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August 9, 2024
Kleinfelder Project No.: 20193610.002A

Laura Krusinski, P.E., Senior Geotechnical Engineer Bridge Program
Maine Department of Transportation
State House Station 16
Augusta, ME 04333-0016

RE: **Geotechnical Design Report**
Replacement of Babson Bridge Over Kitteredge Brook (Bridge No. 5244)
Route 3\198, WIN 23515.00
Mount Desert, Hancock County, Maine

Dear Ms. Krusinski,

This Geotechnical Design Report presents the results of our geotechnical evaluation for the proposed integral abutment foundations for the complete replacement of Bridge No. 5244 in Mount Desert, Hancock County, Maine. Our scope of work included performing subsurface explorations, geotechnical laboratory testing, engineering analyses, and preparation of this report providing a summary of subsurface conditions at the site and foundation recommendations.

We appreciate the opportunity to provide geotechnical engineering services for this project. If you have any questions regarding this report or if we can be of further assistance, please do not hesitate to contact the undersigned at 713-320-0366.

Respectfully submitted,

KLEINFELDER, INC.

Russell L. Thomas, Jr., P.E.
Principal Geotechnical Engineer
Maine PE License No. 18478

STATE OF MAINE
RUSSELL L. THOMAS, JR.
No. 18478
8/9/2024
LICENSED
PROFESSIONAL ENGINEER

Attachments

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Geotechnical Design Report

For:

Replacement of Babson Bridge Over Kitteredge Brook (Bridge No. 5244) Route 3\198, WIN 23515.00 Mount Desert, Hancock County, Maine

Prepared for:

Maine Department of Transportation
Bridge Program
State House Station 16
Augusta, ME 04333

Prepared by:

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August 9, 2024

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1 INTRODUCTION

1.1 General

Kleinfelder conducted two phases of subsurface exploration programs including soil borings, rock coring, geotechnical laboratory testing, and geotechnical analyses to evaluate the suitability of the subsurface conditions to support the proposed bridge foundations. The purpose of this Geotechnical Design Report (GDR) is to summarize our findings and engineering analyses, and to provide geotechnical recommendations and construction considerations to be used for the design of the proposed bridge.

This report was prepared in general accordance with the provisions of the 2020 AASHTO LRFD Bridge Design Specifications (AASHTO LRFD, 9th Edition) and the 2003 Edition of the MaineDOT Bridge Design Guide, with revisions through June 2018 (MaineDOT BDG).

Conclusions and recommendations presented in this report are based on our understanding of the proposed project as presented below, subsurface conditions encountered at the locations of the soil borings, and our engineering analysis based on field and geotechnical laboratory test data. The recommendations presented herein should not be extrapolated to other areas or used for other projects. This report is subject to the Limitations included herein, and the Limitation Section included as **Section 7**. Kleinfelder has also included an information sheet prepared by GBA (the Geoprofessional Business Association, Inc.) and recommend that individuals using this report read the limitations along with the GBA document found in **Appendix D**.

Unless otherwise specified, elevations (EL) in this report are referenced to the North American Vertical Datum of 1988 (NAVD88). A site location plan is presented as **Figures 1 and 2**.

1.2 Background Information

Our understanding of the proposed project is based on the following documents:

- Proposal
- Preliminary Geotech Report
- Highway bridge inspection report

- Hydrology report

1.3 Site and Project Description

MaineDOT Bridge No. 5244, also called the Babson Bridge, carries Sound Drive (Route 3, or Route198) over the tidal area of Kitteredge Brook in Mount Desert, Maine. Kitteredge Brook, also known as Meadow Brook, is a tidal stream connected to Somes Sound.

We understand Babson Bridge was originally built in 1949 and reconstructed in 1977 with a single span length of about 21 feet. The roadway on the existing bridge has two opposing 11-foot-wide traffic lanes with 2-foot-wide shoulders and a sidewalk on the southern side for a total width of about 25 feet. The top of the existing bridge is at approximately EL +11 feet. The superstructure consists of a 12-inch-thick reinforced concrete slab supported on two abutments. The abutments consist of dry-stacked granite blocks with reinforced concrete caps.

At the time of our field study, the abutments were in generally poor condition. There appears to be movement of the stones and mild voids throughout. A cracked stone was observed in the row of stones under the concrete cap on the south end of the west abutment. There were signs of water getting behind the west abutment, which likely caused of some of the observed deterioration. Record drawings with substructure information were not available at the time of report preparation.

The MaineDOT, Bridge Maintenance Division, maintains an on-line repository of current inventory reports for public bridge structures throughout the State. The latest inventory report for Babson Bridge No. 5244 states the following:

- Classification
  - Owner: MaineDOT
  - Max Span Length: 17.6 feet
  - Federal Bridge Indicator: No
- Age and Condition (i.e., National Bridge Inventory Rating, defined in FHWA 1995)
  - Deck Condition: 4 – Poor Condition, Advanced Deterioration
  - Superstructure Condition: 4 – Poor Condition, Advanced Deterioration
  - Substructure Condition: 4 – Poor Condition, Advanced Deterioration
  - Year Built: 1949
  - Channel Condition: 6 – Bank Slump, Widespread Minor Damage

- Approach Condition: 8 – Equal to Present Desirable Criteria
- Annual Average Daily Traffic: 8099

- Inspection and Appraisal
  - Date of Inspection: September 28, 2021
  - Federal Sufficiency Rating: 26.6%

It should be noted that according to FHWA (1995), a sufficiency rating under 49% indicates the bridge structure is functionally obsolete (or a bridge that has a poor configuration and/or design) and is eligible for replacement or rehabilitation.

Results of a scour analysis was presented in a report prepared by NorthStar Hydro for MaineDOT, titled "Final Design Hydrology, Hydraulics and Scour Report, Babsons Bridge over Meadow Brook, Route 198, Bridge No. 5244", dated October 28, 2022. Therein, an evaluation of the scour potential was performed and based upon the predicted and measured velocity of the water passing beneath the bridge, the observed bedding material, and the bridge geometry. It should be noted that the bridge spans tidal waters where velocities increase when tides change rapidly, such as a surge event and when tides are low, usually mid-tide. In addition, the analysis, performed using HFWA HEC-18, assumed scour would be limited to contraction and long-term bed scour. Results indicate scour with general contraction scour ranging from 4 to 5 feet if the bedding material is exposed at the channel bed surface, which represents a 10-year and 100-year surge flood event, respectively. As such an unsupported length of 5 feet was used for design of pile foundations herein.

Based on visual observations at the time of our field study, the roadway showed signs of settlement, lateral soil spreading, and pavement cracking behind both abutments due to undermining from the voids in the abutment below the highest annual tide elevation. The east abutment was bulging towards Kitteredge Brook. The side slopes on the approaches showed visual evidence of scour above the existing rip rap along with a large washout on one side of the road at the low point. We understand the existing bridge site is subject to strong tidal flows.

1.4 Proposed Construction

The Maine Department of Transportation (MaineDOT) is currently considering the complete replacement of the bridge. The proposed bridge replacement is anticipated to include lengthening from about 18 feet to 56 feet and raising the bridge to assist in alleviating current scour/damming concerns and to account for sea level rise from about EL +9.8 feet to EL +13.87 to +14.67 feet. The design of the bridge superstructure is not included in the current scope of services.

We understand MaineDOT prefers to construct a new bridge supported on integral abutments while minimizing impacts to the causeway. Following communication between our (Kleinfelder's) Structural Engineer and MaineDOT, we understand the preferred foundation (substructure) alternative for support of the new abutment and wingwalls is a deep foundation system comprised of steel H-piles.

We understand the proposed replacement structure will be a single-span bridge, comprised of integral abutment and wingwalls founded on a deep foundation system consisting of both driven and pre-augured steel H-Piles. Design of the replacement bridge superstructure will be part of the detail-build portion of the project, wherein the contractor picks one of the three (3) superstructure options listed below, and completes the design as part of the construction project:

1. Composite Tub Girder (G-Beams)
2. Type D, Precast Northeast Extreme Tee (NEXT) beam
3. Type F, Precast NEXT beam

The loading conditions assumed herein are based upon the heaviest superstructure option, Type F, Precast NEXT beam.

The bridge replacement abutments are proposed to be constructed behind the existing abutments. We understand the replacement bridge will be raised from the existing vertical profile from approximately EL +10.8 feet to about EL +14.67 and EL +13.87 at Abutments 1 and 2, respectively. The span length will be increased to about 56 feet in order to accommodate the 100-year design flood and associated sea level rise. 1.75:1 (horizontal to vertical) riprap slopes will be placed in front of the new integral abutments for scour protection. The top of existing streambed is approximately EL -0.5 feet. Based on the anticipated scour contraction of 5 feet, the bottom of the structural earth excavation and heavy riprap protection will extend to a depth of approximately EL -5.5 feet on both sides of the bridge.

Based on discussions with the Structural Engineer, we understand the maximum factored pile load is 310 kips at the Strength - I Limit State and 210 kips at the Service - I Limit State, not considering the lateral soil loading.

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1.5 Project Authorization

Kleinfelder has performed the geotechnical engineering services described herein in accordance with our proposal dated February 11, 2022. Our work on this project was authorized by the agreement between MaineDOT and Kleinfelder dated March 3, 2022.

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2 GEOLOGIC SETTING

According to the Maine Geologic Survey (2016), the landscape of the Mount Desert Island is dominated by linear valleys carved out of northwest-trending fracture zones. The surface of the Island as seen today is a result of significant erosion remnants of the Appalachian Mountain chain and was originally at the center of a magma chamber two miles beneath a volcano. More recently, the Island has been sculpted by repeated glacial cycles, resulting in rounded, streamlined peaks separated by elongated lakes in the glacial troughs. Offshore, these rounded islands are separated by glacial troughs flooded by ocean waters.

With respect to the project site, the surficial geology of the western side of the project site generally consists of a thin layer of glacial, colluvial, and/or residual materials overlying bedrock. On the eastern side of the project site, the surficial soils are part of the Presumpscot Formation described as fine-grained marine mud that generally consists of silt and clay with sand lenses and commonly contains gravel drop-stones.

The Coastal Landslides Hazard map for the Southwest Harbor Quadrangle (2005) indicates that the bridge lies along an area mapped as Low Coastal Bluffs. The low coastal bluffs are described as shoreline with a sedimentary bluff less than 20 feet in height and is immediately adjacent to the shoreline. In general, low coastal bluffs are not at risk of failing in the form of a landside.

The Bedrock Geology of Mount Desert Island (2018) indicates that both sides of the project site is underlain by intrusive rock of the Somesville Granite Formation. The Somesville Granite Formation is composed of pink to light gray, fine-grained to porphyritic biotite granite which contains varying portions of quartz and equant, perthitic alkali-feldspar phenocrysts. Biotite is the dominant mafic mineral, whereas hornblende is scarce to absent. Many accessory minerals are included.

A summary of the surficial geology is shown on **Figure 3**, attached to this report. Local and regional topography provided by the USGS is shown on **Figure 4**, attached to this report.

3 FIELD EXPLORATION

Two (2) subsurface investigation programs (Preliminary Design and Final Design borings) including soil borings, rock coring and geotechnical laboratory testing were performed by Kleinfelder at the project site. The purpose of our field studies was to evaluate the subsurface conditions near the proposed abutments, perform field testing, and obtain soil samples for additional laboratory testing (as appropriate).

The first subsurface exploration program was performed in May of 2019; the second subsurface exploration program was performed in May of 2022. The actual locations of the explorations performed at the site are shown on the **Figure 5**, attached. The borings were laid out by a Kleinfelder engineer. The as-drilled locations shown on the plans where subsequently surveyed by MaineDOT. As such, the accuracy of the locations and elevations shown on the boring logs may be considered survey-grade.

3.1 Preliminary Design Subsurface Exploration Program - May 2019

Kleinfelder conducted a subsurface exploration program at the project site between May 13 and 15, 2019. The subsurface exploration program consisted of two soil borings (BB-MDMB-101 and BB-MDMB-102). The explorations were performed by New England Boring Contractors (NEBC) of Hermon, Maine.

Boring BB-MDMB-101 and BB-MDMB-102 were advanced to depths of approximately 42.5 and 34.0 feet below ground surface (bgs), respectively and were terminated in bedrock. The boring termination depths correspond to elevations of approximately EL -31.3 and EL -23.0 feet, respectively.

3.2 Final Design Subsurface Exploration Program - May 2022

Kleinfelder conducted a second subsurface exploration program at the project site between May 25 and 26, 2022. The subsurface exploration program consisted of two borings (BB-MDMB-201 and BB-MDMB-202). The explorations for the second field study were also performed by NEBC.

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Boring BB-MDMB-201 and BB-MDMB-202 were advanced to depths of approximately 47.5 and 38.0 feet below ground surface (bgs), respectively and were terminated in bedrock. The boring termination depths correspond to elevations of approximately EL -35.2 and EL -27.2 feet, respectively.

3.3 Drilling Methods

3.3.1 Standard Drilling

The borings for both field studies were advanced with a truck-mounted Mobile B-53 drill rig using drive and wash drilling techniques, utilizing a 4-inch inside diameter HW casing. Standard Penetration Tests (SPT), in general accordance with the American Society for testing and Materials (ASTM) designation D1586, were performed during drilling. Standard penetration testing entails driving an approximately 1.38-inch ID (approximately 2-inch outside diameter - OD) split spoon sampler into a soil layer using a 140-lb weight (hammer) dropped freely from a height of 30 inches and recording the number of hammer blows (blow count) for 4 consecutive advancements of the split spoon measuring 6 inches each, for a total advancement of the split spoon of 24 inches. The number of blows required to penetrate the second and third six-inch-intervals was recorded and designated the “N-value” or “penetration resistance.” The N-value, when properly evaluated, is an indication of soil strength and foundation support capability. In general, SPTs were performed starting at the ground surface, at standard five-foot intervals.

For this project, an automatic hammer was used during standard penetration testing. NEBC utilized a calibrated automatic drop hammer to generate consistent energy transfer to the sampler. The hammer was calibrated on July 19, 2018, and had a hammer efficiency ratio of 92.3 percent.

Upon completion of field testing, measured N-values were corrected to an equivalent (60%) energy ratio, N_{60}, by the following equation. Both field and corrected N-values are presented on the boring logs.

$$N_{60} = N \times \frac{ER}{60}$$

Where:

N = Uncorrected N-value measured during drilling

N_{60} = Blow count value corrected to an equivalent (60%) energy ratio

ER = Drill rod energy ratio, expressed as a percentage, for the system used

3.3.2 Rock Coring

Refusal materials were evaluated in each of the four borings. Refusal materials are materials that cannot be penetrated with the drive and wash drilling methods described above. Refusal, thus indicated, may result from boulders, lenses, ledges, or layers of relatively hard rock underlain by partially weathered rock or residual soil; refusal may also represent the surface of relatively continuous bedrock. Core drilling procedures are required to penetrate refusal materials and determine their character and continuity.

As stated above, HW casing was used to keep the boreholes from caving. Refusal materials were cored in general accordance with ASTM D 2113 using a diamond impregnated core bit fastened to the end of a hollow, double-tube core barrel The core barrel used was NQ-2 sized, measuring approximately 72 inches in length and had an outer diameter of about 2 inches. The core barrel was rotated at high speeds while the cuttings were brought to the surface by circulating drilling fluids (typically bentonite mud). During coring, our field engineer recorded the penetration resistance (measured in minutes per foot of penetration).

Core samples of the material penetrated were protected and retained in the swivel-mounted inner tube. Upon completion of each core run, the core barrel was brought to the surface, the recovered material was measured, then removed and placed in boxes for storage. The core samples were then returned to our laboratory where the refusal material was identified, and the percent core recovery and rock quality designation (RQD) was determined by a geotechnical engineer or geologist.

The percent core recovery was the ratio of the sample length obtained within the core barrel compared to the length cored, expressed as a percent. The rock quality designation was obtained by summing only those pieces of recovered core which are 4 inches or longer and are at least moderately hard, then dividing by the total length cored, also expressed as a percent. The percent core recovery and the RQD are related to soundness and continuity of the refusal material. Further details concerning rock coring are presented on the appropriate boring logs.

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3.3.3 Borehole Abandonment

Groundwater monitoring wells were not installed as part of either exploration program. Upon completion of drilling, the borings were backfilled with drill cuttings, sand, and gravel. Asphalt cold patch was then used to repair the pavement surface.

3.4 Drilling Oversight

A Kleinfelder geotechnical engineer performed full-time drilling oversight during the subsurface exploration programs. The soil samples recovered during the standard penetration testing and the refusal materials recovered during rock coring, were logged, and described in general accordance with MaineDOT's guidelines and procedures. A copy of MaineDOT's Geotechnical Section, Key to Soil and Rock Descriptions and Terms, Field Identification Information, dated January 2020, used herein has been included in **Appendix A**, attached to this report.

Soil and rock samples collected during drilling of the borings were stored in sealed glass jars or rock core boxes and brought to the Kleinfelder office for review by a senior geotechnical engineer Select samples were then delivered to a geotechnical laboratory for testing.

3.5 Boring Logs

The attached soil boring logs represent our interpretation of the field drilling log, engineering examination of the field samples, and results of laboratory testing. Therefore, the boring logs contain both factual and interpretive information. Lines delineating subsurface strata on the boring logs are intended to group soils having similar engineering properties and characteristics. They should be considered approximate as the actual transition between soil types (strata) may be gradual. Descriptions of the soil and rock encountered in the explorations are included in the boring logs provided in **Appendix A**.

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4 LABORATORY TESTING

Geotechnical laboratory testing was performed on selected soil and rock samples from the 2019 and 2022 borings to substantiate field classifications and evaluate engineering properties of the soil and rock. The results of laboratory tests are included on the boring logs. Copies of the lab test reports are also included in **Appendix B**. A summary of the number and type of tests performed is provided in the table below:

Table 1 – Laboratory Test Schedule

Lab Test	Test Designation	Number of Tests Performed	
		Preliminary Design	Final Design
Gradation - Particle Size Analysis	ASTM D422	4	-
Gradation -Particle Size Distribution	ASTM D6913	-	4
Chloride	ASTM D1411	-	1
Bulk Density/Compressive Strength of Rock Core	ASTM D7012 Method C/ASTM D4543	1	-
Elastic Moduli/Compressive Strength of Intact Rock Core	ASTM D7012	-	2

4.1 Geotechnical Laboratory Testing for Preliminary Design

Four grain size analyses (ASTM D422) and one bulk density and compressive strength of rock test (ASTM D7012 Method C) were performed on selected soil samples and rock cores respectively, by GeoTesting Express, Inc. of Acton, Massachusetts.

4.2 Geotechnical Laboratory Testing for Final Design

Four grain size analyses (ASTM D6913) and two compressive strength of rock tests (ASTM D7012) were performed on selected soil samples and rock cores respectively, by Thielsch Engineering of Cranston, Rhode Island.

In addition to the index testing, a corrosion test suite to evaluate soil corrosion potential for steel and/or concrete construction was also performed on one composite soil sample from the borings. The corrosion test suite included Chloride analysis (ASTM D1411) only.

5 SUBSURFACE CONDITIONS

Subsurface conditions encountered during Kleinfelder's field exploration program are described below, in general order of their occurrence. For more detailed information, please refer to the boring logs found in **Appendix A**. A longitudinal profile graphically depicting Kleinfelder's interpretation of the subsurface conditions between borings is presented on the Interpretative Subsurface Profile on **Figure 6.**

The generalized subsurface conditions (soil stratigraphy) described below summarize trends observed in the borings. Actual subsurface conditions between borings will vary. As discussed above, the strata boundaries shown on the boring logs are based on our interpretations. Actual transitions may be gradual and should be considered approximate.

5.1 Surficial Material

Borings were advanced through approximately 6.5 to 9 inches of bituminous pavement at the ground surface.

5.2 Undocumented Fill

Undocumented fill material was encountered below the bituminous pavement at all borings. The existing fill layer extended to depths between approximately 15.0 feet (EL -4.2) and 9.3 feet (EL +1.7 feet) bgs. The fill generally consisted of light brown and dark gray sand with varying amounts of silt and gravel. Fill materials in the borings contained possible cobbles and boulders which were noted on the boring logs.

The SPT N-values (raw, uncorrected) in the fill layer ranged from 9 to greater than 100 blows per foot (bpf), indicating loose to very dense material. The high blow counts encountered in the fill layer could be the result of cobbles and boulders.

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5.3 Buried Organic Matter

A layer of buried organic-laden soil was encountered within and under the Undocumented Fill layer in Borings BB-MDMB-101 and BB-MDMB-201 respectively, near the Western Abutment (Abutment 1) at an approximate depth of 10 feet bgs (EL +1.2 to EL +2.3 feet). The portion of the existing fill layer below the buried organic soil layer in Boring BB-MDMB-101 is described as "Organic-laden" in **Table 2** and **Table 3**.

The thickness of the buried organic-laden soil varied between approximately 2.5 feet and 5.0 feet and extended to about EL -1.3 and EL -2.7 feet, respectively. The sample was described as a dark brown to light gray silty sand with trace amounts of wood fragments and organic fines. The SPT N-value (raw, uncorrected) in this layer varied between 9 and 10 bpf, indicating a loose relative density.

5.4 Glacial Deposits

Naturally occurring glacial deposits were encountered below the buried organic-laden soil in Boring BB-MDMB-201 and below the Fill layer in the other borings and extended to the top of bedrock.

The thickness of the glacial deposits varied between approximately 8.5 feet and 21.5 feet and extended to about EL -10.6 and EL -24.2 feet. The layer generally consisted of light gray/brown, fine to coarse sand, with greater than 40 percent by weight of silt and varying amounts of gravel A sample, 5D between an approximate depth of 21 and 23 feet bgs (EL -10.8 to EL -12.8 feet) from this layer in Boring BB-MDMB-101 was described as a slightly plastic, hard Silt.

The SPT N-values (raw, uncorrected) within the Glacial Deposits layer ranged from 11 to 64 bpf, indicating medium dense to very dense material.

In general, the Glacial Deposits encountered in Borings BB-MDMB-101 and BB-MDMB-201, which are on the western side of the Site, was in general agreement with the mapped geology detailed above in **Section 2**. However, Borings BB-MDMB-102 and BB-MDMB-202. which were advanced on the eastern side of the Site was not consistent with mapped geology which anticipated the natural material to consist mostly of silt or clay.

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Table 2 – Design Conditions at Abutment No. 1 (West)

Stratum	Top Elevation (feet)	Bottom Elevation (feet)	Top Depth (feet, bgs)	Bottom Depth (feet, bgs)	Thickness (feet)
Existing Asphalt	11.2	10.5	0.0	0.7	0.7
Existing Fill	10.5	1.2	0.7	10.0	9.3
Organic-Laden	1.2	-3.8	10.0	15.0	5.0
Glacial Deposits	-3.8	-17.5	15.0	28.7	13.7
Granite/Syenite Bedrock	-17.5	-	28.7	-	-

Table 3 – Design Conditions at Abutment No. 2 (East)

Stratum	Top Elevation (feet)	Bottom Elevation (feet)	Top Depth (feet, bgs)	Bottom Depth (feet, bgs)	Thickness (feet)
Existing Asphalt	11.0	10.4	0.0	0.6	0.6
Existing Fill	10.4	-4.0	0.6	15.0	14.4
Glacial Deposits	-4.0	-10.5	15.0	21.5	6.5
Granite/Syenite Bedrock	-10.5	-	21.5	-	-

5.5 Bedrock

Bedrock was encountered below the glacial deposits and was confirmed via rock coring in the borings as presented below in **Table 4** and **Table 5**.

The presumed top of bedrock behind the existing Abutment 1/West Abutment (as per Borings BB-MDMB-201 and BB-MDMB-101), was encountered between approximate depths of 36.5 feet (EL -24.2 feet) and 31.0 feet (EL -19.8 feet).

The presumed top of bedrock behind the existing Abutment 2/East abutment (as per Borings BB-MDMB-102 and BB-MDMB-202), was encountered between approximate depths of 21.6 feet (EL -10.6 feet) and 23.5 feet (EL -12.7 feet).

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Hence, it can be inferred that the bedrock slopes steeply in the longitudinal direction, from an approximate depth of 28.7 feet (EL -17.5 feet) at Abutment 1 on the West to approximately 23.5 feet deep (EL-12.7 feet) at Abutment 2 on the East.

Table 4 – Presumed Top of Rock

Boring	Approximate Ground Surface Elevation (feet)	Depth to Top of Rock (feet)	Approximate Top of Rock Elevation
BB-MDMB-101	EL +11.18	31.0	EL -19.8
BB-MDMB-201	EL +12.27	36.5	EL -24.2
BB-MDMB-102	EL +10.96	21.6	EL -10.6
BB-MDMB-202	EL +10.77	23.5	EL -12.7

Assume top of bedrock at Abutment No. 1 to be at EL -19.8 feet and at Abutment No. 2 to be at EL -10.5 feet.

The recovered cored bedrock was described as pink with black mottles, hard to moderately hard, fine to medium grained, slightly weathered granite of the Somesville Formation, with horizontal to steep, very close to moderately close, open to healed fractures. The rock core recovery ranged between 90 and 100%; the Rock Quality Designation (RQD) values ranged from 0 to 53% which corresponds to a Rock Quality of very poor (RQD < 25%) to fair (51% < RQD < 75%).

The percent recovery and unconfined compressive strengths of the bedrock core runs are summarized in the table below.

Table 5 – Rock Core Summary

Boring	Core Number	Elevation (feet)	Recovery	RQD	Unconfined Compressive Strength (psi)
BB-MDMB-101	R2	EL -20.82 to EL -25.82	90%	31%	12,115
BB-MDMB-101	R3	EL -26.32 to EL -31.32	90%	33%	
BB-MDMB-102	R1	EL -13.04 to EL -18.04	100%	25%	
BB-MDMB-102	R2	EL -18.04 to EL -23.04	92%	53%	
BB-MDMB-201	R1	EL -25.23 to EL-30.23	100%	33%	1,906

Table 5 – Rock Core Summary

Boring	Core Number	Elevation (feet)	Recovery	RQD	Unconfined Compressive Strength (psi)
BB-MDMB-201	R2	EL -30.23 to EL -35.23	100%	26%	
BB-MDMB-202	R1	EL -16.73 to EL -21.73	100%	20%	
BB-MDMB-202	R2	EL -22.23 to EL -24.9	100%	8%	1,481
BB-MDMB-202	R3	EL -24.9 to EL -27.23	100%	0%	

Detailed descriptions of the rock core and RQD values for each core run are shown on the exploration logs in **Appendix A**.

Unconfined compressive strength tests performed in general accordance with ASTM standard method D7012 on rock samples from the borings showed that the uniaxial, unconfined compressive strength of the rock ranges between 213.3 ksf (1481 psi) and 1744.6 ksf (12,115 psi). The lower end test results were due to the failure shear plane occurring along a healed fracture (BB-MDMB-201) and an axial splitting failure/bulging failure mode with subvertical failure plane (BB-MDMB-202).

5.6 Groundwater

Groundwater was measured during drilling, in the Preliminary Design Borings (May 2019) and in the Final Design Borings (May 2022). Groundwater was measured inside the casing before resuming drilling of the borings. Groundwater readings are provided on the boring logs and the table below.

Table 6 – Groundwater Level Measurements

Boring	Date	Approximate Ground Surface Elevation	Depth to Water	Approximate Water Elevation
BB-MDMB-101*	05/14/2019	EL 11.18	5 ½ ft	EL 5.68
BB-MDMB-102*	05/15/2019	EL 10.96	5.8 ft	EL 5.16
BB-MDMB-201**	05/26/2022	EL 12.27	15 in	EL 11.02
BB-MDMB-202**	05/25/2022	EL 10.77	12 in	EL 9.77
* - Drilling was paused overnight. Readings were taken 24 hours from the start of drilling. ** - Readings were taken on the same day that drilling was started.				

It should be noted that Kitteredge Brook is tidally influenced, and water was introduced during drilling therefore, the water levels indicated may not represent stabilized ground water conditions. Long term groundwater information is not available.

The groundwater information included in this report is based on observations made during drilling and may not represent the actual groundwater level, as additional time may be required for the groundwater levels to stabilize. The groundwater level presented in this report only represents the conditions encountered at the time and location of the explorations. It should be noted that water and groundwater levels fluctuate due to local and regional factors including, but not limited to, tidal influence, snow melt, precipitation events, seasonal changes, periods of wet or dry weather, site topography, well pumping, nearby construction, or other below grade activities, such as excavation, dewatering, infiltration basins, etc.

6 GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

6.1 Summary

The recommendations in this report are developed in general accordance with the AASHTO Load and Resistance Factor Design Bridge Design Specifications, Ninth Edition, 2020 (AASHTO LRFD) and the MaineDOT Bridge Design Guide, 2003 with 2018 updates (MaineDOT Bridge Guide).

Spread footings are not feasible for this project. Spread footings would need to bear below the scour elevation evaluated for this project. Per the project's hydraulics report, the scour elevation will be approximately El.-5.75 (approximately 5 feet of scour below the existing stream bed at the bridge structure), hence, excavation for shallow foundation would have to extend deeper than approximately 17 feet below existing grade. Therefore, deep foundations are recommended for the proposed bridge abutments and wingwalls. Following communication with the Kleinfelder Structural Engineer and MaineDOT, we understand that the preferred foundation alternative for the bridge abutments and wingwall is steel driven H-piles/deep foundations.

The following sections provide detailed descriptions of our analyses and/or assumptions. In short, we are recommending HP 14x73 be used for both abutments. However, piles for Abutment No. 1 (West) should be installed using traditional pile driving methods to top of rock when fitted with a suitable rock point (similar to Rock Injector Point HP-80500 from Associated Pile & Fitting) and at least one pile monitored by a PDA during driving/installation. Piles for Abutment No. 2 should be predrilled to a depth sufficient to create a 5-ft-long rock socket into the underlying granitic bedrock, installed with a device such that the center of the pile has about 3 inches of separation from the bottom of the rock socket, and then grouted in-place using 4,500 psi, or stronger, grout.

6.2 Design Groundwater Elevation

The FEMA Flood Insurance Rate Map (FIRM) places the Babson Bridge project site within "Zone AE – Special Flood Hazard Area." A Base Flood Elevation (BFE) of EL +10 feet was defined for the site.

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Bridge #5244 conveys flow from the Kitteredge Brook under Route 198 in Mount Desert, Maine. The outlet is a tidal stream connected to Somes Sound. The Final Design Hydrology, Hydraulics, and Scour Report prepared by Northstar Hydro,Inc. of Winthrop, Maine dated October 28, 2022, indicates that Sea Level Rise (SLR) of up to 4 feet could overtop the existing bridge assuming current estimates of potential rise for 100-years. The report also includes the following tide information based on the NOAA Tide Prediction Station # 8413564, Southwest Harbor, predicted based on Bar Harbor indicates the following information:

Table 7 – Tide Elevations

Tide Level	Elevation
Highest Astronomical Tide, 2019	EL +7.72 feet
Mean High-High Water (MHHW)	EL +5.40 feet
Mean Lower Low Water (MLLW)	EL -5.57 feet
10-year tidal surge with 4 feet of sea level rise (SLR)	EL +12.5 feet

Hence, we recommend that a design groundwater elevation of EL +10 feet be used.

6.3 Frost Considerations

Based on the Maine Design Freezing Index Map (Figure 5.1, Maine DOT BDG), the design freezing index for the Mount Desert, Maine area is approximately 1,050 freezing degree-days. An assumed water content of 10% was used for coarse grained soils. Based on Section 5.2.1 of the MaineDOT BDG and based on the subsurface soils encountered, the pile supported integral abutments shall be embedded a minimum of 68 inches for protection against seasonal frost. Riprap thickness is not to be considered as contributing to the overall thickness of soils required for frost protection.

Considering this, we recommend the bottom of the pile cap should be at least 6 feet below finished grade to provide frost protection, or deeper as needed based upon the structural design of the unsupported length under scour conditions.

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6.4 Lateral Earth Pressures

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows:

- Angle of internal friction (ϕ) of 32 degrees;
- Total unit weight (γ) of 125 pcf; and
- Soil-concrete interface friction angle (δ) of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, Kp, of 6.89. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil based on LRFD Tables 3.11.6.4-1 and 3.11.6.4-2.

The abutment design shall include a drainage system behind the abutment to mitigate excessive hydrostatic pressures. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 “Granular Borrow for Underwater Backfill.”

Slopes in front of the pile supported integral abutments should be constructed with riprap and erosion control geotextile. The riprap slopes should not exceed 1.75:1(H:V) in accordance with MaineDOT Standard Detail 610(03). The 1.75H:1V riprap slopes shall “toe-in” at least 1 foot below the streambed.

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6.5 Design Considerations Related to Bedrock Type

The 2020 AASHTO LRFD subdivides rock into hard rock and soft rock. Article C.10.7.3.2.1 of 2020 AASHTO LRFD states that "*a definition of hard rock that relates to measurable rock characteristic has not been widely accepted*". However, FHWA manual NHI Course No. 132021, "Design and Construction of Driven Pile Foundations Volume I" states that the transition between soft and hard rock can be defined by an unconfined compressive strength (evaluated in accordance with ASTM standard method D7012) of the rock between 200 and 1,000 kilo-pound per square foot (ksf). Kulhawy proposed a value of the unconfined compressive strength of the rock of 400 ksf as the boundary between soft and hard rock. In absence of local experience, this value can be used to approach the design of piles bearing on rock.

We understand the proposed H-piles for Abutment No. 1 will extend to bedrock, while the H-piles for Abutment No. 2 will be grouted in-place within a pre-drilled, 5-ft-long rock socket. The design of H-piles driven onto hard rock (or grouted in-place within a suitable length of rock socket) is governed by structural aspects: a pile driven onto hard rock or socketed into rock will fail structurally before the hard rock fails. Piles driven to soft rock will penetrate rock, therefore the design of H-piles driven into soft rock is governed by geotechnical aspects. The FHWA manual NHI Course No. 132021 states that design of piles driven into soft rock could be approached using static capacity analysis (SCA) methods. However, "*degradation of the weak rock, reduction in shaft resistance due to shattering of the rock structure from driving adjacent piles, and formation of an enlarged hole around the pile hamper analytical methods*." Therefore, static, and dynamic field testing should be used to confirm nominal geotechnical pile resistances at the time of pile installation.

6.6 H-piles for Integral Abutments

Based on our review of preliminary design drawings, we understand that Abutments No. 1 and No. 2 are planned to be integral abutments, each supported on a single row of steel H-piles. The piles at Abutment No. 1 are planned to bear on hard rock The piles at Abutment No. 2 are planned to be pre-drilled a defined distance into hard rock and then grouted in-place.

The MaineDOT Bridge Design Guide prefers that sections HP 10x42, HP 12x53, HP 14x73, and HP 14x89 be considered in our analyses, and that H-piles used for bridge foundations be

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composed of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 ksi.

Table 8 – Estimated Pile Lengths

Location	Approximate Top of Pile Embedded into Integral Abutment Cap	Approximate Scour Contraction	Approx. Top of Rock	Estimated Pile Length**
Abutment No. 1 (West)	EL +6 feet	EL -6 feet	EL -19.9 feet	30 feet
Abutment No. 2 (East)	EL +6 feet	EL -6 feet	EL -10.5 feet	25 feet*

*Assumes 5-ft rock socket below top of rock.
**Lengths rounded to nearest whole 5-ft interval.

The above estimates do not account for variations between borings or for areas outside the boring locations. Additional lengths may be required.

Per MaineDOT design methodology, an initial pile area may be estimated based on a relationship between the maximum factored resistance of the factored vertical dead and live load applied to the superstructure divided by 80 percent of the member yield strength. Using this method, the pile section must have an area of at least 15.5 square inches.

