piles extending 10 ft into highly weathered rock. We assumed 1/8" section loss due to corrosion of the steel pile.
- Water Table at El +326 ft.

4.0 DESIGN INPUT FOR LATERAL ANALYSES

4.1 Pile Cap Properties

Table 1 Pile Cap Properties

Bottom of Concrete Cap Elevation (ft)	Concrete Cap Midplane Elevation (ft)	Young's Modulus (ksi)	Poisson's Ratio	Thickness (ft)	Unit Weight of Concrete (pcf)	Pile Cap Dimensions (ft)	
						Xp Direction	Yp Direction
325	326.5	4030	0.3	3	150	10	59.4

ksi = kips per square inch; pcf = pounds per cubic foot.

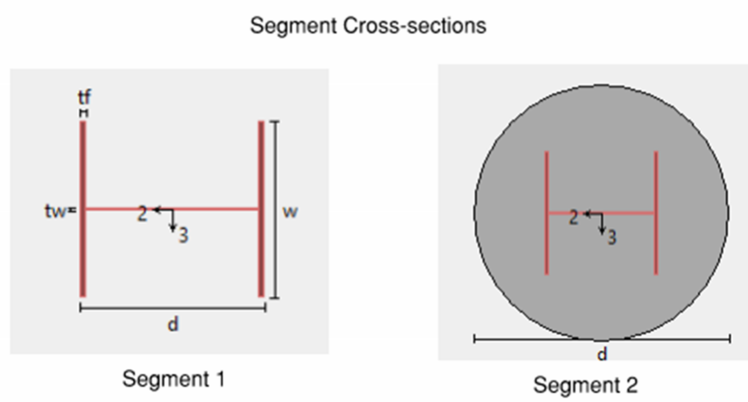
4.2 Pile Properties

Table 2 Pile Properties

Pile Type	Length (ft)		Unit Weight (pcf)		HP 14X89 Section Dimension (After 1/8" section loss)
	Segment 1	Segment 2	Segment 1	Segment 2	
Pile Type 1	17.5	10	490	150	Width = 14.625 in Web Thickness = 0.495 in Depth = 13.75 in Flange Thickness = 0.495 in
Pile Type 2	10.5	10			
Pile Type 3	3.5	10			

Note:

- Pile head to pile cap connection is assumed fixed.
- Pile types correspond to different pile lengths according to the estimated variable top of rock elevations. Segment 1 consists of HP 14X89 section and Segment 2 consists of HP 14X89 section embedded in a 10-ft-long by 30-in-diameter grouted rock socket.
- For steel pile; Yield stress = 50 ksi, Young's Modulus = 29,000 ksi and poisson's ratio = 0.3.
- For rock socket; Compressive strength of grout = 4 ksi , concrete modulus = 3605 ksi and Poisson's ratio = 0.2.



4.3 Rock Properties

We analyzed the foundation considering scour down to the estimated top of bedrock. Overburden soils are ignored. Table 3 provides rock properties used in FB-MultiPier for the subsurface profiles presented below.

Table 3 Soil and Rock Properties used in FB-MultiPier Analyses.

	Layer 1
	Bedrock
Total Unit Wt. (pcf)	175
Friction Angle (deg)	-
Unconfined Compressive Strength (psf)	1,249,420
GSI	59
mi	7
Intact Modulus (ksi)	740
Soil Type	Rock
Lateral Soil Model	Massive Rock
Axial Soil Model	Driven Pile (McVay)
Nominal Unit Side Friction (psf)	13,000
Torsional Soil Model	Hyperbolic
Shear Modulus (ksi)	301
Tip Soil Model	Driven Pile Mcvay
Small Strain Shear Modulus (ksi)	301
Poisson's Ratio	0.23
Nominal Tip Resistance (kips)	2,433

deg = degrees; pcf = pounds per cubic foot; pci = pounds per cubic inch; psf = pounds per square foot; ksi = kips per square inch.

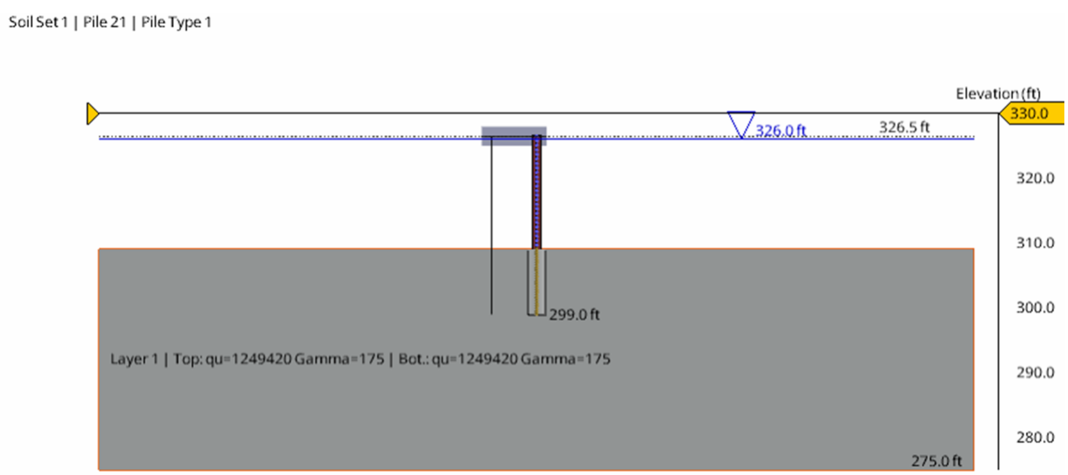
4.4 Subsurface Profile:

Table 4 provides the soil layer data used to create the subsurface profiles presented below.

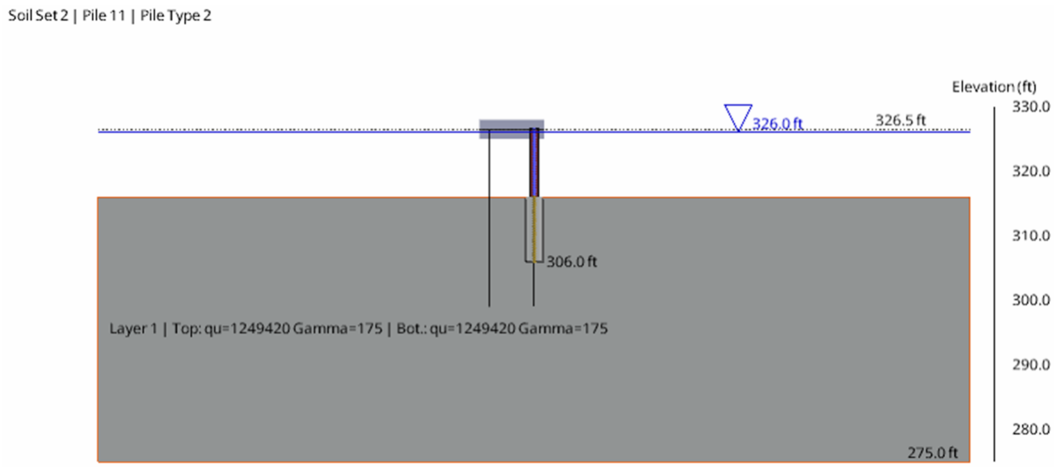
Table 4 Soil Layer used in FB-Multipier Analyses.

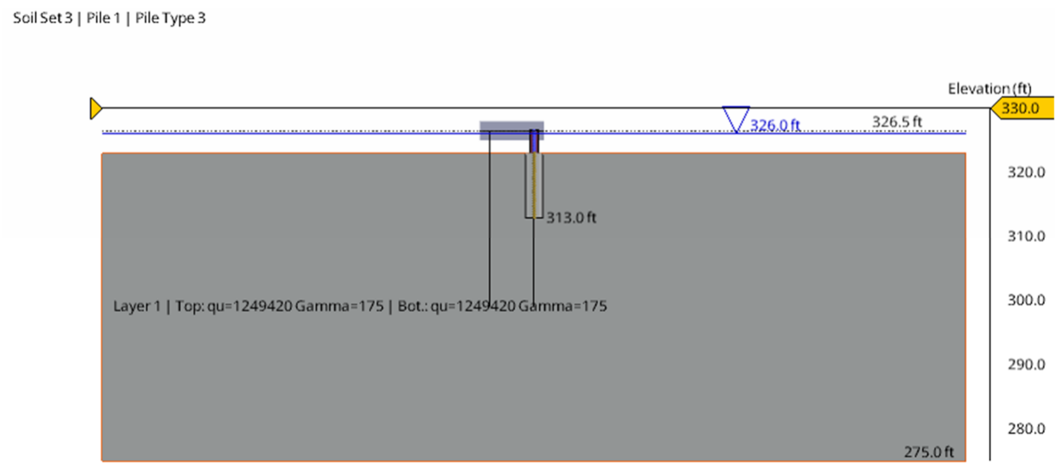
	Soil Set 1 (Based on BB-202)	**Soil Set 2 (interpolated between the two borings)**	**Soil Set 3 (Based on BB-103)**
Top of Bedrock El.	309	316	323
GWT El.	326	326	326
Applicable Piles	Pile 17 through Pile 24	Pile 9 through Pile 16	Pile 1 through Pile 8

Soil profiles developed for this analysis are provided below.



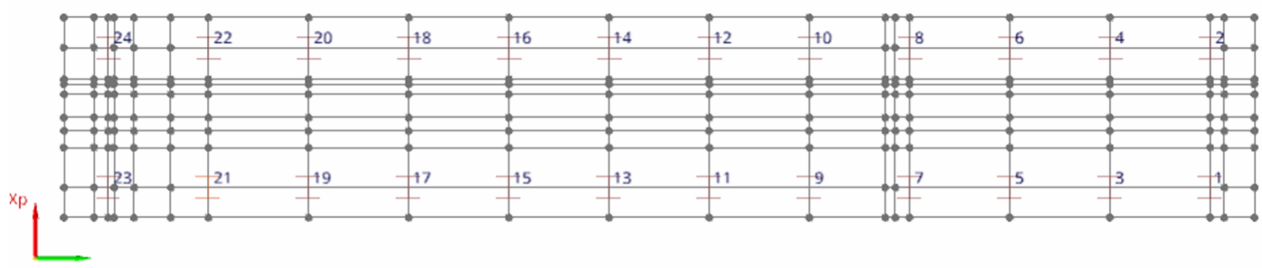
	Client	Thornton Tomasetti			Page	5
	Project	MaineDOT Dover Bridge Final Design			Pg. Rev.	1
GEI Consultants	By	M. Ahmed	Chk.	N. Betancur	App.	
	Date	8/25/2025	Date	8/25/2025	Date	
Project No.	2305541	Document No.	N/A			
Subject	Abutment No 2 Lateral Analysis Calculation Package					



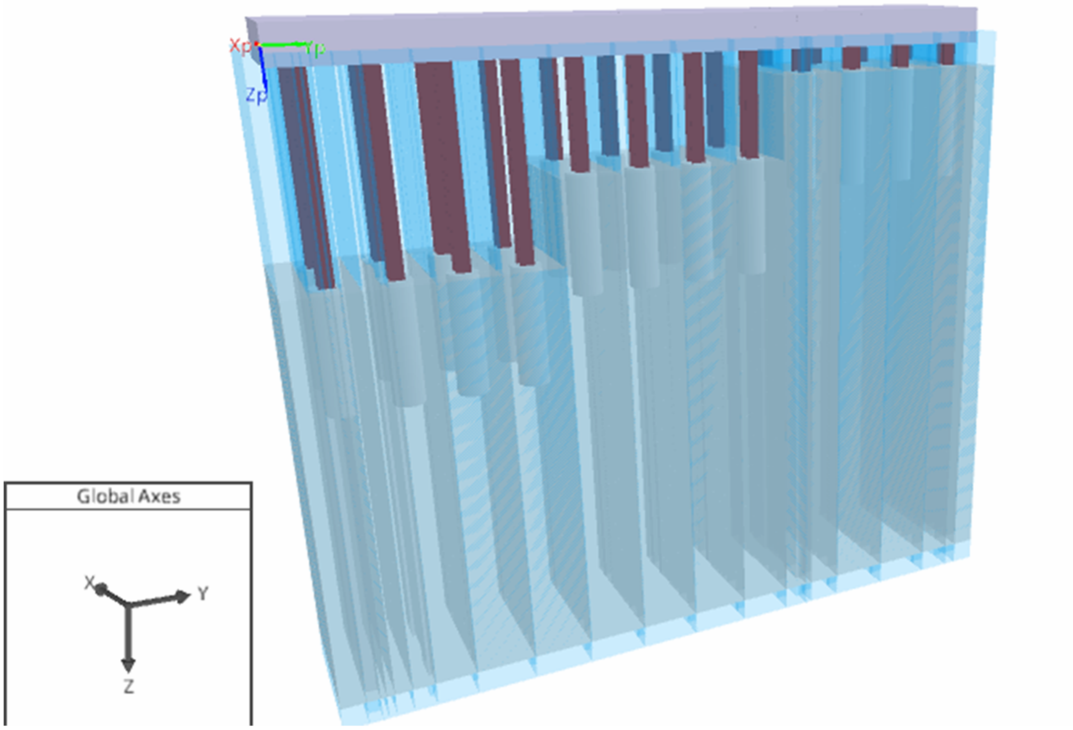


4.5 Layout Diagram

Total of 24 HP14 x 89 Piles (2 rows of 12 piles each) were used to model the abutment (see pile properties for more detail).



