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GEOTECHNICAL DESIGN REPORT

THOMPSONS BRIDGE NO. 2848 OVER DECKER COVE

MAINE DOT WIN 18748.00

SOUTHPORT, MAINE

Prepared for:
Stantec
Scarborough, Maine

February 2019
09.0025910.01

Prepared by:
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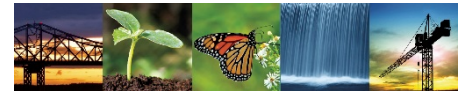
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VIA EMAIL

February 27, 2019
File No. 09.0025910.01

Mr. Daniel Taylor, P.E.
Stantec
482 Payne Road Scarborough Court
Scarborough, Maine 04074

Re: Geotechnical Design Report
Replacement of Thompsons Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00
Southport, Maine

Dear Dan:

We are pleased to provide this Geotechnical Design Report (GDR) to Stantec for the replacement of Bridge No. 2848 over Decker Cove in Southport, Maine. Our services were provided in accordance with Master Services Agreement 40839 between Stantec and GZA, and Stantec Task Order JN 195311466, dated November 8, 2018, which incorporates GZA's proposal No. 09.P000040.18, dated November 1, 2018, and the attached *Limitations* included in **Appendix A**. GZA is providing geotechnical engineering services as a Subconsultant to Stantec, who is under contract with Maine Department of Transportation for design of the proposed bridge replacement.

It has been a pleasure serving Stantec on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Nicholas Williams, P.E.
Assistant Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer



Andrew R. Blaisdell, P.E.
Associate Principal

NVW/ARB/CLS:erc

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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Thompsons Bridge No. 2848 over Decker Cove in Southport, Maine. Our work was completed in accordance with Master Services Agreement 40839 between Stantec and GZA, and Stantec Task Order JN 195311466, dated November 8, 2018, which incorporates GZA's proposal No. 09.P000040.18, dated November 1, 2018, and the attached *Limitations* included in **Appendix A** of this report.

GZA is providing geotechnical engineering services as a Subconsultant to Stantec, who is under contract with MaineDOT for design of the proposed bridge replacement.

GZA prepared a Preliminary Geotechnical Design Basis memorandum dated October 19, 2018. The evaluations and recommendations presented herein supersede the recommendations presented in our October 19, 2018 memorandum.

1.1 BACKGROUND

The existing Thompson Bridge No. 2848 carries Route 238 over Decker Cove (Atlantic Ocean) in Southport, at the location shown on **Figure 1**. The existing bridge was constructed in the 1930s and consists of a 192'-6" non-continuous four-span bridge with a steel superstructure and open steel grid deck. The bridge is supported on timber pile pier bents. The abutments consist of timber piles with wood lagging backwalls in a spill-through configuration. The abutment piles are reportedly doweled into rock. We understand the bridge needs replacement due primarily to the superstructure's poor condition with significant section loss and also due to the deteriorating condition of the timber substructure units.

We also understand that a replacement bridge is planned to be constructed on the existing alignment, incorporating a full closure and detour. The replacement bridge is proposed to be 192 feet long, with two, 96-foot spans, as shown on **Figure 2**. The superstructure will be comprised of four precast concrete bulb tee girders with concrete closure pours between the girders and pavement over the concrete. The replacement bridge is planned to be supported by semi-integral abutments with rigid end diaphragms and a center pile-supported bent pier. The new pier is planned to be located immediately north of the existing center pier.

Each abutment will be supported by spread footings bearing on tremie seals over bedrock. The center pier will be supported by driven H-piles installed through fiber-reinforced polymer (FRP) pipes embedded below the mudline, which will be filled with concrete after pile installation. Based on information provided by Stantec, we understand the maximum factored axial pile load is 600 kips for the strength loading condition and the 4-pile configuration.

The road will be widened by about 6 feet, with all of the widening occurring to the west (left). Less than 1 foot of new fill is planned over the existing approach embankment to facilitate a vertical curve on the replacement bridge. Up to 6 feet of new fill placement will be required under the left shoulder of the widened approaches.



1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and provide final geotechnical design recommendations and construction considerations for bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions, traffic and boring access;
- Coordinated and observed preliminary and final subsurface exploration programs, consisting of a total of nine test borings, to evaluate subsurface conditions;
- Conducted a laboratory testing program to evaluate classification and engineering properties of the site soil and bedrock;
- Completed geotechnical evaluations for soil and bedrock properties; approach embankment design considerations; frost susceptibility and drainage of approach embankments; AASHTO load and resistance factors associated with geotechnical design elements; spread footing bearing and lateral capacity; nominal resistance of pile foundations; pile drivability; lateral pile design parameters; and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for spread footings, driven piles, lateral earth pressures and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2016 consisting of five test borings and a final design exploration program in 2018 consisting of four test borings. The approximate as-drilled boring locations were estimated by tape measurement from existing bridge components, and are shown on **Figure 2**. Ground surface and mudline elevations were estimated based on the bridge deck level at El. 13, as indicated on the plans.

New England Boring Contractors of Hermon, Maine provided drilling services and coordinated utility clearance. MaineDOT personnel cut a hole in the metal grate bridge deck at borings BB-SDC-102 through -104, BB-SDC-201 and -204 prior to drilling and patched the holes upon completion. All borings were drilled using 3- and 4-inch driven casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot typical intervals in the overburden using a 24-inch-long, 1-3/8-inch inside-diameter sampler. Bedrock cores were obtained using NX or NQ2 wire-line coring equipment in each test boring. Photographic logs of the recovered rock core specimens are included in **Appendix C**. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**. Elevations referenced in this report are in feet and refer to the National American Vertical Datum of 1988 (NAVD88). Additional details of each program are described below.

2.1 PRELIMINARY BORINGS

GZA completed five (5) test borings (BB-SDC-101 through BB-SDC-105) between May 24 and May 27, 2016. One boring was drilled behind each abutment (BB-SDC-101 and -105) and three were drilled through the



existing bridge deck at potential new pier locations (BB-SDC-102 through -104). The borings were drilled using a track-mounted drill rig. The abutment borings were backfilled with cuttings and asphalt cold patch.

The borings were drilled to depths of approximately 29 to 64 feet below ground surface (bgs) and terminated approximately 10 to 22 feet into bedrock. SPT sampling was conducted using a safety hammer operated by a rope and cathead. The land-based borings were backfilled with soil cuttings and/or sand and patched with cold patch.

2.2 FINAL BORINGS

GZA completed four (4) supplemental test borings (BB-SDC-201 through BB-SDC-204) between November 13 and December 27, 2018. One boring was drilled in front of each abutment (BB-SDC-201 and -204) and two were drilled in the river to the left and right of the proposed center pier (BB-SDC-202 and -203). The bridge borings were drilled using a track-mounted drill rig, and the river borings were conducted using a barge-mounted drill rig.

The borings were drilled to depths of approximately 21 to 78 feet bgs and terminated approximately 11 to 33 feet into bedrock. Land-based SPT sampling was completed with an automatic hammer, and water-based SPT sampling was completed using a safety hammer operated by a rope and cathead. The automatic hammer was NEBC Drill Rig No. 23, which had a rated hammer efficiency factor of 0.869 at the time of drilling.

Field vane shear tests were conducted to estimate the undrained shear strength of cohesive soils encountered in borings BB-SDC-202 and BB-SDC-203. Pairs of vane shear tests were completed at each test interval.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete a soil and rock testing program to assess the gradation and engineering characteristics of the soil and the strength of the bedrock. The testing program consisted of:

3.1 SOIL

- 10 gradation analysis / AASHTO Classification / Unified Soil Classification System / Frost Classification assessments;
- 5 sets of Atterberg Limits;
- 14 moisture content tests;
- 2 organic content tests; and
- 2 hydrometer tests.



3.2 ROCK

- 8 unconfined compressive strength / secant modulus tests; and
- 4 point-load tests (2 axial and 2 diametrical).

Results of the testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available surficial geology data¹, the surficial geologic unit mapped along the shoreline in the area consists of glacial till (heterogeneous mixture of sand, silt, clay and stones) with bedrock outcrops. There are also discontinuous glacial-marine deposits of the Presumpscot Formation in the site vicinity, described as silt, clay and sand.