Table 9 – Initial Section Size Based on Required Area

Pile Size	Minimum Size Required	A_s, Section Area*	Acceptable?
HP 10x43	15.5 in^2	12.4 in^2	No
HP 12x53	15.5 in^2	15.5 in^2	Yes
HP 14x73	15.5 in^2	21.4 in^2	Yes
HP 14x89	15.5 in^2	26.1 in^2	Yes

*Per Nucor Skyline, 2024.

6.6.1 Strength Limit State Design – Driven H-Piles

Pile foundations should be designed so that the available factored geotechnical and drivability resistance is greater than the factored loads applied to the pile at the strength limit state.

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Based on our review of both MaineDOT's Bridge Design Guide and AASHTO's LRFD Bridge Design Specifications, design of individual pile foundations that bear on or in hard rock should consider the factored structural pile resistance, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance, and the factored geotechnical resistance.

Pile groups should be designed to resist lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the abutments, along with the anticipated depth of scour contraction during the design flood event. The pile group resistance should provide adequate foundation resistance using the resistance factors discussed below:

- With respect to strength limit state design, an axial resistance factor of 0.50 should be applied for severe driving conditions and an axial resistance factor of 0.60 should be applied for good driving conditions.

- Because the H-piles will be subjected to lateral loading, the piles must also be evaluated for resistance against combined axial compression and flexure in accordance with LRFD Articles 6.9.2.2 and 6.15.2. For combined loading, an axial resistance factor of 0.70 and 1.0 should be applied to the combined axial and flexural resistance of the upper zone of the pile and lower zone of the pile, respectively.

Load combinations that do exceed the lateral load limits established for the service limit state should be evaluated by the Geotechnical Designer by means of a project-specific pile lateral load analysis using LPILE® software. Buckling analyses of piles should be performed by the Structural Designer. Piles should also be checked for resistance against combined axial loads and flexure per LRFD 6.15.2 and 6.9.2.2. Pile resistance should be determined for compliance with the LRFD interaction equation.

The maximum factored axial pile load should not exceed the lesser of the factored geotechnical resistance and factored structural resistance for a single pile. In accordance with LRFD Article 6.5.4.2, the factored pile loads should not exceed the factored structural resistance using the resistance factors provided in BDG 5.7.2 H-Piles. If greater loads result, more piles, or larger piles, should be considered.

Structural Resistance. The nominal axial compressive structural resistance for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The nominal axial structural

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compressive resistance subject to the combined axial compression and flexure shall be evaluated based on unbraced lengths and effective length factors as determined from LPile once structural loads are available. The nominal axial structural resistance should be evaluated based on combined axial compression and flexure.

The structural axial resistance for selected H-pile sections were calculated using the resistance factors discussed above. The unbraced pile lengths and effective length factors in these evaluations were assumed. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance based on unbraced lengths and effective length factors determined from LPile.

Geotechnical Resistance. Per LRFD Article 10.7.3.2.3 – *Piles Driven to Hard Rock*, “the nominal resistance (axial geotechnical resistance) of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal, is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified [above] in Article 6.5.4.2 and 6.15 for severe driving conditions. A pile driving acceptance criteria shall be developed that will prevent pile damage (see WEAP analysis below).”

Drivability Analyses. A pile drivability analysis was performed using the GRLWEAP computer program to determine a suitable hammer energy range with acceptable end-of-drive blow counts without exceeding the limiting compressive stresses within the piles during driving. Installation of the four typical steel H-pile sections preferred by MaineDOT were simulated using DELMAG diesel hammers within different rated energy ranges. Per MaineDOT Standard Specifications (2020, Division 500), pile stresses during normal driving should not exceed 90% of the specified yield stress of the pile material. Furthermore, per MaineDOT Bridge Design Guide (2003, Section 5.7.2.1), H- piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel, with a minim yield stress of 50 ksi. Therefore, the maximum driving stresses within a steel H-pile must not exceed 45 ksi during installation.

Per LRFD Table 10.5.5.2.3-1, we have assumed that at least one pile driven at Abutment No. 1 will be monitored by a PDA (pile dynamic analyzer) during installation. As such, a resistance factor of 0.65 was available when determining the factored resistance axial compression under dynamic loading.

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Summary of Strength Limit State Design. The table below provides a summary of the factored axial compressive structural, geotechnical, and drivability resistances of the selected H-pile sections with regards to strength limit state design, and the controlling condition.

Table 10 – Factored Axial Pile Resistance at Strength Limit State for Abutment No. 1

Pile Section	Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Controlling Axial Pile Resistance
HP 10 X 42	310	310	182	182
HP 12 X 53	388	388	234	234
HP 14 X 73	535	535	325	325*
HP 14 X 89	653	653	413	413*

*Values greater than the factored load applied to the superstructure for each pile (vertical dead and live), which as stated above, is 310 kips for this bridge project.

6.6.2 Strength Limit State Design – Predrilled H-Piles

The piles planned for support of Abutment No. 2 will not be driven. Rather, they are intended to be predrilled and then grouted into place. We recommend predrilling be completed with temporary steel casing and a “normal” drill bit so that the top of hard rock can be identified in the field based on performance criteria agreed to by the project stakeholders. Once the top of rock is identified and casing is set, a rock-coring bit should be used to advance the excavation a distance of 5 feet, or more, into hard rock.

Upon completion of coring activities, the drilling tools should be removed along with any drilling mud. Clean water should be injected into the bottom of the excavation to wash out any remaining cuttings and/or drilling mud so that a visual inspection of the rock socket might be accomplished. Once clean of debris and drilling mud, the remaining water should also be removed from the excavation. The open excavation should allow an experienced technician to use a mirror to direct sunlight down the hole for a visual inspection. The purpose of this visual inspection is to evaluate the need for additional coring due to poor quality rock.

The steel casing used above hard rock should only be removed after the steel H-pile has been placed into the excavation and grouted in-place. During removal, the top of wet grout should drop. The installation Contractor should be ready to add additional grout as needed to keep the top of grout at the planned elevation.

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Because the piles at Abutment No. 2 will not be driven and not subject to damage. The rock sockets will also be predrilled. Due to the strength of materials within the rock socket, which are significantly larger that the overburden soils, the geotechnical resistance provided herein is based on the rock socket alone, ignoring the overburden soils.

Furthermore, the difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing was considered when estimating axial compressive resistance of piles embedded in rock. If end bearing in rock were to be used as part of the axial compressive resistance, the contribution of skin friction in the rock socket would have to be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value. In this case, the available end bearing is much less than skin friction. As such, we will ignore end bearing and rely solely upon skin friction.

A resistance factor of 0.6 was used when considering the geotechnical resistance limit state of the steel H-piles grouted into a 5-foot-long rock socket. (This resistance factor is the practical limit presented in LRFD Table 10.5.5.2.3-1.)

Table 11 – Factored Axial Pile Resistance at Strength Limit State for Abutment No. 2

Pile Section	Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Controlling Axial Pile Resistance
HP 10 X 42	434	801	n/a	434*
HP 12 X 53	542	952	n/a	542*
HP 14 X 73	749	1118	n/a	749*
HP 14 X 89	913	1129	n/a	913*

*Values greater than the factored load applied to the superstructure for each pile (vertical dead and live), which as stated above, is 310 kips for this bridge project.

6.6.3 Service and Extreme Limit State Design

The design of H-piles at the service limit state must also consider tolerable transverse and longitudinal movement of piles and pile group movement considering changes in soil conditions due to scour based on the design flood event. For the service limit state, resistance factors of $\varphi = 1.0$ should be used for extreme events in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be evaluated at the Service I load combination and a resistance factor, φ of 0.65.

Extreme limit state design shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the design flood event can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\varphi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal axial geotechnical pile resistance at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. A summary of the calculated factored axial structural, geotechnical, and drivability resistances of selected H-piles for the extreme and service limit states are provided in the following table.

Table 12 – Factored Axial Pile Resistance at Service and Extreme Strength Limit State for Abutment No. 1

Pile Section	Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Structural Resistance	Controlling Axial Pile Resistance
HP 10 X 42	434	1456	280	280*
HP 12 X 53	542	1730	360	360*
HP 14 X 73	749	2033	500	500*
HP 14 X 89	913	2053	635	635*

*Values greater than the factored services load applied to the superstructure for each pile (vertical dead and live), which as stated above, is 210 kips for this bridge project.

Table 13 – Factored Axial Pile Resistance at Service and Extreme Strength Limit State for Abutment No. 2

Pile Section	Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Controlling Axial Pile Resistance
HP 10 X 42	434	1456	n/a	434*
HP 12 X 53	542	1730	n/a	542*

Table 13 – Factored Axial Pile Resistance at Service and Extreme Strength Limit State for Abutment No. 2

Pile Section	Factored Axial Pile Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Controlling Axial Pile Resistance
HP 14 X 73	749	2033	n/a	749*
HP 14 X 89	913	2053	n/a	913*

*Values greater than the factored services load applied to the superstructure for each pile (vertical dead and live), which as stated above, is 210 kips for this bridge project.

6.7 Lateral Resistance

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on loading conditions, the pier size, and the engineering properties of the pier. This type of analysis is typically performed by the Structural Engineer utilizing a computer analysis program and usually requires a trial-and-error procedure to appropriately size the piers and meet project tolerances.

To assist the design engineer in this procedure, we are providing the following soil parameters for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally loaded piles, the LPILE® software. The soil-related parameters required for input into the LPILE® software are summarized in the table below:

Table 14 – Lateral Load Parameters for use in L-Pile Analysis at Abutment No. 1

Depth Range, ft		Stratum	P-Y Model	Effective Unit Weight, pcf	Friction Angle, deg	K, pci	Uniaxial Comp. Str. of Rock, psi
Top	Bottom						
0	10	Existing Fill	Sand (Reese)	52.6	28	20	--
10	15	Buried Organic Matter	Sand (Reese)	42.6	26	20	--
15	28.7	Glacial Deposits	Sand (Reese)	62.6	32	65	--
28.7	--	Granite	Strong Rock (Vuggy Limestone)	97	--	--	1481
(If used)		Soil Type 4	Sand (Reese)	62.6	32	65	--

Table 15 – Lateral Load Parameters for use in L-Pile Analysis at Abutment No. 2

Depth Range, ft		Stratum	P-Y Model	Effective Unit Weight, pcf	Friction Angle, deg	K, pci	Uniaxial Comp. Str. of Rock, psi
Top	Bottom						
0	15	Existing Fill	Sand (Reese)	52.6	28	20	--
15	21.5	Glacial Deposits	Sand (Reese)	62.6	32	65	--
21.5	--	Granite	Strong Rock (Vuggy Limestone)	97	--	--	1481
(If used)		Soil Type 4	Sand (Reese)	62.6	32	65	--

The values presented above for subgrade modulus and the strain at 50% are based on recommended values for the L-Pile Plus program for the strength of materials encountered in our borings and are not necessarily based on laboratory test results.

The parameters presented in the above table do **not** include factors of safety. We recommend that a factor of safety of at least 2 be introduced to the analysis by doubling the applied lateral loads and moments.

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6.8 Installation Considerations

6.8.1 Wave Equation Analysis of Piles (WEAP)

The contractor performing the work should perform a hammer-specific WEAP analysis to determine the exact pile-hammer configuration to keep stresses within the given stress limits during driving. Piles should be driven to termination criteria determined by either wave equation or dynamic testing of the first pile driven.

As prescribed by Article C.10.7.3.2.3, pile tip protection (driving shoes) should be used for piles driven to bedrock at Abutment No. 1. Due to sloping bedrock at the site, driving shoes with teeth should be used to reduce the likelihood of the piles "walking" across the sloping bedrock.

6.8.2 Recommended Field Testing

As stated above, we recommend that dynamic pile load tests be performed using a Pile Dynamic Analyzer (PDA) with signal matching (CAPWAP) to check that the required nominal axial capacity of the pile has been achieved and that driving stresses do not exceed 90% of the yield stress of the pile during driving.

We recommend that PDA with signal matching at the beginning of redrive (BOR) be performed on a minimum of 2 piles at Abutment No. 1. The nominal resistance to be achieved during pile driving (to be verified by PDA) should be the required design pile axial nominal resistance divided by a resistance factor of 0.65.

6.9 Embankment Settlement

As stated above, we understand the vertical curve will be raised such that about 3 feet of fill will be required to reach finished grade at each abutment. As such, we evaluated the effect of raising grade 3 feet along each approach embankment using the Modified Hough Method as presented in FHWA's Soils and Foundations Reference Manual – Volume I (Samtani, 2006a). Results suggests that both approach embankments will experience about 1/2 inch of immediate settlement due the new fill required.

We anticipate most of this settlement will occur at the time of construction. That is, the 1/2 inch of settlement will most likely have already been accounted for by the time fine grading is undertaken and the planned pavement surface is installed. The design team should not expect any appreciable long-term settlement at either approach.

6.10 Integral Abutment Settlement

For abutments supported on piles bearing on bedrock or in a rock socket, settlement will result from elastic compression of the piles. Settlement of the bedrock under pile loads is negligible.

6.11 Slope Stability Analysis

Based on Kleinfelder's experience with similar structures and conditions, global stability of the new abutments is not a concern.

6.12 Seismic Considerations

Seismic analysis was not performed for the bridge per MaineDOT Section 5.2.5 which states that seismic analysis is not required for single-span bridges or any bridge in Seismic Zone 1 or SDC A.

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7 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by the Client and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The work performed was based on project information provided by Client. If Client does not retain Kleinfelder to review any plans and specifications, including any revisions or modifications to the plans and specifications, Kleinfelder assumes no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field to the plans and specifications, Client must obtain written approval from Kleinfelder's engineer that such changes do not affect our recommendations. Failure to do so will vitiate Kleinfelder's recommendations.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to confirm that the recommendations of this report are properly incorporated in the design of this project, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify

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our recommendations if variations in the soil conditions are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for placement of pavements;

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FIGURES

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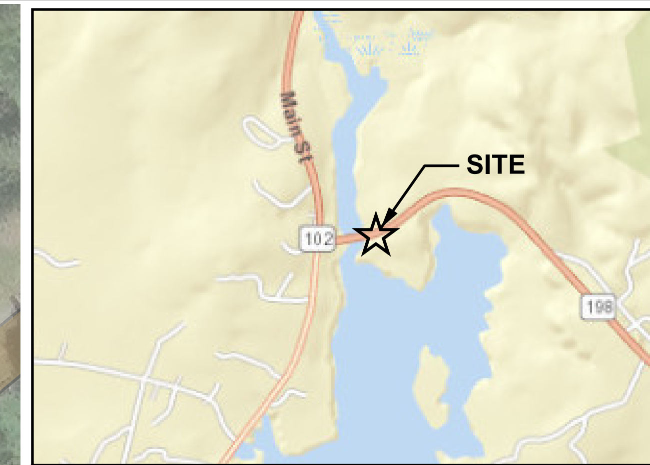

VICINITY MAP

NOT TO SCALE

NOTES:
1. BASE MAPPING AND VICINITY MAP CREATED FROM LAYERS COMPILED BY ESRI PRODUCTS.
COORDINATE SYSTEM: NAD 1983 2011 STATEPLANE MAINE EAST FIPS 1801

LEGEND

★ APPROXIMATE SITE LOCATION

N

0 100 200

1" = 100 SCALE IN FEET

GIS FILE PATH: \\azrgisstorp03\working_clients_Automated_Exploration Plans\20193610.002A_ReplacementofBabsonBridgeWIN23515_20240411_spanda\APRX\
GIS FILE NAME: gINT_20240411_2211_sp
PLOTTED: 4/11/2024 10:56 PM BY: SPANDA

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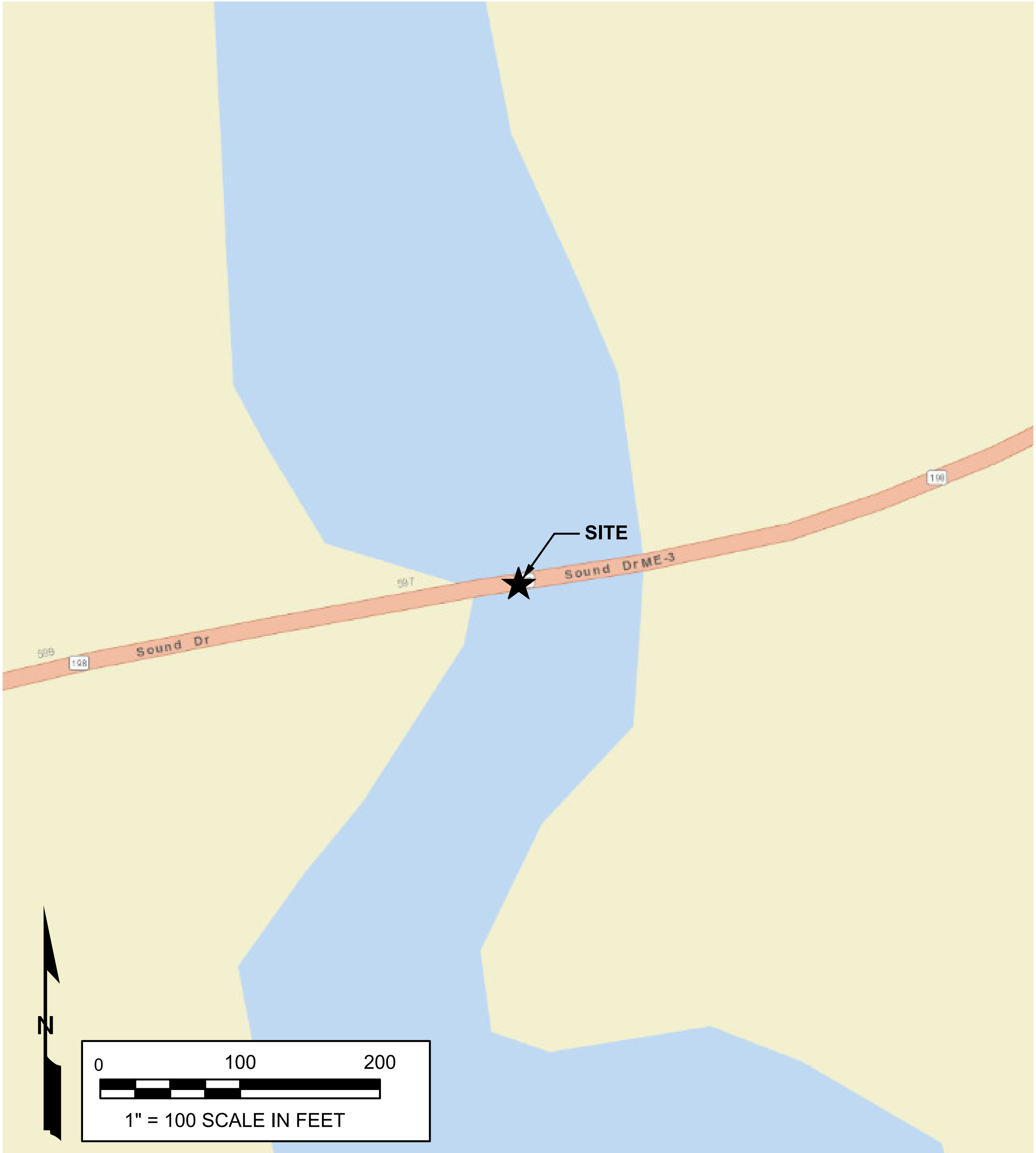
PROJECT NO. 20193610.002A		SITE LOCATION	FIGURE
DRAWN BY:	SPANDA	Replacement of Babson Bridge (WIN 23515) Sound Drive Over Kitteredge Brook Mount Desert, Maine	1
CHECKED BY:	KW		
DATE:	04-11-2024		



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CHECKED BY:	KW
DATE:	04-11-2024

SITE LOCATION

Replacement of Babson Bridge (WIN 23515)
Sound Drive Over Kitteredge Brook
Mount Desert, Maine

FIGURE
2

GIS FILE PATH: \\azrgisstorp03\working_clients_Automated_Exploration Plans\20193610.002A_ReplacementofBabsonBridgeWIN23515_20240411_spanda\APRX\
GIS FILE NAME: gINT_20240411_2211_sp
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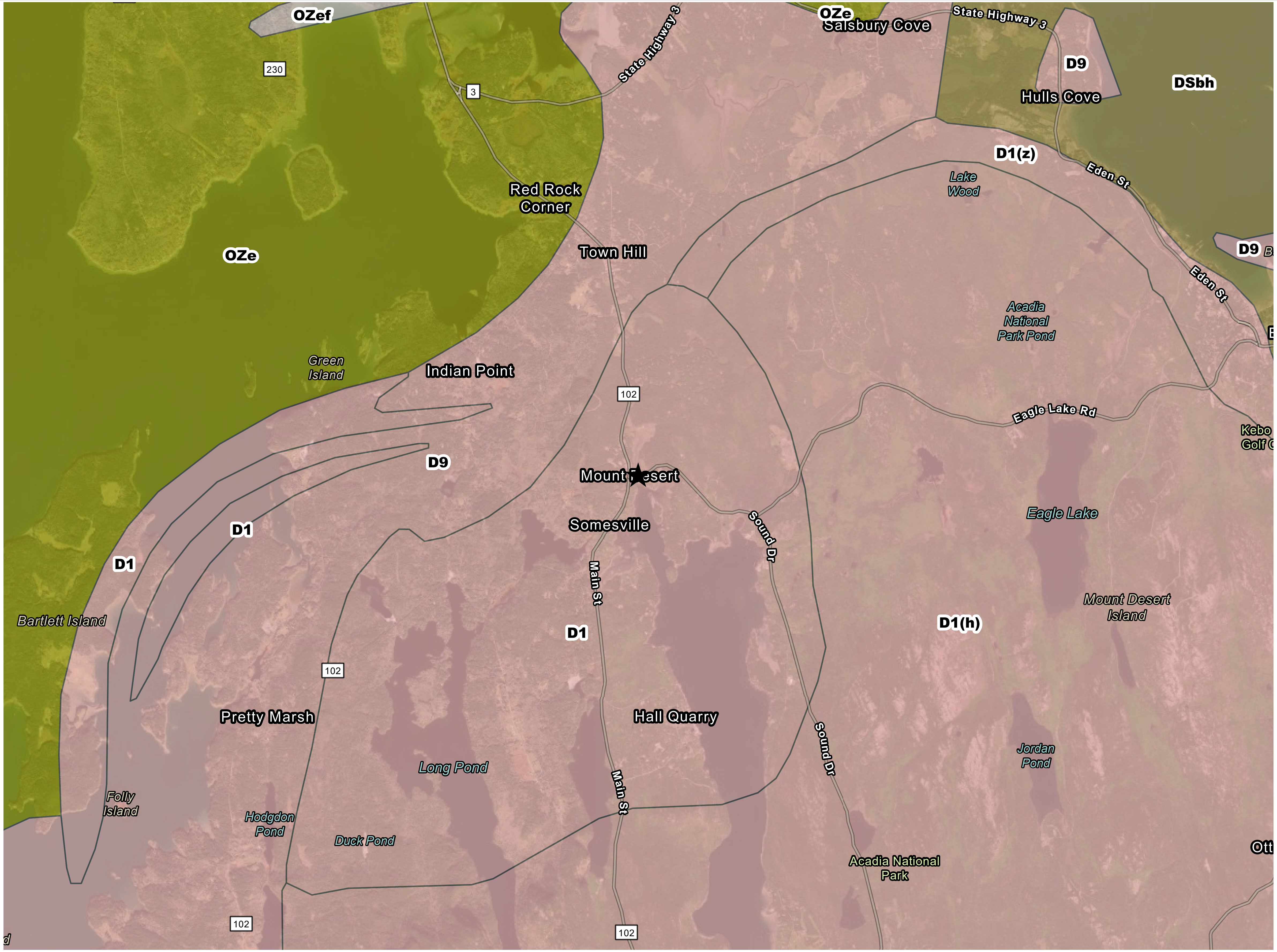

LEGEND

★ APPROXIMATE SITE LOCATION

USGS GEOLOGY

IGNEOUS, INTRUSIVE

METAMORPHIC AND SEDIMENTARY,

METAMORPHIC, OTHER

NOTES:
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THE STATE GEOLOGIC MAP COMPILATION OF THE CONTERMINOUS UNITED STATES WAS DEVELOPED BY THE U.S. GEOLOGICAL SURVEY MINERAL RESOURCES PROGRAM (MRP).

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1" = 5,000 SCALE IN FEET

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PROJECT NO. 20193610.002A		SURFICIAL GEOLOGY	FIGURE
DRAWN BY:	SPANDA	Replacement of Babson Bridge (WIN 23515)	3
CHECKED BY:	KW	Sound Drive Over Kitteredge Brook	
DATE:	04-11-2024	Mount Desert, Maine	

GIS FILE PATH: \\azrgisstorp03\working_clients_Automated_Exploration Plans\20193610.002A_ReplacementofBabsonBridgeWIN23515_20240411_spanda\APRX\
GIS FILE NAME: gINT_20240411_2211_sp
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LEGEND

★ APPROXIMATE SITE LOCATION

NOTES:
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N

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PROJECT NO. 20193610.002A		USGS TOPOGRAPHIC FIGURE	FIGURE
DRAWN BY:	SPANDA	Replacement of Babson Bridge (WIN 23515) Sound Drive Over Kitteredge Brook Mount Desert, Maine	4
CHECKED BY:	KW		
DATE:	04-11-2024		

GIS FILE PATH: \\azrgisstorp03\working_clients_Automated_Exploration Plans\20193610.002A_ReplacementofBabsonBridgeWIN23515_20240411_spanda\APRX\
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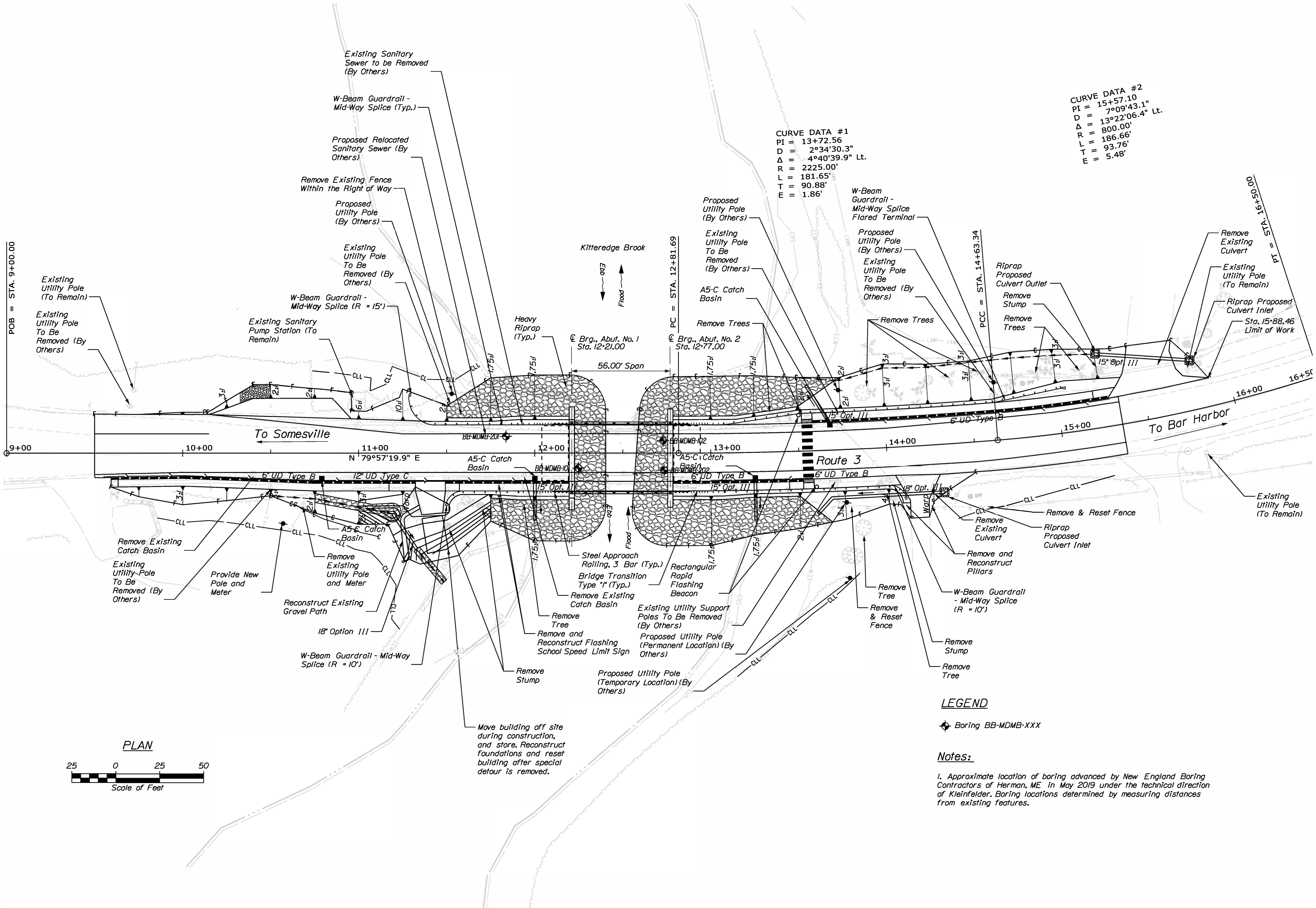


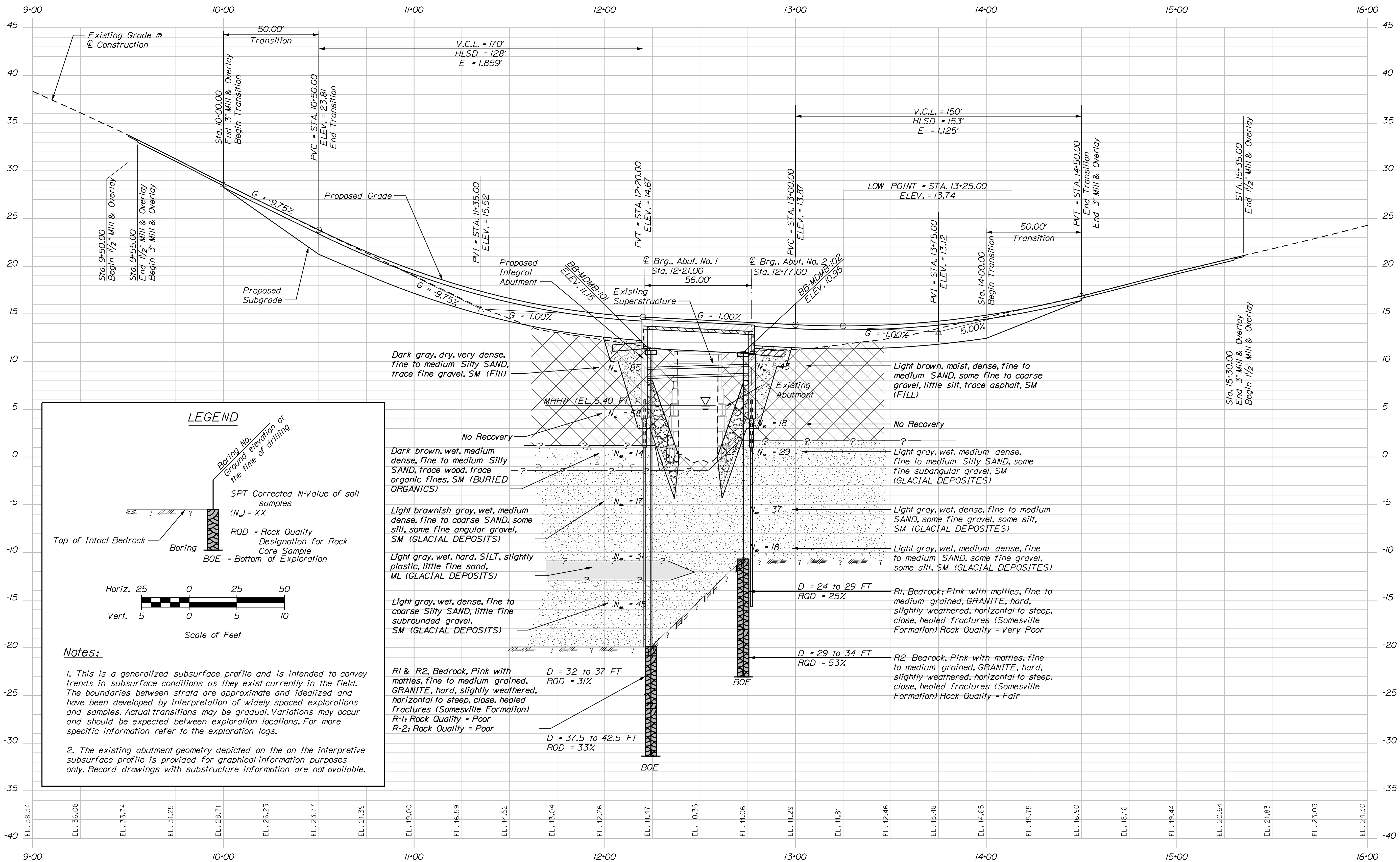
LEGEND

Boring BB-MDMB-XXX

Notes:

1. Approximate location of boring advanced by New England Boring Contractors of Herman, ME in May 2019 under the technical direction of Kleinfelder. Boring locations determined by measuring distances from existing features.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO. 20193610.002A	BORING LOCATION PLAN	FIGURE 5
	DRAWN BY: KW CHECKED BY: RLT DATE: 7/30/2024	REPLACEMENT OF BABSON BRIDGE OVER KITTEREDGE BROOK (BRIDGE NO. 5244) ROUTE 3\198, WIN 23515.00 MOUNT DESERT, MAINE	



PROFILE

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PROJECT NO. 20193610.002A	INTERPRETIVE SUBSURFACE PROFILE	FIGURE
DRAWN BY: KW	REPLACEMENT OF BABSON BRIDGE OVER KITTEREDGE BROOK (BRIDGE NO. 5244) ROUTE 3\198, WIN 23515.00 MOUNT DESERT, MAINE	6
CHECKED BY: RLT		
DATE: 7/30/2024		

Summary of Subsurface Conditions at Abutment No. 1 (West)
Maine Department of Transportation (MaineDOT)
Replacement of Babson's Bridge (Bridge Number 5244, WIN 23515.00), Sound Drive, Mount Desert, Maine

Boring ID	Top of Boring El. (ft)	Bottom of Boring El. (ft)	Boring Depth (ft)	GW Depth. (ft)	GW El. (ft)	Asphalt		Upper Fill		Buried Organics		Lower Fill		Glacial Deposits		Bedrock	
						Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)
BB-MDMB-101	11.2	-31.3	42.5	5.5	5.7	11.2	0.6	10.6	9.4	1.2	2.5	-1.3	2.5	-3.8	16	-19.8	11.5
BB-MDMB-201	12.3	-35.2	47.5	1.3	11.0	12.3	0.8	11.5	9.2	2.3	5.0	-	-	-2.7	21.5	-24.2	11

Notes:

"-": Indicates stratum or groundwater was not encountered

Soil strata in the general order of their occurrence in the borings are presented from left to right in this table

The groundwater levels herein only represent the conditions encountered at the location and time indicated. Water is introduced in the boreholes during drilling using drive and wash drilling techniques. Therefore, the groundwater measurements may not be representative of actual field groundwater conditions. Furthermore, groundwater levels fluctuate due to local and regional factors including seasonal changes, well pumping, and periods of wet or dry weather, nearby construction dewatering, infiltration basins, etc.