Center to center Pile Spacing along Yp direction	60	in
Center to center Pile Spacing along Xp direction	84	in



4.6 Load Combinations

Thornton Tomasetti provided loads from the superstructure and approach slab along with section details for abutment No.2 by email dated 4/29/2025. Based on Abutment No. 2 plan and elevation on the progress set drawings dated 02/25/2024, we calculated the total load per linear foot of the footing to be applied to the FBMP model. We considered 5 load combinations for our analysis and calculated loads using appropriate load factors for each load combination as shown below

Load Combination 1	Service I	=DC(SS)*1+DW(SS)*1+LL(SS)*1+DC*1+LL*1+EV*1+EH*1+LS*1
Load Combination 2	Strength I (no Superstructure)	=DC*1.25+LL*1.75+EV*1.35+EH*1.5+LS*1.75
Load Combination 3	Strength I (w/ superstructure)	=DC(SS)*1.25+DW(SS)*1.5+LL(SS) *1.75+DC*1.25+ LL*1.75+EV*1.35+EH*1.5+LS*1.75
Load Combination 4	Extreme I (EQ) (Pae+0.5Pir)	=DC*1+DW*1+LL*1+EV*1+EH*1+LS*1+0.5*EQ PIR *1
Load Combination 5	Extreme I (EQ) (0.5Pae+Pir)	=DC*1+DW*1+LL*1+EV*1+0.5*EH*1+0.5* LS*1+EQ PIR *1

Load factors in red font

Load combinations 2 and 3 correspond to Strength I limit state without and with the load from superstructure, respectively. We evaluated two cases for Extreme Events, load combinations 4 and 5, considering 100 percent of seismic earth pressure and 50 percent of wall inertial force and vice versa as per 2020 AASHTO LRFD Bridge Design Specifications, Ninth Edition, Section 11.6.5.

Loads provided and detailed load calculation is shown in Attachment 1: Load Calculation for FB-Multipier. We modeled loads in FB-MultiPier as distributed loads per linear foot along the pile cap.

4.7 FB-MultiPier Generated Loads

FB-MultiPier internally calculated self-weight and buoyant weight of submerged elements. A self-weight factor of 1, 1.25, and 1 were assigned for Service I (Load Combination 1), Strength I (Load Combinations 2 and 3), and Extreme Event I (EQ) (Load Combinations 4 and 5), respectively. In addition, FB-MultiPier was allowed to internally calculate group effect reduction factors (p-y multipliers) to account for interaction of the group of piles relative to the pile spacing in both directions, longitudinal and transverse.

5.0 RESULTS

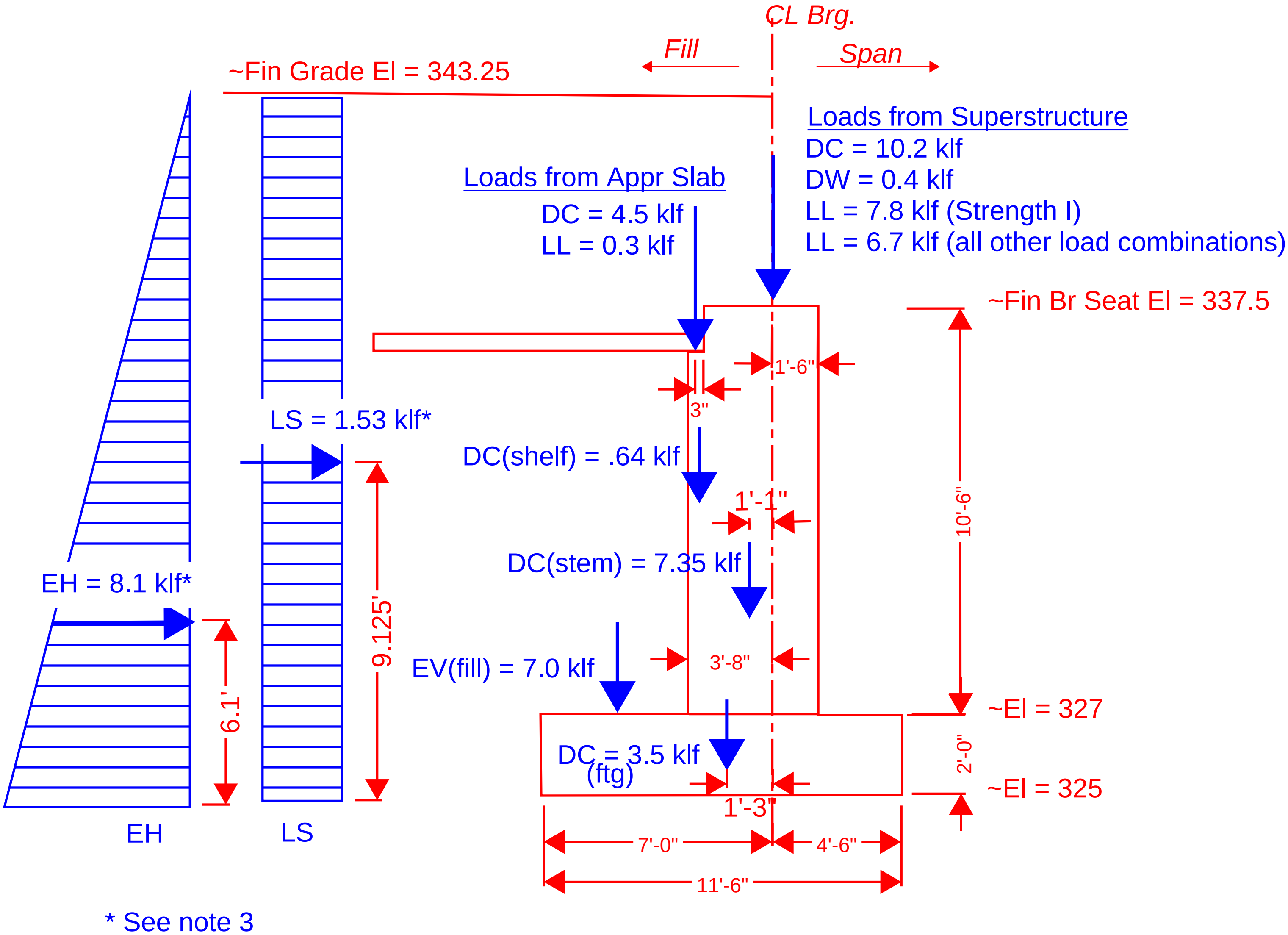
Table 5 provides a summary of the maximum reactions obtained for each load combination. Graphical output curves of deflection, bending moment, and shear along the piles is provided in Attachment 2.

Table 5 FBMP Results

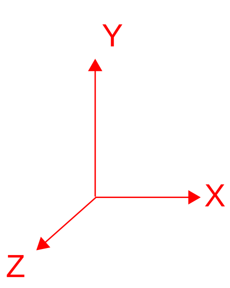
Pile Type	Load Combination	Longitudinal Deflections (inches)	Transverse Deflections (inches)	Max. Moment @ Longitudinal Axis (Kip-ft)	Max. Moment @ Transverse Axis (Kip-ft)	Max. Shear along Longitudinal Axis (Kip)	Max. Shear along Transverse Axis (Kip)	Max Axial Force (Kips) [Compression]
	FBMP Sign Convention	Lateral X	Lateral Y	Moment 2	Moment 3	Shear 2	Shear 3	Axial
Pile Type 1	LC 1	-0.44	-0.04	-4	88	16	-1	118
	LC 2	-0.57	-0.05	-6	140	26	-1	108
	LC 3	-0.59	-0.06	-7	143	26	-1	176
	LC 4	-0.47	-0.04	-4	113	21	-1	87
	LC 5	-0.31	-0.03	-3	76	14	-1	70
Pile Type 2	LC 1	-0.25	-0.04	8	126	23	-2	163
	LC 2	-0.33	-0.05	14	196	36	-3	130
	LC 3	-0.34	-0.05	14	200	37	-3	242
	LC 4	-0.26	-0.04	11	158	29	-2	106
	LC 5	-0.18	-0.03	7	106	19	-1	89
Pile Type 3	LC 1	-0.08	-0.03	48	178	-80	20	140
	LC 2	-0.10	-0.04	68	-241	-110	29	138
	LC 3	-0.11	-0.05	70	246	-113	30	204
	LC 4	-0.08	-0.03	54	197	-90	23	110
	LC 5	-0.05	-0.02	36	133	-61	15	87

ATTACHMENT 1: LOAD CALCULATION FOR FB-MULTIPIER

This loading diagram was originally provided to GEI by Thornton Tomasetti and further revised by GEI during design of the abutment 2 foundation. Unfactored superstructures loads are unchanged from those provided to GEI. Substructure DC, EV, EH, and LS loads were revised by GEI based on final design geomtery.



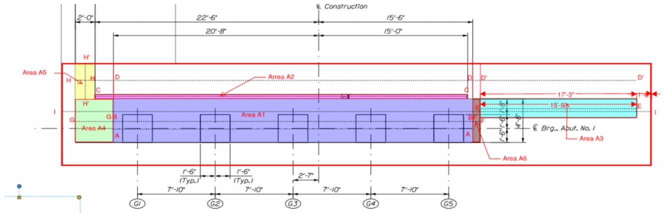
Section - Abutment No. 2
Not to Scale



Notes:
1. All loads are unfactored
2. Loads are based on geometry assumptions shown in sketch.
Loads will change with any changes to proposed geometry.
3. Horizontal Earth Pressure (EH) and Live Load Surcharge (LS) were calculated based on the following:
   EH = 0.36*135pcf*18.25'*18.25'/2 = 8.1 klf
   LS = 0.31*135pcf*2'*18.25' = 1.53 klf

Loads to GEI 4/29/2025

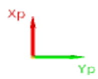
GEI Consultants	Project: MaineDOT Dover Bridge Final Design Project No: 2305541 Subject: Abutment No 2 Load Calculation	Prepared By: S. Poudyal Date: 5/1/2025 Checked By: N. Betancur Date: 5/30/2025



Geometry:

Final Grade Elevation	343.25	ft
Final Bridge Seat Elevation	337.5	ft
Top of Pile Cap/footing Elevation	328	ft
Bottom of Pile Cap/footing Elevation	325	ft

Unit wt of Concrete 150 pcf



Stem — Area A1

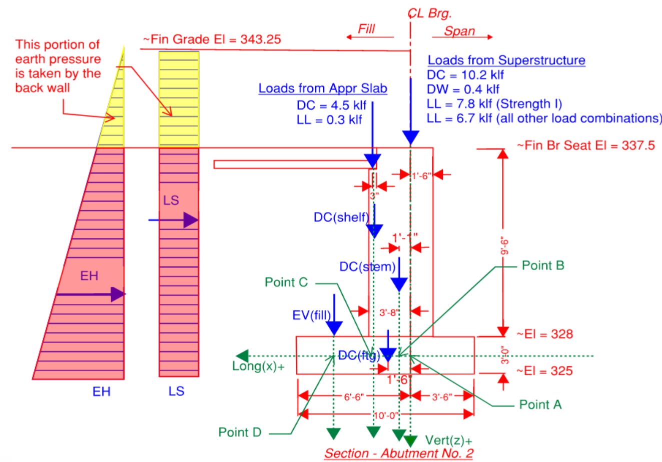
width of Stem	4.67	ft	(Fixed)
Height of Stem	9.5	ft	
Weight of Stem wall per linear foot, DC (Stem)	6.65	klf	

Shelf — Area A2

width of Shelf	0.5	ft	(Fixed)
Indent depth on Stem for approach slab	2	ft	(From Progress set)
Height of shelf	7.5	ft	
Weight of shelf wall per linear foot, DC (Shelf)	0.5625	klf	

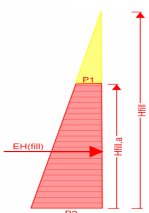
Footing

Heel Width	2.83		
Stem + Shelf width	5.17		(Fixed)
Toe Width	2.00		
Total width of Footing	10.00	ft	
Height of Footing	3	ft	
Weight of footing per linear foot, DC (Footing)	4.4985	klf	

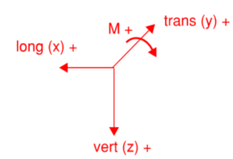


Backfill soil

H_{fill}	18.25	ft	
Height of fill under the top of bridge seat, H_{fill},a	12.5		
γ_{fill}	125	pcf	
$\Phi_{fill,}$	32	deg	
$k_{a\ (Rankine)}$	0.31		
kh	0.089		
Kae	0.34		
Equivalent height of soil for live load surcharge , H_{eq}	2	ft	(MaineDOT Bridge Design Guide Table 3-4 Section 3.6.8)