Based on available bedrock geology data², the site is mapped near the contact of the Devonian Syenite formation and the Cape Elizabeth formation. The syenite is characterized as an unmetamorphosed, igneous syenite intrusion of Devonian age. The Cape Elizabeth formation is described as thinly-bedded slate and heavier beds of schist and quartzite of Ordovician to Precambrian age.

4.2 SUBSURFACE PROFILE

Five soil units were encountered in the test borings beneath surficial pavement and overlying bedrock: Fill, River Bottom Deposit, Marine Deposit, Glacial Till, and Decomposed Rock. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. The subsurface conditions are also shown in relation to the bridge alignment on the interpretive subsurface profile in **Figure 3**, and in relation to the center pier in the interpretive subsurface cross-section in **Figure 4**.

¹ Smith, G.W., 1976, "Surficial Geology of the Boothbay Harbor Quadrangle," (Open File No. 76-33), Maine Geological Survey, Department of Conservation, scale 1:24,000.

² Osberg, P.H., Hussey, A.M., and Boone, G.M., 1985, "Bedrock geologic map of Maine." Maine Geological Survey, Department of Conservation, scale 1:500,000.



GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	6 to 19	Brown/Black/Red/Gray, loose to very dense, fine to coarse SAND, trace to some Gravel, trace Silt. Trace wood, brick, and shell fragments. (USCS: SW-SM, SP-SM, SM, SP, or GP) <ul style="list-style-type: none"> • MaineDOT Frost Classification = 0-II • A 2-foot thickness of possible Rock Fill (GRAVEL, little to some Sand, probable cobbles/boulders) was encountered in boring BB-SDC-105 at 10 to 12 feet bgs. • Approximate 2-foot thick riprap encountered in borings BB-SDC-201 and BB-SDC-204; included in total fill thickness. <i>Encountered directly overlying Bedrock in all abutment/approach borings (BB-SDC-101, BB-SDC-105, BB-SDC-201, and BB-SDC-204).</i>
River Bottom Deposit	7 to 9	Dark Brown/Dark Gray, very soft organic SILT to organic CLAY, little Sand, trace Gravel, with shells and wood (USCS: OL, OH). Dark brown, clayey fine to medium SAND at mudline. <ul style="list-style-type: none"> • 4 field vanes: Peak Undrained Shear Strength (Su) = 357-558 psf, Residual Su = 89-125 psf. • 3 Atterberg limits: Liquid Limit (LL) = NP-80, Plasticity Index (PI) = NP-40, water content = 46.9-77.2 percent. <i>Encountered in all water borings (BB-SDC-102, BB-SDC-103, BB-SDC-104, BB-SDC-202, and BB-SDC-203).</i>
Marine Deposit	7 to 13	Gray, stiff to soft, silty CLAY, trace fine Sand (USCS: CL). <ul style="list-style-type: none"> • 3 field vanes: Peak Undrained Shear Strength (Su) = 201-982 psf, Residual Su = 22-124 psf. • 2 Atterberg limits: Liquid Limit (LL) = 33-37, Plasticity Index (PI) = 14-16, water content = 33.8-39.2 percent. <i>Encountered in borings BB-SDC-102, BB-SDC-103, BB-SDC-202, and BB-SDC-203; not encountered in approach borings.</i>
Glacial Till	4 to 32	Gray/Brown, medium dense to dense, fine to coarse SAND with varying amounts of Gravel, Silt, and clay (USCS: SM, SC or SP). <ul style="list-style-type: none"> • Penetrated 0.2 to 5.0 feet of possible cobbles/boulders in borings BB-SDC-103, -202, and -203 using roller cone or core barrel <i>Encountered in all water borings (BB-SDC-102, BB-SDC-103, BB-SDC-104, BB-SDC-201, and BB-SCD-202)</i>
Decomposed Rock	3 to 14	Variable ranging <u>from</u> : Tan, very dense, sandy GRAVEL, little Silt, with remnant bedrock structure, <u>to</u> : Very soft to hard, fresh to highly weathered Grey PHYLLITE/SCHIST fractured into gravel. <i>See additional discussion below.</i> <i>Encountered in borings BB-SDC-102, BB-SDC-103, BB-SDC-202, and BB-SDC-203.</i> <u>Estimated Top of Decomposed Rock:</u> Pier 1: Approx. El. -50.6 to El. -58.3
Top of Bedrock Elevation		<u>Encountered Top of Rock:</u> Abutment 1: Approx. El. -1.2 to El. -4.4 Pier 1: Approx. El. -61.0 to El. -64.1 Abutment 2: Approx. El. -5.4 to El. -5.9

4.2.1 Decomposed Rock

Decomposed rock descriptions and encountered locations are described in the table above. Split spoons driven within the stratum generally met refusal within 2 to 5 inches. After split spoon refusals, cores were attempted to recover more of the material for classification which resulted in brown wash water return, little to no recovery and 0 percent Rock Quality Designation (RQD). The recovered decomposed rock in the spoons and cores had remnant bedrock structure, but was easily broken with a field knife or by hand.



Some harder rock pieces fractured into gravel were collected in the core samples and increases in resistance during the advancement of the roller cone showed variable hardness within the stratum. The transition from decomposed rock to rock was interpreted to occur at the depth where the RQD first exceeds zero.

4.2.2 Bedrock

Bedrock was cored beneath the decomposed rock in each test boring and was primarily described as Schist and Pegmatite. Photographic logs of the recovered rock core specimens are included in **Appendix C**.

Schist was encountered in all nine test borings. Schist was described as moderately hard to hard, fresh to slightly weathered, fine to medium-grained and grey and black. Joints are extremely close to widely spaced, low angle to moderately dipping, planar to undulating, rough, fresh to decomposed, and partially open to open, with occasional silt and sand infilling. The RQD of the Schist ranged from 0 to 100 percent. The upper 1 to 3 feet of rock in borings BB-SDC-103, -104, and -202 was described as Phyllite.

The upper portion of the Schist encountered in BB-SDC-103, BB-SDC-202, and BB-SDC-203 (at Pier 1), beneath the decomposed rock, varied in hardness and weathering from the other borings. Portions of the recovered core in these borings were described as very soft to medium soft and moderately to highly weathered. The weathered Schist appeared to be capped by 0 to 4.8 feet of harder Schist or Phyllite and extended from approximately El. -61 to El. -77 and was underlain by hard Pegmatite and fresh hard Schist.

Pegmatite was encountered as intrusions in the metamorphic rocks (schist and phyllite) in seven borings (BB-SDC-102, -103, -105, and -201 through -204) and was described as hard, fresh to slightly weathered, coarse to very coarse-grained and pink, orange, tan, and grey. Joints are extremely close to wide, low to high angle, planar to undulating, rough, fresh to decomposed, and partially open to open, with occasional sand infilling. The RQD ranged from 0 to 100 percent, with an average of 74 percent.

The bedrock elevations interpreted from the borings indicate the top of rock is relatively level at the abutment locations, and slopes steeply from both abutments down toward the center of Decker Cove. The average inclination of the bedrock surface from both abutments toward the center of the cove appears to be approximately 1.5H:1V to 2H:1V. The actual inclination of the bedrock surface is unknown, but it is likely stepped with steeper and flatter portions, rather than following a consistent inclination.

Unconfined compressive strength (UCS) testing was conducted on eight samples of fresh to slightly weathered rock, the results of which are summarized in the following table. Weathered Schist from BB-SDC-103, -202, and -203 was not tested and is expected to be weaker than the test results presented below.