Boring ground surface elevations (El.) are based on a drawing titled "Babson Bridge - Kitteredge Brook - Mount Desert - Hancock County - Boring Location Plan " prepared by Kleinfelder dated January 2024

Boring BB-MDMB-101 drilled by NEBC of Hermon, Maine between May 13, 2019 and May 14, 2019 using drive and wash with casing drilling techniques

Boring BB-MDMB-201 drilled by NEBC of Hermon, Maine on May 26, 2022 using drive and wash with casing drilling techniques

Summary of Subsurface Conditions at Abutment No. 2 (East)
Maine Department of Transportation (MaineDOT)
Replacement of Babson's Bridge (Bridge Number 5244, WIN 23515.00), Sound Drive, Mount Desert, Maine

Boring ID	Top of Boring El. (ft)	Bottom of Boring El. (ft)	Boring Depth (ft)	GW Depth. (ft)	GW El. (ft)	Asphalt		Fill		Glacial Deposits		Bedrock	
						Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)	Stratum Top El. (ft)	Stratum Thickness (ft)
BB-MDMB-102	11.0	-23.0	34.0	5.8	5.2	11.0	0.6	10.4	8.7	1.7	12.3	-10.6	12.4
BB-MDMB-202	10.8	-27.2	38.0	1.0	9.8	10.8	0.6	10.2	14.4	-4.2	8.5	-12.7	14.5

Notes:

"-": Indicates stratum or groundwater was not encountered

Soil strata in the general order of their occurrence in the borings are presented from left to right in this table

The groundwater levels herein only represent the conditions encountered at the location and time indicated. Water is introduced in the boreholes during drilling using drive and wash drilling techniques. Therefore, the groundwater measurements may not be representative of actual field groundwater conditions. Furthermore, groundwater levels fluctuate due to local and regional factors including seasonal changes, well pumping, and periods of wet or dry weather, nearby construction dewatering, infiltration basins, etc.

Boring ground surface elevations (El.) are based on a drawing titled "Babson Bridge - Kitteredge Brook - Mount Desert - Hancock County - Boring Location Plan " prepared by Kleinfelder dated January 2024

Boring BB-MDMB-102 drilled by NEBC of Hermon, Maine between May 14, 2019 and May 15, 2019 using drive and wash with casing drilling techniques

Boring BB-MDMB-202 drilled by NEBC of Hermon, Maine on May 25, 2022 using drive and wash with casing drilling techniques

Summary of L-Pile Parameters for Abutment 1 & 2
Maine Department of Transportation (MaineDOT)
Replacement of Babson's Bridge (Bridge Number 5244, WIN 23515.00), Sound Drive, Mount Desert, Maine

NAVD 88 Elevation Range (ft)		Depth Range (ft)		Stratum	LPILE Soil/Rock Type	Effective Unit Weight (pcf)	Undrained Cohesion (psf)	Friction Angle (deg)	Non-Default Strain Factor Ɛ50	Non-Default k value (pci)	Uniaxial Compressive Strength For Rock (psi)
Top	Bottom	Top	Bottom								
Abutment 1 (based on BB-MDMB-101 and BB-MDMB-201)											
11.2	1.2	0	10	Existing Fill	Sand (Reese)	52.6		28		20	
1.2	-3.8	10	15	Buried Organics	Sand (Reese)	42.6		26		20	
-3.8	-17.5	15	28.7	Glacial Deposits	Sand (Reese)	62.6		32		65	
-17.5	-	28.7	-	Granite/Syenite Bedrock	Strong Rock (Vuggy Limestone)	97					1481
Abutment 2 (based on BB-MDMB-102 and BB-MDMB-202)											
11	-4	0	15	Existing Fill	Sand (Reese)	52.6		28		20	
-4	-10.5	15	21.5	Glacial Deposits	Sand (Reese)	62.6		32		65	
-10.5	-	21.5	-	Granite/Syenite Bedrock	Strong Rock (Vuggy Limestone)	97					1481

Notes: (1) Design groundwater elevation assumed to be at El. 10 feet
(2) Soil profiles based on subsurface conditions encountered in test borings BB-MDMB-101 and BB-MDMB-201 for Abutment 1 and BB-MDMB-102 and BB-MDMB-202 for Abutment 2.
(3) Elevations are based on the subsurface data obtained from the referenced test borings

APPENDIX A

LOGS OF BORINGS ADVANCED BY KLEINFELDER

MaineDOT Soil Description Key
Soil Boring Logs and Rock Core Logs – Preliminary Design Borings (2019)
Soil Boring Logs and Rock Core Logs – Final Design Borings (2022)

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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-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-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: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive, Mount Desert, ME

Boring No.: BB-MDMB-101
WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	11.18	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/13/2019 - 5/14/2019	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	Sta. 12+24.78, 8.57' RT, N 195600.9542, E 2177864.8225	**Casing ID/OD:**	HW-4"	**Water Level*:**	5.5 ft at 7:50 AM on 5/14/19
Hammer Efficiency Factor:	0.923	**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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN HW	10.6	6.5" BITUMINOUS PAVEMENT — 0.5	
	1D	24/10	1.00 - 3.00	23/11/44/27	55	85			1D: Top 5": Dark gray, dry, very dense, fine to medium Silty SAND, trace fine gravel, SM (FILL) Bottom 5": Light brown, moist, very dense, fine to medium SAND, little silt, little fine to coarse subangular gravel, SP-SM (FILL)	
5	2D	24/0	5.00 - 7.00	14/29/9/12	38	58			2D: No Recovery Drilling action was slow from 6 to 10 feet. Observed rock fragments in the drill water due to possible cobbles and boulders.	
10	3D	24/6	10.00 - 12.00	9/5/4/6	9	14		1.2	— 10.0 3D: Dark brown, wet, medium dense, fine to medium Silty SAND, trace wood, trace organic fines, SM (BURIED ORGANICS)	
	R1	24/24	12.50 - 14.50				NQ-2	-1.3	— 12.5 R1: Boulder: min/ft: 2.0, 1.8 Switched to 3" casing at 14.5 feet and roller bit to 15 feet and sampled	
15	4D	24/6	15.00 - 17.00	6/4/7/16	11	17	SPIN HW	-3.8	— 15.0 4D: Light brownish gray, wet, medium dense, fine to coarse SAND, some silt, some fine angular gravel, SM (GLACIAL DEPOSITS)	% Silt = 27.7 % Sand = 41.1 % Gravel = 31.2 A-2-4
20									Encountered possible cobbles at 20 feet. Spin casing down to 21 feet and sampled at 21 feet.	
	5D	24/17	21.00 - 23.00	17/10/10/15	20	31			5D - Top 12": Similar to 4D, except dense (GLACIAL DEPOSITS)	
								-10.8	— 22.0 Bottom 5": Light gray, wet, hard, SILT, slightly plastic, little fine sand, ML (GLACIAL DEPOSITS)	
								-12.8	— 24.0 Assumed lithology change	
25										

Remarks:

Truck Mounted Mobile B-53 Drill Rig
NH License Plate No. 4368

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: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive, Mount Desert, ME

Boring No.: BB-MDMB-101
WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	11.18	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/13/2019 - 5/14/2019	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	Sta. 12+24.78, 8.57' RT, N 195600.9542, E 2177864.8225	**Casing ID/OD:**	HW-4"	**Water Level*:**	5.5 ft at 7:50 AM on 5/14/19
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							SPIN HW				
	6D	24/12	26.00 - 28.00	5/12/17/20	29	45				6D: Light gray, wet, dense, fine to coarse Silty SAND, little fine subrounded gravel, SM (GLACIAL DEPOSITS)	% Silt = 40.4 % Sand = 42.6 % Gravel = 17 A-4
30											
								-19.8		31.0	
										Observed pink rock fragment in the drill cuttings at about 31 feet. Presumed top of rock.	
	R2	60/54	32.00 - 37.00	RQD = 31%			NQ-2			R2: Bedrock: Pink with black mottles, fine-medium grained, GRANITE, hard, slightly weathered, horizontal to steep, close, healed fractures Somesville Formation, RQD = 31%, poor, Rec = 54"/60" (90%) Rock core rates: 31 ft-32 ft (2:00), 32 ft-33 ft (2:00), 33 ft-34 ft (1:30), 34 ft-35 ft (1:48), 35 ft-36 ft (1:54)	
35											
										Core barrel jammed at about 37 feet. Roller bit from 37 to 37.5 feet and cored at 37.5 feet.	
	R3	60/54	37.50 - 42.50	RQD = 33%						R3: Bedrock: Pink with black mottles, fine-medium grained, GRANITE, hard, slightly weathered, horizontal to steep, close, healed fractures Somesville Formation, RQD = 33%, poor, Rec = 54"/60" (90%) Rock core rates: 37.5 ft-38.5 ft (5:00), 38.5 ft-39.5 ft (1:48), 39.5 ft-40.5 ft (1:48), 40.5 ft-41.5 ft (2:0), 41.5 ft-42.5 ft (1:48)	
40											
								-31.3		42.5	
										Bottom of Exploration at 42.5 feet below ground surface. Backfilled borehole with drill cuttings and 10.5 bags of gravel. Restored ground surface with asphalt cold patch.	
45											
50											

Remarks:

Truck Mounted Mobile B-53 Drill Rig
NH License Plate No. 4368

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Boring No.: BB-MDMB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive
Mount Desert, ME

Boring No.: BB-MDMB-102

WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	10.96	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/14/2019 - 5/15/2019	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	Sta. 12+72.89, 7.05' LT, N 195624.7329, E 2177909.4615	**Casing ID/OD:**	HW-4"	**Water Level*:**	5.8 ft at 7:50 AM on 5/15/19
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected

T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN HW	10.4		6.5" BITUMINOUS PAVEMENT 0.5	
	1D	24/12	1.00 - 3.00	37/17/11/17	28	43				1D: Light brown, moist, dense, fine to medium SAND, some fine to coarse gravel, little silt, trace asphalt, SM (FILL)	% Silt = 19.7 % Sand = 52.5 % Gravel = 27.8 A-1-b
										Drill rig chattered from 3 to 6 feet on possible cobbles and boulders.	
5											
	2D	24/0	6.00 - 8.00	9/5/7/10	12	18				2D: No Recovery, angular rock fragment in tip of spoon Drill rig chattered from 6 to 9.3 feet.	
	3D	24/4	9.30 - 11.30	5/12/7/11	19	29		1.7		9.3 3D: Light gray, wet, medium dense, fine to medium Silty SAND with Gravel, some fine subangular gravel, SM (GLACIAL DEPOSITS)	
10											
15	4D	24/6	15.00 - 17.00	4/12/12/13	24	37				4D: Light gray, wet, dense, fine to medium SAND, some fine gravel, some silt, SM (GLACIAL DEPOSITS)	% Silt = 26.2 % Sand = 45.2 % Gravel = 30.6 A-2-4
	5D	19/15	19.00 - 20.58	6/2/10/(60/1)	12	18				5D: Similar to 4D except medium dense, some fine gravel, angular rock fragment at tip of spoon (GLACIAL DEPOSITS)	
20											
								-10.6		21.6 Observed pink rock fragments in the drill cuttings from 21.6 to 24 feet. Presumed top of rock at 21.6 feet.	
25	R1	60/60	24.00 - 29.00	RQD = 25%			NQ-2			R1: Bedrock: Pink with black mottles, fine to medium-grained, GRANITE, hard, slightly weathered, horizonal to steep, close, healed	

Remarks:

Truck Mounted Mobile B-53 Drill Rig
NH License Plate No. 4368

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive, Mount Desert, ME

Boring No.: BB-MDMB-102
WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	10.96	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	M. Chea	**Rig Type:**	Mobile Drill Truck-Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/14/2019 - 5/15/2019	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	Sta. 12+72.89, 7.05' LT, N 195624.7329, E 2177909.4615	**Casing ID/OD:**	HW-4"	**Water Level*:**	5.8 ft at 7:50 AM on 5/15/19
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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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.
25							NQ-2			fractures Somesville Formation, RQD = 25%, very poor, Rec = 60"/60" (100%) Rock core rates: 24 ft-25 ft (6:30), 25 ft-26 ft (3:24), 26 ft-27 ft (2:54), 27 ft-28 ft (2:54), 28 ft-29 ft (2:36)	
	R2	60/55	29.00 - 34.00	RQD = 53%						R2: Bedrock: Pink with black mottles, fine to medium-grained, GRANITE, hard, slightly weathered, horizonal to steep, close, healed fractures Somesville Formation, RQD = 53%, fair, Rec = 55.2"/60" (92%) Rock core rates: 29 ft-30 ft (2:42), 30 ft-31 ft (1:54), 31 ft-32 ft (2:12), 32 ft-33 ft (1:54), 33 ft-34 ft (1:30)	
30											
								-23.0		34.0 **Bottom of Exploration at 34.0 feet below ground surface.** Backfilled borehole with drill cuttings and 5 bags of gravel. Restored ground surface with asphalt cold patch.	
35											
40											
45											
50											

Remarks:

Truck Mounted Mobile B-53 Drill Rig
NH License Plate No. 4368

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive, Mount Desert, Maine

Boring No.: BB-MDMB-201
WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	12.27	**Auger ID/OD:**	
Operator:	Tom C & Mark T	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	N. Jamba	**Rig Type:**	Mobile Drill Truck - Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/26/2022 - 5/26/2022	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	STA 11+83.60, 9.51' LT, N 195611.5835, E 2177821.1094	**Casing ID/OD:**	HW-4"	**Water Level*:**	15-in at 8:30 AM on 5/26/22
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140lb. Hammer
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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_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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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							SPIN HW			9" BITUMINOUS PAVEMENT	
								11.5		0.8	
	1D	24/20	1.00 - 3.00	27/74/49/27	123	189				Dark gray, very dense, fine to medium SAND, little silt, little gravel. (FILL)	
	2D	24/17	3.00 - 5.00	5/5/5/7	10	15				Light gray, medium dense, fine to medium SAND, little silt, little gravel. (FILL)	
5	3D	24/16	5.50 - 7.50	3/6/3/4	9	14				Light gray, moist, medium dense, fine to medium SAND, little silt, little gravel. (FILL)	
10								2.3		10.0	
	4D	24/12	10.00 - 12.00	1/6/4/9	10	15				Light gray, medium dense, fine to medium SAND, some silt, trace wood. (BURIED ORGANICS)	
15								-2.7		15.0	
	5D	24/14	15.00 - 17.00	25/27/19/28	46	71				Light brown, wet, very dense, fine to coarse SAND, some gravel, some silt (GLACIAL DEPOSITS).	Silt=21.4% Sand=51.9% Gravel=26.7% A-1-b (SM)
20	6D	24/16	20.00 - 22.00	18/31/29/29	60	92				Light brown, wet, very dense, fine to coarse SAND, some gravel, some silt (GLACIAL DEPOSITS).	
25											

Remarks:

Truck-Mounted Mobile B-53 Drill Rig

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Boring No.: BB-MDMB-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babson Bridge (#5244) over Kitteredge Brook
Location: Sound Drive, Mount Desert, Maine

Boring No.: BB-MDMB-201
WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	12.27	**Auger ID/OD:**	
Operator:	Tom C & Mark T	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	N. Jamba	**Rig Type:**	Mobile Drill Truck - Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/26/2022 - 5/26/2022	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	STA 11+83.60, 9.51' LT, N 195611.5835, E 2177821.1094	**Casing ID/OD:**	HW-4"	**Water Level*:**	15-in at 8:30 AM on 5/26/22
Hammer Efficiency Factor: 0.923		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	7D	24/15	25.00 - 27.00	20/25/28/30	53	82	SPIN HW			Light gray, wet, very dense, fine to coarse SAND, some silt, some gravel (GLACIAL DEPOSITS).	Silt=24.8% Sand=51.4% Gravel=23.8% A-1-b (SM)
30	8D	24/14	30.00 - 32.00	21/32/32/23	64	98				Light gray, wet, very dense, fine to coarse SAND, some silt, some gravel (GLACIAL DEPOSITS).	
35	9D	24/12	35.00 - 37.00	20/31/29/50	60	92				Light gray, wet, very dense, fine to coarse SAND, some silt, some gravel (GLACIAL DEPOSITS).	
								-24.2		36.5	
	R1	60/60	37.50 - 42.50	RQD = 33%			NQ-2			Observed pink rock fragment in the drill cuttings at about 36.5 feet. Presumed top of rock at 36.5 feet bgs. R1: Bedrock: Pink with black mottles, fine grained, GRANITE, moderately hard, slightly weathered, horizontal to steep, very close to moderately close, healed fractures, Somesville Formation, RQD = 33%, poor, Rec = 60"/60"(100%) Rock core rates: 37.5 ft-38.5 ft (2:34), 38.5 ft-39.5 ft (1:58), 39.5 ft-40.5 ft (2:07), 40.5 ft-41.5 ft (2:10), 41.5 ft-42.5 ft (2:34)	
40											
	R2	60/60	42.50 - 47.50	RQD = 26%						R2: Pink with black mottles, fine grained, GRANITE, moderately hard, slightly weathered, horizontal to steep, very close to moderately close, healed fractures, Somesville Formation, RQD = 26%, poor, Rec = 60"/60"(100%) Rock core rates: 42.5 ft-43.5 ft (3:50), 43.5 ft-44.5 ft (2:10), 44.5 ft-45.5 ft (2:20), 45.5 ft-46.5 ft (2:30), 46.5 ft-47.5 ft (2:20)	
45											
								-35.2		47.5	
										Bottom of Exploration at 47.5 feet below ground surface. Backfilled borehole with drill cuttings and 5 bags of gravel. Restored ground surface with asphalt cold patch.	
50											

Remarks:

Truck-Mounted Mobile B-53 Drill Rig

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Babson Bridge (#5244) over Kitteredge Brook Location: Sound Drive, Mount Desert, Maine	Boring No.: BB-MDMB-202 WIN: 23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	10.77	**Auger ID/OD:**	
Operator:	Tom C & Mark T	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	N. Jamba	**Rig Type:**	Mobile Drill Truck - Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/25/2022 - 5/25/2022	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	STA 12+73.43, 9.79' RT, N 195608.2506, E 2177912.9315	**Casing ID/OD:**	HW-4"	**Water Level*:**	12-in at 10:05 AM on 5/25/22
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = Weight of Rods or Casing
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SPIN HW	10.2		7" BITUMINOUS PAVEMENT 0.5	
	1D	24/10	1.00 - 3.00	15/18/3/8	21	32		8.8		Dark gray, dense, fine to medium SAND, little silt, little gravel, trace asphalt. (FILL) 2.0 Drill rig chattered from 2 to 5 feet on possible cobbles and boulders.	
5	2D	24/0	5.00 - 7.00	5/9/6/5	15	23		5.8		5.0 No Recovery	
10	3D	24/10	10.00 - 12.00	5/7/6/14	13	20				Light gray, medium dense, fine to medium SAND, little silt, little gravel. (FILL)	
15	4D	24/6	15.00 - 17.00	10/19/25/40	44	68		-4.2		15.0 Light gray, wet, very dense, gravelly SAND, trace silt (GLACIAL DEPOSITS). Drill rig chattered from about 19 to 20 feet. Observed sand and rock fragments in the drill cuttings.	Silt=9.3% Sand=51.4% Gravel=39.3% A-1-a (SW-SM)
20	5D	24/15	20.50 - 22.50	15/24/24/22	48	74				Light gray, wet, very dense, Silty SAND, some gravel. (GLACIAL DEPOSITS) Rock fragment at tip of the spoon.	Silt=38.2% Sand=40.4% Gravel=21.4% A-4(0) (SM)
								-12.7		23.5 Observed pink rock fragments in the drill cuttings from 23.5 to 27.5 feet. Presumed top of rock at 23.5 feet bgs.	
25											

Remarks:

Truck-Mounted Mobile B-53 Drill Rig

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Babson Bridge (#5244) over Kitteredge Brook	**Location:** Sound Drive Mount Desert, Maine	**Boring No.:**	BB-MDMB-202
		WIN:	23515.00

Driller:	New England Boring Contractor	**Elevation (ft.)**	10.77	**Auger ID/OD:**	
Operator:	Tom C & Mark T	**Datum:**	NAVD88	**Sampler:**	SPT/Split spoon
Logged By:	N. Jamba	**Rig Type:**	Mobile Drill Truck - Mounted	**Hammer Wt./Fall:**	140lbs/30"
Date Start/Finish:	5/25/2022 - 5/25/2022	**Drilling Method:**	Drive and Wash/Coring	**Core Barrel:**	NQ (2" OD)
Boring Location:	STA 12+73.43, 9.79' RT, N 195608.2506, E 2177912.9315	**Casing ID/OD:**	HW-4"	**Water Level*:**	12-in at 10:05 AM on 5/25/22
Hammer Efficiency Factor:	0.923	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							SPIN HW				
	R1	60/60	27.50 - 32.50	RQD = 20%			NQ-2			R1: Bedrock: Pink with black mottles, fine-grained, GRANITE, moderately hard, slightly weathered, horizontal to steep very close to moderately close, open fractures Somesville Formation, RQD = 20%, very poor, Rec = 60"/60"(100%) Rock core rates: 27.5 ft-28.5 ft (2:10), 25.5 ft-29.5 ft (1:51), 29.5 ft-30.5 ft (1:97), 30.5 ft-31.5 ft (1:35), 31.5 ft-32.5 ft (2:05)	
30											
	R2	32/32	33.00 - 35.67	RQD = 8%						R2: Pink with black mottles, fine grained, GRANITE, moderately hard, slightly weathered, horizontal to steep, very close to close, healed fractures Somesville Formation, RQD = 8%, very poor, Rec = 32"/32" (100%) Rock core rates: 33.0 ft-34.0 ft (1:30), 34.0 ft-35.0 ft (2:10), 35.0 ft-35.7 ft (1:50).	
35	R3	27/27	35.70 - 37.95	RQD = 0%						R3: Pink with black mottles, fine-grained, GRANITE, moderately hard, slightly weathered, horizontal to steep, very close, healed fractures, Somesville Formation, RQD = 0%, very poor, Rec = 27"/ 27"(100%) Rock core rates: 35.7 ft-36.7 ft (1:10), 36.7 ft-38.0 ft (2:30).	
								-27.2		38.0 **Bottom of Exploration at 38.0 feet below ground surface.** Backfilled borehole with drill cuttings and 5 bags of gravel. Restored ground surface with asphalt cold patch.	
40											
45											
50											

Remarks:

Truck-Mounted Mobile B-53 Drill Rig

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

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

APPENDIX B

LABORATORY TEST RESULTS

Preliminary Design (Geo Testing Express):
Grain Size Analysis of Soil Samples, Bulk Density and Compressive Strength of Rock Cores

Final Design (Thielsch Engineering Inc.):
Grain Size Analysis and Chloride Analysis of Soil Samples, Compressive Strength of Rock Cores

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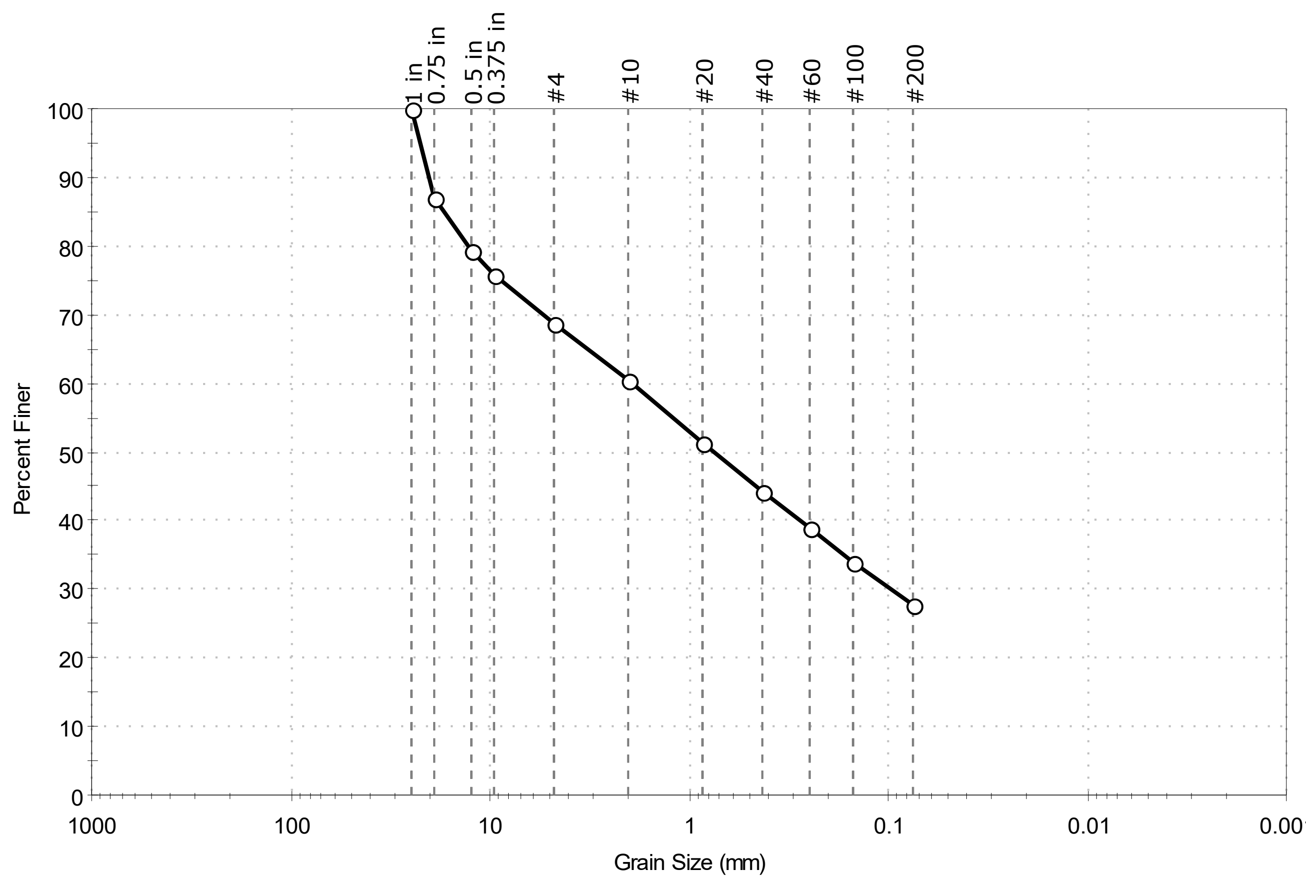
Client:	Kleinfelder, Inc.			
Project:	Replace Babsun's Bridge on Sound Dr			
Location:	Mt. Desert, ME		Project No:	GTX-310167
Boring ID:	BB-MDMB-101	Sample Type: jar	Tested By:	ckg
Sample ID:	S-4	Test Date: 06/29/19	Checked By:	bfs
Depth :	15-17	Test Id: 510080		
Test Comment:	---			
Visual Description:	Moist, olive silty sand with gravel			
Sample Comment:	---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	31.2	41.1	27.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	87		
0.5 in	12.50	79		
0.375 in	9.50	76		
#4	4.75	69		
#10	2.00	60		
#20	0.85	51		
#40	0.42	44		
#60	0.25	39		
#100	0.15	34		
#200	0.075	28		

Coefficients

D_{85} =17.0801 mm	D_{30} =0.0966 mm
D_{60} =1.9126 mm	D_{15} =N/A
D_{50} =0.7505 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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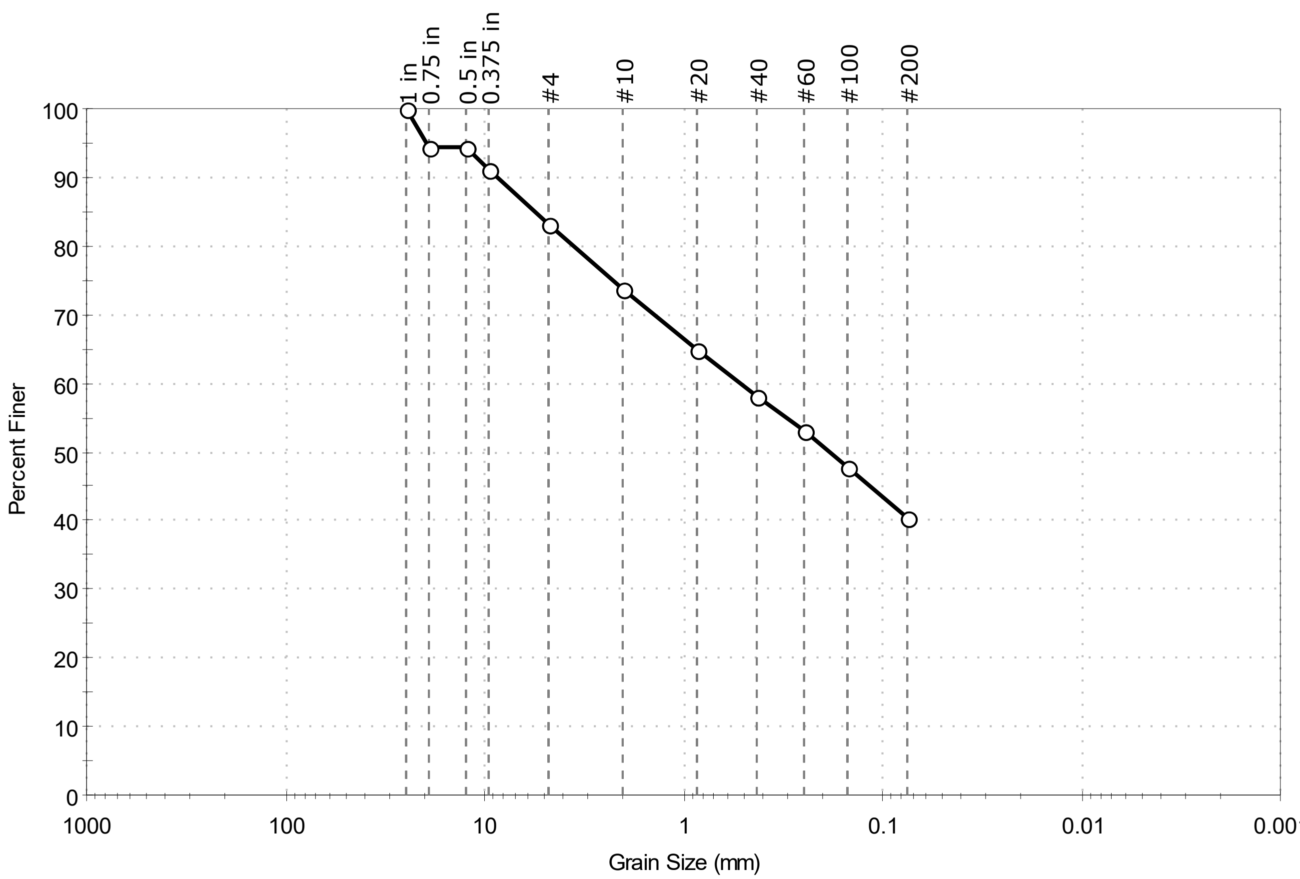
Client:	Kleinfelder, Inc.		
Project:	Replace Babsun's Bridge on Sound Dr		
Location:	Mt. Desert, ME	Project No:	GTX-310167
Boring ID: BB-MDMB-101	Sample Type: jar	Tested By:	ckg
Sample ID: S-6	Test Date: 06/29/19	Checked By:	bfs
Depth : 26-28	Test Id: 510081		
Test Comment:	---		
Visual Description:	Moist, olive brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	17.0	42.6	40.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	94		
0.375 in	9.50	91		
#4	4.75	83		
#10	2.00	74		
#20	0.85	65		
#40	0.42	58		
#60	0.25	53		
#100	0.15	48		
#200	0.075	40		

Coefficients

D_{85} =5.6113 mm — D_{30} =N/A

D_{60} =0.5154 mm — D_{15} =N/A

D_{50} =0.1868 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

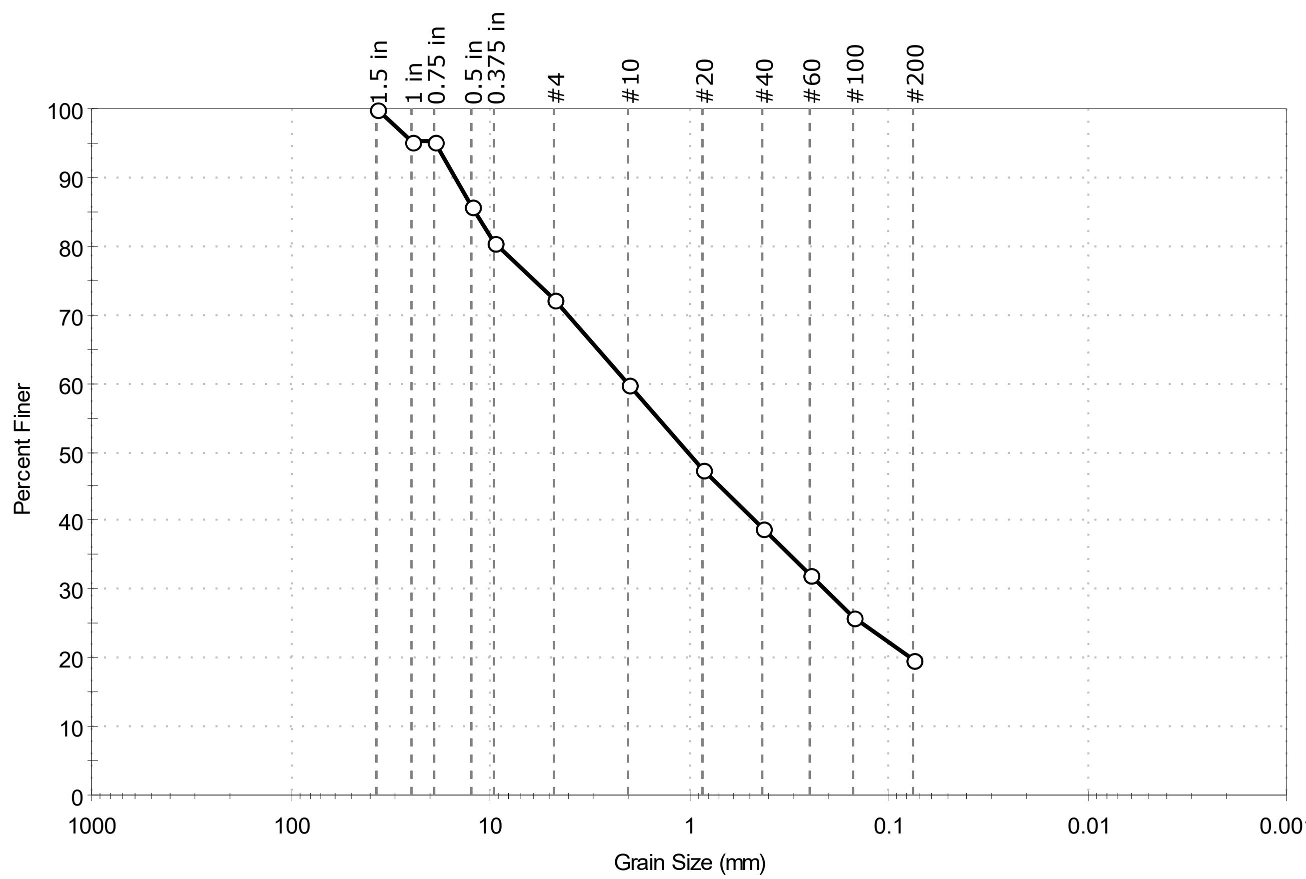
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 7/3/2019 12:24:20 PM

Client: Kleinfelder, Inc.			
Project: Replace Babsun's Bridge on Sound Dr			
Location: Mt. Desert, ME		Project No:	GTX-310167
Boring ID: BB-MDMB-102	Sample Type: jar	Tested By:	ckg
Sample ID: S-1	Test Date: 07/02/19	Checked By:	bfs
Depth : 1-3	Test Id: 510082		
Test Comment: ---			
Visual Description: Moist, dark yellowish brown silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	27.8	52.5	19.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	95		
0.75 in	19.00	95		
0.5 in	12.50	86		
0.375 in	9.50	81		
#4	4.75	72		
#10	2.00	60		
#20	0.85	48		
#40	0.42	39		
#60	0.25	32		
#100	0.15	26		
#200	0.075	20		