Sign Convection



Unfactored Earth Pressures on abutment

	Service I Strength I	Extreme I (EQ)	
Ptop= K * γfill * (Hfill-Hfill,a)	*0.22*	*0.24*	ksf
Pbottom =K * γfill * Hfill	*0.71*	*0.78*	ksf
EH(fill) = 0.5(Ptop+Pbottom)*Hfill,a	*5.81*	*6.38*	klf
LS(fill) = K * γfill* Heq*Hfill,a	0.97	1.06	klf

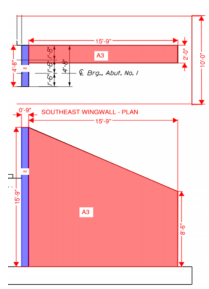
Vertical earth pressure on the heel (excluding heel at the back of Area A3)

Width of Heel	2.83	ft
$EV(fill)_{Stem}$	5.39	klf

WingWall-SouthEast

Area A3

End Height closer to the abutment, H1	15.75	ft	
End Height away from the abutment, H2	8.5	ft	
thickess of wingwall	2	ft	
Weight of Wingwall per linear foot at two ends, DC (Ww,1)	4.73	klf	@ H1
	2.55	klf	@ H2



Unfactored Earth Pressures on Wingwall-Southeast

	Service I Strength I	Extreme I (EQ)	
$EH(fill)_{Ww,1}$ = 0.5*K*γfill*H^2 at two ends	*6.81*	*7.47*	klf
	2.56	*2.81*	klf
$EV(fill)_{Ww,1}$ at two ends	6.56		klf
	3.54		klf
$LS(fill)_{Ww,1}$ = K * γfill* Heq*Hfill,H1	1.45	1.59	klf
$LS(fill)_{Ww,1}$ = K * γfill* Heq*Hfill,H2	0.89	0.98	

SouthEast Kicker Block

Area A6

Height	15.75	ft
Width	4.67	ft
Weight of kicker block per linear foot, DC (Kb,1)	11.03	klf

Discuss

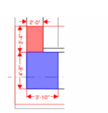
Unfactored Earth Pressures on Kicker Block -Southeast

	Service I Strength I	Extreme I (EQ)	
$EH(fill)_{Kb,1}$ = 0.5*K*γfill*H^2	6.81	7.47	klf
$EV(fill)_{Kb,1}$	6.56		klf
$LS(fill)_{Kb,1}$ = K * γfill* Heq*Hfill	1.45	1.59	klf

NorthWest Kicker Block

Portion Along the footing — Area A4

Height	16.67	ft
Width	4.67	ft
Weight of kicker block per linear foot, DC (Kb,2)	11.66666667	klf



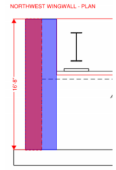
Unfactored Earth Pressures on Kicker Block-Northwest

	Service I Strength I	Extreme I (EQ)	
$EH(fill)_{Kb,2}$ = 0.5*K*γfill*H^2	*7.49*	*8.22*	klf
$EV(fill)_{Kb,2}$	5.39		klf
$LS(fill)_{Kb,2}$ = K * γfill* Heq*Hfill	1.52	1.67	klf

NorthWest Wingwall (L shaped)

Portion Perpendicular to footing — Area A5

Height	16.67	ft
Thickness	3.33	ft
Width along footing dirn	2.00	ft
Weight of wingwall per linear foot, DC (Ww,2)	0.52725	klf



Unfactored Earth Pressures on Wingwall-Northwest (perpendicular to footing)

	Service I Strength I	Extreme I (EQ)	
$EH(fill)_{Ww,2}$ = 0.5*K*γfill*H^2	*7.49*	*8.22*	klf
EV for fill behind this portion included in $EV(fill)_{Abutment}$			klf
$LS(fill)_{Ww,2}$ = K * γfill* Heq*Hfill	1.52	1.67	klf

GEI Consultants

Project: MaineDOT Dover Bridge Final Design
Project No: 2305541
Subject: Abutment No 2 Load Calculation

Prepared By: S. Poudyal
Date: 5/1/2025
Checked By: N. Betancur
Date: 5/30/2025

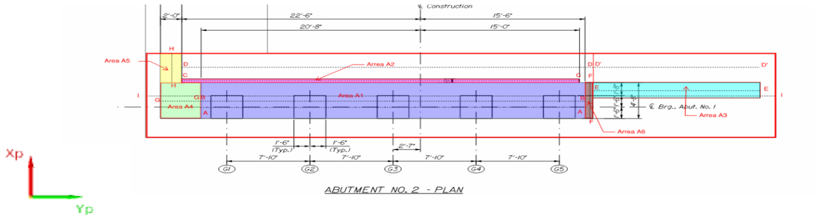
Forces per linear foot of the footing (in Kips/foot)

	Load Cases		Forces (kips/ft)			Line of application	Load Factors				Moment Arms		Remarks
			Service	Strength	Extreme		Load Case	Service	Strength I (max)	Extreme Event (EQ)	About Longitudinal Axis	About Transverse Axis	
Superstructure	1	DC (SS)	10.20	10.20		Acting vertically downward along the C/L of the bridge	DC	1	1.25	1	0	0	Applied at point A (along Line A-A) i.e, C/L of the bearings and @ mid plane of pile cap
	2	DW (SS)	0.40	0.40			DW	1	1.5	1	0	0	
	3	LL (SS) *	6.70	7.80			LL (SS) *	1	1.75	1	0	0	
Stem	4	DC (stem)	6.65	6.65	6.65	Acting vertically downward along the C/L of the Stem	DC	1	1.25	1	0	0	Applied at point B (along line B-B) i.e, C/L of the stem @ mid plane of pile cap
	5	Kh* DC (stem)**	0.00	0.00	-0.59	Acting horizontally (-Xp dirn) at 1/2 of the height of the stem	EQ PIR	0	0	1	0	-6.25	
Approach Slab	6	DC (AS)	4.50	4.50	4.50	Acting vertically downward along the C/L of the Shelf	DC	1	1.25	1	0	0	Applied at point C (along line C-C) i.e, C/L of the shelf @ mid plane of pile cap
	7	LL (AS)	0.30	0.30	0.30		LL	1	1.75	1	0	0	
	8	Kh* DC (AS)**	0.00	0.00	-0.40	Acting horizontally (-Xp dirn) at 1/2 of the height of the indent on the top of the stem	EQ PIR	0	0	1	0	-10	
Shelf	9	DC (shelf)	0.56	0.56	0.56	Acting vertically downward along the C/L of the Shelf	DC	1	1.25	1	0	0	
	10	Kh* DC (shelf)**	0.00	0.00	-0.05	Acting horizontally (-Xp dirn) at 1/2 of the height of the shelf	EQ PIR	0	0	1	0	-5.25	
Fill	11	EV (Fill)	5.39	5.39	5.39	Acting vertically downward along the C/L of the footing heel	EV	1	1.35	1	0	0	Resolved at point D (line D-D) i.e, C/L of the footing heel @ mid plane of pile cap
	12	Kh* EV (fill)**	0.00	0.00	-0.48	Acting horizontally (-Xp dirn) at 1/2 of the height of the backfill soil	EQ PIR	0	0	1	0	-9.1	
	13	EH (Fill)	-5.81	-5.81	-6.38	Acting horizontally (-Xp dirn)at trapezoidal centroid of the height of the backfill soil below bridge seat	EH	1	1.5	1	0	-3.7	
	14	LS (Fill)	-0.97	-0.97	-1.06	Acting horizontally (-Xp dirn) at 1/2 of the height of the backfill soil below the bridge seat	LS	1	1.75	1	0	-4.8	
WingWall-SouthEast	15	DC Ww,1	4.73 2.55	4.73 2.55	4.73 2.55	Acting vertically downward along the C/L of the wingwall,	DC	1	1.25	1	0	0.0	Applied along line E-E i.e, C/L of Ww1 @ mid plane of pile cap
	16	Kh *DC Ww,1**	- -	- -	-0.42 -0.23	Acting horizontally (-Xp dirn) at 1/2 of the height of the backfill	EQ PIR	-	-	1	0 0	-9.4 -5.8	
	17	EV(fill)Ww,1	6.56 3.54	6.56 3.54	6.56 3.54	Acting vertically downward along the C/L of the footing	EV	1	1.35	1	0 0	0 0	Resolved at point D (line D'-D') i.e, C/L of the footing heel @ mid plane of pile cap
	18	Kh* EV(fill)Ww,1**	- -	- -	-0.58 -0.31	Acting horizontally (-Xp dirn) at 1/2 of the height of the backfill	EQ PIR	-	-	1	0 0	-9.4 -5.8	
	19	EH (Fill) Ww,1	-6.81 -2.56	-6.81 -2.56	-7.47 -2.81	Acting horizontally (-Xp dirn) at 1/3 of the height of the backfill soil	EH	1	1.5	1	0 0	-4.8 -2.3	
	20	LS (Fill) Ww,1	-1.45 -0.89	-1.45 -0.89	-1.59 -0.98	Acting horizontally (-Xp dirn) at 1/2 of the height of the backfill soil at each end of wingwall	LS	1	1.75	1	0 0	-7.9 -4.3	
SouthEast Kicker Block	21	Kb,1 DC	11.03	11.03	11.03	Acting vertically downward along the C/L of the kickerblock ,along the footing	DC	1	1.25	1	0	0.0	Applied along line F-F i.e, C/L of Kb,1 @ mid plane of pile cap
	22	Kh *DC Kb,1**	-	-	-0.98	Acting horizontally (-Xp dirn) at 1/2 of the height of Kb,1	EQ PIR	-	-	1	0.0	-9.4	
	23	EV (Fill) Kb,1	6.56	6.56	6.56	Acting vertically downward along the C/L of the footing heel at the back of kicker block	EV	1	1.35	1	0.0	0.0	Resolved at point D (line D-D') i.e, C/L of the footing heel behind the kicker block@ mid plane of pile cap
	24	Kh*EV (Fill) Kb,1**	-	-	-0.583	Acting horizontally (-Xp dirn) at 1/2 of the height of Kb,1	EQ PIR	0	0	1	0.0	-9.4	
	25	EH (Fill) Kb,1	-6.81	-6.81	-7.47	Acting horizontally (-Xp dirn) at 1/2 of the height of Kb,1	EH	1	1.5	1	0.0	-4.8	
	26	LS (Fill) Kb,1	-1.45	-1.45	-1.59	Acting horizontally (-Xp dirn) at 1/2 of the height of Kb,1	LS	1	1.75	1	0.0	-7.9	
NorthWest Kicker Block	27	DC Kb,2	11.67	11.67	11.67	Acting horizontally (-Xp dirn) at the CG of the wall portion,along the footing	DC	1	1.25	1	0	0.0	Applied along line G-G i.e, C/L of Kb,2 @ mid plane of pile cap
	28	Kh* DC Kb,2**	-	-	-1.038333333	Acting horizontally (-Xp dirn) at 1/2 of the height of the wingwall portion	EQ PIR	-	-	1	0	-9.8	
	29	EV (Fill) Kb,2	5.39	5.39	5.39	Acting vertically downward along the C/L of the footing heel at the back of kicker block	EV	1	1.35	1	0	0.0	Resolved at point D (line H-D) i.e, C/L of the footing heel behind kb,2 @ mid plane of pile cap
	30	Kh*EV(Fill) Kb,2	-	-	-0.480127188	Acting horizontally (-Xp dirn) at 1/2 of the height of the kicker block	EQ PIR	-	-	1	0.0	-9.8	
	31	EH (Fill) Kb,2	-7.49	-7.49	-8.22	Acting horizontally (-Xp dirn) at 1/3 of the height of fill	EH	1	1.5	1	0.0	-5.1	
	32	LS (Fill) Kb,2	-1.52	-1.52	-1.67	Acting horizontally (-Xp dirn) at 1/2 of the height of the fill	LS	1	1.75	1	0.0	-8.3	
NorthWest Wingwall (L shaped)	33	DC Ww,2	0.53	0.53	0.53	Acting vertically downward along the C/L of the portion of wingwall,along the footing	DC	1	1.25	1	0	0.0	Applied along line H-H i.e, C/L of the portion of wingwall (Area A4) @ mid plane of pile cap
	34	kh* DC Ww,2**	-	-	-0.04692525	Acting horizontally (-Xp dirn) at 1/2 of the height of the wingwall portion	EQ	-	-	1	0.0	-9.8	
	35	EV (Fill) Ww,2	EV for fill behind this portion included in EV(fill)Abutment										
	36	Kh* EV(Fill) Ww,2											
	37	EH(fill) Ww,2	-7.49	-7.49	-8.22	Acting horizontally (-Yp dirn) at 1/3 of the height of the fill	EH	1	1.5	1	5.1	0.0	Applied along line H'-H' i.e, C/L of the portion of wingwall (Area A4) perpendicular to the pile cap@ mid plane of pile cap
	38	LS (Fill) Ww,2	-1.52	-1.52	-1.67	Acting horizontally (-Yp dirn) at 1/2 of the height of the fill	LS	1	1.75	1	8.3	0.0	
Footing	39	DC (Footing)					Taken care by FBMP as self weight						
	40	kh* DC (Footing)**	-	-	-0.4003665	Acting horizontally (-Xp dirn) at 1/2 of the height of the footing	EQ	-	-	1	0	0	Applied along line I-I i.e, C/L of thefooting @ mid plane of pile cap