SUMMARY OF BEDROCK STRENGTH TEST RESULTS							
Boring	Depth below Existing Ground (ft bgs)	Depth below Top of Rock (ft bgs)	Elevation (ft NAVD 88)	Unconfined Compressive Strength (psi)	Secant Modulus @ 50% of Failure Stress (ksi)	Unit Weight (pcf)	Rock Type
BB-SDC-101	17.5	0.1	-4.5	11,469	1,920	165	SCHIST
BB-SDC-102	23.4	2.4	-37.6	7,551	4,700	170	SCHIST
BB-SDC-102	28.7	7.7	-42.9	6,321	2,470	163	PEGMATITE
BB-SDC-104	20.3	1.8	-29.3	12,835	4,660	164	SCHIST
BB-SDC-105	20.1	1.2	-7.1	12,063	3,980	162	PEGMATITE
BB-SDC-105	22.7	3.8	-9.7	9,459	3,750	171	SCHIST
BB-SDC-201	10.9	5.0	-6.2	7,340	3,890	181	SCHIST
BB-SDC-204	11.1	0.9	-6.3	5,689	1,840	164	PEGMATITE

Axial and diametral point load testing were conducted on Schist from BB-SDC-102 and Phyllite from BB-SDC-104. The size-corrected point load index, $I_{s(50mm)}$, ranged from 795 to 3,346 psi, which suggests an unconfined compressive strength well in excess of 10,000 psi based on typical correlation factors between $I_{s(50mm)}$ and UCS.

4.2.3 Groundwater

The test borings were drilled using drive-and-wash techniques, which introduce water into the borehole during drilling. As a result, stabilized groundwater levels were not determined at the proposed abutment locations. Considering the proximity to the tidal cove, groundwater levels are anticipated to fluctuate within a range from few feet above high tide level, to near low tide level.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater will occur due to variations in season, tide, precipitation, and other factors. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations based on the 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition with Interims, (AASHTO), and the MaineDOT Bridge Design Guide, 2003 Edition, with updates through 2014 (MaineDOT BDG). Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

The proposed embankment widening will require new fills up to 6 feet high, and reconstructed slopes. Approach embankment side slope inclinations will be 2H:1V or flatter above El. 7 and between 1.5H:1V and 1.75H:1V below El. 7, where they will be covered with riprap, as described previously. The plans indicate that riprap is planned to be keyed in approximately 3 feet below mudline level.



The subsurface conditions beneath the approaches include fill over bedrock. Therefore, it is GZA’s opinion that post-construction embankment settlement will be negligible.

The existing fills in the area of the bridge will not be subject to deep-seated rotational instability due to the shallow rock. Stability of the slopes will be controlled by the surficial granular soil and existing riprap, which will be covered by riprap. Standard riprap slope construction details call for the riprap to be keyed into the soil at the base of the slope, as is shown on the plans. Considering that the toe of the slopes will likely bear on bedrock in some locations, especially on the west side where the widening will occur, keying in the toe of the riprap may not be practical. Excavating a keyway in rock and buttressing the toe of the riprap are also considered impractical due to difficult access at the mudline.

It is noted that the existing slopes are frequently between 1.5H:1V and 1.75H:1V, and there are no signs of distress. To limit the potential for inadequate key embedment to achieve toe stability, we conclude that the entire embankment widening below El. 7 should be completed using riprap. This will limit the potential for the new fill to be less stable than the existing material and is a small enough volume that the cost is not increased significantly.

5.3 EVALUATION OF FOUNDATION TYPES

5.3.1 Abutment Foundations

Test borings BB-SDC-101, -105, -201, and -204 revealed approximately 14 to 20 feet of fill beneath the existing roadway overlying bedrock. Rock is too shallow to construct an integral abutment foundation with conventional driven piles. Rock-socketed piles, spun piles, micropiles or drilled shafts are technically feasible, but the construction cost is anticipated to be greater than a spread footing alternative. Based on the shallow depths to bedrock, spread footings built within cofferdams bearing on competent bedrock are the preferred alternative for abutment support.

5.3.2 Pier Foundations

Pier foundation alternatives were influenced by the subsurface conditions interpreted from borings BB-SDC-103, -202 and -203. The interpreted soil layering is presented on **Figure 4**, and the properties developed for the soil strata summarized in the table below.

SUBSURFACE STRATA SUMMARY, PIER 1		
Soil Unit	Estimated Thickness (feet)	
	Minimum	Maximum
River Bottom Deposit (very soft to soft Organic Silt/Clay)	8	9
Marine Deposit (soft Clay)	7	13
Glacial Till (medium dense to dense Sand, Gravel)	18	26
Decomposed Rock (very dense Gravel, Variable Bedrock)	3	14
Depths below Mudline to:		
Decomposed Rock	39	42
Bedrock	45	53

A bent pier configuration was selected by Stantec for Pier 1. Conventional foundation types for bent piers in the water include driven pipe piles or drilled shafts. However, due to concern with the corrosion of



steel piles in salt water using conventional corrosion protection technologies (e.g., epoxy coating, cathodic protection), driven steel H-piles encased in concrete-filled fiber reinforced polymer (FRP) shells have been selected for this project to enhance durability.

5.4 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was evaluated in accordance with the AASHTO LRFD, along with consideration of the 2011 AASHTO Guide Specifications for LRFD Bridge Design (Seismic Guide Specification).

Article C3.4.2.2 of the Seismic Guide Specification suggests two scenarios for depth of ground motion determination for a bridge, where the depth of ground motion corresponds to the top of the considered soil/bedrock profile for site class determination. The first applies to “short bridges with a limited number of spans” and indicates that the motion will primarily be controlled by the abutments. The Seismic Guide Specification recommends that the site class be determined at the base of the approach fill for this scenario. The second scenario applies to “long bridges” and provides recommendations for determining the site class for a bent pier. The recommendation for bent piers is to use a ground motion that is between 4 and 7 pile diameters beneath the ground surface, or at a depth of significant impedance (i.e., transition from weak to strong soil). For this project, the “long bridge” scenario would result in higher seismic motions since soil would be considered in the profile, while the abutments will be founded on rock and would lead to a site class corresponding to rock. An excerpt containing Article C3.4.2.2 is included in **Appendix E**.

Stantec indicated that due to the semi-integral abutment design, the seismic response of the bridge may be significantly influenced by the center pier. Therefore, the site classification is based on the “long bridge” scenario and the soil profile at the pier.

At Pier 1, 19 feet below the mudline (roughly 7 pile diameters of the FRP casing), the soil changes from soft cohesive soil to medium dense to dense glacial till, with an average SPT N value of approximately 20, which is within the range of 15 to 50 specified for Site Class D. Therefore, we recommend that the site be assigned to Site Class D.

5.5 LOAD AND RESISTANCE FACTORS

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), and live load surcharge (LS) loads, using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design. Load factors are not provided for passive earth pressure because this is considered a resistance in AASHTO LRFD. A load factor of 1.5 may be applied to the passive soil reaction used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck, consistent with the load factor provided for active earth pressure in AASHTO Table 3.4.1-2.

The recommended LRFD resistance factors for strength limit state design of foundations were derived from LRFD Tables 10.5.5.2.2-1, 10.5.5.2.3-1 and 10.5.5.2.4-1 and are presented in the following table.



GEOTECHNICAL RESISTANCE FACTORS – STRENGTH LIMIT STATE			
Foundation Resistance Type	Method/Condition	Resistance Factor (ϕ)	AASHTO Reference
Bearing	Footing on Rock	0.45	10.5.5.2.2-1
Sliding	Footing on Rock	0.8	10.5.5.2.2-1
Axial Pile Resistance	Driven Pile, Installed with Dynamic Testing	0.65	10.5.5.2.3-1

Resistance factors for service and extreme limit state design should be taken as 1.0.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor $\phi_c=0.50$, per AASHTO LRFD Article 10.7.3.2.3 for hard driving condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor $\phi_c=0.7$ and the flexural resistance factor $\phi_f=1.0$ should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

5.6 SPREAD FOOTING DESIGN CONSIDERATIONS

5.6.1 Footing Bearing Resistance

Nominal and factored bearing resistances were calculated for bedrock-bearing footings using the Rock Mass Rating- (RMR-) based empirical correlation presented in “Foundations on Rock,” by Duncan Wyllie. RMR was evaluated in accordance with Table 10.4.6.4-1 of the 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO). The current (8th) Edition of the AASHTO Design Specifications does not include the RMR formulation included in the previous version (6th Edition). However, Articles C10.4.6.4 and 10.6.2.6.2 of the 8th Edition refer to RMR-based design procedures for footings on rock, so the 6th Edition methodology was followed.