Coefficients

D_{85} =11.9786 mm D_{30} =0.2077 mm
D_{60} =2.0057 mm D_{15} =N/A
D_{50} =1.0036 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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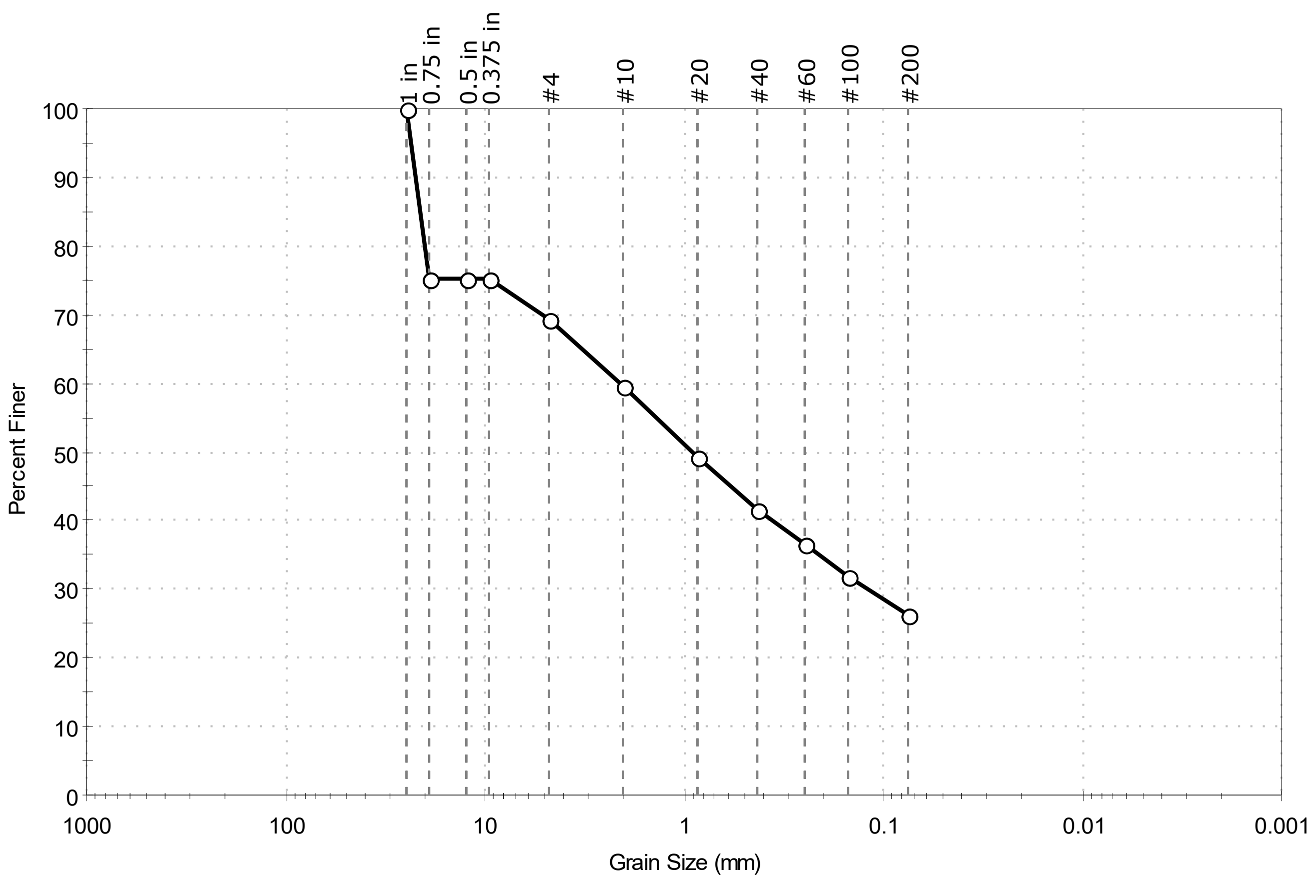
Client:	Kleinfelder, Inc.				
Project:	Replace Babsun's Bridge on Sound Dr				
Location:	Mt. Desert, ME			Project No:	GTX-310167
Boring ID:	BB-MDMB-102	Sample Type:	jar	Tested By:	ckg
Sample ID:	S-4	Test Date:	06/29/19	Checked By:	bfs
Depth :	15-17	Test Id:	510083		
Test Comment:	---				
Visual Description:	Moist, olive silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	30.6	43.2	26.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	75		
0.5 in	12.50	75		
0.375 in	9.50	75		
#4	4.75	69		
#10	2.00	60		
#20	0.85	49		
#40	0.42	42		
#60	0.25	37		
#100	0.15	32		
#200	0.075	26		

Coefficients

D_{85} =21.1583 mm D_{30} =0.1201 mm

D_{60} =2.0794 mm D_{15} =N/A

D_{50} =0.9035 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 : ROUNDED

Sand/Gravel Hardness : HARD

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Client:	Kleinfelder, Inc.				
Project:	Replace Babsun's Bridge on Sound Dr				
Location:	Mt. Desert, ME			Project No:	GTX-310167
Boring ID:	BB-MDMB-101	Sample Type:	cylinder	Tested By:	tlm
Sample ID:	C-2	Test Date:	06/24/19	Checked By:	jsc
Depth :	33-33.5	Test Id:	510084		
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-MDMB-101	C-2	33.14-33.47	162	12115	1	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:	Kleinfelder, Inc.	Test Date:	6/21/2019
Project Name:	Replace Babsun's Bridge on Sound Dr	Tested By:	cmh
Project Location:	Mt. Desert, ME	Checked By:	jsc
GTX #:	310167		
Boring ID:	BB-MDMB-101		
Sample ID:	C-2		
Depth:	33.14-33.47 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	3.93	3.93	3.93
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	518.67		
Bulk Density, lb/ft^3	162	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.0	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

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

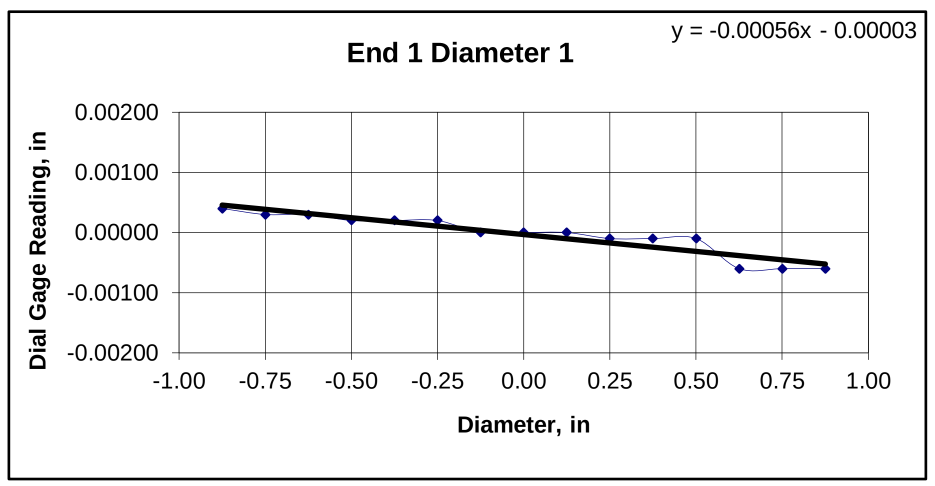
Difference between max and min readings, in:
0° = 0.00100 90° = 0.00100

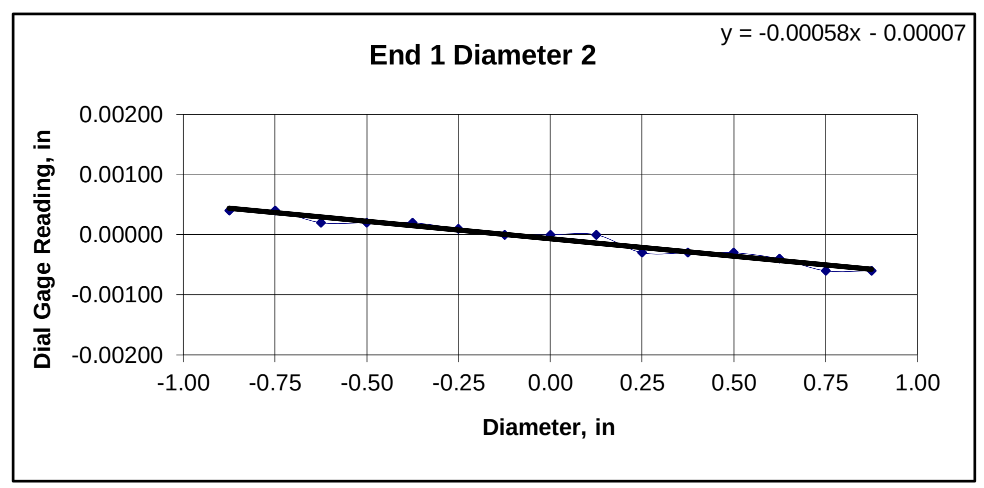
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00030	0.00030	0.00020	0.00020	0.00010	0.00010	0.00000	0.00000	-0.00010	-0.00030	-0.00040	-0.00050	-0.00070	-0.00070
Diameter 2, in (rotated 90°)	0.00040	0.00040	0.00030	0.00030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

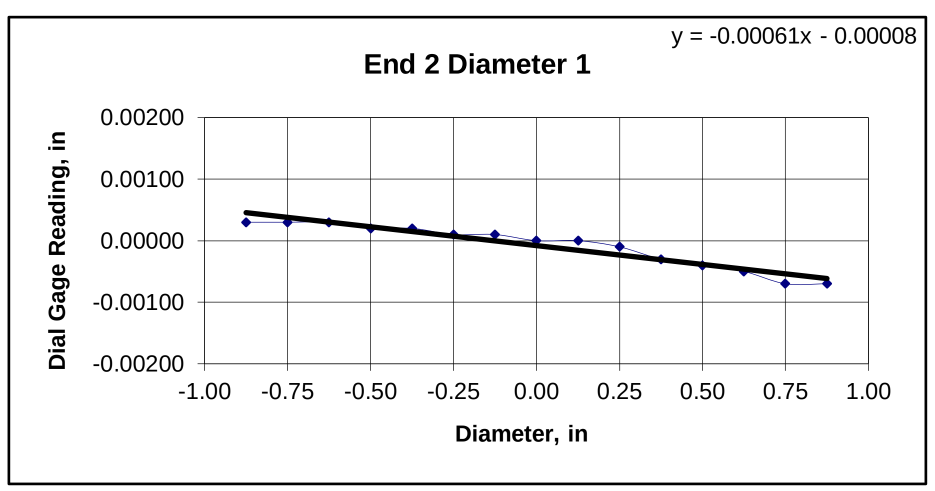
Difference between max and min readings, in:
0° = 0.001 90° = 0.001

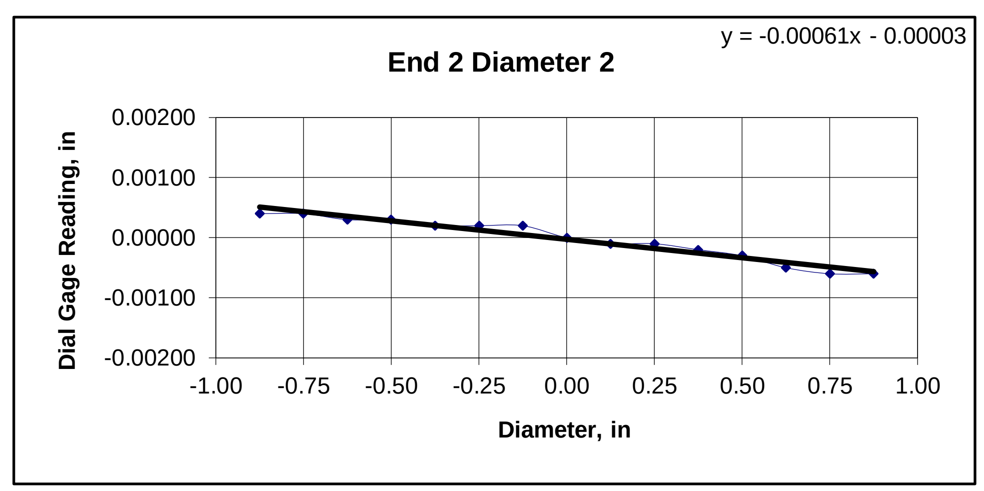
Maximum difference must be < 0.0020 in. Difference = ± 0.00050

Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	Slope of Best Fit Line	0.00056
	Angle of Best Fit Line:	0.03209
End 2:	Slope of Best Fit Line	0.00061
	Angle of Best Fit Line:	0.03503
Maximum Angular Difference:		0.00295

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	Slope of Best Fit Line	0.00058
	Angle of Best Fit Line:	0.03323
End 2:	Slope of Best Fit Line	0.00061
	Angle of Best Fit Line:	0.03520
Maximum Angular Difference:		0.00196

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	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00100	1.990	0.00050	0.029	YES
END 2					
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00100	1.990	0.00050	0.029	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Kleinfelder, Inc.
Project Name:	Replace Babsun's Bridge on Sound Dr
Project Location:	Mt. Desert, ME
GTX #:	310167
Test Date:	6/24/2019
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-MDMB-101
Sample ID:	C-2
Depth, ft:	33.14-33.47

No Photo Available

After cutting and grinding

After break

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: Kleinfelder Boston, MA PM: Nerivaldo Jamba Assigned By: Nerivaldo Jamba Collected By: Client	Project Information: **Mount Desert, Maine** **ME** Kleinfelder Project Number: 20193610-002A Summary Page: 1 of 1 Report Date: 06.24.22

LABORATORY TESTING DATA SHEET, Report No.: 7422-F-139

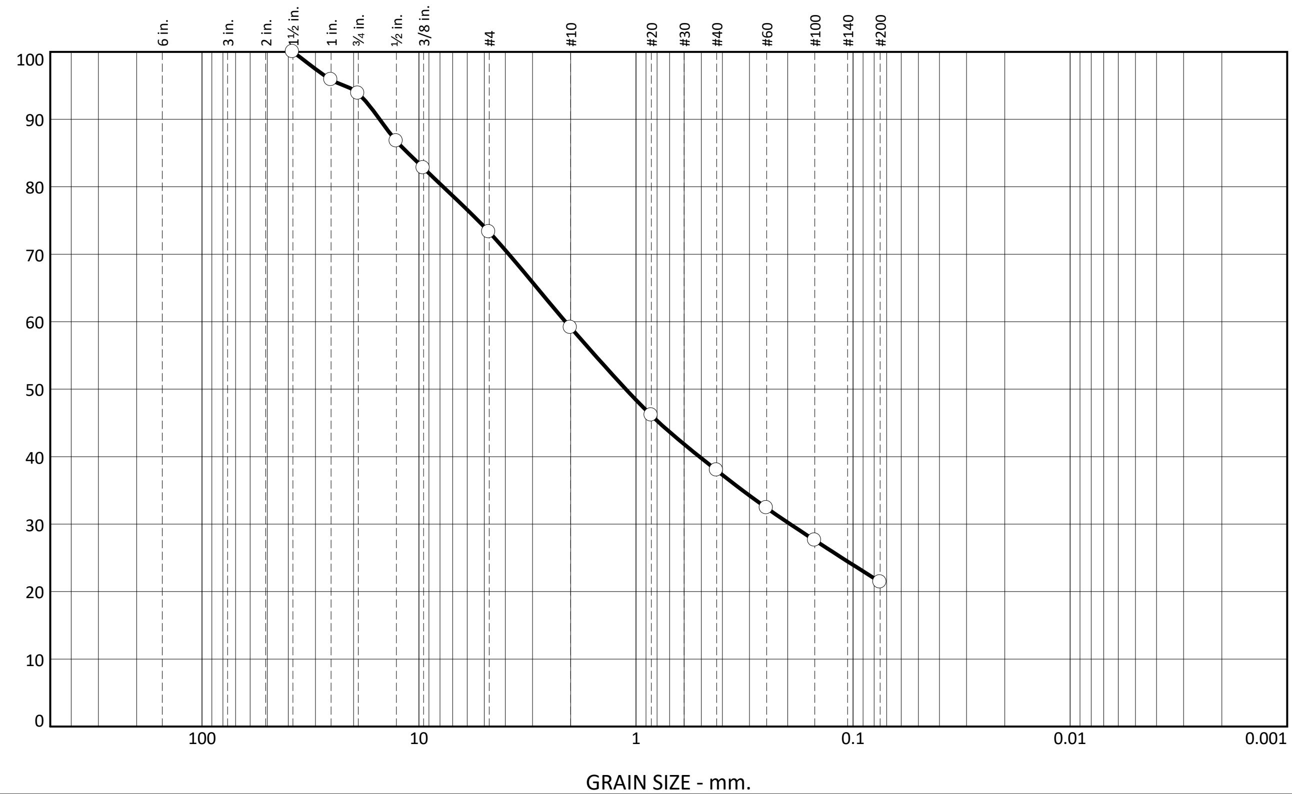
Source/Boring No.	Material/Sample No.	Depth (ft)	Laboratory No.	Identification Tests						Corrosivity Tests								Laboratory Log and Soil Description
				As Received Moisture Content %	LL %	PL %	Gravel %	Sand %	Fines %	Resistivity (Mohms-cm)	Sulfate (mg/kg)	Chloride (mg/kg)	Sulfide (mg/kg)	Redox Potential (mv)	pH	Electrical Resist. As Received Ohm-cm @ 60°F	Electrical Resist. Saturated Ohm-cm @ 60°F	
				D2216	D4318		D6913			EPA						G57		
BB-201	S-4	10-12	22-S-2162															Sample not received
BB-201	S-5	15-17	22-S-2163				26.7	51.9	21.4									Olive silty sand with gravel
BB-201	S-7	25-27	22-S-2164				23.8	51.4	24.8									Olive silty sand with gravel
BB-202	S-3	10-12	22-S-2165									237						Chloride Only
BB-202	S-4	15-17	22-S-2166				39.3	51.4	9.3									Olive well-graded sand with silt and gravel
BB-202	S-5	20-22	22-S-2167				21.4	40.4	38.2									Olive silty sand with gravel

Date Received: *06.09.22* Reviewed By: Date Reviewed: *06.24.22*

This report only relates to items inspect and/or tested. No warranty, expressed or implied, is made.
This report shall not be reproduced, except in full, without prior written approval from the Agency, as defined in ASTM E329.

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.2	20.5	14.2	21.1	16.6	21.4	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	95.9			
3/4"	93.8			
1/2"	86.8			
3/8"	82.8			
#4	73.3			
#10	59.1			
#20	46.1			
#40	38.0			
#60	32.4			
#100	27.6			
#200	21.4			

* (no specification provided)

Material Description

Olive silty sand with gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 15.1737 D_{85}= 11.2371 D_{60}= 2.1127
D_{50}= 1.1147 D_{30}= 0.1949 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: Boring Depth: 15-17'
Sample Number: BB-201 / S-5

Sample Date: 06.22.22

Thielsch Engineering Inc.

Cranston, RI

Client: Kleinfelder
Project: Mount Desert, Maine
Maine
Project No: 20193610-002A

Figure 22-S-2163

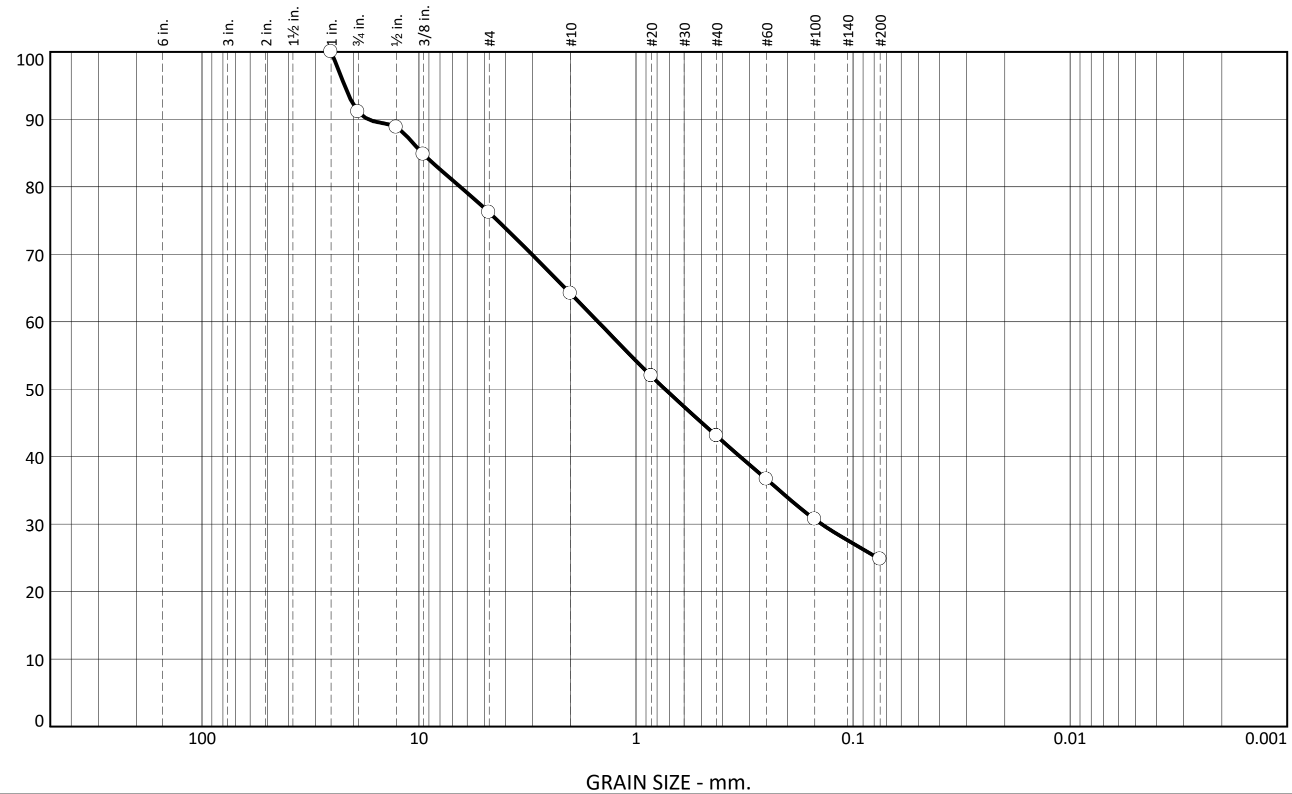
Tested By: SL / JF / FR

Checked By:

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	8.9	14.9	12.0	21.1	18.3	24.8	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	91.1			
1/2"	88.8			
3/8"	84.8			
#4	76.2			
#10	64.2			
#20	51.9			
#40	43.1			
#60	36.6			
#100	30.7			
#200	24.8			

* (no specification provided)

Material Description

Olive silty sand with gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 17.1459 D_{85}= 9.6745 D_{60}= 1.4970
D_{50}= 0.7347 D_{30}= 0.1396 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-1-b

Test Remarks

Source of Sample: Boring Depth: 25-27'
Sample Number: BB-201 / S-7

Sample Date: 06.22.22

Thielsch Engineering Inc.

Cranston, RI

Client: Kleinfelder
Project: Mount Desert, Maine
Maine
Project No: 20193610-002A

Figure 22-S-2164

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

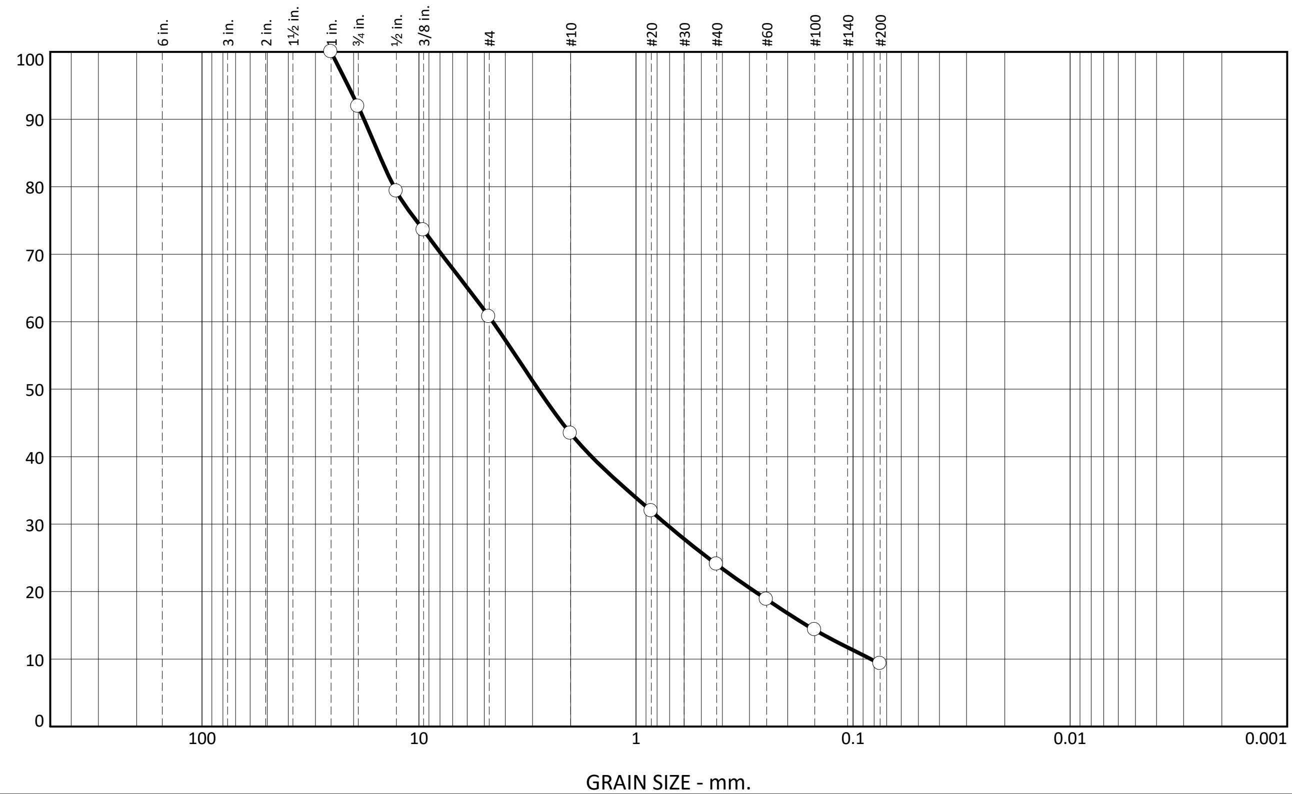
Tested By: SL / JF / FR

Checked By:

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	8.1	31.2	17.3	19.4	14.7	9.3	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1"	100.0			
3/4"	91.9			
1/2"	79.3			
3/8"	73.6			
#4	60.7			
#10	43.4			
#20	31.9			
#40	24.0			
#60	18.8			
#100	14.3			
#200	9.3			

* (no specification provided)

Material Description

Olive well-graded sand with silt and gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 17.9145 D_{85}= 15.3662 D_{60}= 4.5750
D_{50}= 2.8262 D_{30}= 0.7258 D_{15}= 0.1633
D_{10}= 0.0828 C_u= 55.24 C_c= 1.39

Classification

USCS= SW-SM AASHTO= A-1-a

Test Remarks

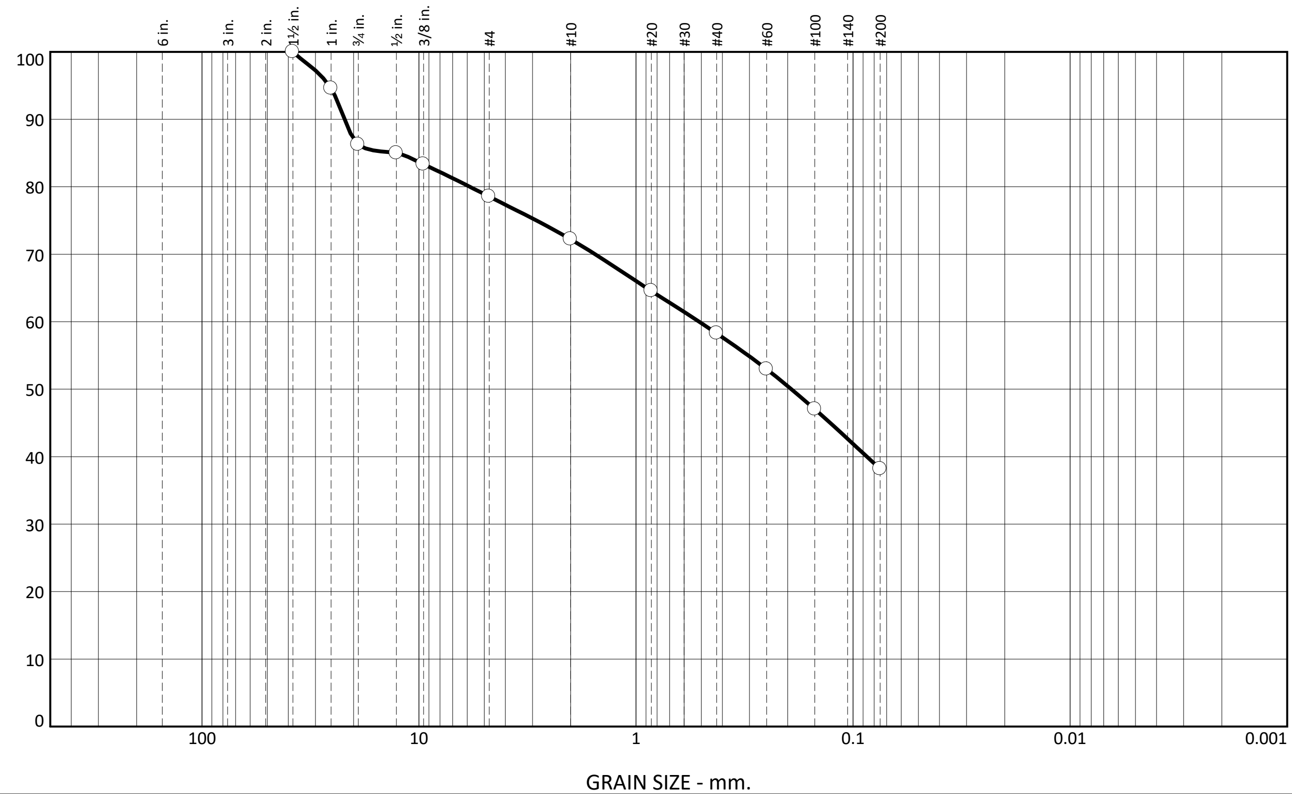
Source of Sample: Boring Depth: 15-17'
Sample Number: BB-202 /S-4 Sample Date: 06.22.22

Thielsch Engineering Inc. Cranston, RI	Client: Kleinfelder Project: Mount Desert, Maine Maine Project No: 20193610-002A Figure 22-S-2166

Tested By: SL / JF / FR Checked By:

Particle Size Distribution Report

ASTM D6913



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.8	7.6	6.4	13.9	20.1	38.2	

Test Results (ASTM D6913)

Sieve Size or Diam. (mm.)	Finer (%)	Spec.* (%)	Out of Spec. (%)	Pct. of Fines
1 1/2"	100.0			
1"	94.6			
3/4"	86.2			
1/2"	85.0			
3/8"	83.3			
#4	78.6			
#10	72.2			
#20	64.6			
#40	58.3			
#60	53.0			
#100	47.0			
#200	38.2			

* (no specification provided)

Material Description

Olive silty sand with gravel

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 22.0250 D_{85}= 12.7052 D_{60}= 0.5119
D_{50}= 0.1924 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Test Remarks

Sample visually classified as non-plastic.

Source of Sample: Boring Depth: 20-22'
Sample Number: BB-202 / S-5

Sample Date: 06.22.22

Thielsch Engineering Inc. Cranston, RI	Client: Kleinfelder Project: Mount Desert, Maine Maine Project No: 20193610-002A Figure 22-S-2167

Tested By: SL / JF / FR Checked By:

These results are for the exclusive use of the client for whom they were obtained. They apply only to the samples tested and are not indicative of apparently identical samples.

CERTIFICATE OF ANALYSIS

Kristina Roland
Thielsch Engineering, Inc.
195 Frances Avenue
Cranston, RI 02910

RE: Klienfelder Mount Desert Maine (20193610-002A)
ESS Laboratory Work Order Number: 22F0392

This signed Certificate of Analysis is our approved release of your analytical results. These results are only representative of sample aliquots received at the laboratory. ESS Laboratory expects its clients to follow all regulatory sampling guidelines. Beginning with this page, the entire report has been paginated. This report should not be copied except in full without the approval of the laboratory. Samples will be disposed of thirty days after the final report has been delivered. If you have any questions or concerns, please feel free to call our Customer Service Department.

Laurel Stoddard
Laboratory Director

REVIEWED
By ESS Laboratory at 2:18 pm, Jun 16, 2022

Analytical Summary

The project as described above has been analyzed in accordance with the ESS Quality Assurance Plan. This plan utilizes the following methodologies: US EPA SW-846, US EPA Methods for Chemical Analysis of Water and Wastes per 40 CFR Part 136, APHA Standard Methods for the Examination of Water and Wastewater, American Society for Testing and Materials (ASTM), and other recognized methodologies. The analyses with these noted observations are in conformance to the Quality Assurance Plan. In chromatographic analysis, manual integration is frequently used instead of automated integration because it produces more accurate results.

The test results present in this report are in compliance with TNI and relative state standards, and/or client Quality Assurance Project Plans (QAPP). The laboratory has reviewed the following: Sample Preservations, Hold Times, Initial Calibrations, Continuing Calibrations, Method Blanks, Blank Spikes, Blank Spike Duplicates, Duplicates, Matrix Spikes, Matrix Spike Duplicates, Surrogates and Internal Standards. Any results which were found to be outside of the recommended ranges stated in our SOPs will be noted in the Project Narrative.

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

SAMPLE RECEIPT

The following samples were received on June 09, 2022 for the analyses specified on the enclosed Chain of Custody Record.

The cooler temperature was not within the acceptance criteria of ≤6°C.

Lab Number	Sample Name	Matrix	Analysis
22F0392-01	BB-202 S-3	Soil	ASTM D1411

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

PROJECT NARRATIVE

No unusual observations noted.

End of Project Narrative.

DATA USABILITY LINKS

To ensure you are viewing the most current version of the documents below, please clear your internet cookies for www.ESSLaboratory.com. Consult your IT Support personnel for information on how to clear your internet cookies.

Definitions of Quality Control Parameters

Semivolatile Organics Internal Standard Information

Semivolatile Organics Surrogate Information

Volatile Organics Internal Standard Information

Volatile Organics Surrogate Information

EPH and VPH Alkane Lists

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

CURRENT SW-846 METHODOLOGY VERSIONS

Analytical Methods

1010A - Flashpoint
6010C - ICP
6020A - ICP MS
7010 - Graphite Furnace
7196A - Hexavalent Chromium
7470A - Aqueous Mercury
7471B - Solid Mercury
8011 - EDB/DBCP/TCP
8015C - GRO/DRO
8081B - Pesticides
8082A - PCB
8100M - TPH
8151A - Herbicides
8260B - VOA
8270D - SVOA
8270D SIM - SVOA Low Level
9014 - Cyanide
9038 - Sulfate
9040C - Aqueous pH
9045D - Solid pH (Corrosivity)
9050A - Specific Conductance
9056A - Anions (IC)
9060A - TOC
9095B - Paint Filter
MADEP 04-1.1 - EPH
MADEP 18-2.1 - VPH

Prep Methods

3005A - Aqueous ICP Digestion
3020A - Aqueous Graphite Furnace / ICP MS Digestion
3050B - Solid ICP / Graphite Furnace / ICP MS Digestion
3060A - Solid Hexavalent Chromium Digestion
3510C - Separatory Funnel Extraction
3520C - Liquid / Liquid Extraction
3540C - Manual Soxhlet Extraction
3541 - Automated Soxhlet Extraction
3546 - Microwave Extraction
3580A - Waste Dilution
5030B - Aqueous Purge and Trap
5030C - Aqueous Purge and Trap
5035A - Solid Purge and Trap

SW846 Reactivity Methods 7.3.3.2 (Reactive Cyanide) and 7.3.4.1 (Reactive Sulfide) have been withdrawn by EPA. These methods are reported per client request and are not NELAP accredited.