* Superstructure LL provided by Thornton Tomasetti as factored loads for Strength I
** Seismic induced horizontal intertial forces on DC and EV components

GEI Consultants	Project: MaineDOT Dover Bridge Final Design Project No: 2305541 Subject: Abutment No 2 Load Calculation	Prepared By: S. Poudyal Date: 5/1/2025 Checked By: N. Betancur Date: 5/30/2025

Longitudinal Loads (Kips/ft)	**Xp**
Transverse Loads (Kips/ft)	**Yp**
Vertical Loads (Kips/ft)	**Zp**
Moment @ Longitudinal Axis (Kip-ft/ft)	**Mxp**
Moment @ Transverse Axis (Kip-ft/ft)	**Myp**



A. Resolving Force 1- 3 about line A-A, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp					
Yp					
Zp	17.30	0.00	27.00	0.00	0.00
Mxp					
Myp					

B. Resolving Force 4, and 5 about line B-B, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.30	-0.59
Yp					
Zp	6.65	8.31	8.31	6.65	6.65
Mxp					
Myp	0.00	0.00	0.00	1.85	3.70

C. Resolving Force 6- 10 about line C-C, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.23	-0.45
Yp					
Zp	5.36	6.85	6.85	5.36	5.36
Mxp					
Myp	0.00	0.00	0.00	2.13	4.27

D. Resolving Force 11-14 about line D-D, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	-6.78	-10.41	-10.41	-7.68	-4.20
Yp					
Zp	5.39	7.28	7.28	5.39	5.39
Mxp					
Myp	25.90	40.01	40.01	30.60	18.59

E. Resolving Force 15-16 about line E-E, Applied load per linear foot (Factored)

End 1

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.21	-0.42
Yp					
Zp	4.73	5.91	5.91	4.73	4.73
Mxp					
Myp	0.00	0.00	0.00	1.97	3.94

End 2

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.11	-0.23
Yp					
Zp	2.55	3.19	3.19	2.55	2.55
Mxp					
Myp	0.00	0.00	0.00	0.65	1.30

F. Resolving Force 17-20 about line D'-D', Applied load per linear foot (Factored)

End 1

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	-8.26	-12.76	-12.76	-9.36	-5.12
Yp					
Zp	6.56	8.85	8.85	6.56	6.56
Mxp					
Myp	43.80	68.56	68.56	50.77	29.49

End 2

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	-3.45	-5.40	-5.40	-3.95	-2.21
Yp					
Zp	3.54	4.78	4.78	3.54	3.54
Mxp					
Myp	9.77	15.60	15.60	11.62	7.17

G. Resolving Force 21 and 22 about line F-F, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.49	-0.98
Yp					
Zp	11.03	13.78	13.78	11.03	11.03
Mxp					
Myp	0.00	0.00	0.00	4.60	9.20

H. Resolving Force 23 and 26 about line D-D', Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	-8.26	-12.76	-12.76	-9.36	-5.12
Yp					
Zp	6.56	8.85	8.85	6.56	6.56
Mxp					
Myp	43.80	68.56	68.56	50.77	29.49

I. Resolving Force 27 and 28 about G-G, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.52	-1.04
Yp					
Zp	11.67	14.58	14.58	11.67	11.67
Mxp					
Myp	0.00	0.00	0.00	5.11	10.21

J. Resolving Force 29 and 32 about line H-D, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	-9.02	-13.91	-13.91	-10.13	-5.43
Yp					
Zp	5.39	7.28	7.28	5.39	5.39
Mxp					
Myp	50.59	79.06	79.06	57.84	32.46

K. Resolving Force 33 and 34 about H-H, Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.02	-0.05
Yp					
Zp	0.53	0.66	0.66	0.53	0.53
Mxp					
Myp	0.00	0.00	0.00	0.23	0.46

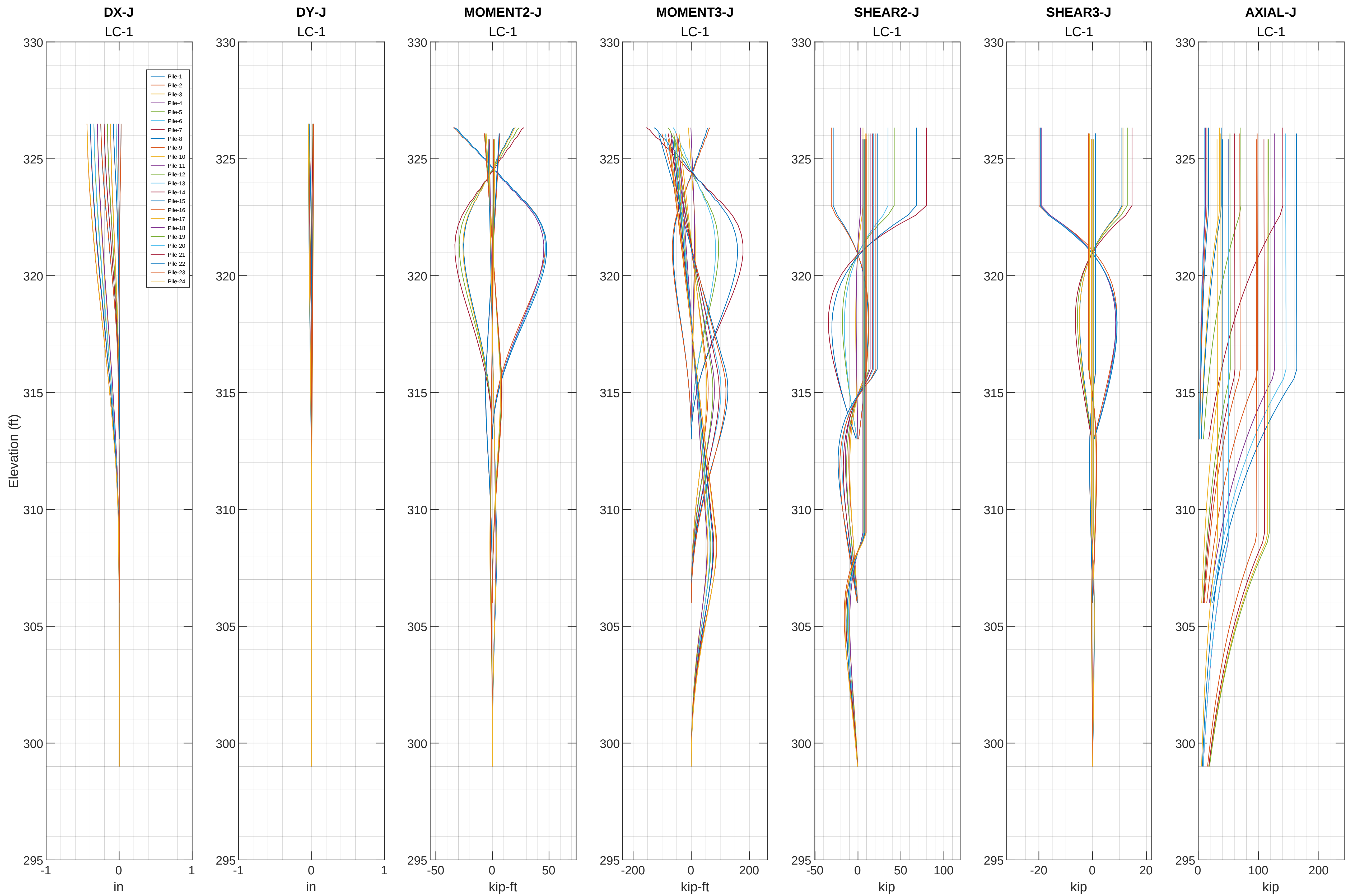
L. Resolving Force 37 and 38 about H'-H', Applied load per linear foot (Factored)

	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp					
Yp	-9.02	-13.91	-13.91	-9.89	-4.95
Zp					
Mxp	-50.59	-79.06	-79.06	-55.48	-27.74
Myp					

M. Resolving Force 40 about I-I, Applied load per linear foot (Factored)

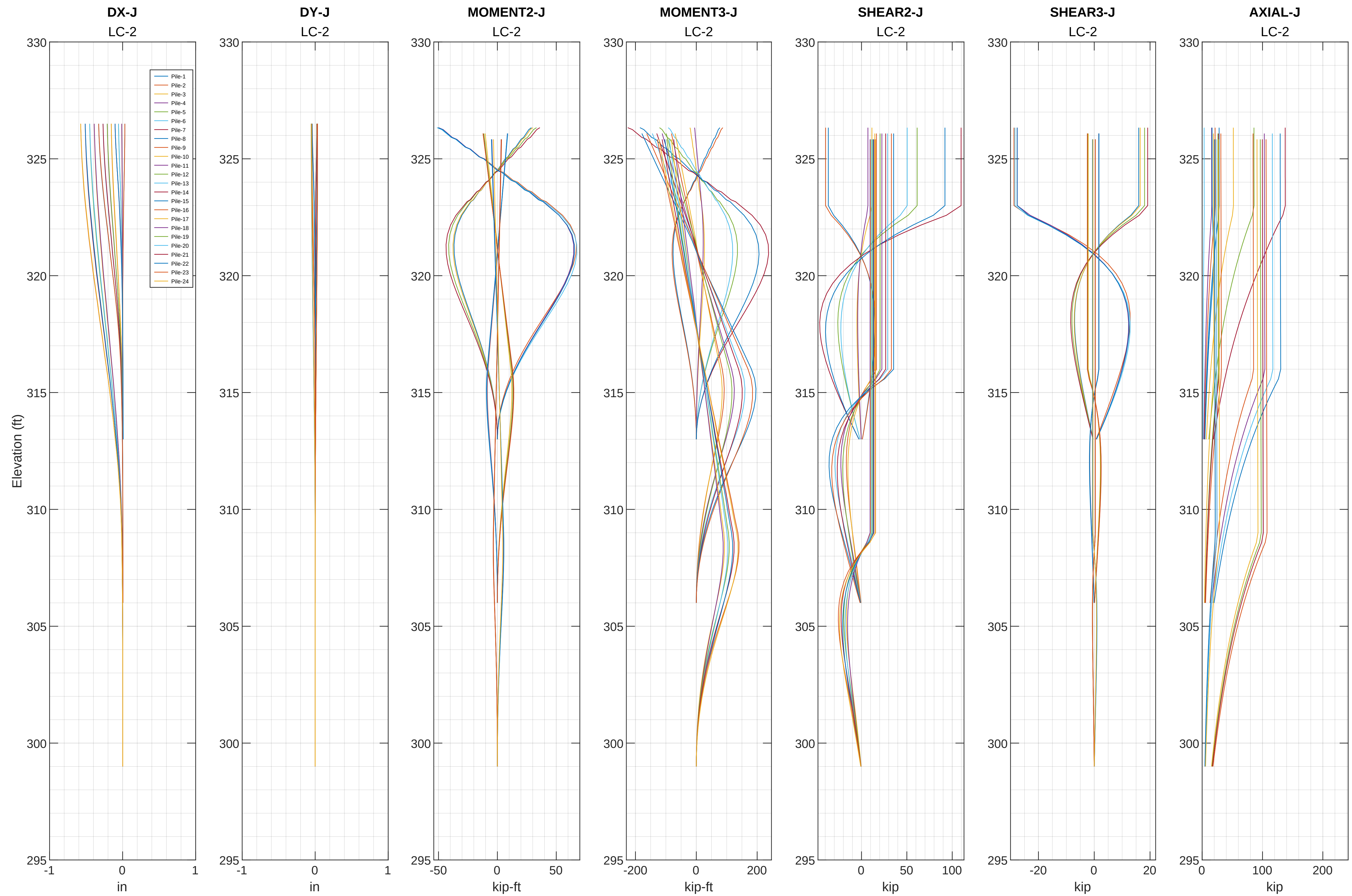
	Service	Strength I (no Superatructure)	Strength I	Extreme Event (EQ) (Pae+0.5Pir)	Extreme Event (EQ) (0.5Pae+Pir)
Xp	0.00	0.00	0.00	-0.20	-0.40
Yp					
Zp					
Mxp					
Myp					