GZA used bedrock data obtained in test borings drilled at or near the proposed abutments to develop foundation design parameters at the abutment locations. The bedrock properties used in the bearing resistance evaluation are presented below:

DESIGN BEDROCK PROPERTIES FOR BEARING RESISTANCE EVALUATION					
Rock Type	RQD (percent)	Unconfined Compressive Strength (ksi)	Rock Mass Rating (RMR)	m	s
Schist/Pegmatite	67	6.7	44	0.31	0.00008

Based on these parameters, the calculated nominal bearing resistance is 60 kips per square foot (ksf), resulting in a factored bearing resistance of 27 ksf for the strength limit state. Supporting calculations are provided in **Appendix E**.

LRFD Article 10.6.2.4.4 indicates that footings bearing on rock with an RMR-based rock quality of Fair or better and designed using LRFD methods are anticipated to experience ½ inch or less of elastic settlement.



5.7 PILE DESIGN CONSIDERATIONS

5.7.1 Pile Type and Loading

We understand four steel HP18x204 H-section piles are planned to support Pier 1 based on the required structural resistance. Stantec provided a maximum factored axial load of 600 kips per pile for the strength condition; therefore, piles should be installed to a nominal axial resistance of at least 923 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65, per LRFD Table 10.5.5.2.3-1, for piles installed using dynamic pile testing with signal-matching analysis to assess nominal geotechnical pile resistance.

5.7.2 Axial Pile Resistance

The axial geotechnical resistance was evaluated using the computer analytical software *APile* by Ensoft. The soft soil is assumed to not contribute to skin friction, and friction is derived in glacial till and possibly decomposed rock.

Considering the presence of the decomposed rock layer, which has the potential to behave similarly either to soil or rock, GZA evaluated two possible conditions to characterize the range of anticipated pile behavior, as summarized below:

1. *Short Pile:* This analysis assumes that the piles will meet practical refusal near the bottom of the glacial till/top of decomposed rock. This would result in a primarily end-bearing pile support condition, which represents the maximum energy transfer to the pile tip and the highest expected driving stress. The results indicate that the piles would have approximately 10 percent side friction at the required nominal resistance in this case.
2. *Long Pile:* This analysis assumes that the decomposed rock will behave similarly to glacial till, and the piles will be driven through most or all of the decomposed rock. This would result in a combination skin friction/end-bearing pile support condition, which should model the largest required hammer size to install the piles. The results indicate that the piles would have approximately 46 percent side friction at the required nominal resistance in this case.

Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by Stantec do not include uplift loading on the piles. By utilizing steel H-piles for support of the pier, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

5.7.3 Preliminary Wave Equation Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an HP 18x204 pile with a nominal geotechnical resistance of 923 kips at the location of the central pier. Analyses were completed using a Delmag D36 diesel hammer with a ram weight of 7,930 pounds and a maximum rated energy of 83,820 foot-pounds (ft-lbs).

The calculated side resistance was used as an input in wave equation analyses, including 10 and 46 percent contribution of skin friction resistance to the maximum factored pile resistance for the Short Pile and Long Pile analyses, respectively. The results are summarized below.



SUMMARY OF WEAP ANALYSES					
Pile Analysis and Type	Embedded Pile Length	Driving System ¹	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
HP 18x204 "Short Pile"	23 feet	Delmag D 36 (83,820 ft-lb, open)	923	37	7
HP 18x204 "Long Pile"	31 feet	Delmag D 36 (83,820 ft-lb, open)	923	30	8

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT preferred range of 6 to 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the required nominal resistance noted. Results of the preliminary wave equation analyses are provided in **Appendix E**.

5.7.4 Lateral Pile Analysis

GZA developed a soil profile for lateral pile evaluations at the center pier. It was assumed that the piles will be driven to the average elevation of the interface between the glacial till and decomposed rock strata (El. -54). This represents the worst case (shortest) pile length relative to achieving pile fixity. It is likely that the piles will penetrate the denser/stronger decomposed rock stratum, which would improve fixity of the pile toe compared to the modeled condition. We understand Stantec is using the analytical software *Group* by Ensoft for lateral pile evaluations. Recommended lateral pile input parameters are presented below.

LATERAL PILE INPUT PARAMETERS – GROUP						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E ₅₀ / UC (psi)	φ' (deg) / Su (psf)	γ _e (pcf)
River Bottom Deposit	Soft Clay	-13	9	E ₅₀ = 0.02	250 psf	32.6
Marine Deposit	Soft Clay	-22	9	E ₅₀ = 0.01	500 psf	47.6
Glacial Till	Reese Sand	-31	23	75	34	67.6
Decomposed Rock (Pile Tip, No Penetration)	--	-54	--	--	--	--

Skin friction resistance should be automatically generated in *Group* based on the provided soil parameters. A unit tip resistance of 350 ksf should be applied to the full box end area of the pile.

5.8 ADDITIONAL FOUNDATION CONSIDERATIONS

5.8.1 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1200, and with low-moisture content (<20 percent) soils, the estimated depth of frost penetration is approximately 5 feet. Consequently, new foundation levels should be set at least 5 feet below ground surfaces exposed to freezing temperatures.



Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-b and A-3 with MaineDOT Frost Classification from I to II, indicating they are considered to exhibit low to moderate frost susceptibility. Since there was no evidence of significant pavement distress or heave, these materials are judged to be suitable for continued use beneath the approach roadway after reconstruction. In accordance with MaineDOT Standards, new backfill placed behind abutments will consist of non-frost-susceptible materials.

5.8.2 Lateral Earth Pressures

The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. In accordance with the requirements of the BDG Section 5.4.3, semi-integral abutment reinforcement is to be designed for active or at-rest earth pressure over the abutment height and wingwalls, and a uniform pressure distribution due to the height of soil behind the superstructure. For the planned relatively short abutment with footing bearing on rock, an at-rest earth pressure coefficient, $K_o=0.47$, is recommended.

The superstructure backwall (end diaphragm) should typically be designed for full passive pressure. A Coulomb passive earth pressure coefficient, $K_{pc}=6.73$, is recommended. Developing full passive pressure assumes that the ratio of lateral movement to backwall 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 Rankine theory passive earth pressure coefficient, $K_{pr}=3.25$.

AASHTO Commentary C3.10.9.1 specifies that multi-span bridges are required to include acceleration-augmented (earthquake-induced) soil pressures for design. The dynamic earth pressure coefficient, K_{AE} , is calculated to be 0.33 for this project, which includes active earth pressure plus a dynamic component. Given that K_{AE} is much lower than the at-rest or passive earth pressure coefficient, the dynamic active earth pressure does not need to be considered for design.

Design lateral earth pressure recommendations are provided in **Section 6.3** of this report.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes that are not riprap-covered should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Soil slopes should be provided with loam and seed for permanent erosion protection. Steeper slopes should be covered with riprap. Riprap should also be provided where the embankment side slopes will be near or below typical water levels, to protect from scour.

As discussed previously, we anticipate that the proposed 1.5H:1V to 1.75H:1V riprap slopes will be underlain directly by bedrock, which would prevent providing a key in accordance with MaineDOT Standard Details. Therefore, we recommend that the entire embankment widening below El. 7 should be completed using riprap. Protective aggregate cushion and geotextile should be placed adjacent to and above the riprap in accordance with the MaineDOT standard details.



6.2 SEISMIC DESIGN

The United States Geological Survey Online Design Maps Tool was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s (Period = 0.0 sec)	0.110 g
S _{Ds} (Period = 0.2 sec)	0.224 g
S _{D1} (Period = 1.0 sec)	0.098 g

Per AASHTO Article 4.7.4.2, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but the minimum requirements apply for superstructure connections and support lengths as specified in AASHTO Articles 4.7.4.4 and 3.10.9.

6.3 ABUTMENT AND WINGWALL DESIGN

- Abutment backfill should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - At-Rest Earth Pressure Coefficient, $K_o = 0.47$ (use for design of abutment and wingwalls)
 - Coulomb Coefficient of Passive Earth pressure, $K_{pc} = 6.73$ (use for design of end diaphragm)
 - Rankine Coefficient of Passive Earth Pressure, $K_{pr} = 3.25$ (use for design of end diaphragm with y/H much less than 0.005 at the discretion of the structural engineer)
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Spread Footing Design

- The proposed abutments should be supported on spread footing foundations bearing on sound, intact bedrock. Footings designed to bear on intact bedrock should be designed using a nominal bearing



resistance, q_n , of 60 ksf. At the strength limit state, footings should be designed for a maximum factored bearing resistance of 27 ksf. A bearing resistance of 27 ksf should be used for service limit state design.