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine
Client Sample ID: BB-202 S-3
Date Sampled: 06/09/22 15:15
Percent Solids: 84

ESS Laboratory Work Order: 22F0392
ESS Laboratory Sample ID: 22F0392-01
Sample Matrix: Soil

Classical Chemistry

Analyte	Results (MRL)	MDL	Method	Limit	DF	Analyst	Analyzed	Units	Batch
Chloride	**237** (20)		ASTM D1411		1	EEM	06/13/22 18:20	mg/kg dry wt dry	DF21320

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

Quality Control Data

Analyte	Result	MRL	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Qualifier
				Classical Chemistry						
Batch DF21320 - General Preparation										
Blank										
Chloride	ND	17	mg/kg dry wt wet							
LCS										
Chloride	48		mg/L	60.00		80	50-150			

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

Notes and Definitions

U	Analyte included in the analysis, but not detected
ND	Analyte NOT DETECTED at or above the MRL (LOQ), LOD for DoD Reports, MDL for J-Flagged Analytes
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference
MDL	Method Detection Limit
MRL	Method Reporting Limit
LOD	Limit of Detection
LOQ	Limit of Quantitation
DL	Detection Limit
I/V	Initial Volume
F/V	Final Volume
§	Subcontracted analysis; see attached report
1	Range result excludes concentrations of surrogates and/or internal standards eluting in that range.
2	Range result excludes concentrations of target analytes eluting in that range.
3	Range result excludes the concentration of the C9-C10 aromatic range.
Avg	Results reported as a mathematical average.
NR	No Recovery
[CALC]	Calculated Analyte
SUB	Subcontracted analysis; see attached report
RL	Reporting Limit
EDL	Estimated Detection Limit
MF	Membrane Filtration
MPN	Most Probable Number
TNTC	Too numerous to Count
CFU	Colony Forming Units

CERTIFICATE OF ANALYSIS

Client Name: Thielsch Engineering, Inc.
Client Project ID: Klienfelder Mount Desert Maine

ESS Laboratory Work Order: 22F0392

ESS LABORATORY CERTIFICATIONS AND ACCREDITATIONS

ENVIRONMENTAL

Rhode Island Potable and Non Potable Water: LAI00179
http://www.health.ri.gov/find/labs/analytical/ESS.pdf

Connecticut Potable and Non Potable Water, Solid and Hazardous Waste: PH-0750
http://www.ct.gov/dph/lib/dph/environmental_health/environmental_laboratories/pdf/OutofStateCommercialLaboratories.pdf

Maine Potable and Non Potable Water, and Solid and Hazardous Waste: RI00002
http://www.maine.gov/dhhs/mecdc/environmental-health/dwp/partners/labCert.shtml

Massachusetts Potable and Non Potable Water: M-RI002
http://public.dep.state.ma.us/Labcert/Labcert.aspx

New Hampshire (NELAP accredited) Potable and Non Potable Water, Solid and Hazardous Waste: 2424
http://des.nh.gov/organization/divisions/water/dwgb/nhelap/index.htm

New York (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: 11313
http://www.wadsworth.org/labcert/elap/comm.html

New Jersey (NELAP accredited) Non Potable Water, Solid and Hazardous Waste: RI006
http://datamine2.state.nj.us/DEP_OPRA/OpraMain/pi_main?mode=pi_by_site&sort_order=PI_NAMEA&Select+a+Site:=58715

Pennsylvania: 68-01752
http://www.dep.pa.gov/Business/OtherPrograms/Labs/Pages/Laboratory-Accreditation-Program.aspx

ESS Laboratory Sample and Cooler Receipt Checklist

Client: Thielsch Engineering, Inc - ESS

Shipped/Delivered Via: ESS Courier

ESS Project ID: 22F0392
Date Received: 6/9/2022
Project Due Date: 6/16/2022
Days for Project: 5 Day

1. Air bill manifest present? No
   Air No.: NA
2. Were custody seals present? No
3. Is radiation count <100 CPM? Yes
4. Is a Cooler Present? Yes
   Temp: 20 Iced with: None
5. Was COC signed and dated by client? Yes

6. Does COC match bottles? Yes
7. Is COC complete and correct? Yes
8. Were samples received intact? Yes
9. **Were labs informed about <u>short holds & rushes</u>?** Yes / No / NA
10. Were any analyses received outside of hold time? Yes / No

11. Any Subcontracting needed? Yes / No
    ESS Sample IDs:
    Analysis:
    TAT:

12. Were VOAs received? Yes / No
    a. Air bubbles in aqueous VOAs? Yes / No
    b. Does methanol cover soil completely? Yes / No / NA

13. Are the samples properly preserved? Yes / No
    a. If metals preserved upon receipt: Date: Time: By/Acid Lot#:
    b. Low Level VOA vials frozen: Date: Time: By:

Sample Receiving Notes:

14. Was there a need to contact Project Manager? Yes / No
    a. Was there a need to contact the client? Yes / No
    Who was contacted? Date: Time: By:

Resolution:

Sample Number	Container ID	Proper Container	Air Bubbles Present	Sufficient Volume	Container Type	Preservative	Record pH (Cyanide and 608 Pesticides)
1	302383	Yes	N/A	Yes	Other Glass	NP	

2nd Review

Were all containers scanned into storage/lab? Initials TS
Are barcode labels on correct containers? Yes / No
Are all Flashpoint stickers attached/container ID # circled? Yes / No / NA
Are all Hex Chrome stickers attached? Yes / No / NA
Are all QC stickers attached? Yes / No / NA
Are VOA stickers attached if bubbles noted? Yes / No / NA

Completed By: [signature] Date & Time: 1009 6/9/22.
Reviewed By: [signature] Date & Time: 16:10 6/9/22

ESS Laboratory

Division of Thielsch Engineering, Inc.
185 Frances Avenue, Cranston, RI 02910-2211
Tel. (401) 461-7181 Fax (401) 461-4486
www.esslaboratory.com

CHAIN OF CUSTODY

ESS LAB PROJECT ID: 22F0392.

Reporting Limits -

Turn Time: Standard X Rush Approved By: ____

State where samples were collected: ME

Is this project for any of the following: (please circle)
MA-MCP CT-RCP RGP DOD Other________

Electonic Deliverable Yes_X__ No____
Format: Excel___ Access___ PDF_X_ Other_____

Project Manager: Kris Roland
Company: Thielsch Engineering
Address: 195 Frances Ave
Cranston, RI 02910

Project # 20193610-002A

Project Name / Client Name:
Kleinfelder
Mount Desert, Maine

Contract Pricing __x___
Special Pricing WO#: ________

Analysis: Chloride ASTM D1411

ESS Lab Sample ID	Date	Collection Time	Grab -G Composite-C	Matrix	Sample Identification	# of Container	Chloride ASTM D1411										Comment #
1	06.09.22	1515	G	S	BB-202 / S-3	1	X										

Preservation Code: 1-NP, 2-HCl, 3-H2SO4, 4-HNO3, 5-NaOH, 6-MeOH, 7-Asorbic Acid, 8-ZnAct, 9-_CH_3OH____

Container Type: P-Poly G-Glass AG-Amber Glass S-Sterile V-VOA

Matrix: S-Soil SD-Solid D-Sludge WW-Wastewater GW-Groundwater SW-Surface Water DW-Drinking Water O-Oil W-Wipes F-Filter

Cooler Present ____Yes ___No

Seals Intact ____ Yes ____No NA: ____

Cooler Temperature: 20 none

Sampled by : N.J.

Comments: Please send reports to kroland@thielsch.com, mcolman@thielsch.com, rroth@thielsch.com

Relinquished by: (Signature)	Date/Time	Received by: (Signature)	Relinquished by: (Signature)	Date/Time	Received by: (Signature)
[signature]	6/9/22 1515	[signature]			
Relinquished by: (Signature)	Date/Time	Received by: (Signature)	Relinquished by: (Signature)	Date/Time	Received by: (Signature)

Please E-mail all changes to Chain of Custody in writing.

Page ____ of ____

THIELSCH ENGINEERING	195 Frances Avenue Cranston RI, 02910 Phone: (401)-467-6454 Fax: (401)-467-2398 thielsch.com *Let's Build a Solid Foundation*	Client Information: Kleinfelder Boston, MA PM: Nerivaldo Jamba Assigned By: Nerivaldo Jamba Collected By: Client	Project Information: **Mount Desert, Maine** Kleinfelder Project Number: 20193610-002A Summary Page: 1 of 1 Report Date: 06.24.22

LABORATORY TESTING DATA SHEET, Report No.: 7422-E-123

				Specimen Data						Compressive Strength Tests								
Boring No.	Sample No.	Depth (ft)	Laboratory No.	Mohs Hard-ness	Diameter (in)	Length (in)	(1) Unit Weight (PCF)	(2) Wet Density (PCF)	Bulk G_s	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) E sec PSI EE+06	(7) Poisson's Ratio	στ PSI	Is_{50} PSI	(8) S_c PSI	Rock Formation or Description or Remarks
BB-201	C-1	37.5-47.5	22-S-2150		1.988	4.642	160.0				1906							Pink Syenite
Break along healed suture																		
BB-202	C-2	28-37.5	22-S-2151		1.982	4.458	155.7				1481	0.65	3.84×10^5	1.00				Pink Syenite
Fresh break																		

Notes

(1) Volume Determined By Measuring Dimensions

(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) PLD=Point Load (diametrical), PLA= Point Load (Axial) ST= Splitting Tensile U= Unconfined Compressive Strength

(4) Taken at Peak Deviator Stress

Notes

(5) Strain at Peak Deviator Stress

(6) Represents Secant Modulus at 50% of Total Failure Stress

(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)

Date Received: *06.09.22* Reviewed By: Date Reviewed: *06.24.22*

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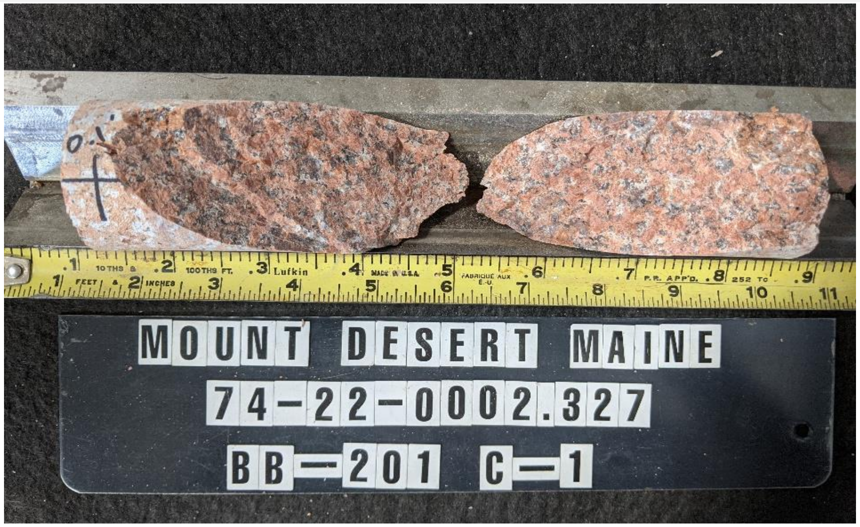
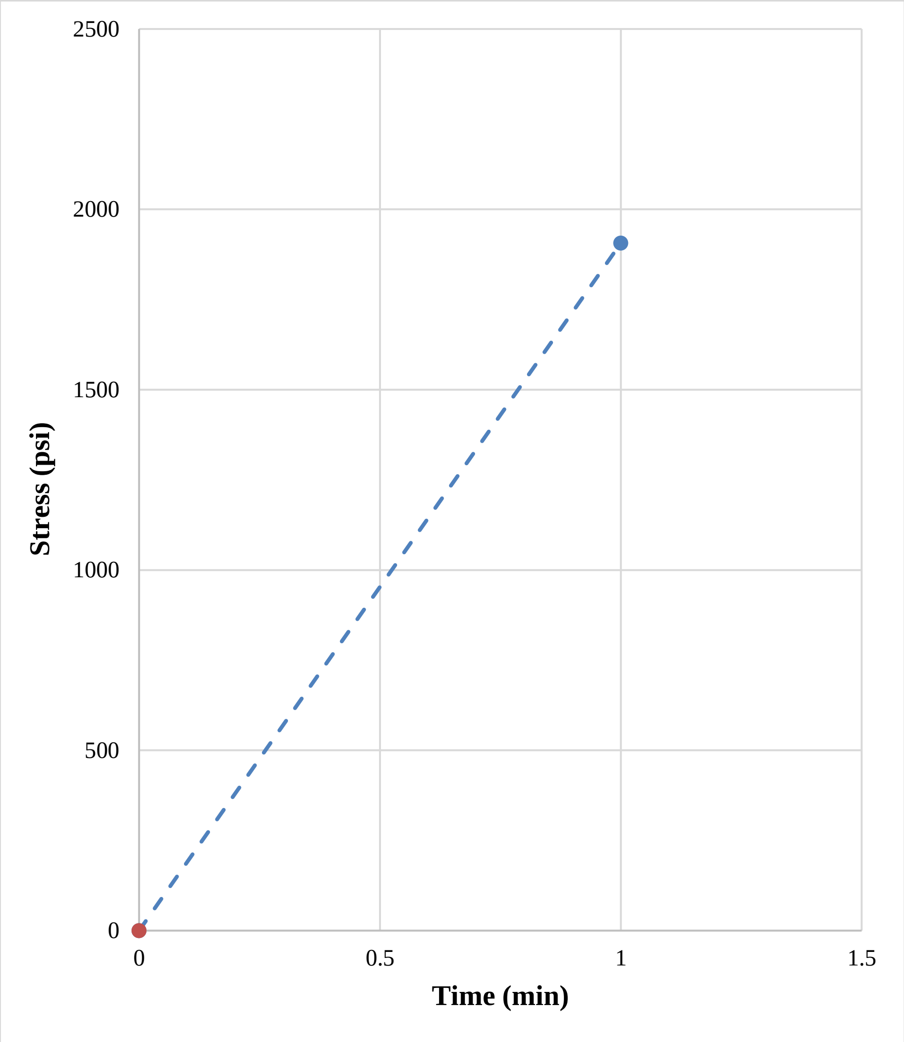
This report shall not be reproduced, except in full, without prior approval from the Agency, as defined in ASTM E329.

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** Kleinfelder Boston, MA PM: Nerivaldo Jamba Assigned by: Nerivaldo Jamba Collected by: Client	**Project Information:** Mount Desert, Maine Project Number: 74-22-0002.327 Technician: AV / AF 06.14.22

ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	BB-201	**Unit Weight (pcf):**	160.0
Sample No.:	C-1	**Failure Stress (psi):**	1,906
Depth (ft):	37.5-47.5	**Failure Mode:**	Fresh
Tested Depth (ft):		**Time to Failure (min):**	1.00
Rock Type:	Syenite		
Features:	Broke along healed suture		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.988	**Poisson's Ratio @ 50%:**	NA
Length, L (in):	4.642	**Strain %:**	NA
L:D Ratio:	2.34	**E sec PSI @ 50%:**	NA



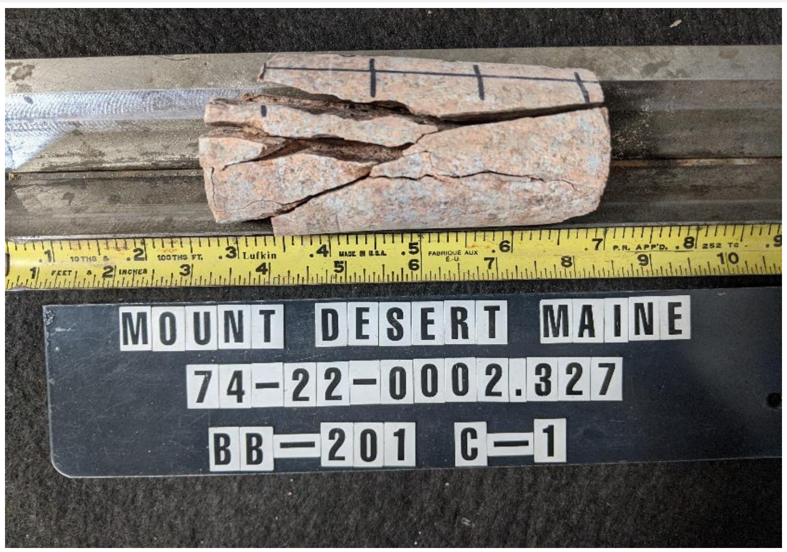


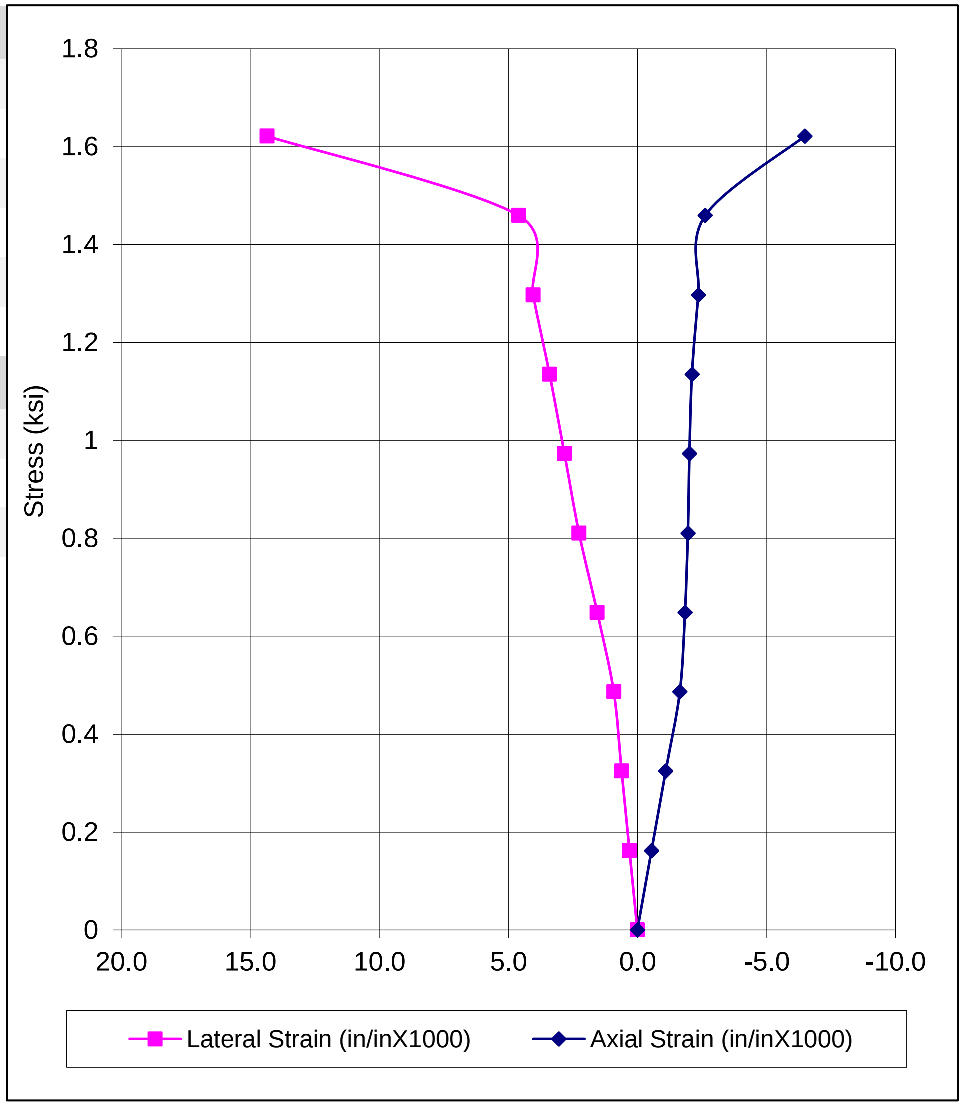
Testing Notes: Healed suture

THIELSCH ENGINEERING	195 Frances Avenue Cranston, Rhode Island 02910 Phone: (401) 467-6454 Fax: (401) 467-2398 www.thielsch.com *Let's Build a Solid Foundation*	**Client Information:** Kleinfelder Boston, MA PM: Nerivaldo Jamba Assigned by: Nerivaldo Jamba Collected by: Client	**Project Information:** Mount Desert, Maine Project Number: 74-22-0002.327 Technician: AV / AF 06.14.22

ASTM D7012 Compressive Strength and Elastic Moduli of Intact Rock Core Specimens

Sample Information		Compressive Test Information	
Boring ID:	BB-202	**Unit Weight (pcf):**	155.7
Sample #:	C-2	**Failure Stress (psi):**	1,481
Depth (ft):	28-37.5	**Failure Mode:**	Fresh
Tested Depth (ft):		**Time to Failure (min)**	1
Rock Type:	Syenite		
Features:	Fresh Break		
Test Specimen Information		**Elastic Moduli Test Information**	
Diameter, D (in):	1.982	**Poisson's Ratio @ 50%:**	1.00
Length, L (in):	4.458	**Strain %:**	0.65
L:D Ratio:	2.25	**E sec PSI @ 50%:**	3.84E+05





Testing Notes:

APPENDIX C

GEOTECHNICAL CALCULATIONS

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Objective: Estimate the Friction Angle for existing fill materials at the Babson's Bridge on Sound Drive based on corrected blow counts at borings BB-MDMB-101 and BB-MDMB-102 in Mount Desert, Maine.

References:

1. AASHTO Standard Specifications for Highway Bridges, 17th Edition - 2002
2. Das, Braja M. Principles of Foundation Engineering, 5th Edition- 2004
3. Correlations of Soil Properties by Michael Carter and Stephen Bentley

Soil Borings: The subsurface conditions are generally based on borings BB-MDMB-101 and BB-MDMB-102. The borings were advanced by New England Boring Contractor between 5/13/2019 and 5/15/2019 and overseen by Kleinfelder professionals. SPT tests were conducted with an automatic hammer. The borings indicated between 9.3 and 12.5 feet of existing fill. The observed existing fill materials consist of medium dense to very dense silty sand with gravel. The high blow counts may be due to the presense of cobbles or boulders in the existing fill.

Calculation:

Soil and Water Properties:

$\gamma.soil := 120\ pcf$ Unit weight of soil

$D.water := 5.5\ ft$ Depth to shallowest groundwater level was observed at about 5.5 feet below ground surface in boring BB-MDMB-101

$\gamma.water := 62.4\ pcf$ Unit weight of water

$\gamma_b := \gamma.soil - \gamma.water = 57.6\ pcf$ buoyant unit weight of soil

Enter Sample Information:

entry	Boring	Sample.no	top.depth (ft)	bottom.depth (ft)	N_{field}	thickness (ft)	hammer
1	"BB–MDMB–101"	"S1"	1	3	55	2	"auto"
2	"BB–MDMB–101"	"S2"	5	7	38	2	"auto"
3	"BB–MDMB–101"	"S3"	10	12	9	2	"auto"
4	"BB–MDMB–102"	"S1"	1	3	28	2	"auto"
5	"BB–MDMB–102"	"S2"	6	8	19	2	"auto"

$i := 0, 1 .. \text{length}(entry) - 1$

Calc By: M. Chea, 10/10/2019

Check By: J. MacGegor, 03/05/2020

Determine vertical effective stress at midpoint of each layer:

Depth to midpoint of each sample

$$Depth.midpoint_i := \mathrm{mean}\left(top.depth_i, bottom.depth_i\right)$$

Vertical effective stress at midpoint of each sample

$$\sigma'_{v_i} := \text{if } Depth.midpoint_i < D.water$$
$$\quad \| \; Depth.midpoint_i \cdot \gamma.soil$$
$$\text{else}$$
$$\quad \| \; D.water \cdot \gamma.soil + \left(Depth.midpoint_i - D.water\right) \cdot \gamma_b$$

Calculate N_{60} by correcting for Field Conditions, as in Das (2004). Conservatively neglect corrections for rod length.

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60} \qquad (2.6)$$

here N_{60} = standard penetration number, corrected for field conditions
N = measured penetration number
η_H = hammer efficiency (%)
η_B = correction for borehole diameter
η_S = sampler correction
η_R = correction for rod length

$n_b := 1.0$ — *For borehole diameter up to 4.7 inches, per Das Table 2.2.2*

$n_s := 1.0$ — *For standard sampler, per Das Table 2.2.3*

$n_{h_i} := 90$ — *From hammer calibration report*

$$N_{60_i} := \frac{\left(N_{field_i} \cdot n_{h_i} \cdot n_b \cdot n_s\right)}{60} = \begin{bmatrix} 82.5 \\ 57 \\ 13.5 \\ 42 \\ 28.5 \end{bmatrix}$$

Calculate $(N_1)_{60}$ by correcting for overburden pressure, as in Das (2004). Conservatively neglect corrections for rod length.

$$C_{N_i} := 0.77 \log\left(\frac{20}{\frac{\left(\sigma'_{v_i}\right)}{2000\ psf}}\right) = \begin{bmatrix} 1.711 \\ 1.358 \\ 1.241 \\ 1.711 \\ 1.331 \end{bmatrix}$$

(by Peck et. al as given in Das (2004) Eqn. 2.14)

$$N1_{60_i} := N_{60_i} \cdot C_{N_i} = \begin{bmatrix} 141.143 \\ 77.421 \\ 16.759 \\ 71.855 \\ 37.945 \end{bmatrix}$$

Average the corrected N-values

$$N1_{60\_avg} := \frac{\left(\sum_{i=0}^{\text{length}(entry)-1} N1_{60_i} \cdot thickness_i\right)}{\sum_{i=0}^{\text{length}(entry)-1} thickness_i} = 69.024$$

Estimate phi based on correlation by Peck, Hanson, and Thornburn (1974) as given in Das (2004), Eqn. 2.17:

$$\phi' := 27.1 + 0.3 \cdot N1_{60\_avg} - 0.00054 \cdot \left(N1_{60\_avg}\right)^2 = 45.235$$

Select Design Phi Value:

$\phi_{design} := 32$

Internal Friction Angle for medium dense to very dense silty sand with gravel estimated based on correlations with corrected N-value, Tables 6.3, 6.4 and 6.5, AASHTO Table 10.4.6.2.4-1, and FHWA GEC No. 7 Table 3.5 below, and accounting for potential variation in site conditions, observed pockets of loose fill, unknown degree of compaction of existing fill.

KLEINFELDER
Bright People. Right Solutions.

The drained friction angle of granular deposits should be determined based on the following correlation.

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

From AASHTO Table 10.4.6.2.4-1

Table 3.5: Correlations Between SPT and CPT Results and Friction Angle of Cohesionless Soils.

(Source: Kulhawy and Maine, 1990).

	In-Situ Test Results	Relative Density	φ' (degrees) (a) [3]	φ' (degrees) (b) [4]
SPT N-Value [1] (blows/300 mm or blows/ft)	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
Normalized CPT cone bearing resistance (q_c/P_a) [2], [4]	< 20	Very Loose	< 30	
	20 to 40	Loose	30 to 35	
	40 to 120	Medium	35 to 40	
	120 to 200	Dense	40 to 45	
	> 200	Very Dense	> 45	

Notes: (1) SPT N-values are field, uncorrected values.
(2) P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.
(3) Range in column (a) from Peck, Hanson, and Thornburn (1974).
(4) Ranges in column (b) and for CPT are from Meyerhof (1956).

From FHWA GEC No. 7 Table 3.5

Table 6.4 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE OF COHESIONLESS SOILS

Material	φ (deg) *Loose*	φ (deg) *Dense*
Uniform sand, round grains	27	34
Well-graded sand, angular grains	33	45
Sandy gravels	35	50
Silty sand	27–33	30–34
Inorganic silt	27–30	30–35

Table 6.5 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE FOR COMPACTED SANDS AND GRAVELS

Soil description	*Class**	*Angle of shearing resistance, φ (deg)*
Well-graded sand-gravel mixtures	GW	>38
Poorly-graded sand gravel mixtures	GP	>37
Silty gravels, poorly graded sand-gravel-silt	GM	>34
Clayey gravels, poorly graded sand-gravel-clay	GC	>31
Well-graded clean sand, gravelly sands	SW	38
Poorly-graded clean sands, gravelly sands	SP	37

* Unified classification system.

From "Correlations of Soil Properties" by Michael Carter and Stephen Bentley

Calc By: M. Chea, 10/10/2019

Check By: J. MacGegor, 03/05/2020

Objective: Estimate the Friction Angle for natural sand deposits at the Babson's Bridge on Sound Drive based on corrected blow counts at borings BB-MDMB-101 and BB-MDMB-102 in Mount Desert, Maine.

References:

1. AASHTO Standard Specifications for Highway Bridges, 17th Edition - 2002
2. Das, Braja M. Principles of Foundation Engineering, 5th Edition- 2004
3. Correlations of Soil Properties by Michael Carter and Stephen Bentley

Soil Borings: The subsurface conditions are generally based on borings BB-MDMB-101 and BB-MDMB-102. The borings were advanced by New England Boring Contractor between 5/13/2019 and 5/15/2019 and overseen by Kleinfelder professionals. SPT tests were conducted with an automatic hammer. The sand layer extended to the top of bedrock which was between 21.6 and 31 below ground surface. The observed sand layer consist of medium dense silty sand, and silt with sand with variable amounts of gravel.

Calculation:

Soil and Water Properties:

$\gamma.soil := 120\ pcf$ Unit weight of soil

$D.water := 5.5\ ft$ Depth to shallowest groundwater level observed at about 5.5 feet below ground surface in boring BB-MDMB-101.

$\gamma.water := 62.4\ pcf$ Unit weight of water

$\gamma_b := \gamma.soil - \gamma.water = 57.6\ pcf$ buoyant unit weight of soil

Enter Sample Information:

$entry$	$Boring$	$Sample.no$	$top.depth$ (ft)	$bottom.depth$ (ft)	N_{field}	$thickness$ (ft)	$hammer$
1	"BB–MDMB–101"	"S4"	15	17	11	2	"auto"
2	"BB–MDMB–101"	"S5"	21	23	20	2	"auto"
3	"BB–MDMB–101"	"S6"	26	28	29	2	"auto"
4	"BB–MDMB–102"	"S3"	9.3	11.3	19	2	"auto"
5	"BB–MDMB–102"	"S4"	15	17	24	2	"auto"
6	"BB–MDMB–102"	"S5"	19	20.6	12	2	"auto"

$i := 0, 1 .. \text{length}(entry) - 1$

Calc By: M. Chea, 10/10/2019

Check By: J. MacGregor 03/05/2020

Determine vertical effective stress at midpoint of each layer:

Depth to midpoint of each sample

$$Depth.midpoint_i := \mathrm{mean}\left(top.depth_i, bottom.depth_i\right)$$

Vertical effective stress at midpoint of each sample

$$\sigma'_{v_i} := \begin{cases} Depth.midpoint_i \cdot \gamma.soil & \text{if } Depth.midpoint_i < D.water \\ D.water \cdot \gamma.soil + \left(Depth.midpoint_i - D.water\right) \cdot \gamma_b & \text{else} \end{cases}$$

Calculate N_{60} by correcting for Field Conditions, as in Das (2004). Conservatively neglect corrections for rod length.

surrounding soil, or

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60} \qquad (2.6)$$

here N_{60} = standard penetration number, corrected for field conditions
N = measured penetration number
η_H = hammer efficiency (%)
η_B = correction for borehole diameter
η_S = sampler correction
η_R = correction for rod length

... based on recommendations by Seed et al.

$n_b := 1.0$ *(For borehole diameter up to 4.7 inches, per Das Table 2.2.2)*

$n_s := 1.0$ *For standard sampler, per Das Table 2.2.3*

$n_{h_i} := 90$ *Based on Hammer calibration report*

$$N_{60_i} := \frac{\left(N_{field_i} \cdot n_{h_i} \cdot n_b \cdot n_s\right)}{60} = \begin{bmatrix} 16.5 \\ 30 \\ 43.5 \\ 28.5 \\ 36 \\ 18 \end{bmatrix}$$

Calculate (N1)60 by correcting for overburden pressure, as in Das (2004). Conservatively neglect corrections for rod length.

$$C_{N_i} := 0.77 \log\left(\frac{20}{\frac{\left(\sigma'_{v_i}\right)}{2000\ psf}}\right) = \begin{bmatrix} 1.155 \\ 1.074 \\ 1.019 \\ 1.256 \\ 1.155 \\ 1.102 \end{bmatrix}$$

(by Peck et. al as given in Das (2004) Eqn. 2.14)

$$N1_{60_i} := N_{60_i} \cdot C_{N_i} = \begin{bmatrix} 19.058 \\ 32.227 \\ 44.336 \\ 35.783 \\ 41.581 \\ 19.83 \end{bmatrix}$$

Average the corrected N-values

$$N1_{60\_avg} := \frac{\left(\sum_{i=0}^{\text{length}(entry)-1} N1_{60_i} \cdot thickness_i\right)}{\sum_{i=0}^{\text{length}(entry)-1} thickness_i} = 32.136$$

Estimate phi based on correlation by Peck, Hanson, and Thornburn (1974) as given in Das (2004), Eqn. 2.17:

$$\phi' := 27.1 + 0.3 \cdot N1_{60\_avg} - 0.00054 \cdot \left(N1_{60\_avg}\right)^2 = 36.183$$

Calc By: M. Chea, 10/10/2019

Check By: J. MacGregor 03/05/2020

Select Design Phi Value:

$\phi_{design} := 34$

Internal Friction Angle for loose to medium dense poorly graded sand, silty sand, and silt with sand with variable amounts of gravel estimated based on correlations with corrected N-value, Tables 6.3, 6.4 and 6.5, AASHTO Table 10.4.6.2.4-1, and FHWA GEC No. 7 Table 3.5 below, and accounting for potential variation in site conditions.

The drained friction angle of granular deposits should be determined based on the following correlation.

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

From AASHTO Table 10.4.6.2.4-1

Table 3.5: Correlations Between SPT and CPT Results and Friction Angle of Cohesionless Soils.

(Source: Kulhawy and Maine, 1990).

	In-Situ Test Results	Relative Density	ϕ' (degrees) (a) [3]	ϕ' (degrees) (b) [4]
SPT N-Value [1] (blows/300 mm or blows/ft)	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
Normalized CPT cone bearing resistance (q_c/P_a) [2], [4]	< 20	Very Loose	< 30	
	20 to 40	Loose	30 to 35	
	40 to 120	Medium	35 to 40	
	120 to 200	Dense	40 to 45	
	> 200	Very Dense	> 45	

Notes: (1) SPT N-values are field, uncorrected values.
(2) P_a is the normal atmospheric pressure = 1 atm ~ 100 kN/m^2 ~ 1 tsf.
(3) Range in column (a) from Peck, Hanson, and Thornburn (1974).
(4) Ranges in column (b) and for CPT are from Meyerhof (1956).

From FHWA GEC No. 7 Table 3.5

Table 6.4 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE OF COHESIONLESS SOILS

Material	ϕ (deg) *Loose*	ϕ (deg) *Dense*
Uniform sand, round grains	27	34
Well-graded sand, angular grains	33	45
Sandy gravels	35	50
Silty sand	27–33	30–34
Inorganic silt	27–30	30–35

Table 6.5 TYPICAL VALUES OF THE ANGLE OF SHEARING RESISTANCE FOR COMPACTED SANDS AND GRAVELS

Soil description	*Class**	*Angle of shearing resistance, ϕ (deg)*
Well-graded sand-gravel mixtures	GW	>38
Poorly-graded sand gravel mixtures	GP	>37
Silty gravels, poorly graded sand-gravel-silt	GM	>34
Clayey gravels, poorly graded sand-gravel-clay	GC	>31
Well-graded clean sand, gravelly sands	SW	38
Poorly-graded clean sands, gravelly sands	SP	37

* Unified classification system.