ATTACHMENT 2: FB-MULTIPIER OUTPUT



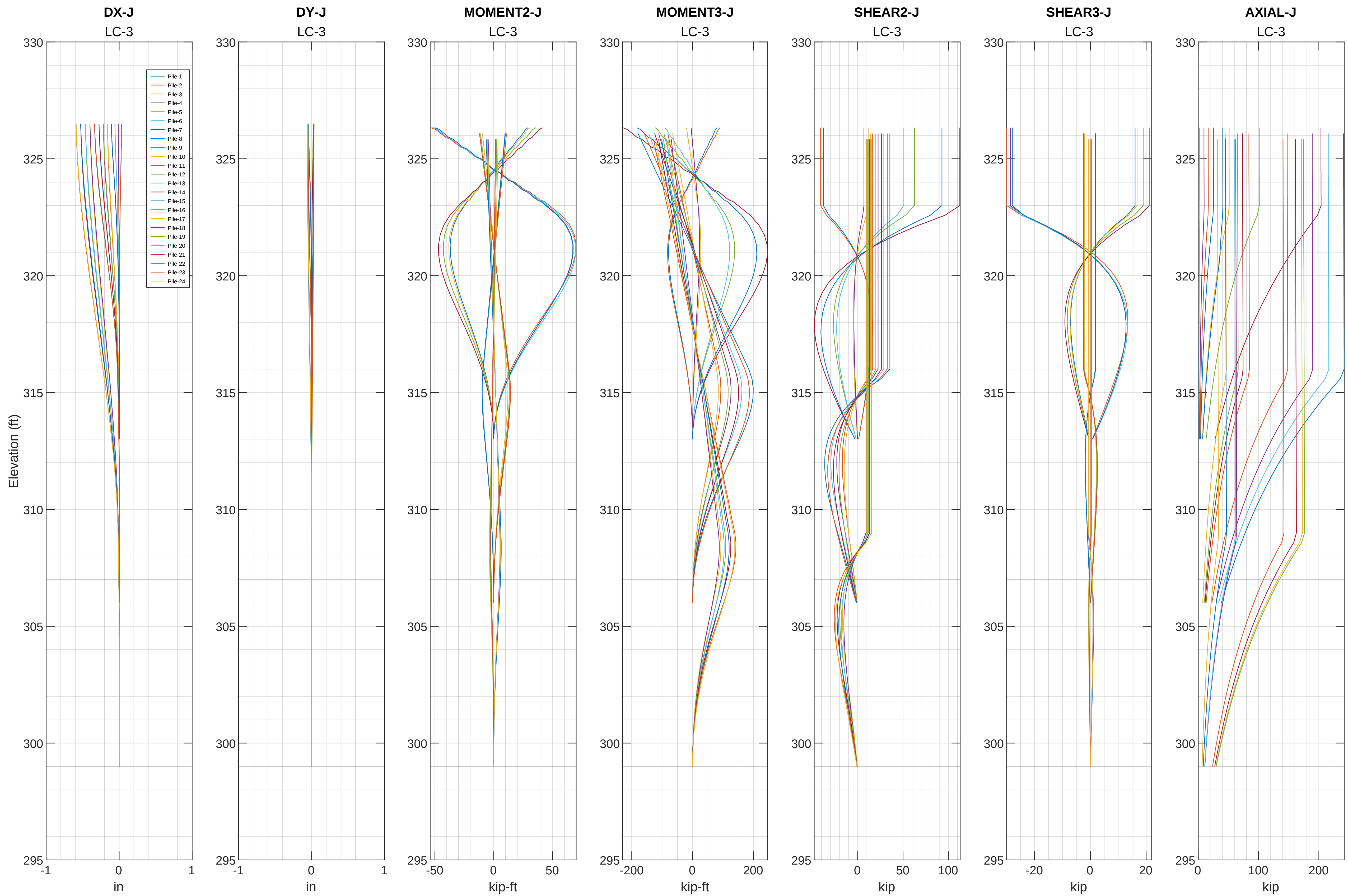

```
Loadcase     : 1
Description  : Service I
```

Dover Bridge #5118 over Piscataquis River WIN 023120.00 Dover-Foxcroft, Maine	GEI Consultants	FB-Multipier Analysis	
Thornton Tomasetti 14 York Street, Portland, ME 04101	Project 2305541	25-Aug-2025	Fig. 1



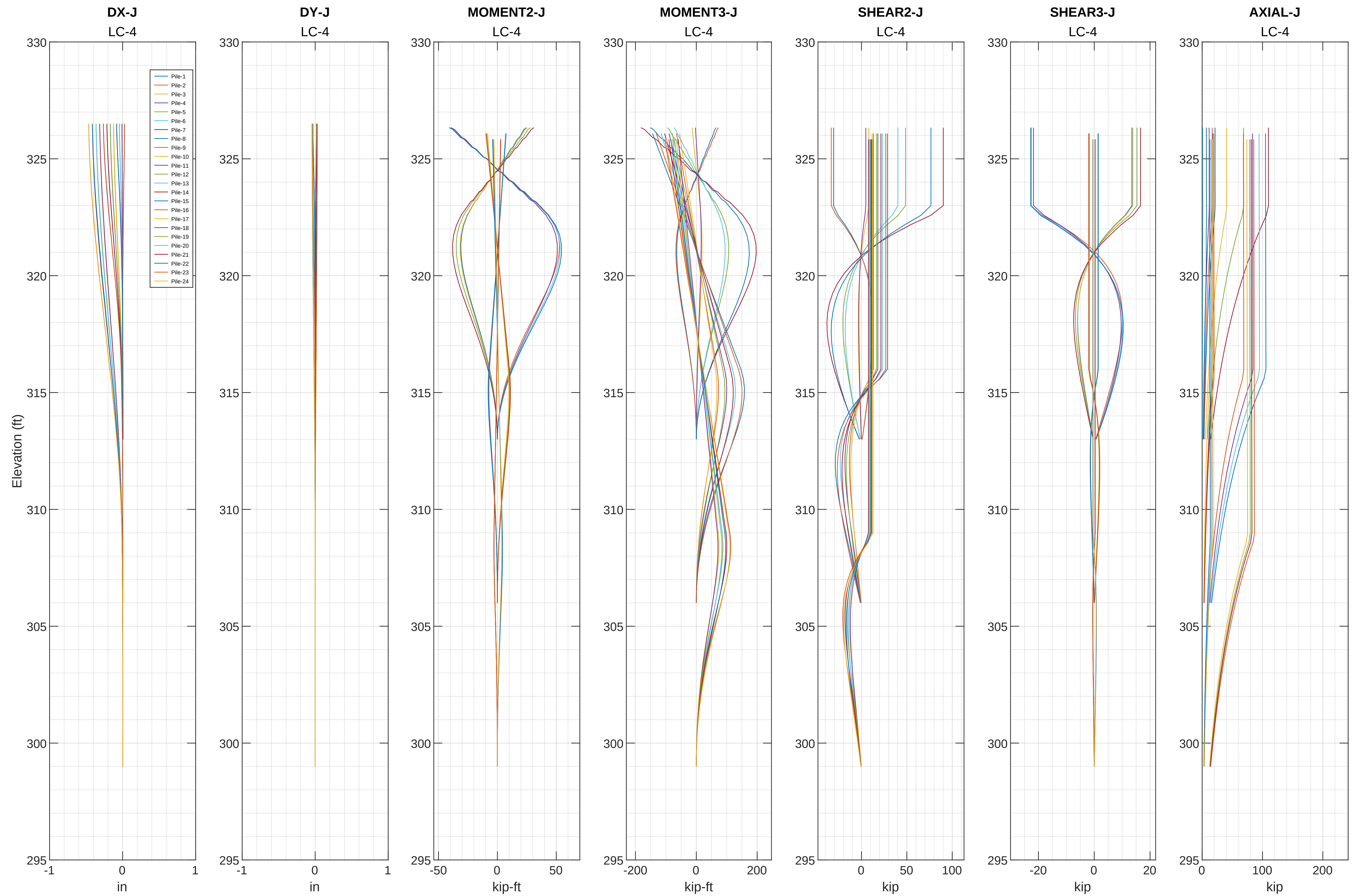

```
Loadcase     : 2
Description  : Strength I (no Superstructure)
```

Dover Bridge #5118 over Piscataquis River WIN 023120.00 Dover-Foxcroft, Maine	GEI Consultants	FB-Multipier Analysis
Thornton Tomasetti 14 York Street, Portland, ME 04101	Project 2305541	25-Aug-2025 Fig. 2




```
Loadcase     : 3
Description  : Strength I (w/ superstructure)
```

Dover Bridge #5118 over Piscataquis River WIN 023120.00 Dover-Foxcroft, Maine	GEI Consultants	FB-Multipier Analysis
Thornton Tomasetti 14 York Street, Portland, ME 04101	Project 2305541	25-Aug-2025 Fig. 3




```
Loadcase     : 4
Description  : Extreme I (EQ) (Pae+0.5Pir)
```

Dover Bridge #5118 over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Thornton Tomasetti
14 York Street, Portland, ME 04101

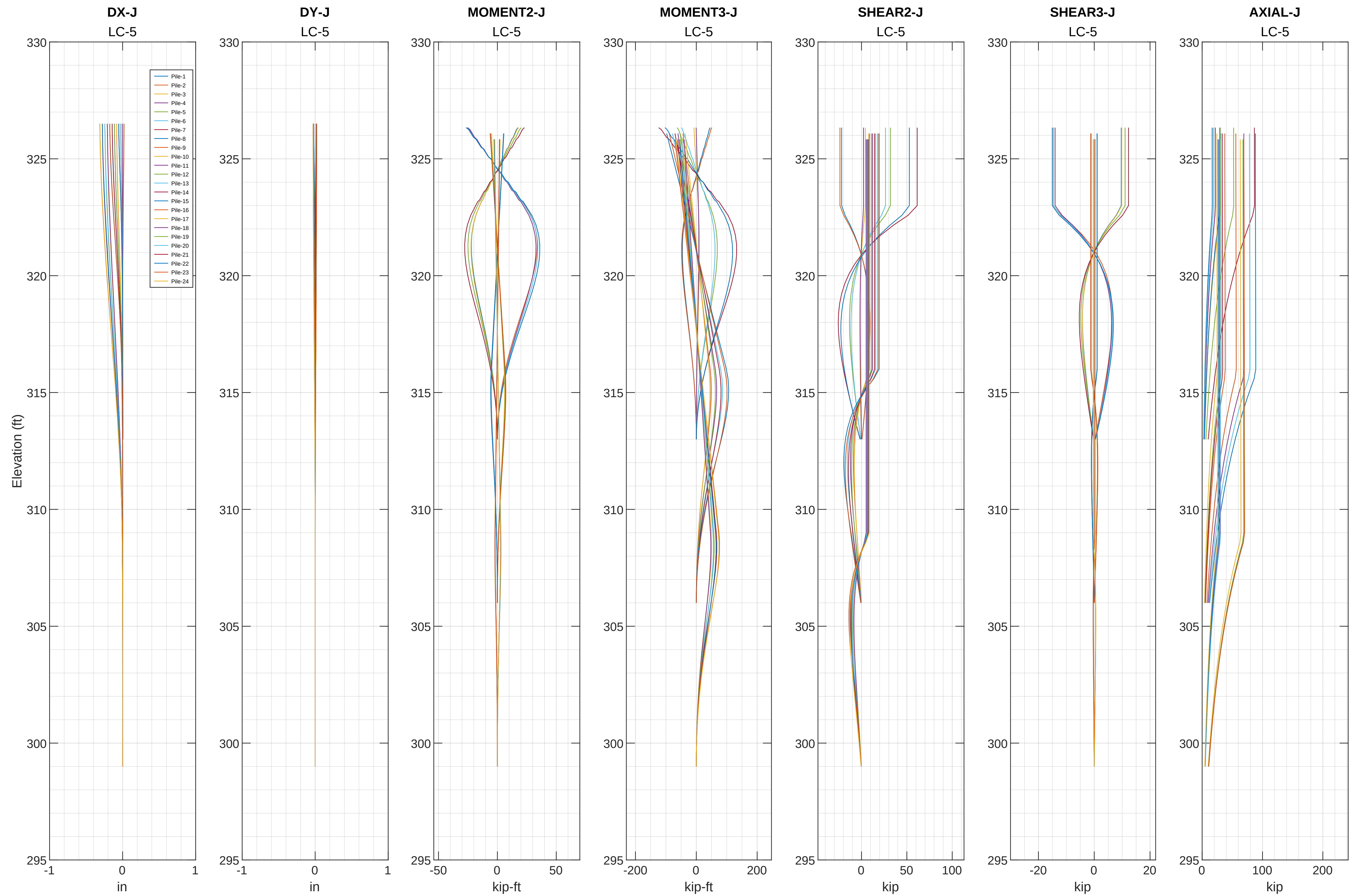
GEI Consultants

Project 2305541

FB-Multipier Analysis

25-Aug-2025

Fig. 4




```
Loadcase     : 5
Description  : Extreme I (EQ) (0.5Pae+Pir)
```

Dover Bridge #5118 over Piscataquis River
WIN 023120.00
Dover-Foxcroft, Maine

Thornton Tomasetti
14 York Street, Portland, ME 04101

GEI Consultants

Project 2305541

FB-Multipier Analysis

25-Aug-2025

Fig. 5

ATTACHMENT 3: ROCK SOCKET AXIAL RESISTANCE

Rock Socket Axial Resistance (Side Resistance)

Purpose:

The purpose of this calculation is to estimate the minimum rock socket length of 2-, 2.5, and 3-foot diameter rock sockets assuming side resistance only.

References:

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. FHWA-NHI-10-016 "Drilled Shafts: Construction Procedures and LRFD Design Methods", 2010.

Select Bedrock Field and Laboratory Data:

Bedrock field and laboratory data obtained from GM2 (2025).