- Spread footings founded on bedrock should be checked for eccentricity with AASHTO Article 10.6.3.3. Eccentricity of the footing reaction at the strength limit state should be limited such that the resultant reaction on the base of the footing is no further than 0.45 B from the centerline of the footing, where B is the footing width perpendicular to the axis of rotation.
- The base resistance against sliding may be based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient ($\tan \delta$) is equal to 0.7 for cast-in-place concrete on sound rock. Therefore, the nominal sliding resistance between footings and bedrock subgrades is equal to the vertical force multiplied by 0.7. The factored sliding resistance coefficient is 0.56 for Strength Limit State.
- The bedrock surface should be cleaned of loose soil or rock at the time of concrete placement for seals, subfooting concrete, or the footing. Bearing surface preparation should be in accordance with **Section 7.2**.
- We understand the footings will be constructed over tremie seals bearing on bedrock. For tremie seal evaluation, the top of bedrock elevation in the borings was found to be within the ranges in the table that follows.

ESTIMATED BEDROCK LEVELS FOR TREMIE SEAL DESIGN	
Foundation Element	Estimated Range in Bedrock Elevation (feet, NAVD 88)
Abutment 1	El. -1 to -5
Abutment 2	El. -5 to -6

It is important to note that the top of intact rock cannot be known for the entire foundation area prior to construction. We expect that intact rock may be encountered above and below the anticipated levels. Some construction-phase engineering should be anticipated to address the potential variability of the encountered conditions.

- If the bedrock level extends above the design bottom of tremie seal elevation, the footing may be raised and vertical reinforcement shortened in the wall subject to review and approval of the Designer to limit overexcavation of bedrock.
- If the exposed bedrock surface after cleaning is below the design footing bearing level, the tremie seal may be thickened to the proposed footing bearing level.
- Concrete used for cofferdam seals, subfootings and footings should consist of Class S Concrete in accordance with MaineDOT Standard Specification Section 502.05.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4H:1V in any direction.
- Rock dowels may be used to supplement the sliding resistance for the footing to resist design lateral loads. The dowels should be grouted a minimum of 2 feet into intact bedrock and embedded at least 1.5 feet into concrete. The unconfined compressive strength of the bedrock should be assumed to be 6.7 ksi for design of rock dowels.
- Dowels should be grouted with a cementitious grout on the MaineDOT Qualified Products List of Grout Materials for Keyways and Anchoring (pre-qualified for anchoring). Epoxy grout should not be used.



- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1.
- Existing substructures should be completely removed at the abutments prior to new foundation construction.

6.4.2 Pile Design

- The proposed pier may be supported on HP18x204 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on or near the bedrock surface.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 923 kips, calculated by dividing the maximum factored pile load of 600 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the piles can be driven to a nominal resistance of 923 kips using a diesel hammer with a rated energy of 83,820 ft-lbs for ASTM A572 Grade 50 HP18x204 piles without exceeding the allowable driving stress of 45 ksi (0.9F_y for 50 ksi steel). The final penetration resistance was 7 to 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the top of rock elevations encountered in the borings and shown on the interpretive cross-section profile (approximately El -61 to El -64). A provision is recommended to be provided in the drawings for extra pile length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into bedrock. However, it is also possible that the piles will achieve nominal pile resistance as shallow as the top of decomposed rock (approximately El. -51 to El. -58).
- The FRP shells should be sized to allow advancement to at least El. -25, which is greater than 10 feet below mudline elevation. If difficult installation is encountered, the FRP length can be shortened and the upper portion cut off after a minimum embedment of 10 feet below mudline has been achieved.
- We recommend that one pile be dynamically tested at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require two restrike tests on the test pile, including one at 24 hours after initial drive and a second at least 48 hours after the first restrike, to assess potential relaxation. In the event that the initial driving and first restrike results do not show a resistance loss, the second restrike test will be waived.
- Piles shall be spliced in accordance with MaineDOT Standard Specification Section 501.047.

7.0 CONSTRUCTION CONSIDERATIONS

This section describes geotechnical-related issues that have the potential to impact design and cost considerations for bridge construction.



7.1 SUPPORT OF EXCAVATION AND DEWATERING

Excavations for abutment foundations will extend up to about 14 to 20 feet below existing grade to expose bedrock. The anticipated bedrock surface elevation ranges from approximately El. -1 to -5 at Abutment 1 and El. -5 to -6 at Abutment 2, corresponding to depths of approximately 1 to 5 feet below mean tide level (El. -0.32) and as deep as mean low tide level (El. -5.15) at Abutment 1, and approximately at or 1 foot below mean low tide level (El. -5.15) at Abutment 2. It is anticipated that 3-sided cofferdams will be required to allow preparation of the bedrock surface, and that a sloped open cut may be feasible on the approach roadway side.

The contractor should be responsible for design of all temporary support of excavation. Design should be completed by a professional engineer registered in the State of Maine. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

At most tide levels, dewatering is anticipated to be feasible by pumping from sumps placed within the excavations, assuming the excavation support system is relatively tight and the sumps are placed in the lowest portions of the bedrock surface. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources to permit final bedrock subgrade preparation and inspection to be completed in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.2 SUBGRADE PREPARATION

Based on our experience on similar projects, we anticipate it may be feasible to complete final bedrock subgrade preparation in-the-dry. However, this may limit work hours based on tidal considerations if relatively watertight excavation support is not used.

We anticipate that the bedrock surface will be variable in terms of elevation, slope and localized weathering. Conventional excavation equipment is anticipated to complete excavations. All soil and loose, decomposed, highly-weathered and fractured bedrock should be removed from the footing bearing surface prior to placement of tremie seals. The prepared bearing surfaces should be checked by visual means in dry conditions. A Special Provision should be prepared to define the project-specific requirements for subgrade preparation and quality assurance/quality control.

The Geotechnical Engineer and Designer should be provided cross-sections showing the prepared rock surface geometry prior to placement of concrete to evaluate whether benching, doweling, or subfooting reinforcement are needed for that foundation location. Based on the test borings we do not anticipate steeply sloping bedrock conditions at the abutments. However, if the exposed bedrock surface is steeper than 4H:1V, then anchoring, doweling, benching or other means should be employed to improve sliding resistance.

7.3 FRP AND PILE INSTALLATION CONSIDERATIONS

We recommend that all H-pile installation activities precede the FRP shell installation. This will eliminate the potential for H-pile installation to alter the location or damage the FRP. Once the H-pile is installed within tolerances, the FRP shell may be placed over the H-pile and advanced to the required depth using a vibratory hammer. The FRP shell should be advanced to a tip elevation of approximately El. -25, 5 feet into the marine clay deposit.



It is possible that the marine clay will be disturbed and remolded following H-pile installation, providing little strength to hold the FRP shell at the desired elevation following vibratory installation. Possible alternatives to address this risk include making the FRP sufficiently long to extend to the top of the glacial till or providing means to hold the FRP in-place. In addition, to install the FRP after H-pile installation, the FRP will have to be long enough to extend above the top of pile elevation for attachment of the vibratory hammer to the top of the FRP. The total length of the FRP shell should provide adequate length to address these issues.

7.4 PILE INSTALLATION CONTROL

We recommend that the H-pile installation be controlled using wave equation analysis and field logging of the pile installation and that final penetration resistance be based on dynamic pile testing with signal matching analysis. As previously noted, the piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.

We recommend dynamic testing with signal matching be completed on a single test pile at one of the production pier pile locations. The test pile should be evaluated at end of initial drive and during restrikes at 24 hours after initial drive and at 48 hours after the first restrike. The intent is to assess potential relaxation. The second restrike test may be waived if the initial drive and first restrike tests are not indicative of relaxation. If relaxation is indicated, it is likely that a higher end-of-drive geotechnical resistance and penetration criteria would be recommended to compensate for resistance lost due to relaxation.