From "Correlations of Soil Properties" by Michael Carter and Stephen Bentley

Calc By: M. Chea, 10/10/2019

Check By: J. MacGregor 03/05/2020

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Strength Limit State Analysis and Service Limit State Evaluation for Driven Piles Design per MaineDOT BDG-2003 and AASHTO LRFD-8

Objectives:

The objective of this generic calculation package as a "proof" of the Maine DOT's Bridge Design Manual methodology in parallel with AASHTO LRFD methods for integral abutments with respect to Strength Limit State Analysis and Service Limit State Evaluations. The calculations herein are an explicit display of the methodologies used by Maine DOT to determine factored axial structural resistances (see Table 5-7), factored lateral resistances and depth to fixity for strength limit state design in sand (see Table 5-8), and factored lateral resistances and depth to fixity for strength limit state design in clay (see Table 5-9). These explicit steps can then be adopted to site-specific conditions with confidence that the Maine DOT BDM is in agreement with AASHTO LRFD methods.

References:

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American Association of State Highway and Transportation Officials (AASHTO). AASHTO LRFD Bridge Design Specifications. 9th Edition. Washington, DC 20001: American Association of State Highway and Transportation Officials (AASHTO), 2020.

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DeLano, John Gordon. “Behavior of Pile-Supported Integral Abutments at Bridge Sites with Shallow Bedrock.” Thesis, University of Maine - Main, 2004.

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Harres, Kent, Lin, Jenn-Shang, Hasanzoi, Marwa. "50 ksi Steel H-pile Capacity", University of Pittsburg Civil and Environmental Engineering for Pennsylvania Department of Transportation, Bureau of Planning and Research, June 2015.

Iowa DOT - Bridges and Structures Bureau. “LRFD Pile Design Examples.” Decatur, Iowa: Iowa Department of Transportation, January 2024.

Guertin Elkerton & Associates. "Bridge Design Guide" (with Updates Through 2018). Maine Department of Transportation, Bridge Program, August 2003.

Abutments Supported on Pile Foundations (Bridge Design Guide, BDG)

For pile supported abutments, the factored load combination causing the maximum and minimum compression in the piles should be determined, and the resulting pile reactions and pile stresses determined. The maximum factored axial pile load should not exceed the lesser of the factored geotechnical resistance and factored structural resistance for a single pile. In accordance with LRFD Article 6.5.4.2, the factored pile loads should not exceed the factored structural resistance using the resistance factors provided in BDG 5.7.2 H-Piles and BDG 5.7.5 Steel Pipe Piles. If greater loads result, more piles, or larger piles, should be considered.

Must determine the "factored geotechnical resistance***" and the "***factored structural resistance***" for a single plie.***
For the Service Limit State, the unfactored lateral pile loads for H-piles should not exceed the lateral loads resistances specified

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in BDG 5.7.2.2

Load combinations that do exceed the lateral load limits established for the service limit state should be evaluated by the Geotechnical Designer by means of a project-specific pile lateral load analysis using LPILE® software. The maximum lateral loads for all piles other than steel H-piles should be evaluated by the Geotechnical Designer. Buckling analyses of piles should be performed by the Structural Designer. Piles should also be checked for resistance against combined axial loads and flexure per LRFD 6.15 and LRFD 6.9.2.2. Pile resistance should be determined for compliance with the LRFD interaction equation.

Where abutments are required in water channels, the bottom of seal should be a minimum of 2 feet below the calculated scour depth from the check flood for scour. Where the calculated scour depth is significant, the Designer may consider designing the deep foundation elements for an unsupported length. The unsupported length should be the vertical distance from the bottom of the seal to the check flood scour depth. In designing deep foundation elements for an abutment with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

In short... Pile foundations should be designed so that the available factored geotechnical and drivability resistance is greater than the factored loads applied to the pile at the strength limit state. Service limit state design of driven pile foundations includes an evaluation of settlement, overall stability, lateral squeeze and lateral movement.

Givens/Input:

Preliminary Design Information:

Design constraints as directed by Maine DOT include the following:

- HP 10x42
- HP 12x53
- HP 14x73
- HP 14x89

"Although HP 14 x 73 pile flanges are non-compact and do not meet the slenderness requirements of LRFD 6.9.4.2, Designers can account for pile slenderness in the design process, and this pile size should still be considered for pile supported integral abutments." Chapter 5 - Substructures, Maine DOT Bridge Manual.

For *Strength Limit State Analysis:*

- Design of the piles should consider the factored structural pile resistance, Pr, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

When piles are driven to hard rock.

- Use Φc = 0.50 for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use Φc = 0.60 for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:
  - Φc = 0.70 for axial resistance
  - Φf = 1.00 for flexural resistance

For *Service Limit State Evaluations*, if piles will be driven to practical refusal in bedrock, settlement will not be a concern.

However, all designs should consider horizontal movement, overall stability, and scour for the design flood event.

Pile Properties: Note, per Section 7.2.1 *Structural Steel*, H-piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 ksi.

$f_y := 50\ ksi$, yield strength of steel (LRFD Articles 10.7.8 Drivability Analysis, and 10.9.3.10.2a Cased Length)

$E_s := 29000\ ksi$, modulus of elasticity for steel used in the H-piles planned for this project

$Section := \begin{bmatrix} HP10x42 \\ HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix}$ $Num := \begin{bmatrix} 1 \\ 2 \\ 3 \\ 4 \end{bmatrix}$ $A_s := \begin{bmatrix} 12.4 \\ 15.5 \\ 21.4 \\ 26.1 \end{bmatrix} in^2$, Area of Section

$d := \begin{bmatrix} 9.7 \\ 11.8 \\ 13.6 \\ 13.8 \end{bmatrix} in$, Depth of Section $b_f := \begin{bmatrix} 10.1 \\ 12.0 \\ 14.6 \\ 14.7 \end{bmatrix} in$, Flange Width $t_f := \begin{bmatrix} 0.420 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$, Flange Thickness

$I_x := \begin{bmatrix} 210 \\ 393 \\ 729 \\ 904 \end{bmatrix} in^4$, moment of inertia major axis $I_y := \begin{bmatrix} 71.7 \\ 127 \\ 261 \\ 326 \end{bmatrix} in^4$, moment of inertia minor axis $r_y := \begin{bmatrix} 2.41 \\ 2.86 \\ 3.49 \\ 3.53 \end{bmatrix} in$, Radius of Gyration Y-Y axis

$A_p := \overrightarrow{d \cdot b_f} = \begin{bmatrix} 98 \\ 142 \\ 199 \\ 203 \end{bmatrix} in^2$, Bottom Area for Plugged Scenarios $r_x := \begin{bmatrix} 4.13 \\ 5.03 \\ 5.84 \\ 5.88 \end{bmatrix} in$, Radius of Gyration X-X axis

$P_p := 2 \cdot d + 2 \cdot b_f = \begin{bmatrix} 40 \\ 48 \\ 56 \\ 57 \end{bmatrix} in$, Perimeter for Plugged Scenarios $t_w := \begin{bmatrix} 0.415 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$, Web Thickness

Analysis:

Determine the nominal and factored Structural Compressive Resistance...

From LRFD Section 6.9.4.2, 8th Edition - Non-slender and Slender Element Cross-Sections

λ_r , width-to-thickness ratio limit as specified in Table 6.9.4.1.1-1, 8th Edition

b , element width as specified in Table 6.9.4.2.1-1, 8th Edition; half-flange width of rolled I-beam section

t_f , element thickness (inches) - for flanges of rolled channels, use average thickness; for circular tubes and round HSS, use the wall thickness of the tube

Slenderness limit for evaluation of H-pile flanges during axial loading conditions...

$0.56 \cdot \sqrt{\frac{E_s}{f_y}} = 13.487$, slenderness limit for 50 ksi steel $\quad b := \frac{b_f}{2}$

$\lambda_r := \frac{b_f}{2 \cdot t_f}$, slenderness ratio for flange of each H-pile size being considered $\quad \lambda_r = \begin{bmatrix} 12.024 \\ 13.793 \\ 14.455 \\ 11.951 \end{bmatrix}$

Slenderness limit for evaluation of H-pile web during axial loading conditions...

$1.49 \cdot \sqrt{\frac{E_s}{f_y}} = 35.884$, slenderness limit for 50 ksi steel

$$k_0 := \frac{\left(d_0 - 9.5\ in\right)}{2} \qquad k_1 := \frac{\left(d_1 - 9.5\ in\right)}{2}$$

$$k_2 := \frac{\left(d_2 - 11.25\ in\right)}{2} \qquad k_3 := \frac{\left(d_3 - 11.25\ in\right)}{2}$$

$\lambda_{r\_web} := \frac{(d - 2 \cdot k)}{t_w}$, slenderness ratio for web of each H-pile size being considered

```
Flange_Check := | for i ∈ ORIGIN..3
                |   if 0.56·√(E_s/f_y) ≥ λ_r_i
                |     out_i ← "Non−slender"
                |   else
                |     out_i ← "Slender"
                | return out
```

```
Web_Check := | for i ∈ ORIGIN..3
             |   if 1.49·√(E_s/f_y) ≥ λ_r_web_i
             |     out_i ← "Non−slender"
             |   else
             |     out_i ← "Slender"
             | return out
```

$$Flange\_Check = \begin{bmatrix} \text{“Non−slender”} \\ \text{“Slender”} \\ \text{“Slender”} \\ \text{“Non−slender”} \end{bmatrix} \qquad Web\_Check = \begin{bmatrix} \text{“Non−slender”} \\ \text{“Non−slender”} \\ \text{“Non−slender”} \\ \text{“Non−slender”} \end{bmatrix}$$

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For Non-slender Elements:

Nominal Compressive Resistance, Pn, shall be determined as follows for elastic flexural buckling:

$E := E_s$, previously defined modulus of steel

$A_g := A_s$, previously defined gross section area

$K := 1.0$, from Table C4.6.2.5-1, 8th Edition, assumed by Maine DOT

$l := 0.1\ ft$, assumed fully braced by Maine DOT

$r_s := r_y$, previously defined radius of gyration

$P_{ecbr\_flex\_non}$, elastic critical buckling resistance based on flexural buckling of non-slender members

$$P_{ecbr\_flex\_non} := \overrightarrow{\left(\frac{\pi^2 \cdot E_s}{\left(\frac{K \cdot l}{r_y}\right)^2} \cdot A_g\right)}$$

6.9.4.1.2—Elastic Flexural Buckling Resistance

The elastic critical buckling resistance, P_e, based on flexural buckling shall be taken as:

$$P_e = \frac{\pi^2 E}{\left(\frac{K\ell}{r_s}\right)^2} A_g \qquad (6.9.4.1.2\text{-}1)$$

where:

A_g = gross cross-sectional area of the member (in.²)
K = effective length factor in the plane of buckling determined as specified in Article 4.6.2.5
ℓ = unbraced length in the plane of buckling (in.)
r_s = radius of gyration about the axis normal to the plane of buckling (in.)

From Article 6.9.4.1.1, 8th Edition...

$F_y := f_y$, previously defined yield strength

P_o , nominal yield resistance

$P_o := F_y \cdot A_g$

$P_{ncr\_flex\_non}$, nominal compressive resistance based on flexural buckling of non-slender members

```
P_ncr_flex_non := | for i ∈ ORIGIN..3
                  |   if P_ecbr_flex_non_i / P_o_i ≥ 0.44
                  |     out_i ← (0.658^(P_o_i / P_ecbr_flex_non_i)) · P_o_i
                  |   else
                  |     out_i ← 0.877 · P_ecbr_flex_non_i
                  | return out
```

- If $\frac{P_e}{P_o} \geq 0.44$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad (6.9.4.1.1\text{-}1)$$

- If $\frac{P_e}{P_o} < 0.44$, then:

$$P_n = 0.877 P_e \qquad (6.9.4.1.1\text{-}2)$$

where:

A_g = gross cross-sectional area of the member (in.²)
F_y = specified minimum yield strength (ksi)
P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kips)
P_o = nominal yield resistance = $F_y A_g$ (kips)

$$P_o = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} kip \qquad P_{ecbr\_flex\_non} = \begin{bmatrix} 14314989 \\ 25199912 \\ 51808364 \\ 64643546 \end{bmatrix} kip \qquad P_{ncr\_flex\_non} = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} kip$$

nominal yield resistance | elastic critical buckling resistance based on flexural buckling of non-slender members | nominal compressive resistance based on flexural buckling of non-slender members

For Non-slender Elements:

Nominal Compressive Resistance, Pn, shall be determined as follows for elastic torsional buckling and flexural-torsional buckling resistance:

$E := E_s$, previously defined modulus of steel

$A_g := A_s$, previously defined gross section area

C_w , warping torsional constant (inches^6)

I_y , moment of inertia about minor axis (in^4)

I_x , moment of inertia about major axis (in^4)

J , St. Vincent torsional constant, (in^4)

Kl , effective length for torsional buckling (in)

K , previously assumed by Maine DOT

l , previously assumed by Maine DOT

For open-section doubly symmetric members, the elastic critical buckling resistance, P_e, based on torsional buckling shall be taken as:

$$P_e = \left[\frac{\pi^2 E C_w}{\left(K_z \ell_z\right)^2} + GJ\right] \frac{A_g}{I_x + I_y} \qquad (6.9.4.1.3\text{-}1)$$

where:

A_g = gross cross-sectional area of the member (in.²)
C_w = warping torsional constant (in.⁶)
G = shear modulus of elasticity for steel = $0.385E$ (ksi)
I_x, I_y = moments of inertia about the major and minor principal axes of the cross-section, respectively (in.⁴)
J = St. Venant torsional constant (in.⁴)
$K_z \ell_z$ = effective length for torsional buckling (in.)

$$C_w := \begin{bmatrix} 1540 \\ 4080 \\ 11200 \\ 14200 \end{bmatrix} in^6 \qquad J := \begin{bmatrix} 0.813 \\ 1.12 \\ 2.01 \\ 3.59 \end{bmatrix} in^4$$

$G := 0.385 \cdot E_s$, shear modulus of elasticity for steel

$P_{ecbr\_tor\_non}$, elastic critical buckling resistance based on torsional buckling of non-slender members

$P_{ncr\_tor\_non}$, nominal compressive resistance based on torsional buckling of non-slender members

$$P_{ecbr\_tor\_non} := \overrightarrow{\left(\left(\frac{\pi^2 \cdot E_s \cdot C_w}{(K \cdot l)^2}\right) + G \cdot J\right) \cdot \left(\frac{A_g}{I_x + I_y}\right)}$$

$$P_{ncr\_tor\_non} := \left\| \begin{array}{l} \text{for } i \in \text{ORIGIN} .. 3 \\ \quad \text{if } \dfrac{P_{ecbr\_tor\_non_i}}{P_{o_i}} \ge 0.44 \\ \quad \quad out_i \leftarrow \left(0.658^{\left(\frac{P_{o_i}}{P_{ecbr\_tor\_non_i}}\right)}\right) \cdot P_{o_i} \\ \quad \text{else} \\ \quad \quad out_i \leftarrow 0.877 \cdot P_{ecbr\_tor\_non_i} \\ \text{return } out \end{array} \right.$$

$$P_{ecbr\_tor\_non} = \begin{bmatrix} 13474222 \\ 24172995 \\ 48121176 \\ 59891496 \end{bmatrix} kip$$

elastic critical buckling resistance based on torsional buckling of non-slender members

$$P_{ncr\_tor\_non} = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} kip$$

nominal compressive resistance based on torsional buckling of non-slender members

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Determine the Factored Structural Compressive Resistance for Non-Slender Elements

From Section 6.9.2.1, 8th Edition - Axial Compression

P_{frc} , factored resistance of components in compression

$P_{frc\_flex\_non}$, factored resistance of components in compression based on flexural load condition of a non-slender member

$P_{frc\_tor\_non}$, factored resistance of components in compression based on torsional load condition of a non-slender member

ϕ_c , resistance factor for compression as specified in Article 6.5.4.2, 8th Edition

$\phi_c := 0.7$

- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_f = 1.00$
- For shear connectors in tension $\phi_{st} = 0.75$

$P_{ncr\_flex\_non}$, nominal compressive resistance based on flexural load condition of a non-slender member

$P_{ncr\_tor\_non}$, nominal compressive resistance based on torsional load condition of a non-slender member

$P_{frc\_flex\_non} := \phi_c \cdot P_{ncr\_flex\_non}$ Equation 6.9.2.1-1, 8th Edition

$P_{frc\_tor\_non} := \phi_c \cdot P_{ncr\_tor\_non}$ Equation 6.9.2.1-1, 8th Edition

$$P_{frc\_flex\_non} = \begin{bmatrix} 434 \\ 542 \\ 749 \\ 913 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on flexural load condition of a non-slender member

$$P_{frc\_tor\_non} = \begin{bmatrix} 434 \\ 542 \\ 749 \\ 913 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on torsional load condition of a non-slender member

Article 6.9.3 - Limiting Slenderness Ratio for Compression Members, 8th Edition

$$\frac{K \cdot l}{r_s} = \begin{bmatrix} 0.5 \\ 0.4 \\ 0.3 \\ 0.3 \end{bmatrix}$$

values are below 120, and are therefore... OK

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From Section 6.9.4.2.2a, 8th Edition - Effective Width of Slender Elements

λ_r , previously defined as slenderness ratio in Table 6.9.4.2.1-1, 8th Edition

b_f , previously defined as flange width

F_{ncr_slen} , nominal compressive resistance of the member calculated from Eq. 6.9.4.2.2-2 using Ag (ksi)

P_{ncr_slen} , nominal compressive resistance of the member calculated from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2 using Ag (ksi)

$P_{ncr\_flex\_slen} := P_{ncr\_flex\_non}$

$P_{ncr\_tor\_slen} := P_{ncr\_tor\_non}$

t_f , previously defined as flange thickness

A_g , previously defined as gross area

$$F_{ncr\_flex\_slen} := \frac{P_{ncr\_flex\_slen}}{A_g}$$

$$F_{ncr\_tor\_slen} := \frac{P_{ncr\_tor\_slen}}{A_g}$$

Table 6.9.4.2.2a-1—Effective Width Imperfection Adjustment Factors, c_1 and c_2

Slender Element	c_1	c_2
All elements supported along two longitudinal edges, except walls of square and rectangular HSS	0.18	1.31
Walls of square and rectangular HSS	0.20	1.38
All other elements	0.22	1.49

$c_1 := 0.22$ $c_2 := 1.49$

F_{el} , elastic local buckling stress

c_1 , effective width imperfection adjustment factor from Table 6.9.4.2.2a-1

c_2 , effective width imperfection adjustment factor from Table 6.9.4.2.2a-1

$$F_{el} := \left(c_2 \cdot \frac{\lambda_r}{\left(\frac{b_f}{t_f} \right)} \right)^2 \cdot f_y$$

b_e , effective width of slender elements

$$b_{e\_flex\_slen} := \left\| \begin{array}{l} \text{for } i \in \text{ORIGIN} .. 3 \\ \quad \left\| \begin{array}{l} \text{if } \dfrac{b_{f_i}}{t_{f_i}} \le \lambda_{r_i} \cdot \left(\dfrac{f_y}{F_{ncr\_flex\_slen_i}} \right)^{0.5} \\ \quad \left\| \; out_i \leftarrow b_{f_i} \right. \\ \text{else} \\ \quad \left\| \; out_i \leftarrow b_{f_i} \cdot \left(1 + c_1 \cdot \left(\dfrac{F_{el_i}}{F_{ncr\_flex\_slen_i}} \right)^{0.5} \right) \cdot \left(\dfrac{F_{el_i}}{F_{ncr\_flex\_slen_i}} \right)^{0.5} \right. \end{array} \right. \\ \text{return } out \end{array} \right.$$

$$F_{ncr\_flex\_slen} = \begin{bmatrix} 7200 \\ 7200 \\ 7200 \\ 7200 \end{bmatrix} ksf \quad F_{ncr\_tor\_slen} = \begin{bmatrix} 7200 \\ 7200 \\ 7200 \\ 7200 \end{bmatrix} ksf \quad F_{el} = \begin{bmatrix} 3996 \\ 3996 \\ 3996 \\ 3996 \end{bmatrix} ksf \quad b_{e\_flex\_slen} = \begin{bmatrix} 8.8 \\ 10.4 \\ 12.7 \\ 12.7 \end{bmatrix} in$$

$$b_{e\_tor\_slen} := \begin{array}{l} \text{for } i \in \text{ORIGIN}..3 \\ \quad \text{if } \dfrac{b_{f_i}}{t_{f_i}} \le \lambda_{r_i} \cdot \left(\dfrac{f_y}{F_{ncr\_tor\_slen_i}}\right)^{0.5} \\ \quad\quad out_i \leftarrow b_{f_i} \\ \quad \text{else} \\ \quad\quad out_i \leftarrow b_{f_i} \cdot \left(1 + c_1 \cdot \left(\dfrac{F_{el_i}}{F_{ncr\_tor\_slen_i}}\right)^{0.5}\right) \cdot \left(\dfrac{F_{el_i}}{F_{ncr\_tor\_slen_i}}\right)^{0.5} \\ \text{return } out \end{array}$$

$$b_{e\_tor\_slen} = \begin{bmatrix} 8.8 \\ 10.4 \\ 12.7 \\ 12.7 \end{bmatrix} in$$

For Slender Elements:

H-piles in question are classified as slender elements cross-sections and shall be subject to requirements of Section 6.9.4.2.2- Slender Element Cross-Sections, 8th Edition. For compression members with slender element cross-section, the Nominal Compressive Resistance, Pn, shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling, or flexural-torsional buckling.

A_{eff} , summation of the effective areas based on effective width

$$A_{eff\_flex} := A_g - \sum_{i=0}^{3} \left(b - b_{e\_flex\_slen}\right) \cdot t_f$$

$$A_{eff\_tor} := A_g - \sum_{i=0}^{3} \left(b - b_{e\_tor\_slen}\right) \cdot t_f$$

$$P_{ncr\_flex\_slen} := \overrightarrow{F_{ncr\_flex\_slen} \cdot A_{eff\_flex}}$$

$$P_{ncr\_tor\_slen} := \overrightarrow{F_{ncr\_tor\_slen} \cdot A_{eff\_tor}}$$

$$A_{eff\_flex} = \begin{bmatrix} 50.4 \\ 53.5 \\ 59.4 \\ 64.1 \end{bmatrix} in^2$$

$$A_{eff\_tor} = \begin{bmatrix} 50.4 \\ 53.5 \\ 59.4 \\ 64.1 \end{bmatrix} in^2$$

$$P_n = F_{cr} A_{eff} \qquad (6.9.4.2.2\text{-}1)$$

in which:

$$F_{cr} = \frac{P_{cr}}{A_g} \qquad (6.9.4.2.2\text{-}2)$$

where:

A_{eff} = summation of the effective areas of the cross-section based on reduced effective widths, b_e, for each slender element in the cross-section = $A_g - \sum(b - b_e)t$, or the effective area determined as specified in Article 6.9.4.2.2b for circular tubes and round HSS (in.^{2})

A_g = total gross cross-sectional area of the member (in.^{2})

b = element width as specified in Table 6.9.4.2.1-1 (in.)

b_e = effective width of the slender element determined as specified in Article 6.9.4.2.2a (in.)

P_{cr} = nominal compressive resistance of the member calculated from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable, using A_g (kip)

t = element thickness (in.)

$$P_{ncr\_flex\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} kip \qquad P_{ncr\_tor\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} kip$$

Determine the Factored Structural Compressive Resistance for Slender Elements

From Section 6.9.2.1, 8th Edition - Axial Compression

P_{frc} , factored resistance of components in compression

ϕ_c , resistance factor for compression as specified in Article 6.5.4.2, 8th Edition

$\phi_c := 0.70$, axial-only resistance for H-piles

$P_{ncr\_flex\_slen}$, nominal compressive resistance - flexural

$P_{ncr\_tor\_slen}$, nominal compressive resistance - torsional

$P_{frc\_flex\_slen} := \phi_c \cdot P_{ncr\_flex\_slen}$ Equation 6.9.2.1-1, 8th Edition

$P_{frc\_tor\_slen} := \phi_c \cdot P_{ncr\_tor\_slen}$ Equation 6.9.2.1-1, 8th Edition

- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles $\phi_c = 0.50$
  - pipe piles $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles $\phi_c = 0.60$
  - pipe piles $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles $\phi_c = 0.70$
  - axial resistance for pipe piles $\phi_c = 0.80$
  - flexural resistance $\phi_f = 1.00$
- For shear connectors in tension $\phi_{st} = 0.75$

$$P_{ncr\_flex\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} \mathit{kip} \qquad P_{ncr\_tor\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} \mathit{kip}$$

$$P_{frc\_flex\_slen} = \begin{bmatrix} 1764 \\ 1872 \\ 2079 \\ 2243 \end{bmatrix} \mathit{kip} \qquad P_{frc\_tor\_slen} = \begin{bmatrix} 1764 \\ 1872 \\ 2079 \\ 2243 \end{bmatrix} \mathit{kip}$$

Factored Axial Pile Resistances at Strength Limit States

$\phi_{c\_severe\_driving} := 0.50$,for axial resistance when subject to severe pile driving conditions

$\phi_{c\_good\_driving} := 0.60$,for axial resistance when subject to good pile driving conditions

$\phi_{c\_axial} := 0.70$,for axial resistance in upper zone

$\phi_{c\_flex} := 1.00$,for flexural resistance in the upper zone

$P_{frc\_flex\_non\_severe} := \phi_{c\_severe\_driving} \cdot P_{ncr\_flex\_non}$

$P_{frc\_flex\_non\_good} := \phi_{c\_good\_driving} \cdot P_{ncr\_flex\_non}$

$P_{frc\_flex\_non\_axial} := \phi_{c\_axial} \cdot P_{ncr\_flex\_non}$

$P_{frc\_flex\_non\_flex} := \phi_{c\_flex} \cdot P_{ncr\_flex\_non}$

$P_{frc\_tor\_non\_severe} := \phi_{c\_severe\_driving} \cdot P_{ncr\_tor\_non}$

$P_{frc\_tor\_non\_good} := \phi_{c\_good\_driving} \cdot P_{ncr\_tor\_non}$

$P_{frc\_tor\_non\_axial} := \phi_{c\_axial} \cdot P_{ncr\_tor\_non}$

$P_{frc\_tor\_non\_flex} := \phi_{c\_flex} \cdot P_{ncr\_tor\_non}$

$P_{frc\_flex\_slen\_severe} := \phi_{c\_severe\_driving} \cdot P_{ncr\_flex\_slen}$

$P_{frc\_flex\_slen\_good} := \phi_{c\_good\_driving} \cdot P_{ncr\_flex\_slen}$

$P_{frc\_flex\_slen\_axial} := \phi_{c\_axial} \cdot P_{ncr\_flex\_slen}$

$P_{frc\_flex\_slen\_flex} := \phi_{c\_flex} \cdot P_{ncr\_flex\_slen}$

$P_{frc\_tor\_slen\_severe} := \phi_{c\_severe\_driving} \cdot P_{ncr\_tor\_slen}$

$P_{frc\_tor\_slen\_good} := \phi_{c\_good\_driving} \cdot P_{ncr\_tor\_slen}$

$P_{frc\_tor\_slen\_axial} := \phi_{c\_axial} \cdot P_{ncr\_tor\_slen}$

$P_{frc\_tor\_slen\_flex} := \phi_{c\_flex} \cdot P_{ncr\_tor\_slen}$

$$P_{frc\_flex\_non\_severe} = \begin{bmatrix} 310 \\ 387 \\ 535 \\ 652 \end{bmatrix} \, kip$$

$$P_{frc\_flex\_non\_good} = \begin{bmatrix} 372 \\ 465 \\ 642 \\ 783 \end{bmatrix} \, kip$$

**confirms Table 5-7 in Maine DOT Bridge Design Guide.

"The factored axial structural axial resistances of selected H-Pile sections are presented in Table 5-7. For the purposes of Table 5-7, the H-piles were assumed fully braced, and an effective length factor (K) of 1.0 was used. *The Structural Designer should recalculate structural resistances for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific LPILE® analyses and recalculate structural resistances*. For preliminary design purposes, however, the resistances provided in Table 5-7 may be used to estimate the factored structural axial resistance of that portion of the pile which is theoretically in pure compression, i.e., that portion below the point of fixity.

Experience in using 50 ksi steel for H-Pile foundations has shown that the factored axial geotechnical resistance frequently governs design. This is particularly apparent for end-bearing piles on poor-quality and/or soft bedrock and for friction piles."

Table 5-7 Factored Axial Structural Resistance of Selected H-Pile Sections

F_y = 50 ksi and fully braced

Pile Section	Factored Axial Structural Resistance	
	Good driving conditions Φ = 0.60 (kips)	Severe driving conditions Φ = 0.50 (kips)
HP 10x42+	372	310
HP 10x57	504	420
HP 12x53+	465	388
HP 12x63	552	460
HP 12x74	654	545
HP 12x84	738	615
HP 14x73+	642	535
HP 14x89+	783	653
HP 14x102	900	750
HP 14x117	1032	860

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Per BDM 5.7.2.1... "The factored geotechnical and drivability resistances should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation , soil setup, lateral spreading and any other site-specific factors, which may affect the pile capacity during and after construction. The factored geotechnical resistance should be determined by applying a resistance, f actor which is dependent on the design method."

Driveability resistances for a single pile in axial compression when a dynamic test is performed pe r Article 10.5.5.2.3-1

$\phi_{c\_driveability} := 0.65$,resistance factor for single pile in axial compression when a dynamic test is performed

$P_{frc\_flex\_non\_dyn} := \phi_{c\_driveability} \cdot P_{ncr\_flex\_non}$

$P_{frc\_tor\_non\_dyn} := \phi_{c\_driveability} \cdot P_{ncr\_tor\_non}$

$P_{frc\_flex\_slen\_dyn} := \phi_{c\_driveability} \cdot P_{ncr\_flex\_slen}$

$P_{frc\_tor\_slen\_dyn} := \phi_{c\_driveability} \cdot P_{ncr\_tor\_slen}$

$$P_{frc\_flex\_non\_dyn} = \begin{bmatrix} 403 \\ 504 \\ 695 \\ 848 \end{bmatrix} \boldsymbol{kip}$$

$$P_{frc\_tor\_non\_dyn} = \begin{bmatrix} 403 \\ 504 \\ 695 \\ 848 \end{bmatrix} \boldsymbol{kip}$$

$$P_{frc\_flex\_slen\_dyn} = \begin{bmatrix} 1638 \\ 1739 \\ 1930 \\ 2083 \end{bmatrix} \boldsymbol{kip}$$

$$P_{frc\_tor\_slen\_dyn} = \begin{bmatrix} 1638 \\ 1739 \\ 1930 \\ 2083 \end{bmatrix} \boldsymbol{kip}$$

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Factored Axial Pile Resistances at Service and Extreme Limit States

Horizontal movement of pile groups induced by lateral loads shall be evaluated for Service Limit State Design. The lateral resistance of a pile is governed by the loading condition, pile stiffness, stiffness of the soil, and the degree of fixity. Service Limit State Design of driven pile foundations includes an evaluation of settlement, overall stability, lateral squeeze, and lateral movement.

The lateral resistance (P_L) and depth to fixity (D_f), for service limit state design for selected H-Pile sections in sand and clay are presented in Table 5-8 and Table 5-9, respectively. The factored lateral resistances presented in Tables 5-8 and 5-9 assume a resistance factor of 1.0 and a maximum lateral deflection of 1/8 inch.

The lateral resistance and depth to fixity presented in Tables 5-8 and Table 5-9 were determined using the computer program LPILE® Plus Version 4, the soil properties stated, a fixed condition at the pile head, an infinitely long pile, an applied axial load equal to As x 0.25 x F_y, a deflection of 1/8", and a friction angle of 32 degrees.

Factored Lateral Resistance and Depth to Fixity for Service Limit State Design

Where the applied lateral load from the Service Limit State Load Combination exceeds that presented in Tables 5-8 and 5-9, or the pile length is less than the depth to fixity shown in the table, a more thorough analysis is recommended, using actual loading and soil conditions. Where soils differ from the conditions assumed in the tables, the Designer should complete a more thorough analysis.

Tables 5-8 and 5-9 present the lateral resistance and depth to fixity for a lateral load applied perpendicular to the pile flange. For conventional abutments and mass piers, H-piles should be oriented with the flange perpendicular to the substructure axis in the direction of the maximum applied lateral load. For conventional abutments and mass piers, where H-piles are oriented with the web perpendicular to the maximum applied lateral load, a thorough analysis of the foundation is recommended, using actual loading and soil conditions (Tables 5-8 and 5-9 do not apply). For integral abutments where the web is oriented perpendicular to the principal axis, the design should be in accordance with Section 5.4.2 Integral Abutments.

Table 5-8 Factored Lateral Resistance and Depth to Fixity for Strength Limit State Design for H-Pile Sections in Sand, φ=1.0

Pile Section	Loose		Medium Dense		Dense	
	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)
HP 10x42+	6.2	24	9.9	20	11.7	18
HP 10x57	7.1	26	11.4	22	13.6	19
HP 12x53+	8.1	28	13.3	24	16.1	20
HP 12x63	8.9	30	14.4	25	17.4	21
HP 12x74	9.4	31	15.6	25	18.9	22
HP 13x60	9.0	31	15.0	25	18.2	21
HP 13x73	9.8	32	16.4	26	20.0	22
HP 13x87	10.6	32	17.7	26	21.7	23
HP 14x73+	10.5	32	17.8	26	21.9	23
HP 14x89+	11.4	33	19.5	27	24.1	24
HP 14x102	12.3	35	20.9	28	25.9	25
HP 14x117	13.1	36	22.3	29	27.0	25

Note: Those marked + are preferred sections. P_L and D_f are determined assuming a friction angle, ϕ, of 32°.