Rock core descriptions and RQDs and Recovery values:

Exploration Number	Run	Penetration (in)	Recovery (in)	RQD (%)	Unconfined compressive Strength (psi)	Rock Type
BB-DFPR-101	R1	57.6	54	83		
	R2	0.2	0.2	0		
	R3	24	18	45		
	R4	48	43	73		
	R5	48	40	46	9,615	Metasiltstone
	R6	60	60	72		
BB-DFPR-102	R1	15.6	12	0		
	R2	60	60	70		
	R3	24	20.4	0		
	R4	48	48	100	7,128	Metasiltstone
	R5	60	60	100		
	R6	36	36	97		
BB-DFPR-103	R1	60	48	20		
	R2	31.2	28.8	51		
	R3	36	36	83		
	R4	51.6	42	0		
	R5	26.4	24	27		
	R6	30	15.6	30		
	R7	50.4	46	79	3,818	Metasiltstone
BB-DFPR-201A	R1	5	3	0		
	R2	7	7	0		
	R3	3	3	0		
	R4	36	30	39	15195	Metasiltstone
	R5	10	10	0		
	R6	20	20	60		
	R7	20	16	65		
	R8	30	29	43		
BB-DFPR-202	R2	41	27	12	6,632	
	R3	48	48	38		
	R4	12	10	0		
	R5	28	28	100		
BB-DFPR-203A	R1	60	58	80	9671	Metasiltstone
	R2	60	58	22		
			Average RQD=	43		

Average qu	8,677	psi
Median qu	8,372	psi
Design qu	**1205**	**ksf**

conservatively selected based on qu values

Calculation:
Assumptions:
1. Abutment to be supported on rock socketed HP piles.
2. HP pile installed in concrete rock socket.
3. Rock socket sizes: 2.0-foot-diameter rock socket
   2.5-foot-diameter rock socket
   3.0-foot-diameter rock socket
4. Axial (compression) capacity obtained from side resistance in bedrock.
5. Bottom of rock socket is cleaned out to ensure removal of loose material before concrete tremie.

Side Resistance Factors:

Service	1.00	
Strength - compression	0.55	*Side Resistance in rock, AASHTO Table 10.5.5.2.4-1 - 0.50 to 0.55 no load test*
Strength - uplift	0.40	*Side Resistance in rock, AASHTO Table 10.5.5.2.4-1 - 0.4 no load test*
Extreme	1.00	*AASHTO Section 10.5.5.3.2 and 10.5.5.3.3 - 1.0 for under extreme event*
Extreme - uplift	0.80	*AASHTO Section 10.5.5.3.2 and 10.5.5.3.3 - 0.8 for uplift under extreme event*

(1) Calculate Rock Socket Side Resistance:

1. AASHTO C10.8.3.5.4a indicates that design based on side wall shear alone should be considered for cases where the base of the shaft hole cannot be cleaned or inspected or where large movements would be required to mobilize resistance in end bearing.

2. Shaft axial resistance contributions from overburden soil is ignored due to scour depth estimate extending to the top of rock.

(A) AASHTO 10.8.3.5.4b, Eqn. 10.8.3.5.4b-1 (qs = unit side resistance for drilled shafts socketed into rock):

$$\frac{qs}{pa} = C * \sqrt{(qu/pa)}$$

Where:

pa= 2.12 ksf - atmospheric pressure
C= 1.00 regression coefficient taken as 1.0 for normal conditions
qu = 1205 ksf - uniaxial compressive strength of rock *[based on 2025 laboratory testing on core samples]*

* ***If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the drilled shaft concrete compressive strength (f'c), f'c shall be substituted for qu. f'c is usually 4 to 5 ksi (576 to 720 ksf). [AASHTO 10.8.3.5.4b]***

f'c = 576 ksf - concrete compressive strength
min(qu,f'c) = 576 ksf

qs (unit side resistance, intact) = 34.9 ksf

(B) AASHTO Eq.10.8.3.5.4b-2 (For fractured rock that caves or needs artificial support during drilling):

$$\frac{qs}{pa} = 0.65 * \alpha_E\sqrt{(qu/pa)}$$

Where:

αE = joint modification factor based on RQD and visual inspection of joint surfaces

From AASHTO Table 10.8.3.5.4b-1, use α_E = 0.55 *based on visual inspection of rock cores and our engineering judgement*

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RDQ (%)	Joint Modification Factor, α_E: Closed Joints	Joint Modification Factor, α_E: Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

Based on the rock core data obtained in the borings (shown above) and visual inspection, jointing appears to be generally closed with no infill, and the average RQD from the rock cores was about 43%.

qs (unit side resistance, fractured rock) = 12.5 ksf

Use q_s = 13 ksf

Nominal Side Resistance, Rs = qs*As where $As = \pi D L_{socket}$

Socket Diameter (ft)	2		2.5		3	
Resistance Factor Str. Limit (Compression)	0.55		0.55		0.55	
Rock Socket Length (feet)	**Nominal Side Resistance**	**Factored Side Resistance, STR (Compression)**	**Nominal Side Resistance**	**Factored Side Resistance, STR (Compression)**	**Nominal Side Resistance**	**Factored Side Resistance, STR (Compression)**
1	82	45	102	56	123	67
5	408	225	511	281	613	337
7	572	314	715	393	858	472
8	653	359	817	449	980	539
9	735	404	919	505	1,103	606
10	817	449	1,021	562	1,225	674
11	898	494	1,123	618	1,348	741
12	980	539	1,225	674	1,470	809
15	1,225	674	1,532	842	1,838	1,011
20	1,634	898	2,042	1,123	2,450	1,348
25	2,042	1,123	2,553	1,404	3,063	1,685
30	2,450	1,348	3,063	1,685	3,676	2,022
40	3,267	1,797	4,084	2,246	4,901	2,695
55	4,492	2,471	5,616	3,089	6,739	3,706

Notes:
1. Assumes no load test to be performed; therefore, per AASHTO Table 10.5.5.2.4-1 use a resistance factor = 0.55.

ATTACHMENT 4: SENSITIVITY ANALYSIS

Abutment 2 - HP14X89 - 2 Rows - Vertical Piles
Rock @ 309 ft

Pile Size =	HP14X89
# Rows =	2
Total # Piles =	24
Pile Lengths =	27.5 ft
Rock Socket Diameter =	30 in
Rock Socket Length =	10 ft

	HP14X89	
	Intact Section	**1/8" Loss 0.125**
Depth, (d) in	13.875	13.75
Web Thickness, (tw) in	0.62	0.495
Width, (w) in	14.75	14.625
Flange Thickness, (tf) in	0.62	0.495

Limit State	Pile Type	Max. Axial		Corresponding Forces				D/C Ratio
				V2	V3	M2	M3	
		Pile #	(kip)	(kip)	(kip)	(kip-ft)	(kip-ft)	
Strength I	1	15	-232.06	39.896	-3.2288	17.638	220.75	0.46

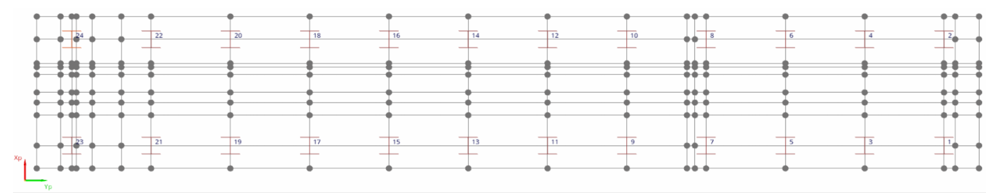
Limit State	Pile Type	Max. Moment M3		Corresponding Forces				D/C Ratio
				Axial	V2	V3	M2	
		Pile #	(kip-ft)	(kip)	(kip)	(kip)	(kip-ft)	
Strength I	1	23	230.26	-171.44	41.895	-3.4618	-19.538	0.41

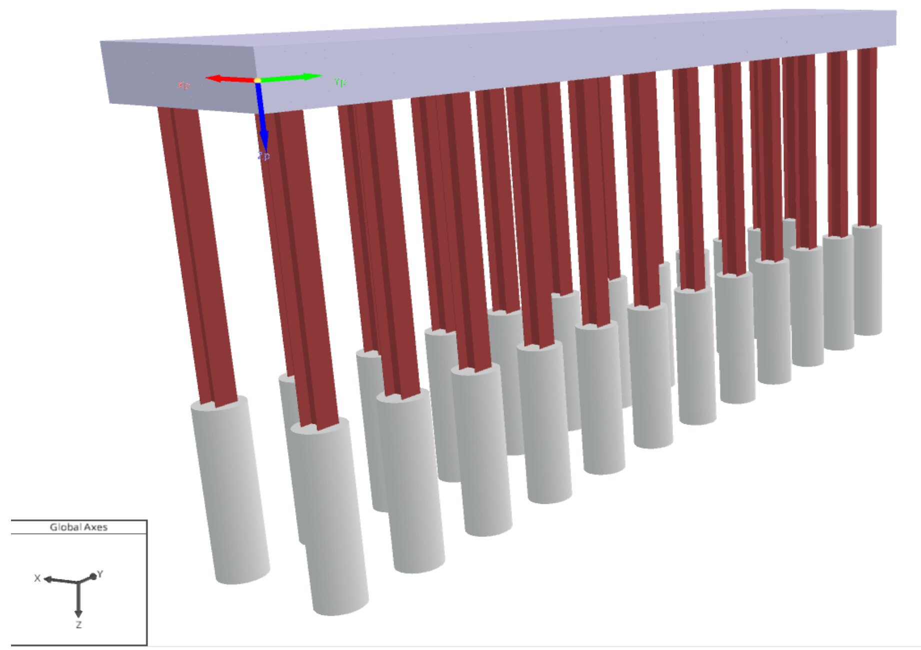
Limit State	Pile Type	Max. Shear V2		Corresponding Forces				D/C Ratio
				Axial	V3	M2	M3	
		Pile #	(kip)	(kip)	(kip)	(kip-ft)	(kip-ft)	
Strength I	1	23	41.895	-171.44	-3.4618	-19.538	230.26	0.41

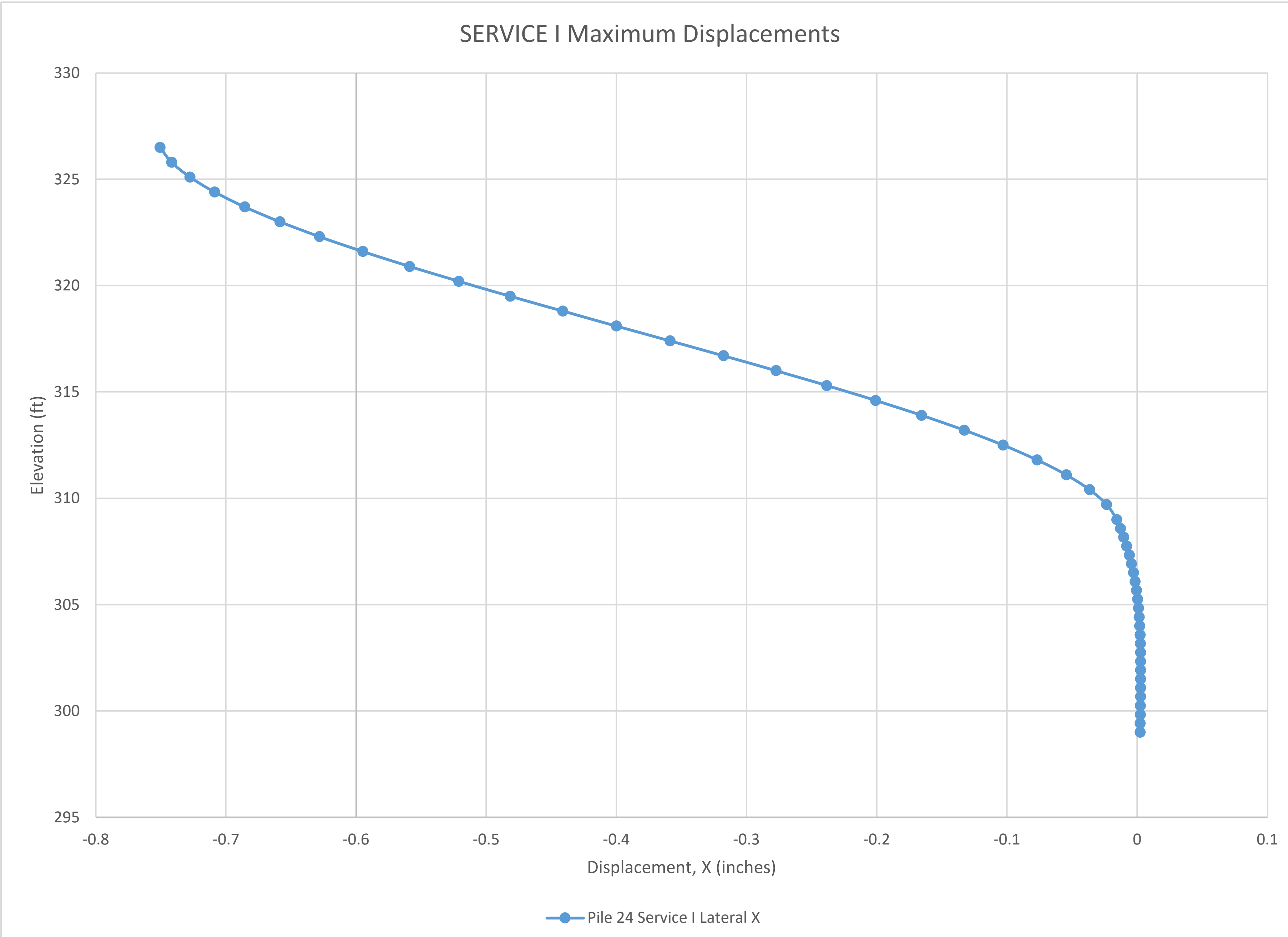
Axial convention: (-) Negative Compression
V2 : Shear along Bridge Longitudinal Axis
V3 : Shear along Bridge Transverse Axis
M2 : Moment about Bridge Longitudinal Axis
M3 : Moment about Bridge Transverse Axis

Moment sign convention according to right hand rule about global Xp and Yp global axes.