7.5 REUSE OF ON-SITE MATERIALS

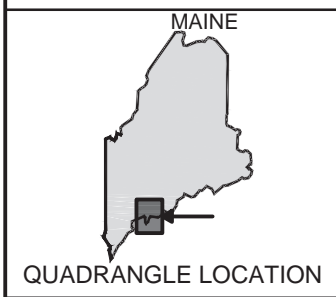
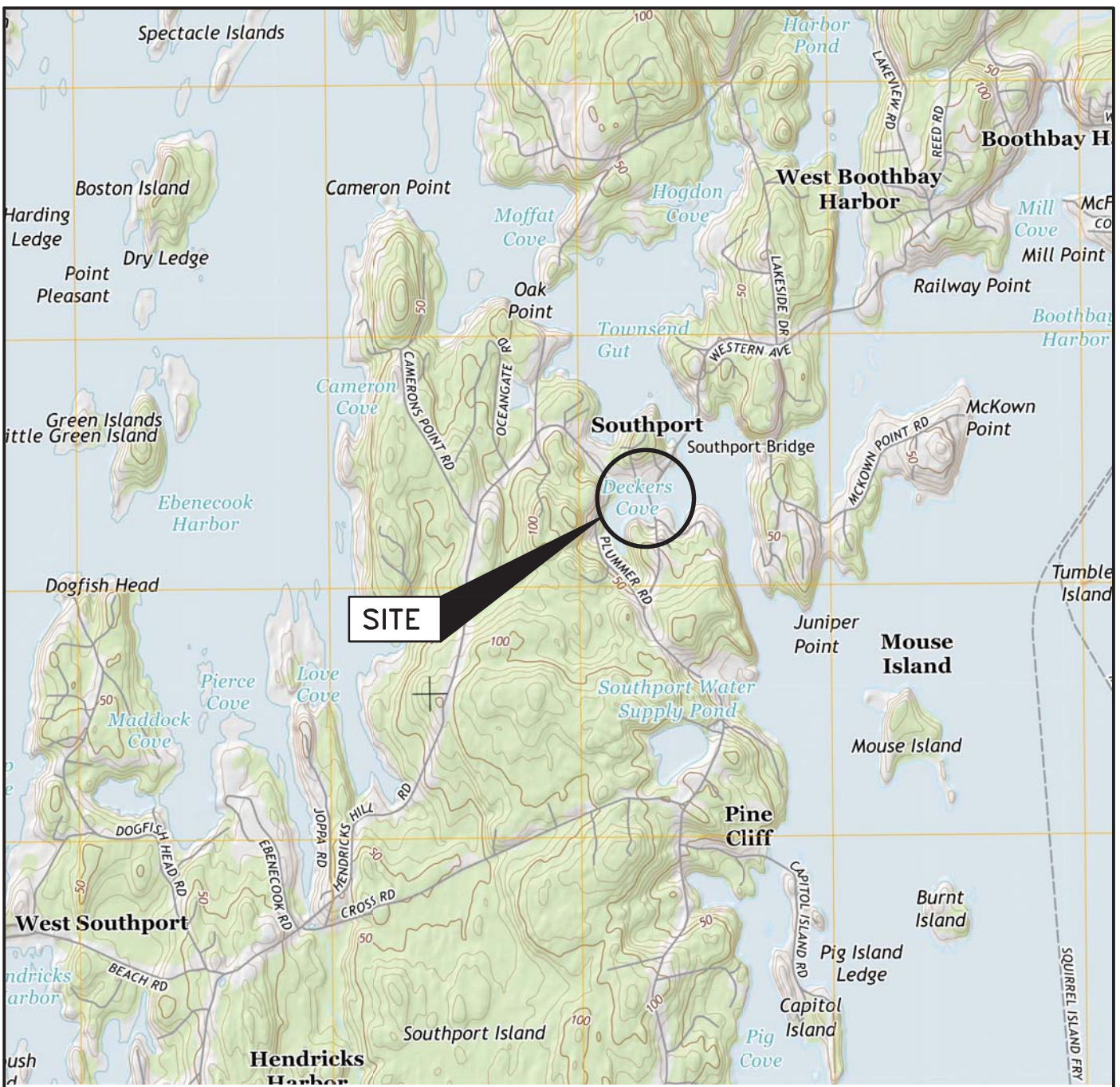
Based on the test boring results, two of the three fill samples tested had less than 10 percent passing the No. 200 sieve, indicating the fill may meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and would be suitable for use as structural backfill. The material is also considered suitable for use as Common Borrow.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.



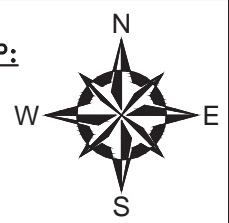
FIGURES

© 2016 - GZA GeoEnvironmental, Inc. GZA-J:\BRANCH\NORWOOD\SOUTHPORT, ME\09.0025910.00_LOCUS MAP_FIG-1.DWG LOCUS JUNE 12, 2014 GARY BASTIEN



SOURCE:
BASE MAP FROM THE FOLLOWING USGS QUADRANGLE MAP:
BOOTHBAY HARBOR (2014)
 DIGITAL TOPOGRAPHIC MAPS PROVIDED BY USGSSTORE.GOV.

CONTOUR ELEVATIONS REFERENCE NAVD 88,
 CONTOURS ARE SHOWN IN FEET AT 10' INTERVALS



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THOMPSON BRIDGE #2848 OVER DECKER COVE
 MAINEDOT WIN 18748.00
 SOUTHPORT, MAINE

PREPARED BY:
 GZA GeoEnvironmental, Inc.
 Engineers and Scientists
 www.gza.com

PREPARED FOR:
 STANTEC

LOCUS MAP

PROJ MGR: ARB	REVIEWED BY: ARB	CHECKED BY: CLS
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS NOTED
DATE: JANUARY 2017	PROJECT NO. 09.0025910.01	REVISION NO. 0