Table 5-9 Factored Lateral Resistance and Depth to Fixity for Service Limit State Design for H-Pile Sections in Clay, φ=1.0, Load Perpendicular to Flange

Pile Section	Soft[1]		Medium Stiff[2]		Stiff[3]	
	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)
HP 10x42+	5.1	22	9.2	18	13.1	16
HP 10x57	5.5	24	10.2	20	14.5	18
HP 12x53+	6.3	26	11.7	21	16.6	19
HP 12x63	6.7	27	12.4	22	17.6	19
HP 12x74	7.1	27	13.1	22	18.7	20
HP 13x60	7.0	27	12.8	22	18.2	19
HP 13x73	7.5	28	13.8	23	19.5	21
HP 13x87	7.9	29	15.6	25	20.7	21
HP 14x73+	8.1	29	14.8	24	21.0	21
HP 14x89+	8.7	31	15.9	25	22.5	22
HP 14x102	9.1	31	16.7	26	23.6	22
HP 14x117	9.5	32	17.5	26	24.8	24

Note: Those marked + are preferred sections.
[1]S_u = 375 psf, [2]S_u = 750 psf, [3]S_u = 1125 psf

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Results:

$$P_o = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} \textit{kip}$$

nominal yield resistance

$$P_{ecbr\_flex\_non} = \begin{bmatrix} 14314989 \\ 25199912 \\ 51808364 \\ 64643546 \end{bmatrix} \textit{kip}$$

elastic critical buckling resistance based on flexural buckling of non-slender members

$$P_{ncr\_flex\_non} = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on flexural buckling of non-slender members

$$P_{ecbr\_tor\_non} = \begin{bmatrix} 13474222 \\ 24172995 \\ 48121176 \\ 59891496 \end{bmatrix} \textit{kip}$$

elastic critical buckling resistance based on torsional buckling of non-slender members

$$P_{ncr\_tor\_non} = \begin{bmatrix} 620 \\ 775 \\ 1070 \\ 1305 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on torsional buckling of non-slender members

$$P_{frc\_flex\_non} = \begin{bmatrix} 434 \\ 542 \\ 749 \\ 913 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on flexural load condition of a non-slender member

$$P_{frc\_tor\_non} = \begin{bmatrix} 434 \\ 542 \\ 749 \\ 913 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on torsional load condition of a non-slender member

$$P_{ncr\_flex\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on flexural load condition of a slender member

$$P_{ncr\_tor\_slen} = \begin{bmatrix} 2520 \\ 2675 \\ 2970 \\ 3205 \end{bmatrix} \textit{kip}$$

nominal compressive resistance based on torsional load condition of a slender member

$$P_{frc\_flex\_slen} = \begin{bmatrix} 1764 \\ 1872 \\ 2079 \\ 2243 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on flexural load condition of a slender member

$$P_{frc\_tor\_slen} = \begin{bmatrix} 1764 \\ 1872 \\ 2079 \\ 2243 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on torsional load condition of a slender member

$$P_{frc\_flex\_non\_dyn} = \begin{bmatrix} 403 \\ 504 \\ 695 \\ 848 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on flexural load condition of a non-slender member when a dynamic test is performed

$$P_{frc\_tor\_non\_dyn} = \begin{bmatrix} 403 \\ 504 \\ 695 \\ 848 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on torsional load condition of a non-slender member when a dynamic test is performed

$$P_{frc\_flex\_slen\_dyn} = \begin{bmatrix} 1638 \\ 1739 \\ 1930 \\ 2083 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on flexural load condition of a slender member when a dynamic test is performed

$$P_{frc\_tor\_slen\_dyn} = \begin{bmatrix} 1638 \\ 1739 \\ 1930 \\ 2083 \end{bmatrix} \textit{kip}$$

factored resistance of components in compression based on torsional load condition of a slender member when a dynamic test is performed

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$$P_{frc\_flex\_non\_severe} = \begin{bmatrix} 310 \\ 387 \\ 535 \\ 652 \end{bmatrix} \, kip$$

factored resistance of components in compression based on flexural load condition of a non-slender member exposed to severe, or hard driving

$$P_{frc\_flex\_non\_good} = \begin{bmatrix} 372 \\ 465 \\ 642 \\ 783 \end{bmatrix} \, kip$$

factored resistance of components in compression based on flexural load condition of a non-slender member exposed to good, or easy driving

$$P_{fdr\_dyn} := \begin{bmatrix} 182 \\ 234 \\ 289 \\ 413 \end{bmatrix}$$

factored driving resistance when a dynamic test is performed, see WEAP analysis appendix and Calc-002f Geotechnical Resistance

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Factored Axial Resistance at Strength Limit State (Structural Resistance)

SR ,structural resistance

$$SR_a := \min\left(P_{o_0}, P_{ecbr\_flex\_non_0}, P_{ncr\_flex\_non_0}, P_{ecbr\_tor\_non_0}, P_{ncr\_tor\_non_0}, P_{frc\_flex\_non_0}, P_{frc\_tor\_non_0}\right)$$

$$SR_b := \min\left(P_{ncr\_flex\_slen_0}, P_{ncr\_tor\_slen_0}, P_{frc\_flex\_slen_0}, P_{frc\_tor\_slen_0}, P_{frc\_flex\_non\_severe_0}\right)$$

$$SR_c := \min\left(P_{frc\_flex\_non\_dyn_0}, P_{frc\_tor\_non\_dyn_0}, P_{frc\_flex\_slen\_dyn_0}, P_{frc\_tor\_slen\_dyn_0}, P_{frc\_flex\_non\_good_0}\right)$$

$$SR_0 := min\left(SR_a, SR_b, SR_c\right)$$

$$SR_d := \min\left(P_{o_1}, P_{ecbr\_flex\_non_1}, P_{ncr\_flex\_non_1}, P_{ecbr\_tor\_non_1}, P_{ncr\_tor\_non_1}, P_{frc\_flex\_non_1}, P_{frc\_tor\_non_1}\right)$$

$$SR_e := \min\left(P_{ncr\_flex\_slen_1}, P_{ncr\_tor\_slen_1}, P_{frc\_flex\_slen_1}, P_{frc\_tor\_slen_1}, P_{frc\_flex\_non\_severe_1}\right)$$

$$SR_f := \min\left(P_{frc\_flex\_non\_dyn_1}, P_{frc\_tor\_non\_dyn_1}, P_{frc\_flex\_slen\_dyn_1}, P_{frc\_tor\_slen\_dyn_1}, P_{frc\_flex\_non\_good_1}\right)$$

$$SR_1 := min\left(SR_d, SR_e, SR_f\right)$$

$$SR_g := \min\left(P_{o_2}, P_{ecbr\_flex\_non_2}, P_{ncr\_flex\_non_2}, P_{ecbr\_tor\_non_2}, P_{ncr\_tor\_non_2}, P_{frc\_flex\_non_2}, P_{frc\_tor\_non_2}\right)$$

$$SR_h := \min\left(P_{ncr\_flex\_slen_2}, P_{ncr\_tor\_slen_2}, P_{frc\_flex\_slen_2}, P_{frc\_tor\_slen_2}, P_{frc\_flex\_non\_severe_2}\right)$$

$$SR_i := \min\left(P_{frc\_flex\_non\_dyn_2}, P_{frc\_tor\_non\_dyn_2}, P_{frc\_flex\_slen\_dyn_2}, P_{frc\_tor\_slen\_dyn_2}, P_{frc\_flex\_non\_good_2}\right)$$

$$SR_2 := min\left(SR_g, SR_h, SR_i\right)$$

$$SR_j := \min\left(P_{o_3}, P_{ecbr\_flex\_non_3}, P_{ncr\_flex\_non_3}, P_{ecbr\_tor\_non_3}, P_{ncr\_tor\_non_3}, P_{frc\_flex\_non_3}, P_{frc\_tor\_non_3}\right)$$

$$SR_k := \min\left(P_{ncr\_flex\_slen_3}, P_{ncr\_tor\_slen_3}, P_{frc\_flex\_slen_3}, P_{frc\_tor\_slen_3}, P_{frc\_flex\_non\_severe_3}\right)$$

$$SR_l := \min\left(P_{frc\_flex\_non\_dyn_3}, P_{frc\_tor\_non\_dyn_3}, P_{frc\_flex\_slen\_dyn_3}, P_{frc\_tor\_slen\_dyn_3}, P_{frc\_flex\_non\_good_3}\right)$$

$$SR_3 := min\left(SR_j, SR_k, SR_l\right)$$

$$SR = \begin{bmatrix} 310 \\ 387 \\ 535 \\ 652 \end{bmatrix} \mathbf{kip}$$,when severe driving conditions control structural resistance

$$P_{frc\_flex\_non\_good} = \begin{bmatrix} 372 \\ 465 \\ 642 \\ 783 \end{bmatrix} \mathbf{kip}$$,when good driving conditions control structural resistance

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$P_o \cdot 0.7 = \begin{bmatrix} 434 \\ 543 \\ 749 \\ 914 \end{bmatrix} \boldsymbol{kip}$,when pre-drilled and grouted in-place (no driving) controls

Results indicate that the methods used herein provide the same results as published by Maine DOT in their Bridge Design Manual for Strength Limit State Design evaluations per AASHTO LRFD Bridge Manual, 8th Edition.

It should be noted that the flange check for slenderness results do not agree with the statements made by Maine DOT and others. Also, only axial compression loads were evaluated, no lateral loads or moments were evaluated or taken into account by the equations/functions relied upon.

Conclusions:

The methodologies, equations, and functions described herein are suitable for site-specific application with all due attention and caution by the user.

Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of the originator and the checker.

Prepared By:	Russell L. Thomas, Jr.		3/30/2024
	Print Name	Signature	Date
Review By:	Dan Kubinski		8/9/2024
	Print Name	Signature	Date

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Draft Pile Design for Western Abutment Using Design Methodology per Maine DOT Bridge Design Guide - Driven H-Piles Grouted In-Place Using Pre-Drilled Rock Sockets

Objectives:

The objective of this calculation package is to evaluate four sizes of steel H-piles for use in the design of an integral abutment bridge per Maine DOT specifications/guidelines, including Strength Limit State and Service Limit State evaluations. In particular, the objective of this calculation package is to evaluate Abutment 2, the eastern-most abutment, where H-piles are planned to be grouted in-place using pre-drilled rock sockets.

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Givens/Input:

Preliminary Design Information (provided by Mr. Keith Wood, P.E., project Structural Engineer for Kleinfelder)

- The bridge structure is planned to be supported on 50 ksi steel H-piles as part of an integral abutment substructure.
- The piles are to be driven to bedrock at Abutment 1 and pre-drilled with a rock socket at Abutment 2. This calculation package considers the evaluation of Abutment 2. Evaluation of Abutment 1 is under separate cover.
- Current design includes a Maximum Strength I Axial Load of 310 kips, and a Service I Axial Load of 210 kips.

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- The bridge generally lies on a West to East alignment . As such, End Bent 1 is located on the western side of Kitteredge Brook and End Bent 2 is located on the eastern side of Kitteredge Brook.
- The bridge is planned to have one, 56-foot long span between the two end bents.
- Cross section indicates that the vertical curve (i.e., roadway alignment) along Route 3 is to be raised about 3 1/2 feet at End Bent 1 and about 3 feet at End Bent 2.
- Considering the condition of the existing abutments, existing slope protection, the planned integral abutments, and improved slope protection, the upper materials are protected while medium dense, fine to coarse SANDS will be exposed along the channel bottom. These soils extend to the top of rock. In addition, the scour report suggests 5 feet of contraction scour could occur. The bottom scour elevation of approximately -6 feet will be used herein (See Figure 31 of Scour Report).
- "Calc-001a Friction Angle for Existing Fill" remains valid and may be relied upon. (phi = 32 degrees)
- "Calc-001b Friction Angle for Natural Sand" remains valid and may be relied upon. (phi = 34 degrees)
- "Calc-002a LRFD Pile Design - Strength Limit State Analysis" remains valid and may be relied upon.

Design constraints as directed by Maine DOT include the following:

- Most commonly used pile sizes:
    - HP 10x42
    - HP 12x53
    - HP 14x73
    - HP 14x89
- For *Strength Limit State Analysis*:
    - Design of the piles should consider the factored structural pile resistance, Pr, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.
    - Both MaineDOT and AASHTO provide factors for axial resistance based on damage sustained to the H-pile during driving. However, neither sources suggest a resistance factor for pre-drilled, grouted-in-place piles. However, AASHTO does provide guidance regarding drilled shafts bearing in rock.
        - If the rock socket is greater than 1.5B and the rock is intact or tightly jointed, then Eq. 10.8.3.5.4c-1 is valid.
        - That is, Qp = 2.5Qu. Where Qp is the nominal unit tip resistance and Qu is the uniaxial compressive strength of intact rock.
        - If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the grout strength, the value of grout compressive strength, f'c, shall be substituted for Qu.
- For *Service Limit State Evaluations*, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability, and scour for the design flood event.

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Assumptions:

- The bridge is to be designed in accordance with AASHTO LRFD Bridge Design Specifications, 9th Edition and Maine DOT's Bridge Design Guide (2003 with updates through 2018).
- Borings were performed per ASTM / Maine DOT standards and guidelines.
- Borings were performed at the locations shown on the provided site plan.
- Standard penetration tests (SPT) reported on soil logs were conducted properly and with in-tolerance, or calibrated, equipment by qualified personnel.
- SPT N-values reported on soil logs are representative of the area(s) of concern and that proper engineering judgment is being applied to the use of measured N-values.
- The equations offered by AASHTO for use in bridge design are accurate and appropriate to apply to this situation.
- The assumptions noted by Maine DOT are a reflection of experience, field testing, and long-term observation of built structures and are also accurate and appropriate to apply to this situation.
- The data gathered during the geotechnical studies by Kleinfelder for the project site are sufficient for determining design parameters and the information they contain is accurate and representative of the project site.
- Published data for the physical properties of steel H-piles by NuCor Skyline are correct:

Pile Properties: Note, per Section 7.2.1 *Structural Steel*, H-piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel.

$f_y := 50\ ksi$, yield strength of steel (LRFD Articles 10.7.8 Drivability Analysis, and 10.9.3.10.2a Cased Length)

$E_s := 29000\ ksi$, modulus of elasticity for steel used in the H-piles planned for this project

$$Section := \begin{bmatrix} \boxed{HP10x42} \\ HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix} \qquad Num := \begin{bmatrix} 1 \\ 2 \\ 3 \\ 4 \end{bmatrix} \qquad A_s := \begin{bmatrix} 12.4 \\ 15.5 \\ 21.4 \\ 26.1 \end{bmatrix} in^2$$

, Area of Section

$$d := \begin{bmatrix} 9.7 \\ 11.8 \\ 13.6 \\ 13.8 \end{bmatrix} in$$

, Depth of Section

$$b_f := \begin{bmatrix} 10.1 \\ 12.0 \\ 14.6 \\ 14.7 \end{bmatrix} in$$

, Flange Width

$$t_f := \begin{bmatrix} 0.420 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$$

, Flange Thickness

$$I_x := \begin{bmatrix} 210 \\ 393 \\ 729 \\ 904 \end{bmatrix} in^4$$

, moment of inertia major axis

$$I_y := \begin{bmatrix} 71.7 \\ 127 \\ 261 \\ 326 \end{bmatrix} in^4$$

, moment of inertia minor axis

$$r_y := \begin{bmatrix} 2.41 \\ 2.86 \\ 3.49 \\ 3.53 \end{bmatrix} in$$

, Radius of Gyration Y-Y axis

$$A_p := \overrightarrow{d \cdot b_f} = \begin{bmatrix} 98 \\ 142 \\ 199 \\ 203 \end{bmatrix} in^2$$

, Bottom Area for Plugged Scenarios

$$r_x := \begin{bmatrix} 4.13 \\ 5.03 \\ 5.84 \\ 5.88 \end{bmatrix} in$$

, Radius of Gyration X-X axis

$$P_p := 2 \cdot d + 2 \cdot b_f = \begin{bmatrix} 40 \\ 48 \\ 56 \\ 57 \end{bmatrix} in$$

, Perimeter for Plugged Scenarios

$$t_w := \begin{bmatrix} 0.415 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$$

, Web Thickness

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Abutments Supported on Pile Foundations (Bridge Design Guide, BDG)

For pile supported abutments, the factored load combination causing the maximum and minimum compression in the piles should be determined, and the resulting pile reactions and pile stresses determined. The maximum factored axial pile load should not exceed the lesser of the factored geotechnical resistance and factored structural resistance for a single pile. In accordance with LRFD Article 6.5.4.2, the factored pile loads should not exceed the factored structural resistance using the resistance factors provided in BDG 5.7.2 H-Piles and BDG 5.7.5 Steel Pipe Piles. If greater loads result, more piles, or larger piles, should be considered.

For the Service Limit State, the unfactored lateral pile loads for H-piles should not exceed the lateral loads resistances specified in 5.7.2.2

Load combinations that do exceed the lateral load limits established for the service limit state should be evaluated by the Geotechnical Designer by means of a project-specific pile lateral load analysis using LPILE® software. The maximum lateral loads for all piles other than steel H-piles should be evaluated by the Geotechnical Designer. Buckling analyses of piles should be performed by the Structural Designer. Piles should also be checked for resistance against combined axial loads and flexure per LRFD 6.15 and 6.9.2.2. Pile resistance should be determined for compliance with the LRFD interaction equation.

Where abutments are required in water channels, the bottom of seal should be a minimum of 2 feet below the calculated scour depth from the check flood for scour. Where the calculated scour depth is significant, the Designer may consider designing the deep foundation elements for an unsupported length. The unsupported length should be the vertical distance from the bottom of the seal to the check flood scour depth. In designing deep foundation elements for an abutment with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

In short... Pile foundations should be designed so that the available factored geotechnical and drivability resistance is greater than the factored loads applied to the pile at the strength limit state. Service limit state design of driven pile foundations includes an evaluation of settlement, overall stability, lateral squeeze and lateral movement.

Design Steps Per Maine DOT (BDG 5.4.2.4.C)

Step 1. Determine the foundation displacements, and the load effects (Pu and Mu) from the superstructure and substructure designs. (Information below provided by Project Structural Engineer.)

$P_u := 310\ kip$,factored applied superstructure vertical dead and live load distributed to each pile.

M_u ,not provided

Current design loads:

- Strength I maximum pile axial load of 310 kips
- Service I maximum pile axial load of 210 kips

Step 2. Determine the magnitude of scour.

The scour report suggests 5 feet of contraction scour could occur below the slope armor/protection. The bottom scour elevation of approximately -6 feet will be used herein (See Figure 31, of Scour Report, p35 of 58)

$Scour_{Elevation} := -6\ ft$

Configuration of Abutment No. 2:

$Top\_of\_Ground := 10.8\ ft$, Top of Ground Elevation Original

$Shelf\_Elevation := 8.0\ ft$, Top of Shelf Elevation

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$TOCap := 10.75\ ft$, Top of Pile Cap Elevation

$Bot\_of\_Conrete\_Jacket := -1\ ft$, Bottom of Concrete Jacket Elevation

$Bot\_of\_Pile\_Cap := 4\ ft$, Bottom of Pile Cap Elevation

$Top\_of\_Pile := 2\ ft + Bot\_of\_Pile\_Cap = 6\ ft$, Top of pile embedded into integral abutment

$Scour_{Elevation} = -6\ ft$, scour elevation

$Top\_of\_Rock_{B\_102} := -10.6\ ft$, Top of rock elevation at Boring B-102, right-most boring

$Top\_of\_Rock_{B\_202} := -12.7\ ft$, Top of rock elevation at Boring B-202, left-most boring

Step 3. Choose preliminary pile size(s):

$$Section := \begin{bmatrix} \boxed{HP10x42} \\ HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix}$$

Step 3.a. Determine the factored applied superstructure vertical dead and live load (Pu) distributed to each pile

$P_u = 310\ kip$

Step 3.b. Select the steel pile strength:

$f_y = 50\ ksi$ As directed by Maine DOT. (per Section 7.2.1 *Structural Steel*, H-piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel)

Step 3.c. Select pile orientation; typically weak axis bending:

As directed by the Structural Engineer and shown on Sheet 21, "Abutment No. 1 Plan and Elevation", the piles are planned to be oriented such that the minor axis is parallel to traffic, while the major axis is perpendicular to traffic. As such, the direction of lateral loads must take this into consideration during later L-pile runs.

Step 3.d. Determine resistance factors for the structural strength for the rock socket at Abutment No. 2:

q_u , unconfined compressive strength of rock core (ksf), Article 10.4.6.4

$q_u := 12115\ psi = 1745\ ksf$

$f'_c := 4500\ psi = 648\ ksf$, assumed strength of cement grout

q_p , nominal unit tip resistance - must limit based on strength of cement grout

$q_p := 2.5 \cdot f'_c = 1620\ ksf$, Equation 10.8.3.5.4c-1 (from LRFD 9)

P_{ntr} , nominal tip resistance for piles in compression on weak rock

$$P_{ntr} := q_p \cdot A_s = \begin{bmatrix} 140 \\ 174 \\ 241 \\ 294 \end{bmatrix} \textit{kip}$$, nominal tip resistance for piles in compression on weak rock

Consider use of skin friction along length of rock socket: (Article 10.8.3.5.4b - Side Resistance)

q_s , nominal unit skin resistance (ksf)

p_a , atmospheric pressure taken as 2.12 ksf

$p_a := 2.12\ \textit{ksf}$, value provided by AASHTO LRFD Article 10.8.3.5.4b

C , regression coefficient taken as 1.0 for normal conditions

$C := 1$, value provided by AASHTO LRFD Article 10.8.3.5.4b

q_u , average unconfined compressive strength of rock core (ksf), Article 10.4.6.4

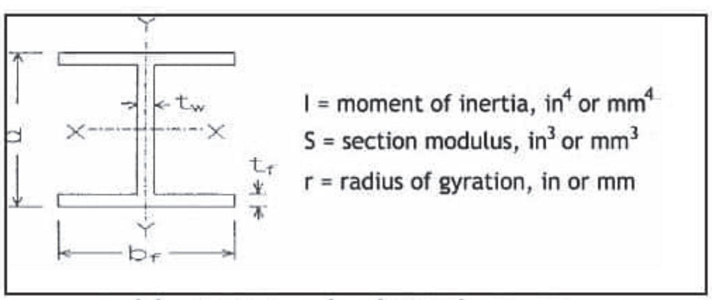
$$q_s := p_a \cdot C \cdot \sqrt[2]{\frac{f'_c}{p_a}}$$, Equation 10.8.3.5.4b-1 (based on Kulhawy et al., 2005)

$q_s = 37.1\ \textit{ksf}$

P_{nsf} , nominal skin friction resistance for piles in rock socket

L_{rs} , length of rock socket (assume 5 foot unless Str. Engr. extends)

r , radius of rock socket (assume 2 inches wider than diagonal width of H-pile to allow for grouting)



$L_{rs} := 5\ \textit{ft}$

$$r := 1\ \textit{in} + \sqrt[2]{(d)^2 + (b_f)^2}$$

$$r = \begin{bmatrix} 15.0 \\ 17.8 \\ 21.0 \\ 21.2 \end{bmatrix} \textit{in}$$

$$P_{nsf} := (2 \cdot \pi \cdot r) \cdot L_{rs} \cdot q_s = \begin{bmatrix} 1456 \\ 1730 \\ 2033 \\ 2053 \end{bmatrix} \textit{kip}$$, nominal skin friction resistance for piles in compression in weak rock socket

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Verify that length of rock socket is at least 1.5B so that Equation 10.8.3.5.4c-1 is valid...

$$Socket\_Check := \left\| \begin{array}{l} \text{for } i \in \textbf{ORIGIN} \,..\, 3 \\ \quad \left\| \begin{array}{l} \text{if } L_{rs} > 1.5 \cdot r_i \\ \quad \left\| \; out_i \leftarrow \text{“"Ok”} \right. \\ \text{else} \\ \quad \left\| \; out_i \leftarrow \text{“Fails Length Check”} \right. \end{array} \right. \\ \text{return } out \end{array} \right. = \begin{bmatrix} \text{“"Ok”} \\ \text{“"Ok”} \\ \text{“"Ok”} \\ \text{“"Ok”} \end{bmatrix}$$

The difference in the deformation required to mobilize skin friction in soil and rock versus what is required to mobilize end bearing shall be considered when estimating axial compressive resistance of "piers" embedded in rock. Where end bearing in rock is used as part of the axial compressive resistance in design, the contribution of skin friction in the rock shall be reduced to account for the loss of skin friction that occurs once the shear deformation along the shaft sides is greater than the peak rock shear deformation, i.e., once the rock shear strength begins to drop to a residual value.

In this case, the available end bearing is much less than skin friction, so we will ignore end bearing and rely solely upon skin friction.

When piles are grouted in-place into pre-drilled rock sockets, they act more like drilled shafts/piers, as such...

- Use Φc = 0.5 for axial resistance along rock socket (lowest value provided in LRFD 10.5.5.2.4-1)

For combined axial and flexural resistance in the upper zone of pile, use:

- Φc = 0.70 for axial resistance
- Φf = 1.00 for flexural resistance

P_{frc} , factored resistance of members in axial compression

$$\phi_c := 0.55$$

$$\phi_f := 1.00$$

$$P_{frc} := P_{nsf} \cdot \phi_c = \begin{bmatrix} 801 \\ 952 \\ 1118 \\ 1129 \end{bmatrix} \textbf{kip}$$, Strength Limit State, member in axial compression

$$P_{frc\_extreme} := P_{nsf} \cdot \phi_f = \begin{bmatrix} 1456 \\ 1730 \\ 2033 \\ 2053 \end{bmatrix} \textbf{kip}$$, geotechnical resistance - Service & Extreme Limit State

Step 3.e. Determine the maximum, required nominal axial pile resistance, Pu/Φf, per Article 6.9.4.1.2

$\phi_{c\_axial} := 0.70$,for axial resistance

$\phi_{c\_flex} := 1.00$,for flexural resistance

P_u ,factored applied superstructure vertical dead and live load distributed to each pile

$P_u = 310\ kip$

R_u ,factored resistance

$R_{u\_axial} := \dfrac{P_u}{\phi_{c\_axial}} = 443\ kip$ $R_{u\_flex} := \dfrac{P_u}{\phi_{c\_flex}} = 310\ kip$

R_{u_max} ,maximum factored resistance

$R_{u\_max} := \max\left(R_{u\_axial}, R_{u\_flex}\right)$

$R_{u\_max} = 443\ kip$

Step 3.f. Estimate an initial pile area using the relationship shown below. This approximation is based on the weak axis bending and an assumed unbraced length of 15 feet.

A_i ,initial pile area using the following approximation:

$A_i := \dfrac{R_{u\_max}}{0.80 \cdot f_y} = 11.071\ in^2$

Area of Section for the Four Common Sizes:

$A_s = \begin{bmatrix} 12.4 \\ 15.5 \\ 21.4 \\ 26.1 \end{bmatrix} in^2$,HP10x42 ,HP12x53 ,HP14x73 ,HP14x89

initial pile size can be HP10x42... however, since End Bent 1 is limited to HP12x53, we will ignore HP10x42 here

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Step 4. Determine the pile unbraced length and maximum moment at the top of the pile by running L-Pile for the design displacement from Step 1, Pu, and live load rotation. Note, the maximum unbraced length is the distance from the top of pile to the point of fixity.

$Top\_of\_Pile = 6\ ft$, Top of pile embedded into integral abutment

$Top\_of\_Rock_{B\_202} := -12.7\ ft$, Top of rock at Boring B-202, left-most boring for End Bent 2

$Point\_of\_Fixity := Top\_of\_Rock_{B\_202} = -12.7\ ft$

$Unbraced\_Length := Top\_of\_Pile - Point\_of\_Fixity = 18.7\ ft$, unbraced length above rock socket

$18.7\ ft = 224.4\ in$, convert to inches for L-pile analysis

$Section := \begin{bmatrix} HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix}$, sections under consideration

Assuming the point-of-fixity is defined by the maximum negative deflection. Determine the depth to fixity below top of pile for each condition modelled.

*** Due to rock socket, point-of-fixity is assumed within the socket and the particular depth is irrelevant. Deflections within the length of pile in the rock socket are insignificant and won't change appreciably with depth. As such, the length of rock sockets are based on factors other than developing a point-of-fixity. Furthermore, with rock sockets of 5 feet or more in rock, we can assume a rigid, fixed position along the entirety of the rock socket with regards to lateral loads.

Step 5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2).

Step 5.a. Calculate slenderness factor for each segment based on unbraced lengths and braced lengths for each scenario...

No need to proceed past this point using assumed behavior because Structural Engineer will model the actual applied moments in both weak and strong axis to determine if plastic deformation at the top of pile meets design criteria.

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Results:

Results of initial trail runs have been communicated to the Structural Engineer. Once final loads a re determined, a more detailed final analysis will be performed.

Conclusions:

None at this time.

Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of t he originator and the checker.

	Print Name	Signature	Date
Prepared By:	Russell L. Thomas, Jr.		3/30/2024
Review By:	Bruce G. Stegman, PE		7/31/2024

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Evaluation of Geotechnical Strength Limit Design - Driven H-piles and WEAP Results

Objectives:

The objective of this calculation package is to evaluate four sizes of steel H-piles for use in the design of an integral abutment bridge per Maine DOT specifications/guidelines regarding Geotechnical Strength Limit State evaluation of WEAP results. In particular, the objective of this calculation package is to evaluate Abutment 1, the western-most abutment, where H-piles are planned to be driven to the top of bedrock.

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Kleinfelder, Inc., "Preliminary Geotechnical Design Report, Replacement of Babson Bridge on Sound Drive in Mount Desert (Bridge No. 5244, WIN 23515.00)", Signed by Massimiliano Rolandi, P.E., Principal Geotechnical Engineer, and Malinda Chea, Staff Professional, dated September 28, 2022.

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Northstar Hydro, "Final Design Hydrology, Hydraulics and Scour Report, Babson Bridge over Meadow Brook, Route 198, Bridge #5244", dated October 28, 2022.

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Givens/Input:

Preliminary Design Information (provided by Mr. Keith Wood, P.E., project Structural Engineer for Kleinfelder)

- The bridge structure is planned to be supported on 50 ksi steel H-piles as part of an integral abutment substructure.
- The piles are to be driven to bedrock at Abutment 1 and pre-drilled with a rock socket at Abutment 2. This calculation package considers the evaluation of Abutment 1. Evaluation of Abutment 2 is under separate

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cover.

- Current design includes a Maximum Strength I Axial Load of 310 kips, and a Service I Axial Load of 210 kips.
- The bridge generally lies on a West to East alignment . As such, End Bent 1 is located on the western side of Kitteredge Brook and End Bent 2 is located on the eastern side of Kitteredge Brook.
- The bridge is planned to have one, 56-foot long span between the two end bents.
- Cross section indicates that the vertical curve (i.e., roadway alignment) along Route 3 is to be raised about 3 1/2 feet at End Bent 1 and about 3 feet at End Bent 2.
- Considering the condition of the existing abutments, existing slope protection, the planned integral abutments, and improved slope protection, the upper materials are protected while medium dense, fine to coarse SANDS will be exposed along the channel bottom. These soils extend to the top of rock. In addition, the scour report suggests 5 feet of contraction scour could occur. The bottom scour elevation of approximately -6 feet will be used herein (See Figure 31 of Scour Report).
- "Calc-001a Friction Angle for Existing Fill" remains valid and may be relied upon. (phi = 32 degrees)
- "Calc-001b Friction Angle for Natural Sand" remains valid and may be relied upon. (phi = 34 degrees)
- "Calc-002a LRFD Pile Design - Strength Limit State Analysis" remains valid and may be relied upon.

Design constraints as directed by Maine DOT include the following:

- Most commonly used pile sizes:

  - HP 10x42
  - HP 12x53
  - HP 14x73
  - HP 14x89

- For *Strength Limit State Analysis*:

  - Design of the piles should consider the factored structural pile resistance, Pr, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

  - Per BDM 5.7.2.1... "The factored geotechnical and drivability resistances should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation, soil setup, lateral spreading and any other site-specific factors, which may affect the pile capacity during and after construction. The factored geotechnical resistance should be determined by applying a resistance, factor which is dependent on the design method."

  - Driveability resistances for a single pile in axial compression when a dynamic test is performed per Article 10.5.5.2.3-1

  - Factor = 0.65 for axial resistance when subject to severe pile driving conditions

- From MaineDOT Standard Specifications, Division 500 - Structures, Section 501 - Foundation Piles:

  - "For the driving system to be acceptable, the number of hammer blows at the required resistance indicated by the wave equation analysis shall be between 3 and 15 blows per inch, and the driving stresses shall not exceed 90% of the specified yield stress of the pile material."

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Assumptions:

- The bridge is to be designed in accordance with AASHTO LRFD Bridge Design Specifications, 9th Edition and Maine DOT's Bridge Design Guide (2003 with updates through 2018).
- Borings were performed per ASTM / Maine DOT standards and guidelines.
- Borings were performed at the locations shown on the provided site plan.
- Standard penetration tests (SPT) reported on soil logs were conducted properly and with in-tolerance, or calibrated, equipment by qualified personnel.
- SPT N-values reported on soil logs are representative of the area(s) of concern and that proper engineering judgment is being applied to the use of measured N-values.
- The equations offered by AASHTO for use in bridge design are accurate and appropriate to apply to this situation.
- The assumptions noted by Maine DOT are a reflection of experience, field testing, and long-term observation of built structures and are also accurate and appropriate to apply to this situation.
- The data gathered during the geotechnical studies by Kleinfelder for the project site are sufficient for determining design parameters and the information they contain is accurate and representative of the project site.
- Published data for the physical properties of steel H-piles by NuCor Skyline are correct:

Pile Properties: Note, per Section 7.2.1 *Structural Steel*, H-piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel.

$f_y := 50\ ksi$, yield strength of steel (LRFD Articles 10.7.8 Drivability Analysis, and 10.9.3.10.2a Cased Length)

$E_s := 29000\ ksi$, modulus of elasticity for steel used in the H-piles planned for this project

$Section := \begin{bmatrix} HP10x42 \\ HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix}$ $Num := \begin{bmatrix} 1 \\ 2 \\ 3 \\ 4 \end{bmatrix}$ $A_s := \begin{bmatrix} 12.4 \\ 15.5 \\ 21.4 \\ 26.1 \end{bmatrix} in^2$, Area of Section

$d := \begin{bmatrix} 9.7 \\ 11.8 \\ 13.6 \\ 13.8 \end{bmatrix} in$, Depth of Section $b_f := \begin{bmatrix} 10.1 \\ 12.0 \\ 14.6 \\ 14.7 \end{bmatrix} in$, Flange Width $t_f := \begin{bmatrix} 0.420 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$, Flange Thickness

$I_x := \begin{bmatrix} 210 \\ 393 \\ 729 \\ 904 \end{bmatrix} in^4$, moment of inertia major axis $I_y := \begin{bmatrix} 71.7 \\ 127 \\ 261 \\ 326 \end{bmatrix} in^4$, moment of inertia minor axis $r_y := \begin{bmatrix} 2.41 \\ 2.86 \\ 3.49 \\ 3.53 \end{bmatrix} in$, Radius of Gyration Y-Y axis

$A_p := \overrightarrow{d \cdot b_f} = \begin{bmatrix} 98 \\ 142 \\ 199 \\ 203 \end{bmatrix} in^2$, Bottom Area for Plugged Scenarios $r_x := \begin{bmatrix} 4.13 \\ 5.03 \\ 5.84 \\ 5.88 \end{bmatrix} in$, Radius of Gyration X-X axis

$P_p := 2 \cdot d + 2 \cdot b_f = \begin{bmatrix} 40 \\ 48 \\ 56 \\ 57 \end{bmatrix} in$, Perimeter for Plugged Scenarios $t_w := \begin{bmatrix} 0.415 \\ 0.435 \\ 0.505 \\ 0.615 \end{bmatrix} in$, Web Thickness

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Results of WEAP Analyses provided in separate cover.

Summary for HP 10x42 sized pile...

Rut ,ultimate capacity (kips)

Mx_CStr ,maximum compression stress (ksi)

Mx_TStr ,maximum tension stress (ksi)

$Blow\_Ct$,blow count (blows/inch of penetration)

$Stroke$,vertical stroke length of hammer (ft)

$Energy$,energy transferred into pile (kip-ft)

$Hammer$,hammer type/designation

Rut (kip)	Mx_CStr (ksi)	Mx_TStr (ksi)	$Blow\_Ct$ $\left(\frac{blows}{ft}\right)$	$Stoke$ (ft)	$Energy$ $(kip \cdot ft)$	$Hammer$	$Acceptable?$
240	42.6	1.6	32	8.6	17.4	D16_32	No
400	44.8	3.8	254	8.4	12.4	D12	No
240	42.9	1.3	29	8.2	18.8	D19_42	No
360	32.3	4.9	9999	9.5	4.8	D5	No
280	42.4	2.6	50	8.9	14.2	D12_32	
280	43.2	3.1	41	9.3	16.9	D12_42	
280	43.5	3.1	40	9.4	17.2	D12_52	
280	43.4	3.1	45	8.8	15.5	D14_42	

Choose typical results for further evaluation... note, the MaineDOT's requirement for blow counts b etween 3 and 15 blows per inch (or 36 and 180 blows per foot).

P_{ndr} ,nominal driving resistance

$P_{ndr_0} := 280\ kip$

Summary for HP 12x53 sized pile...

Rut (kip)	Mx_CStr (ksi)	Mx_TStr (ksi)	$Blow\_Ct$ $\left(\frac{blows}{ft}\right)$	$Stoke$ (ft)	$Energy$ $(kip \cdot ft)$	$Hammer$	$Acceptable?$
320	44.4	1.4	45	9.4	17.6	$D16\_32$	
400	36.2	4.8	207	8.0	11.4	$D12$	No
320	44.8	1.3	42	8.9	19.2	$D19\_42$	
400	28.5	5.2	9999	9.4	4.8	$D5$	No
360	43.4	3.9	72	9.65	14.3	$D12\_32$	
360	44.3	4.3	58	10.0	17.0	$D12\_42$	
360	44.5	4.3	57	10.1	17.2	$D12\_52$	
360	44.5	3.7	64	9.5	15.8	$D14\_42$	

Choose typical results for further evaluation... note, the MaineDOT's requirement for blow counts b etween 3 and 15 blows per inch (or 36 and 180 blows per foot).

$$P_{ndr_1} := 360 \ kip$$

Summary for HP 14x73 sized pile...