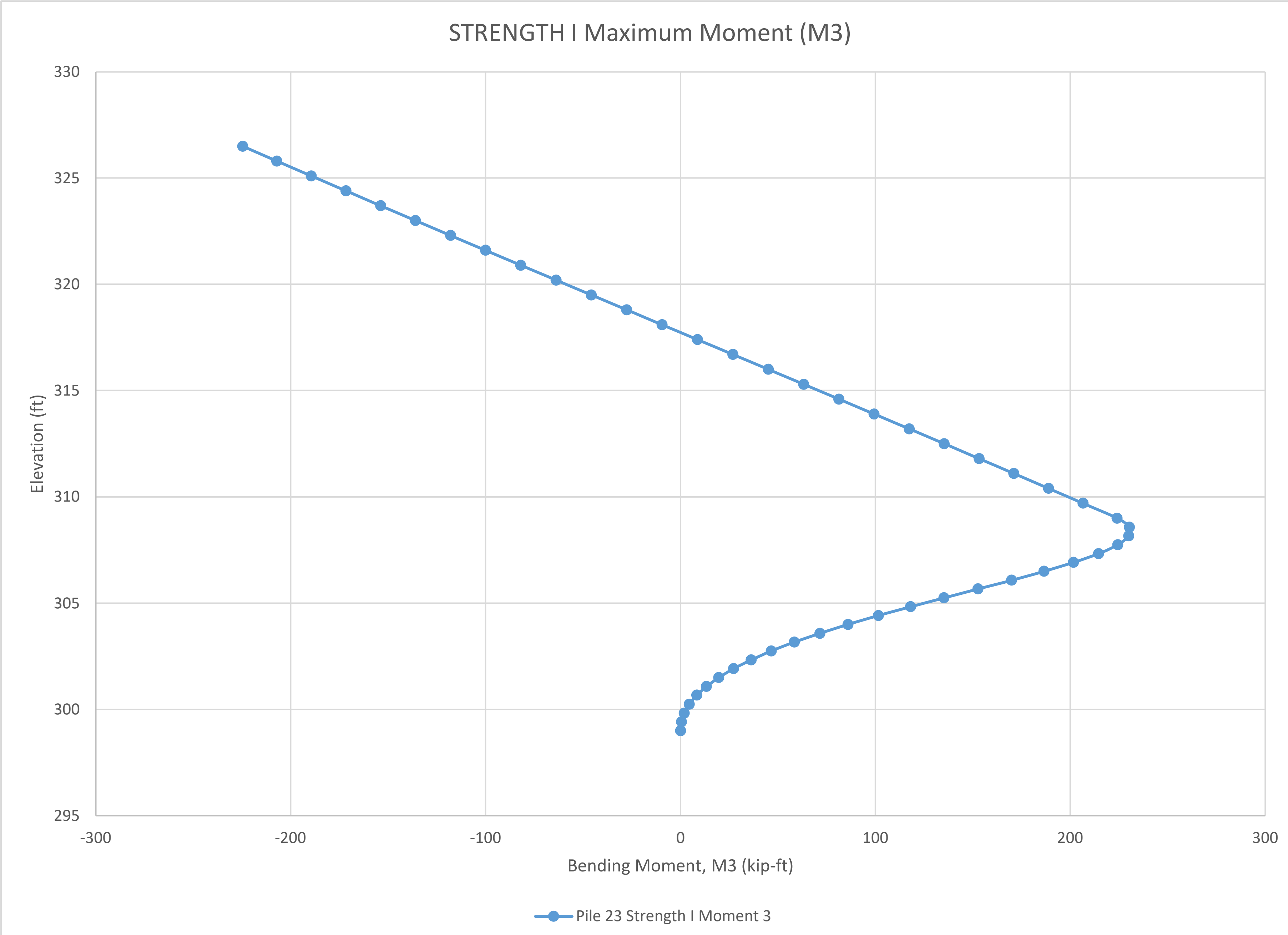
Limit State	Pile Type	Lateral Displacements and Rotations							
		X (in)	Pile #	Y (in)	Pile #	θx (rad)	Pile #	θy (rad)	Pile #
Service I	1	-0.75062	24	-0.15428	2	-0.00108	22	0.0049637	23

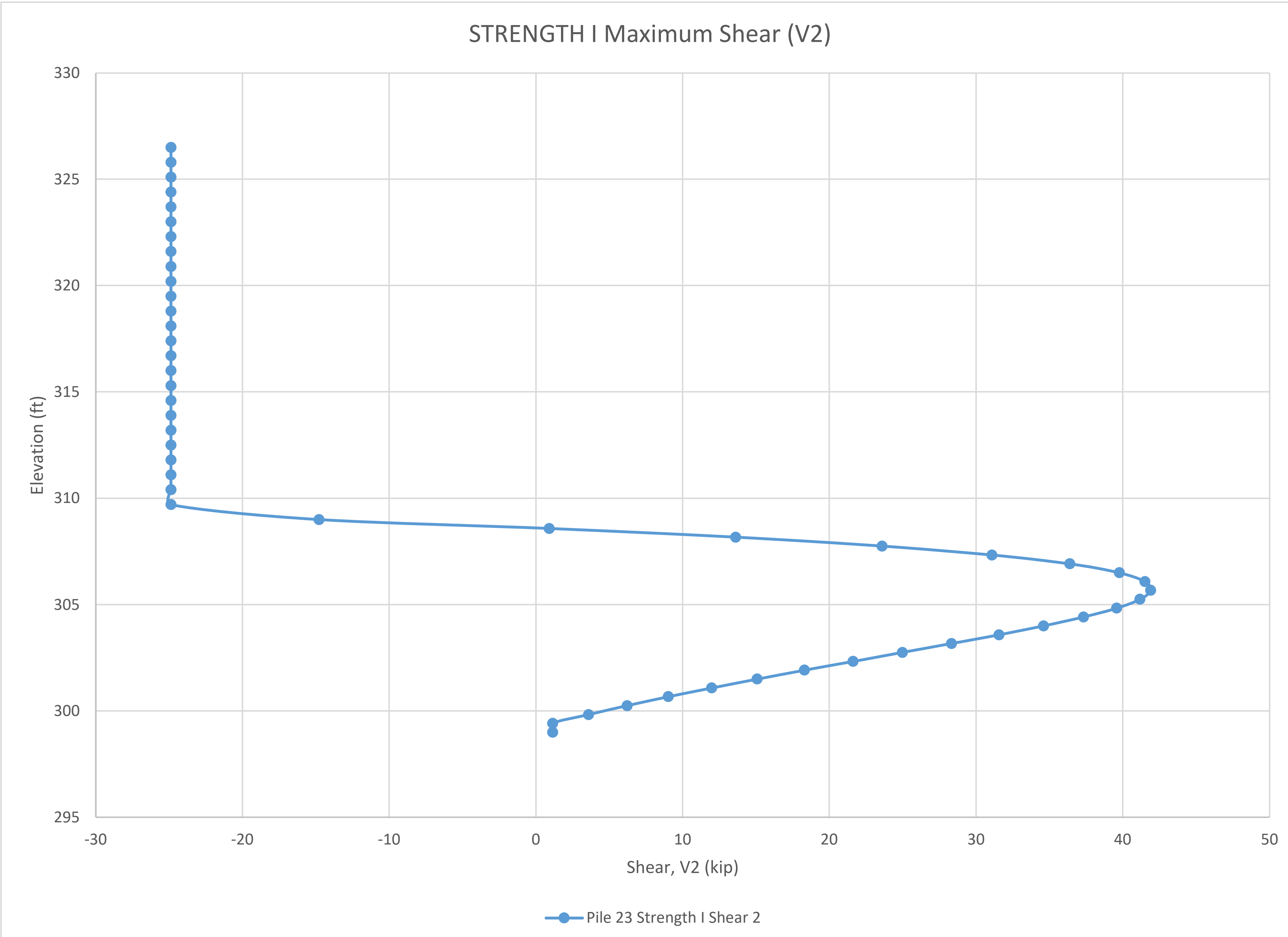






SERVICE I Maximum Displacements
Elevation (ft)
330
325
320
315
310
305
300
295
-0.8
-0.7
-0.6
-0.5
-0.4
-0.3
-0.2
-0.1
0
0.1
Displacement, X (inches)
Pile 24 Service I Lateral X


STRENGTH I Maximum Moment (M3)
Elevation (ft)
330
325
320
315
310
305
300
295
-300
-200
-100
0
100
200
300
Bending Moment, M3 (kip-ft)
Pile 23 Strength I Moment 3



STRENGTH I Maximum Shear (V2)
330
325
320
315
310
305
300
295
Elevation (ft)
-30
-20
-10
0
10
20
30
40
50
Shear, V2 (kip)
Pile 23 Strength I Shear 2

Abutment 2 - HP14X89 - 2 Rows - Vertical Piles
Rock @ 307 ft

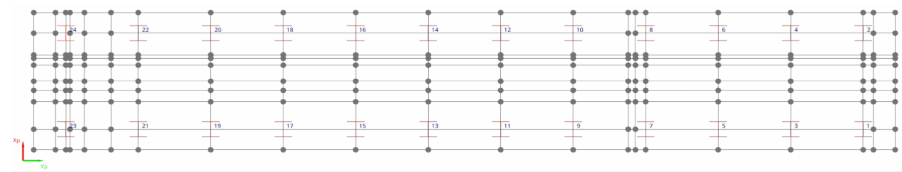
Pile Size =	HP14X89	
# Rows =	2	
Total # Piles =	24	
Pile Lengths =	27.5 ft	
Rock Socket Diameter =	30 in	
Rock Socket Length =	10 ft	

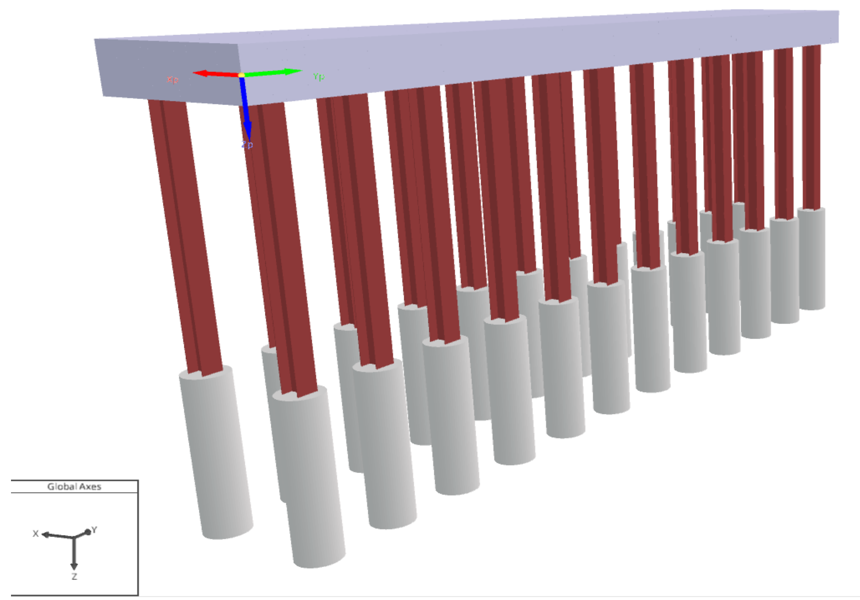
	HP14X89	
	Intact Section	**1/8" Loss 0.125**
Depth, (d) in	13.875	13.75
Web Thickness, (tw) in	0.62	0.495
Width, (w) in	14.75	14.625
Flange Thickness, (tf) in	0.62	0.495

Limit State	Pile Type	Max. Axial		Corresponding Forces				
				V2	V3	M2	M3	D/C Ratio
		Pile #	(kip)	(kip)	(kip)	(kip-ft)	(kip-ft)	
Strength I	1	15	-239.1	44.33	-3.5902	19.699	246.25	0.49

Limit State	Pile Type	Max. Moment M3		Corresponding Forces				
				Axial	V2	V3	M2	D/C Ratio
		Pile #	(kip-ft)	(kip)	(kip)	(kip)	(kip-ft)	
Strength I	1	23	256.73	-179.88	46.525	-3.817	-21.519	0.45