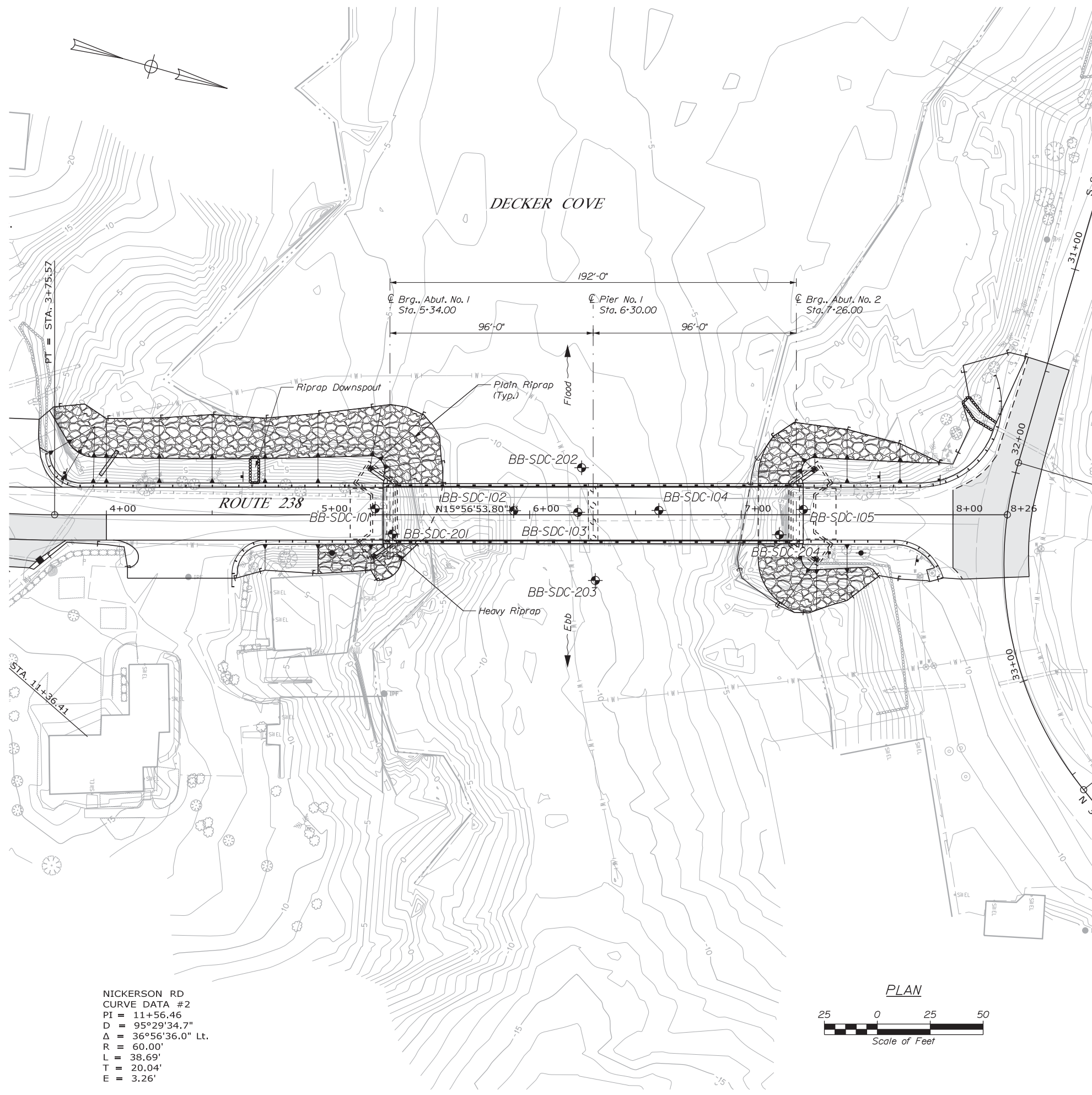
FIGURE
1
 SHEET NO. 1 OF 1

Date: 2/27/2019

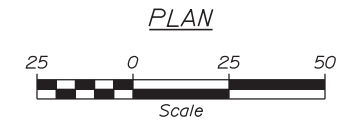
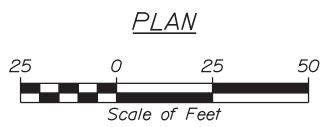
Username: dataylor

Division: BRIDGE

Filename: ... \001\BRIDGE\MSTA\006_BLP.DGN



NICKERSON RD
 CURVE DATA #2
 PI = 11+56.46
 D = 95°29'34.7"
 Δ = 36°56'36.0" Lt.
 R = 60.00'
 L = 38.69'
 T = 20.04'
 E = 3.26'



NOTES

- 1) Base map developed from electronic files provided by Stantec on August 16, 2016 (Files included 002_GeneralPlan.dgn, BDPLAN.dgn, contours.dgn, and 3DTopo_10aug16.dgn) and on January 24, 2019 (bridge.dgn).
- 2) The as drilled locations of the test borings were estimated by taping to existing bridge structural elements and should be considered approximate.
- 3) BB-SDC-100 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between May 24 and May 27, 2016.
- 4) BB-SDC-200 series bridge borings were performed by New England Boring Contractors and observed by GZA personnel between November 13, 2018 and December 27, 2018.

LEGEND

BB-SDC-204 Location and designation of cased wash boring

THOMPSONS BRIDGE NO. 2848 OVER DECKER COVE
 MAINEDOT WIN 18748.00
 SOUTHPORT, ME

BORING LOCATION PLAN

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: STANTEC	
PROJ MGR: NWV	REVIEWED BY: ARB	CHECKED BY: CLS	FIG 2
DESIGNED BY: NWV	DRAWN BY: NWV	SCALE: AS SHOWN	SHEET NO. 1 OF 1
DATE: 2/21/19	PROJECT NO. 09.0025910.01	REVISION NO. 0	

STATE OF MAINE DEPARTMENT OF TRANSPORTATION		STP-1874(800)	
THOMPSONS BRIDGE DECKER COVE LINCOLN COUNTY		BORING LOCATION PLAN	
SOUTHPORT		SHEET NUMBER	
DATE: 02/21/2019		WIN 18748.00	
SIGNATURE: Andrew R. Blaisdell		BRIDGE NO. 2848	
P.E. NUMBER: 10957		BRIDGE PLANS	
DATE: 02/21/2019		18748.00	
DATE: FEB 2019		DATE: FEB 2019	
BY: NWV		BY: CLS	
L. TIMBERLAKE		L. TIMBERLAKE	
DESIGN-DETAILED		CHECKED-REVIEWED	
DESIGN-DETAILED		DESIGN-DETAILED	
REVISIONS 1		REVISIONS 2	
REVISIONS 3		REVISIONS 4	
FIELD CHANGES		FIELD CHANGES	
DATE: FEB 2019		DATE: FEB 2019	
SIGNATURE: Andrew R. Blaisdell		SIGNATURE: Andrew R. Blaisdell	
P.E. NUMBER: 10957		P.E. NUMBER: 10957	
DATE: 02/21/2019		DATE: 02/21/2019	
DATE: FEB 2019		DATE: FEB 2019	
BY: NWV		BY: CLS	
L. TIMBERLAKE		L. TIMBERLAKE	
DESIGN-DETAILED		CHECKED-REVIEWED	
DESIGN-DETAILED		DESIGN-DETAILED	
REVISIONS 1		REVISIONS 2	
REVISIONS 3		REVISIONS 4	
FIELD CHANGES		FIELD CHANGES	