Rut (kip)	Mx_CStr (ksi)	Mx_TStr (ksi)	$Blow\_Ct$ $\left(\frac{blows}{ft}\right)$	$Stoke$ (ft)	$Energy$ $(kip \cdot ft)$	$Hammer$	$Acceptable?$
450	44.9	2.8	74	10.2	17.9	$D16\_32$	
						$D12$	
450	44.9	1.9	98	9.6	18.9	$D19\_42$	
						$D5$	
527	44.6	3.7	149	10.8	15.3	$D12\_32$	
508	44.9	3.6	102	11.1	17.6	$D12\_42$	
500	44.9	3.6	98	11.1	17.8	$D12\_52$	
497	44.9	2.7	108	10.5	16.5	$D14\_42$	

Choose typical results for further evaluation... note, the MaineDOT's requirement for blow counts b etween 3 and 15 blows per inch (or 36 and 180 blows per foot).

$$P_{ndr_2} := 500 \ kip$$

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Summary for HP 14x89 sized pile...

Rut (kip)	Mx_CStr (ksi)	Mx_TStr (ksi)	$Blow\_Ct$ $\left(\frac{blows}{ft}\right)$	$Stoke$ (ft)	$Energy$ $(kip \cdot ft)$	$Hammer$	$Acceptable?$
555	44.9	2.5	107	10.8	19.1	D16_32	▯
▯	▯	▯	▯	▯	▯	D12	
544	44.5	2.1	94	10.2	20	D19_42	
▯	▯	▯	▯	▯	▯	D5	
▯	▯	▯	▯	▯	▯	D12_32	
655	44.5	5.6	174	11.4	17.9	D12_42	
655	44.9	5.6	170	11.6	18.2	D12_52	
635	44.7	4.2	176	10.9	17.1	D14_42	

Choose typical results for further evaluation... note, the MaineDOT's requirement for blow counts between 3 and 15 blows per inch (or 36 and 180 blows per foot).

$$P_{ndr_3} := 635\ kip$$

Geotechnical Resistance of Driven Pile (per GRLWEAP and observed soil conditions)

$$P_{ndr} = \begin{bmatrix} 280 \\ 360 \\ 500 \\ 635 \end{bmatrix} kip$$

Per BDM 5.7.2.1... "The factored geotechnical and drivability resistances should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation, soil setup, lateral spreading and any other site-specific factors, which may affect the pile capacity during and after construction. The factored geotechnical resistance should be determined by applying a resistance, factor which is dependent on the design method."

Driveability resistances for a single pile in axial compression when a dynamic test is performed per Article 10.5.5.2.3-1

$\phi_{c\_dyn} := 0.65$,resistance factor for single pile in axial compression when a dynamic test is performed

$P_{fdr\_dyn} := \overrightarrow{\phi_{c\_dyn} \cdot P_{ndr}}$,factored driving resistance when a dynamic test is performed

$\phi_s := 1.0$,resistance factor for single pile in axial compression - service and extreme limit state

$P_{fdr} := \phi_s \cdot \overrightarrow{P_{ndr}}$,factored driving resistance

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Results:

The factored axial pile [geotechnical] resistances for the four pile sections are provided below with respect to Strength Limit State, Service Limit State, and Extreme Limit State designs when a dynamic test is performed.

$$P_{fdr\_dyn}=\begin{bmatrix}182\\234\\325\\413\end{bmatrix}\ kip$$,Strength Limit State for HP 10x42, HP 12x53, HP 14x73, HP 14x89

$$P_{fdr}=\begin{bmatrix}280\\360\\500\\635\end{bmatrix}\ kip$$,Service and Extreme Limit States for HP 10x42, HP 12x53, HP 14x73, HP 14x89

Conclusions:

Results indicate that the piles at Abutment No 1 (West) need to be HP 14x73 or HP 14x89 to achieve the 310-kip factored load when a dynamic test is performed. Each of the four pile sections evaluated meet ting 210-kip service load requirement.

Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of t he originator and the checker.

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	Print Name	Signature	Date
Review By:	Bruce G. Stegman, PE		7/31/2024
	Print Name	Signature	Date

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Lateral Earth Pressure Parameters for Abutment Walls and Wingwalls (Abutment 1)

Objectives:

The objective of this calculation package is to determine the static and seismic Rankine earth pressure coefficients and equivalent fluid pressures for the integral abutment walls and wingwalls at Abutment 1, as part of the Babson Bridge Replacement Project in Mount Desert, Maine.

References:

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E. G. Diaz-Segura, “The Coefficient of Earth Pressure at Rest,” Electronic Journal of Geotechnical Engineering, vol. 21, no. Bundle 05, Art. no. Bund. 05, Jan. 2016.

J. Jaky, “Pressure in Silos,” Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering, vol. 1, pp. 103–107, Jun. 1948.

J. Jaky, “The Coefficient of Earth Pressure at Rest,” Journal Society Hungarian Architecture and Engineering, vol. 78, pp. 355–388, 1944.

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R. Michalowski, “Coefficient of Earth Pressure at Rest,” Journal of Geotechnical and Geoevnironmental Engineering, vol. 131, no. 11, pp. 1429–1433, Nov. 2005.

NAVFAC, “Foundations & Earth Structures Design Manual 7.2.” Department of the Navy Naval Facilities Engineering Systems Command, Sep. 1986.

W. J. M. Rankine, “On the Stability of Loose Earth,” Philosophical Transactions of the Royal Society of London, vol. 147, pp. 9–27, Jun. 1856.

Givens/Input:

Choose an effective unit weight of soil and an effective internal angle of friction to model. The proposed backfill input is based on Soil Type IV (Granular underwater backfill) per Maine DOT BDG Section 3.6.1 - we assume that the zone of soil contained within the the bearing zone of the abutment will be excavated and replaced with granular borrow (MaineDOT Specification 703.19 “Granular Borrow for Underwater Backfill"). Design groundwater elevation is assumed to be at EL +10 feet. The in-situ soils are based on our interpretation of boring logs BB-MDMB-101 and BB-MDMB-201.

Chose design values for *total unit weight* for backfill materials...

$$\gamma := \begin{bmatrix} 125 \\ 115 \\ 105 \\ 125 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

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Chose design values for *effective internal angle of fricion* for backfill materials...

$$\phi' := \begin{bmatrix} 32 \\ 28 \\ 26 \\ 32 \end{bmatrix} deg$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

Chose design values for *interface fricion anlge* between concrete and backfill materials... (see Table 3-3, Interface Friction Angle, Concrete to Soil for various soil types.)

$$\delta := \begin{bmatrix} 24 \\ 21 \\ 19 \\ 24 \end{bmatrix} deg$$

,granular underwater backfill, proposed - assumed soil type 3
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

Chose design values for *effective cohesion* for backfill materials...

$$c' := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

Chose design values for *angle of backfill from horizontal* assuming flat, or horizontal backfill surface...

$$\beta := \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{bmatrix} deg$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

Assumptions:

$\gamma_w := 62.43\ pcf$,unit weight of water (ignore changes in temperature and/or atmospheric conditions)

per Rankine...

"The resistance to displacement by sliding along a given plane in a loose granular mass, is equal to the normal pressure exerted between the parts of the mass on either side of that plane, multiplied by a specific constant (or coefficient of friction)."

"The forces which balance each other in or upon a given body or structure being distinguished into two systems, called respectively *active* and *passive*, which stand to each other in the relation of cause and effect, then will the passive forces be the least which are capable of balancing the active forces, consistently with the physical condition of the body or structure."

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That the chosen soil properties are sufficiently representative of the soil type(s) in question.

Per MaineDOT Bridge Design Guide (BDG):

Lateral Earth Pressure Considerations

- Per Section 3.6.4, for walls with a total wall height, H, greater than or equal to 5 feet, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

- Per Section 3.6.4, at-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g. rigid frame bridges) or to cantilever walls where the heel is restrained and the base/stem connection prevents rotation of the stem, or for buried structures.

- Per Section 3.6.5.2, the Rankine theory is recommended to be used for the design of any yielding walls of the following types: gravity shaped walls and integral abutments, semi gravity walls, prefabricated modular walls with steep back faces (20 deg or less measured from vertical), and cantilever walls and abutments with short heels.

- Per Section 3.6.5.2, interface friction between the wall backface and the backfill is not considered in Rankine's theory. The resultant lateral earth load due to the weight of the backfill should be assumed to act at a height of H/3 above the base of the wall, where H is the total wall height, measured along a vertical plane extending from the ground surface above the back of the footing down to the bottom of the footing.

- Per Section 3.6.9, the resistance due to passive earth pressure in front of walls should be neglected unless the wall extends well below the depth of frost penetration, scour, or other types of potential disturbance, such as utility trench excavation in front of the wall. Neglecting this passive earth pressure is due to the consideration that the soil may be removed during future construction, which will eliminate its contribution to wall stability.

- Per Section 5.4.2.11, integral abutments must rotate to develop full passive earth pressure reistance, i.e., the ratio of lateral abutment movement to abutment height (y/H), exceeds 0.005.

Drainage and Hydrostatic Considerations

- Per Section 3.6.2, retained earth should be drained and the development of hydrostatic water pressure eliminated by the use of a free-draining backfill such as crushed rock (less than 5 percent passing a No. 200 sieve), gravel drains, or other drainage systems.

- Per Section 3.6.2, if retained earth is not allowed to drain, or if the groundwater levels differ on opposites sides of the wall, the effect of hydrostatic water pressure should be added to the earth pressure. Pore water pressures should be added to the effective horizontal stresses in determining total lateral earth pressure on the wall.

- Per Section 3.6.2, abutment walls/wingwalls should be designed for a minimum differential water pressure due to a 3 foot head of water in the backfill soil above the weepholes.

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- Per Section 5.4.2.13, drainage behind integral abutments shall be backfilled with granular borrow for underwater backfill. A proper draininge system as described in MaineDOT BDG, Section 5.4.1.9 should be provided to eliminate hydrostatic pressure and control erosion of the underside of the abutment embankment slope protection.

Live Load Surcharge Considerations

- Per Section 3.6.8, a live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height, H, behind the back of the wall. H, is defined as the total wall height measured along a vertical plane extending from the bottom of the footing up to the ground surface at the back of the wall.

- Per Section 3.6.8, the additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by a height of soil, Heq, defined in MaineDOT BDG Table 3-4.

Seismic Considerations

Per Section 3.6.3, where applicable, the effects of wall inertia and amplification of active earth pressure by earthquake should be considered. The Mononobe-Okabe method should be used to determine equivalent static pressures for seismic loads on walls and abutments as described in Section 3.7.3 Substructure. If the soils are saturated, liquefaction should be evaluated and addressed per Section 3.7.4.2. (This calculation to be under separate cover, if requested by the Structural Engineer.)

Functions:

Submerged or boyant unit weight relationship...

$$\gamma' := \gamma - \gamma_w$$

Earth Pressure Coefficients when B=0:

At-Rest:

$$K_o(\phi') := 1 - \sin(\phi')$$

Active:

$$K_a(\phi') := \frac{1 - \sin(\phi')}{1 + \sin(\phi')}$$

Passive:

$$K_p(\phi') := \frac{1 + \sin(\phi')}{1 - \sin(\phi')}$$

Equivalent Fluid Pressures:

At-Rest:

$$EFP_o(\gamma', \phi') := \gamma' \cdot (1 - \sin(\phi'))$$

Active:

$$EFP_a(\gamma', \phi') := \gamma' \cdot \left(\frac{1 - \sin(\phi')}{1 + \sin(\phi')}\right)$$

Passive:

$$EFP_p(\gamma', \phi') := \gamma' \cdot \left(\frac{1 + \sin(\phi')}{1 - \sin(\phi')}\right)$$

$$Ultimate\_Friction\_Coef := \tan(\delta)$$

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Results:

$$\gamma' := \gamma - \gamma_w = \begin{bmatrix} 62.57 \\ 52.57 \\ 42.57 \\ 62.57 \end{bmatrix} \mathit{pcf}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

$$K_o(\phi') = \begin{bmatrix} 0.47 \\ 0.53 \\ 0.56 \\ 0.47 \end{bmatrix} \qquad \overrightarrow{EFP_o(\gamma', \phi')} = \begin{bmatrix} 29 \\ 28 \\ 24 \\ 29 \end{bmatrix} \mathit{pcf}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

$$K_a(\phi') = \begin{bmatrix} 0.31 \\ 0.36 \\ 0.39 \\ 0.31 \end{bmatrix} \qquad \overrightarrow{EFP_a(\gamma', \phi')} = \begin{bmatrix} 19 \\ 19 \\ 17 \\ 19 \end{bmatrix} \mathit{pcf}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

$$K_p(\phi') = \begin{bmatrix} 3.25 \\ 2.77 \\ 2.56 \\ 3.25 \end{bmatrix} \qquad \overrightarrow{EFP_p(\gamma', \phi')} = \begin{bmatrix} 204 \\ 146 \\ 109 \\ 204 \end{bmatrix} \mathit{pcf}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

$$Ultimate\_Friction\_Coef = \begin{bmatrix} 0.45 \\ 0.38 \\ 0.34 \\ 0.45 \end{bmatrix}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-201
,buried organic-laden soil (soil type 1), BB-201, BB-101
,glacial till (soil type 4), BB-101 & 201

Conclusions:

The values shown should be reflected in the report text.

Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of the originator and the checker.

	Print Name	Signature	Date
Prepared By:	Shradha Panda	Shradha Panda	05/31/2024
Review By:	Russell L. Thomas, Jr.	[signature]	7/17/2024

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Lateral Earth Pressure Parameters for Abutment Walls and Wingwalls (Abutment 2)

Objectives:

The objective of this calculation package is to determine the static and seismic Rankine earth pressure coefficients and equivalent fluid pressures for the integral abutment walls and wingwalls at Abutment 2, as part of the Babson Bridge Replacement Project in Mount Desert, Maine.

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R. Michalowski, “Coefficient of Earth Pressure at Rest,” Journal of Geotechnical and Geoevnironmental Engineering, vol. 131, no. 11, pp. 1429–1433, Nov. 2005.

NAVFAC, “Foundations & Earth Structures Design Manual 7.2.” Department of the Navy Naval Facilities Engineering Systems Command, Sep. 1986.

W. J. M. Rankine, “On the Stability of Loose Earth,” Philosophical Transactions of the Royal Society of London, vol. 147, pp. 9–27, Jun. 1856.

Givens/Input:

Choose an effective unit weight of soil and an effective internal angle of friction to model. The proposed backfill input is based on Soil Type IV (Granular underwater backfill) per Maine DOT BDG Section 3.6.1 - we assume that the zone of soil contained within the the bearing zone of the abutment will be excavated and replaced with granular borrow (MaineDOT Specification 703.19 “Granular Borrow for Underwater Backfill"). Design groundwater elevation is assumed to be at EL +10 feet. The in-situ soils are based on our interpretation of boring logs BB-MDMB-102 and BB-MDMB-202.

Chose design values for *total unit weight* for backfill materials...

$$\gamma := \begin{bmatrix} 125 \\ 115 \\ 125 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

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Chose design values for *effective internal angle of fricion* for backfill materials...

$$\phi' := \begin{bmatrix} 32 \\ 28 \\ 32 \end{bmatrix} deg$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

Chose design values for *interface fricion anlge* between concrete and backfill materials... (see Table 3-3, Interface Friction Angle, Concrete to Soil for various soil types.)

$$\delta := \begin{bmatrix} 24 \\ 21 \\ 24 \end{bmatrix} deg$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

Chose design values for *effective cohesion* for backfill materials...

$$c' := \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

Chose design values for *angle of backfill from horizontal* assuming flat, or horizontal backfill surface...

$$\beta := \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} deg$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

Assumptions:

$\gamma_w := 62.43\ pcf$,unit weight of water (ignore changes in temperature and/or atmospheric conditions)

per Rankine...

"The resistance to displacement by sliding along a given plane in a loose granular mass, is equal to the normal pressure exerted between the parts of the mass on either side of that plane, multiplied by a specific constant (or coefficient of friction)."

"The forces which balance each other in or upon a given body or structure being distinguished into two systems, called respectively *active* and *passive*, which stand to each other in the relation of cause and effect, then will the passive forces be the least which are capable of balancing the active forces, consistently with the physical condition of the body or structure."

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That the chosen soil properties are sufficiently representative of the soil type(s) in question.

Per MaineDOT Bridge Design Guide (BDG):

Lateral Earth Pressure Considerations

- Per Section 3.6.4, for walls with a total wall height, H, greater than or equal to 5 feet, the horizontal movement of the top of the wall due to structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

- Per Section 3.6.4, at-rest earth pressures are usually limited to bridge abutments to which superstructures are fixed prior to backfilling (e.g. rigid frame bridges) or to cantilever walls where the heel is restrained and the base/stem connection prevents rotation of the stem, or for buried structures.

- Per Section 3.6.5.2, the Rankine theory is recommended to be used for the design of any yielding walls of the following types: gravity shaped walls and integral abutments, semi gravity walls, prefabricated modular walls with steep back faces (20 deg or less measured from vertical), and cantilever walls and abutments with short heels.

- Per Section 3.6.5.2, interface friction between the wall backface and the backfill is not considered in Rankine's theory. The resultant lateral earth load due to the weight of the backfill should be assumed to act at a height of H/3 above the base of the wall, where H is the total wall height, measured along a vertical plane extending from the ground surface above the back of the footing down to the bottom of the footing.

- Per Section 3.6.9, the resistance due to passive earth pressure in front of walls should be neglected unless the wall extends well below the depth of frost penetration, scour, or other types of potential disturbance, such as utility trench excavation in front of the wall. Neglecting this passive earth pressure is due to the consideration that the soil may be removed during future construction, which will eliminate its contribution to wall stability.

- Per Section 5.4.2.11, integral abutments must rotate to develop full passive earth pressure reistance, i.e., the ratio of lateral abutment movement to abutment height (y/H), exceeds 0.005.

Drainage and Hydrostatic Considerations

- Per Section 3.6.2, retained earth should be drained and the development of hydrostatic water pressure eliminated by the use of a free-draining backfill such as crushed rock (less than 5 percent passing a No. 200 sieve), gravel drains, or other drainage systems.

- Per Section 3.6.2, if retained earth is not allowed to drain, or if the groundwater levels differ on opposites sides of the wall, the effect of hydrostatic water pressure should be added to the earth pressure. Pore water pressures should be added to the effective horizontal stresses in determining total lateral earth pressure on the wall.

- Per Section 3.6.2, abutment walls/wingwalls should be designed for a minimum differential water pressure due to a 3 foot head of water in the backfill soil above the weepholes.

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- Per Section 5.4.2.13, drainage behind integral abutments shall be backfilled with granular borrow for underwater backfill. A proper draininge system as described in MaineDOT BDG, Section 5.4.1.9 should be provided to eliminate hydrostatic pressure and control erosion of the underside of the abutment embankment slope protection.

Live Load Surcharge Considerations

- Per Section 3.6.8, a live load surcharge should be applied when traffic loads are located within a horizontal distance equal to one-half of the wall height, H, behind the back of the wall. H, is defined as the total wall height measured along a vertical plane extending from the bottom of the footing up to the ground surface at the back of the wall.

- Per Section 3.6.8, the additional lateral earth pressure due to live load should be modeled by a surcharge load equal to that applied by a height of soil, Heq, defined in MaineDOT BDG Table 3-4.

Seismic Considerations

Per Section 3.6.3, where applicable, the effects of wall inertia and amplification of active earth pressure by earthquake should be considered. The Mononobe-Okabe method should be used to determine equivalent static pressures for seismic loads on walls and abutments as described in Section 3.7.3 Substructure. If the soils are saturated, liquefaction should be evaluated and addressed per Section 3.7.4.2. (This calculation to be under separate cover, if requested by the Structural Engineer.)

Functions:

Submerged or boyant unit weight relationship...

$$\gamma' := \gamma - \gamma_w$$

Earth Pressure Coefficients when B=0:

At-Rest:

$$K_o(\phi') := 1 - \sin(\phi')$$

Active:

$$K_a(\phi') := \frac{1 - \sin(\phi')}{1 + \sin(\phi')}$$

Passive:

$$K_p(\phi') := \frac{1 + \sin(\phi')}{1 - \sin(\phi')}$$

Equivalent Fluid Pressures:

At-Rest:

$$EFP_o(\gamma', \phi') := \gamma' \cdot (1 - \sin(\phi'))$$

Active:

$$EFP_a(\gamma', \phi') := \gamma' \cdot \left(\frac{1 - \sin(\phi')}{1 + \sin(\phi')}\right)$$

Passive:

$$EFP_p(\gamma', \phi') := \gamma' \cdot \left(\frac{1 + \sin(\phi')}{1 - \sin(\phi')}\right)$$

$$Ultimate\_Friction\_Coef := \tan(\delta)$$

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Results:

$$\gamma' := \gamma - \gamma_w = \begin{bmatrix} 62.57 \\ 52.57 \\ 62.57 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

$$K_o(\phi') = \begin{bmatrix} 0.47 \\ 0.53 \\ 0.47 \end{bmatrix} \qquad \overrightarrow{EFP_o(\gamma', \phi')} = \begin{bmatrix} 29 \\ 28 \\ 29 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

$$K_a(\phi') = \begin{bmatrix} 0.31 \\ 0.36 \\ 0.31 \end{bmatrix} \qquad \overrightarrow{EFP_a(\gamma', \phi')} = \begin{bmatrix} 19 \\ 19 \\ 19 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

$$K_p(\phi') = \begin{bmatrix} 3.25 \\ 2.77 \\ 3.25 \end{bmatrix} \qquad \overrightarrow{EFP_p(\gamma', \phi')} = \begin{bmatrix} 204 \\ 146 \\ 204 \end{bmatrix} pcf$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

$$Ultimate\_Friction\_Coef = \begin{bmatrix} 0.45 \\ 0.38 \\ 0.45 \end{bmatrix}$$

,granular underwater backfill, proposed
,existing fill (b/w soil types 1 & 2), BB-102
,glacial till (soil type 4), BB-102 & 202

Conclusions:

The values shown should be reflected in the report text.

Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of t he originator and the checker.

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	Print Name	Signature	Date
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Settlement Analysis and Service Limit State Evaluation for Driven Piles Design per MaineDOT BDG-2003, AASHTO LRFD-9, and FHWA

Objectives:

The objective of this calculation package is to 1) evaluate the settlement potential at the approach embankments due to new vertical curve (raising grade about 3 feet), and 2) evaluate the settlement potential for driven piles at both abutments due to applied service limit load of 210 kips.

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Samtani, Naresh C., and Edward A. Nowatzki. 2006b. “Soils and Foundations Reference Manual - Volume II.” NHI Course No. 132012 FHWA-NHI-06-089. Washington, D.C. 20590: National Highway Institute, U.S. Department of Transportation, Federal Highway Administration.

US Army Corps of Engineers, Engineering and Design. 1993. Design of Pile Foundations. Technical Engineering and Design Guides as Adapted from the US Army Corps of Engineers, No. 1. ASCE Press.

Givens/Input:

Based on Sheet 3, Profile of the preliminary design documents, we understand both approaches are planned to be raised about 3 feet.

h_f , height of new fill (feet) $h_f := 3\ ft$

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γ_f , unit weight of new fill

$\gamma_f := 125\ pcf$

Based on soil borings performed at the project site... design material properties include the following:

Location	Soil_Type	USCS	Avg_N1_{60}	Bearing_Index	Dry_Unit_Wt (pcf)	Layer_Thickness (ft)
Abutment_1	Existing_Fill	SM	87	388	131	4.3
Abutment_1	Existing_Fill	SM	14	63	115	5
Abutment_1	Orgaic_Mtl	SM	44	146	103	5
Abutment_1	Glacial_Till	SM	61	220	113	13.7
Abutment_1	Granite		100	505	137	
Abutment_2	Existing_Fill	SM	35	116	101	4.4
Abutment_2	Existing_Fill	SM	22	81	116	5
Abutment_2	Existing_Fill	SM	52	178	105	5
Abutment_2	Glacial_Till	SM	43	142	115	6.5
Abutment_2	Granite		100	505	137	

H , height of layer

H_{1A} , height of layer; 1 = Abutment No.; A = layer i.d. from top down.

$H_{1A} := 4.3\ ft$ $\gamma_{1A} := 131\ pcf$ $C_{1A} := 388$ $H_{2A} := 4.4\ ft$ $\gamma_{2A} := 101\ pcf$ $C_{2A} := 116$

$H_{1B} := 5\ ft$ $\gamma_{1B} := 115\ pcf$ $C_{1B} := 63$ $H_{2B} := 5\ ft$ $\gamma_{2B} := 116\ pcf$ $C_{2B} := 81$

$H_{1C} := 5\ ft$ $\gamma_{1C} := 103\ pcf$ $C_{1C} := 146$ $H_{2C} := 5\ ft$ $\gamma_{2C} := 105\ pcf$ $C_{2C} := 178$

$H_{1D} := 13.7\ ft$ $\gamma_{1D} := 113\ pcf$ $C_{1D} := 220$ $H_{2D} := 6.5\ ft$ $\gamma_{2D} := 115\ pcf$ $C_{2D} := 142$

Pile Properties: Note, per Section 7.2.1 *Structural Steel*, H-piles used for bridge foundations should be composed of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 ksi.

$E_s := 29000\ ksi$, modulus of elasticity for steel used in the H-piles planned for this project

$P := 210\ kip$

$$Section := \begin{bmatrix} HP10x42 \\ HP12x53 \\ HP14x73 \\ HP14x89 \end{bmatrix} \qquad A_s := \begin{bmatrix} 12.4 \\ 15.5 \\ 21.4 \\ 26.1 \end{bmatrix} in^2$$

$$L_1 := 4.3\ ft + 5\ ft + 5\ ft + 13.7\ ft = 28\ ft$$

$$L_2 := 4.4\ ft + 5\ ft + 5\ ft + 6.5\ ft + 5\ ft = 25.9\ ft$$

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, Area of Section

Assumptions:

- Application of the modified Hough Method for computing immediate settlements of embankments is appropriate for these conditions.
- Borings performed for the Babson Bridge project are representative of the controlling site conditio ns.
- SPT N-values obtained during drilling were performed accurately and with calibrated equipment.

$\gamma_{water} := 62.34\ pcf$

Functions:

δ , deflection

$$\delta_1 := \frac{P \cdot L_1}{A_s \cdot E_s} = \begin{bmatrix} 0.196 \\ 0.157 \\ 0.114 \\ 0.093 \end{bmatrix} in \qquad \delta_2 := \frac{P \cdot L_2}{A_s \cdot E_s} = \begin{bmatrix} 0.182 \\ 0.145 \\ 0.105 \\ 0.086 \end{bmatrix} in$$

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Analyses/Calculations:

Change in stress due to new fill:

$\Delta\sigma_f$, change in stress due to new fill

$\Delta\sigma_f := h_f \cdot \gamma_f$ $\quad \Delta\sigma_f = 375\ psf$

Determine stress at midpoint for each layer of soil assuming groundwater at top of original ground. . .

$$\sigma_{1A} := \frac{H_{1A}}{2} \cdot (\gamma_{1A} - \gamma_{water}) = 147.619\ psf \qquad \sigma_{1A} := 200\ psf$$

$$\sigma_{1B} := \frac{H_{1B}}{2} \cdot (\gamma_{1B} - \gamma_{water}) + H_{1A} \cdot (\gamma_{1A} - \gamma_{water})$$

$$\sigma_{1C} := \frac{H_{1C}}{2} \cdot (\gamma_{1C} - \gamma_{water}) + H_{1B} \cdot (\gamma_{1B} - \gamma_{water}) + H_{1A} \cdot (\gamma_{1A} - \gamma_{water})$$

$$\sigma_{1D} := \frac{H_{1D}}{2} \cdot (\gamma_{1D} - \gamma_{water}) + H_{1C} \cdot (\gamma_{1C} - \gamma_{water}) + H_{1B} \cdot (\gamma_{1B} - \gamma_{water}) + H_{1A} \cdot (\gamma_{1A} - \gamma_{water})$$

$$\sigma_{2A} := \frac{H_{2A}}{2} \cdot (\gamma_{2A} - \gamma_{water}) = 85.052\ psf \qquad \sigma_{2A} := 200\ psf$$

$$\sigma_{2B} := \frac{H_{2B}}{2} \cdot (\gamma_{2B} - \gamma_{water}) + H_{1A} \cdot (\gamma_{2A} - \gamma_{water})$$

$$\sigma_{2C} := \frac{H_{2C}}{2} \cdot (\gamma_{2C} - \gamma_{water}) + H_{2B} \cdot (\gamma_{2B} - \gamma_{water}) + H_{2A} \cdot (\gamma_{2A} - \gamma_{water})$$

$$\sigma_{2D} := \frac{H_{2D}}{2} \cdot (\gamma_{2D} - \gamma_{water}) + H_{2C} \cdot (\gamma_{2C} - \gamma_{water}) + H_{2B} \cdot (\gamma_{2B} - \gamma_{water}) + H_{2A} \cdot (\gamma_{2A} - \gamma_{water})$$

Determine settlement at midpoint for each layer...

$$\Delta H_{1A} := \frac{H_{1A}}{C_{1A}} \cdot \log\left(\frac{\sigma_{1A} + \Delta\sigma_f}{\sigma_{1A}}\right) \qquad \Delta H_{2A} := \frac{H_{2A}}{C_{2A}} \cdot \log\left(\frac{\sigma_{2A} + \Delta\sigma_f}{\sigma_{2A}}\right)$$

$$\Delta H_{1B} := \frac{H_{1B}}{C_{1B}} \cdot \log\left(\frac{\sigma_{1B} + \Delta\sigma_f}{\sigma_{1B}}\right) \qquad \Delta H_{2B} := \frac{H_{2B}}{C_{2B}} \cdot \log\left(\frac{\sigma_{2B} + \Delta\sigma_f}{\sigma_{2B}}\right)$$

$$\Delta H_{1C} := \frac{H_{1C}}{C_{1C}} \cdot \log\left(\frac{\sigma_{1C} + \Delta\sigma_f}{\sigma_{1C}}\right) \qquad \Delta H_{2C} := \frac{H_{2C}}{C_{2C}} \cdot \log\left(\frac{\sigma_{2C} + \Delta\sigma_f}{\sigma_{2C}}\right)$$

$$\Delta H_{1D} := \frac{H_{1D}}{C_{1D}} \cdot \log\left(\frac{\sigma_{1D} + \Delta\sigma_f}{\sigma_{1D}}\right) \qquad \Delta H_{2D} := \frac{H_{2D}}{C_{2D}} \cdot \log\left(\frac{\sigma_{2D} + \Delta\sigma_f}{\sigma_{2D}}\right)$$

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Results:

$\Delta\sigma_f$, change in stress due to new fill

$\Delta\sigma_f = 375\ psf$

σ_{ij} , stress at middle of layer before adding fill

$\sigma_{1A} = 200\ psf$ $\sigma_{2A} = 200\ psf$

$\sigma_{1B} = 426.9\ psf$ $\sigma_{2B} = 300.4\ psf$

$\sigma_{1C} = 660.2\ psf$ $\sigma_{2C} = 545.1\ psf$

$\sigma_{1D} = 1108.9\ psf$ $\sigma_{2D} = 822.8\ psf$

ΔH_{ij} , settlement at middle of layer due to adding fill

$\Delta H_{1A} = 0.061\ in$ $\Delta H_{2A} = 0.209\ in$

$\Delta H_{1B} = 0.261\ in$ $\Delta H_{2B} = 0.261\ in$

$\Delta H_{1C} = 0.08\ in$ $\Delta H_{2C} = 0.077\ in$

$\Delta H_{1D} = 0.095\ in$ $\Delta H_{2D} = 0.09\ in$

Determine total settlement anticipated due to new fill placement...

$$\Delta H_{1\_total} := \Delta H_{1A} + \Delta H_{1B} + \Delta H_{1C} + \Delta H_{1D} = 0.50\ in$$

$$\Delta H_{2\_total} := \Delta H_{2A} + \Delta H_{2B} + \Delta H_{2C} + \Delta H_{2D} = 0.64\ in$$

Conclusions:

The above analysis suggests that both approach embankments will experience about 1/2 inch of immediate settlement due to placement of new fill required to raise finished grade. Most likely, the majority of this settlement will occur at the time of construction and upon completion of fill placement, the 1/2 inch of settlement would most likely have already been made up. The design team should not expect any appreciable long-term settlement at either approach.

Furthermore, piles at each abutment will be supported by the underlying bedrock. As such, settlement will be limited to elastic deformation of the piles themselves rather than load-induced settlements. The values determined herein may be considered negligible.

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Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of the originator and the checker.

	Print Name	Signature	Date
Prepared By:	Russell L. Thomas, Jr.		7/05/2024
Review By:	Bruce G. Stegman, PE		7/31/2024

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Evaluation of Frost Penetration Depth per MaineDOT BDG-2003

Objectives:

The objective of this calculation package is to determine the frost penetration depth at the project site.

References:

Guertin Elkerton & Associates. 2003. “Bridge Design Guide (with Updates Through 2018).” Maine Department of Transportation, Bridge Program.

Kleinfelder, Inc., "Preliminary Geotechnical Design Report, Replacement of Babson Bridge on Sound Drive in Mount Desert (Bridge No. 5244, WIN 23515.00)", Signed by Massimiliano Rolandi, P.E., Principal Geotechnical Engineer, and Malinda Chea, Staff Professional, dated September 28, 2022.

Kleinfelder, Inc., "Mount Desert Hancock County Babson Bridge Over Kitteredge Brook Route 3/198 Project No. 23515.00 Project Length 0.11 mi. Bridge No. 5244", Prepared for State of Maine Department of Transportation, Designed by Keith Wood, PE, December 2022.

Givens/Input:

The project is located on Mount Desert Island, Maine. The primary near-surface soil type is SAND

From the Bridge Design Guide (Section 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.

Assumptions:

- Application of the modified Hough Method for computing immediate settlements of embankments is appropriate for these conditions.
- Borings performed for the Babson Bridge project are representative of the controlling site conditions.
- SPT N-values obtained during drilling were performed accurately and with calibrated equipment.
- Coarse grained soil types - Silty Fine to Medium SAND and boulders used to model frost penetration with respect to Figure 5-1 and Table 5-1.

Analyses/Calculations:

Step 1: Determine the design freeze index from Figure 5-1

$$DFI := 1050$$

Step 2: Determine average moisture content of upper soils. No lab results available. Assumed value from Table 5-1.

$$w := 0.10 \qquad w = 10\%$$

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Step 3: Interpolate between known values in Table 5-1.

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

DFI ,design freeze index

f_d ,depth of frost penetration

$DFI_{d1} := 1000$ $\quad f_{d1} := 66.3\ in$

$DFI_{d2} := 1100$ $\quad f_{d2} := 69.8\ in$

$$f_d := \left(\frac{f_{d2} - f_{d1}}{DFI_{d2} - DFI_{d1}}\right) \cdot \left(DFI - DFI_{d1}\right) + f_{d1}$$

Results:

$f_d = 68.05\ in$

Conclusions:

The above analysis is consistent with other published sources and may be relied upon with a full understanding of the assumptions listed herein.

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Closure:

The calculation set, however prepared, shall be wet signed or contain the electronic signature of the originator and the checker.

	Print Name	Signature	Date
Prepared By:	Shradha Panda	Shradha Panda	05/31/2024
Review By:	Russell L. Thomas, Jr.		7/17/2024

APPENDIX D

GBA INSERT

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will <u>not</u> likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will <u>not</u> be adequate to develop geotechnical design recommendations for the project.

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do <u>not</u> rely on an executive summary. Do <u>not</u> read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are <u>not</u> final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are <u>Not</u> Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will <u>not</u> of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are <u>not</u> building-envelope or mold specialists.***

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