Limit State	Pile Type	Max. Shear V2		Corresponding Forces				
				Axial	V3	M2	M3	D/C Ratio
		Pile #	(kip)	(kip)	(kip)	(kip-ft)	(kip-ft)	
Strength I	1	23	46.525	-179.88	-3.817	-21.519	256.73	0.45

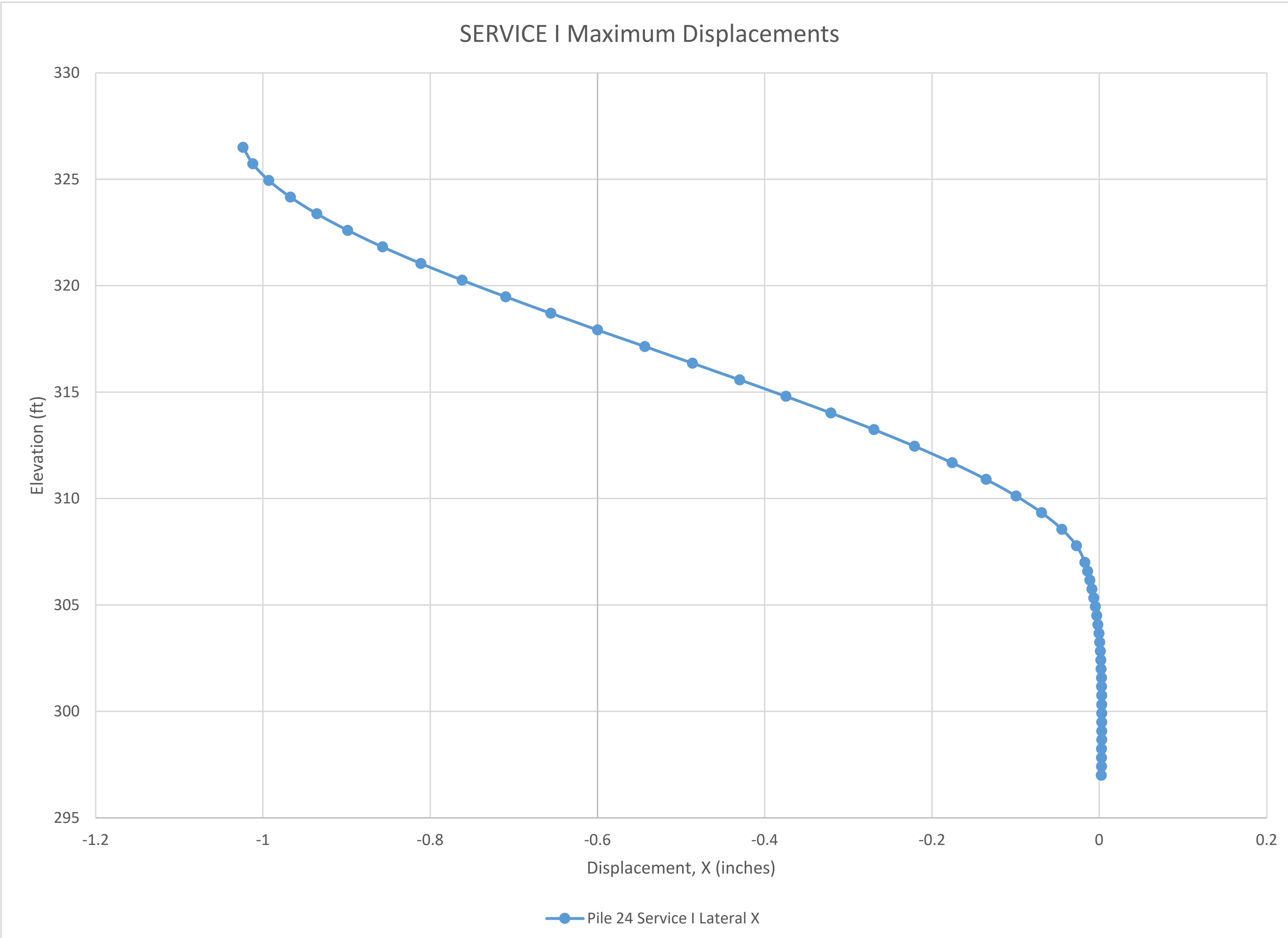


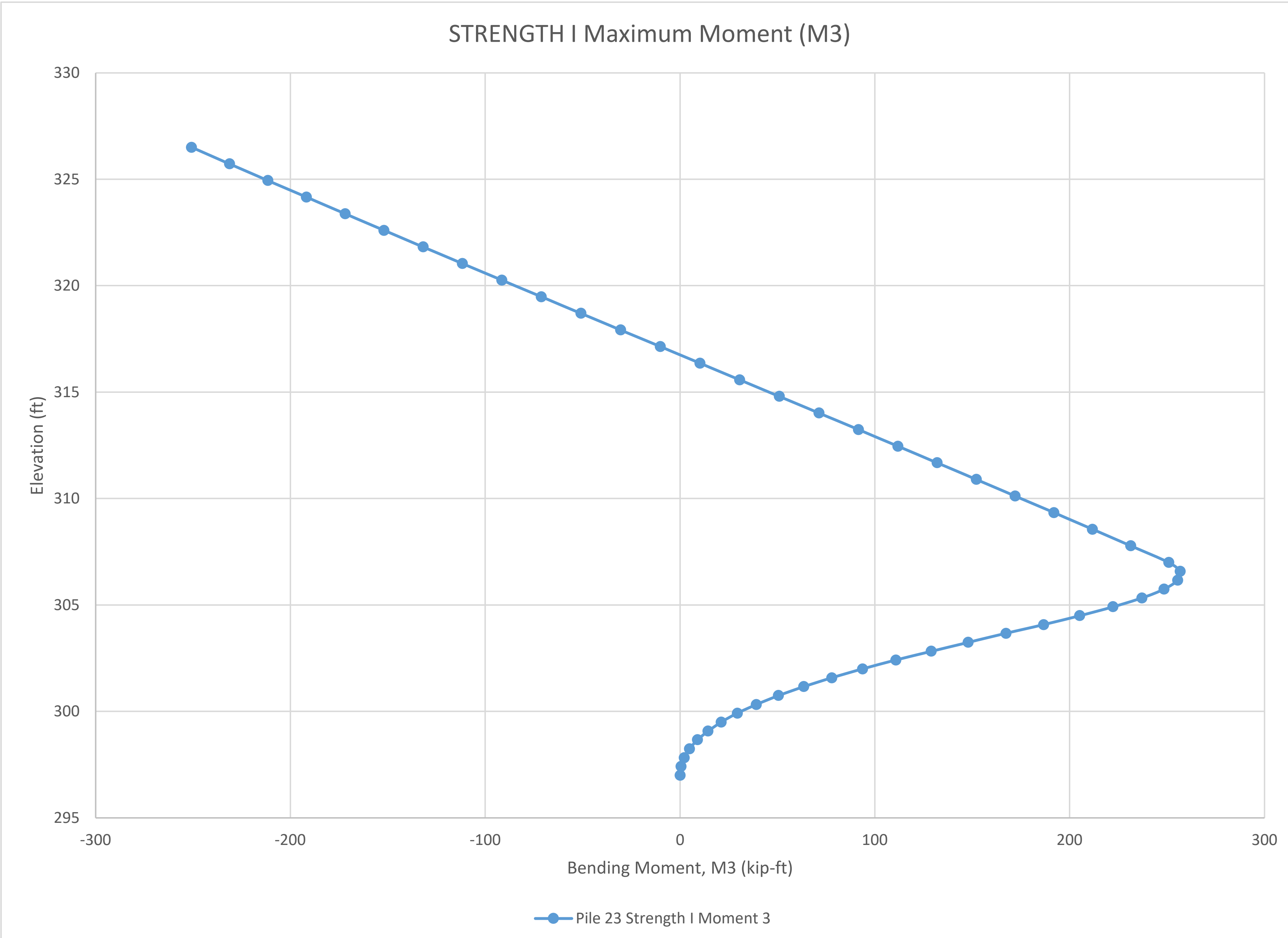


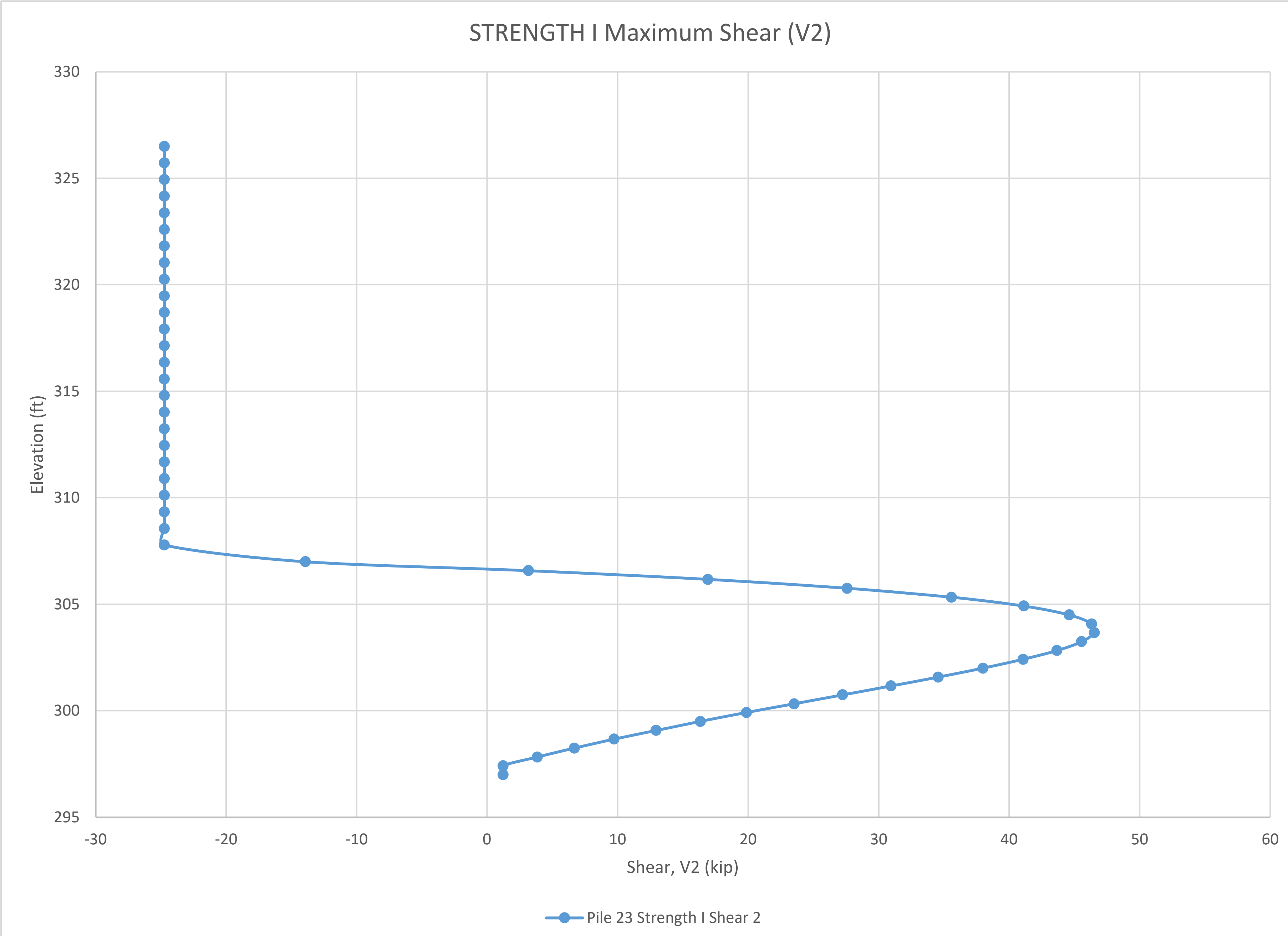
Axial convention: (-) Negative Compression
V2 : Shear along Bridge Longitudinal Axis
V3 : Shear along Bridge Transverse Axis
M2 : Moment about Bridge Longitudinal Axis
M3 : Moment about Bridge Transverse Axis

Moment sign convention according to right hand rule about global Xp and Yp global axes.

Limit State	Pile Type	Lateral Displacements and Rotations							
		X (in)	Pile #	Y (in)	Pile #	θx (rad)	Pile #	θy (rad)	Pile #
Service I	1	-1.0238	24	-0.21327	2	-0.00135	22	0.0061259	23


SERVICE I Maximum Displacements
Elevation (ft)
330
325
320
315
310
305
300
295
-1.2
-1
-0.8
-0.6
-0.4
-0.2
0
0.2
Displacement, X (inches)
Pile 24 Service I Lateral X


STRENGTH I Maximum Moment (M3)
Elevation (ft)
330
325
320
315
310
305
300
295
-300
-200
-100
0
100
200
300
Bending Moment, M3 (kip-ft)
Pile 23 Strength I Moment 3



STRENGTH I Maximum Shear (V2)
Elevation (ft)
330
325
320
315
310
305
300
295
-30
-20
-10
0
10
20
30
40
50
60
Shear, V2 (kip)
Pile 23 Strength I Shear 2