6
OF 43

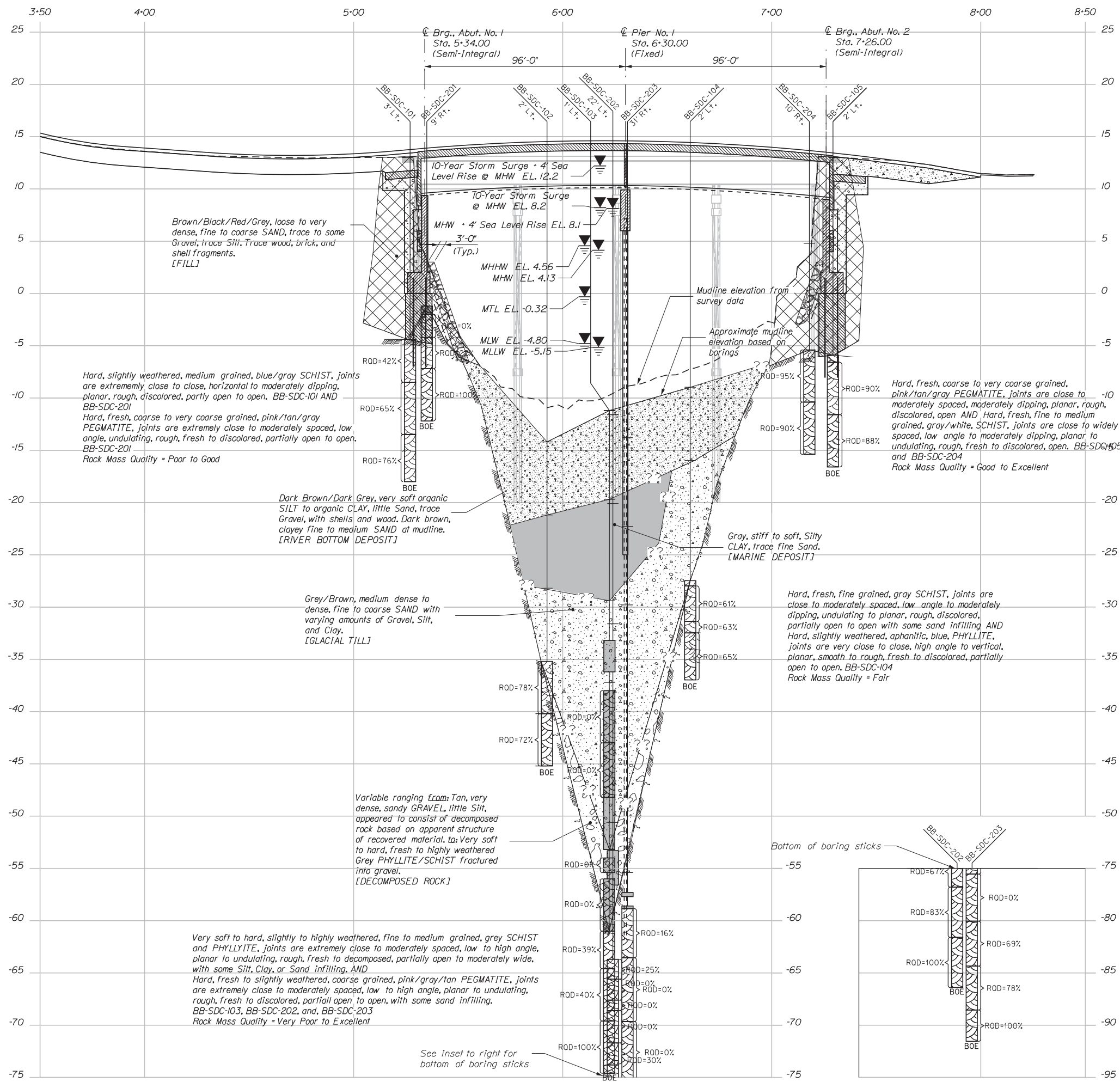
Date: 2/27/2019

Username: dataylor

Division: BRIDGE

Filename: ... \00\BRIDGE\MSTA\007_ISP.DGN

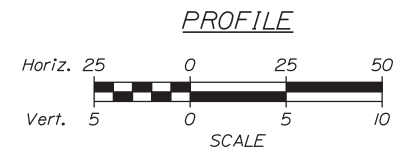
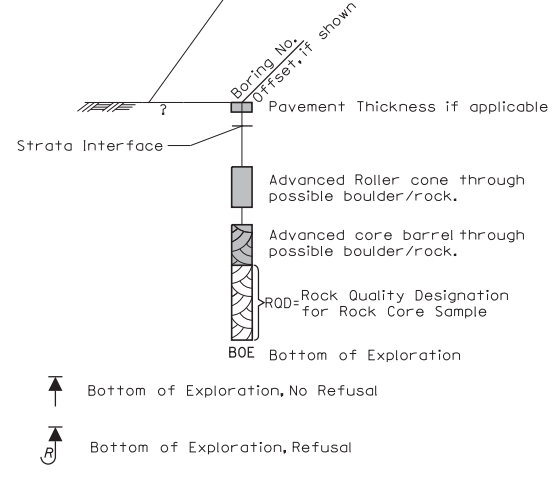
ELEVATION, FEET (NAVD 88)



NOTES

- 1) Base map developed from electronic files provided by Stantec on January 24, 2019 (Files included profiles.dgn).
- 2) The as drilled locations of the test borings were estimated by taping to existing bridge structural elements and should be considered approximate.
- 3) Interpreted top of rock considers general trend of ledge lines from 1933 plans between borings BB-SDC-101 and -102 and between borings BB-SDC-104 and -105.
- 4) This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

LEGEND



THOMPSONS BRIDGE NO. 2848 OVER DECKER COVE
MAINEDOT WIN 18748.00
SOUTHPORT, ME

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: **GZA GeoEnvironmental, Inc.**
Engineers and Scientists
www.gza.com

PREPARED FOR: STANTEC

PROJ MGR: NVW	REVIEWED BY: ARB	CHECKED BY: CLS	FIG
DESIGNED BY: NVW	DRAWN BY: NVW	SCALE: AS SHOWN	3
DATE: 2/21/19	PROJECT NO. 09.0025910.01	REVISION NO. 0	

SHEET NO. 1 OF 1

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-1874(800)
WIN 18748.00
BRIDGE NO. 2848
BRIDGE PLANS

THOMPSONS BRIDGE
DECKER COVE
LINCOLN COUNTY
SOUTHPORT
INTERPRETIVE SUBSURFACE
PROFILE

DATE: FEB 2019
BY: NVW
L. TIMBERLAKE
DESIGN-DETAILED
CHECKED-REVIEWED
DESIGN-DETAILED
DESIGN-DETAILED
REVISIONS 1
REVISIONS 2
REVISIONS 3
REVISIONS 4
FIELD CHANGES

PROF. MANAGER: Andrew R. Blaisdell
SIGNATURE: Andrew R. Blaisdell
P.E. NUMBER: 10957
DATE: 02/21/2019

SHEET NUMBER: 3
OF 43

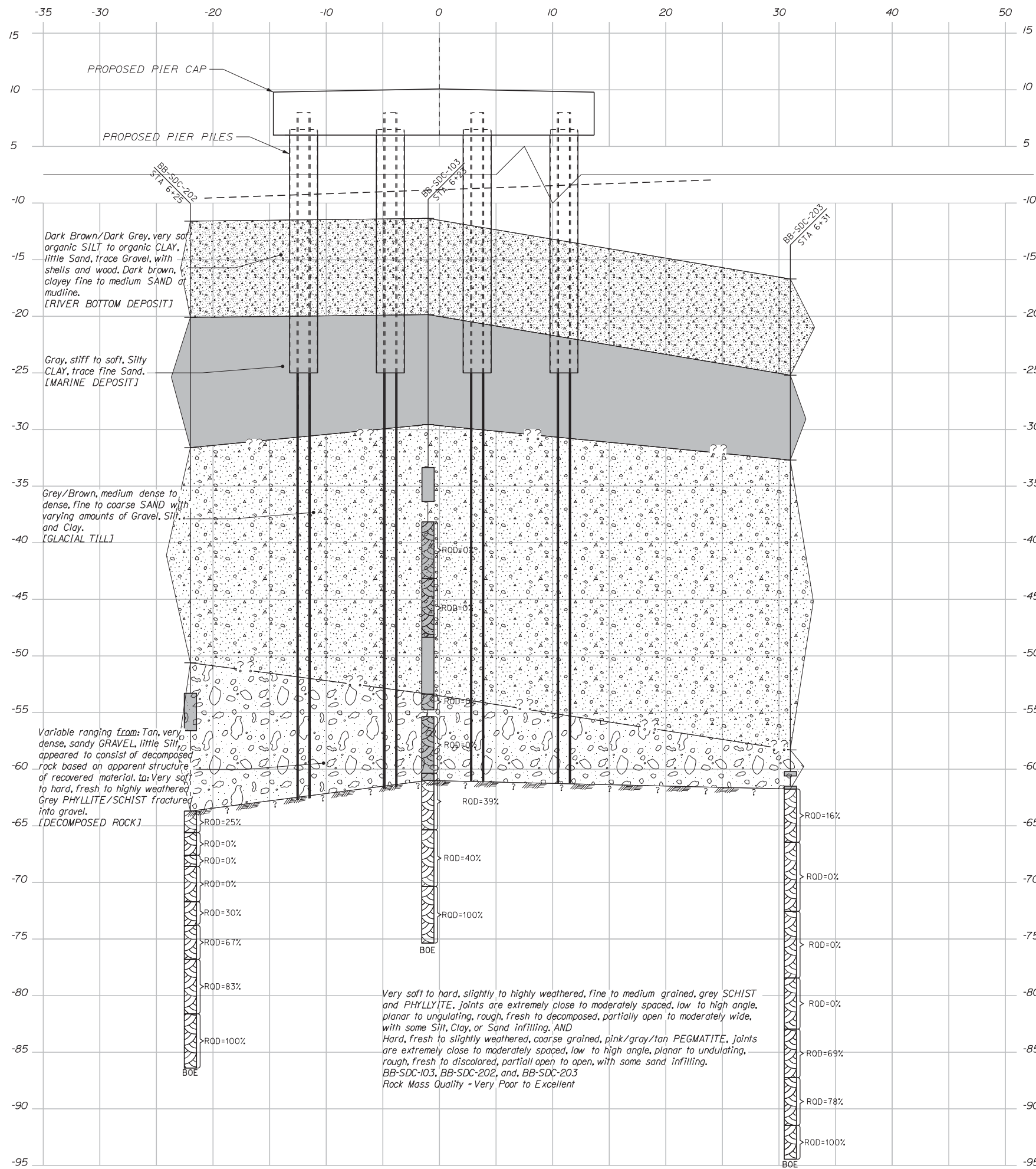
Date: 2/27/2019

Username: dataylor

Division: BRIDGE

Filename: ... \00\BRIDGE\MSTA\007_ISP.DGN

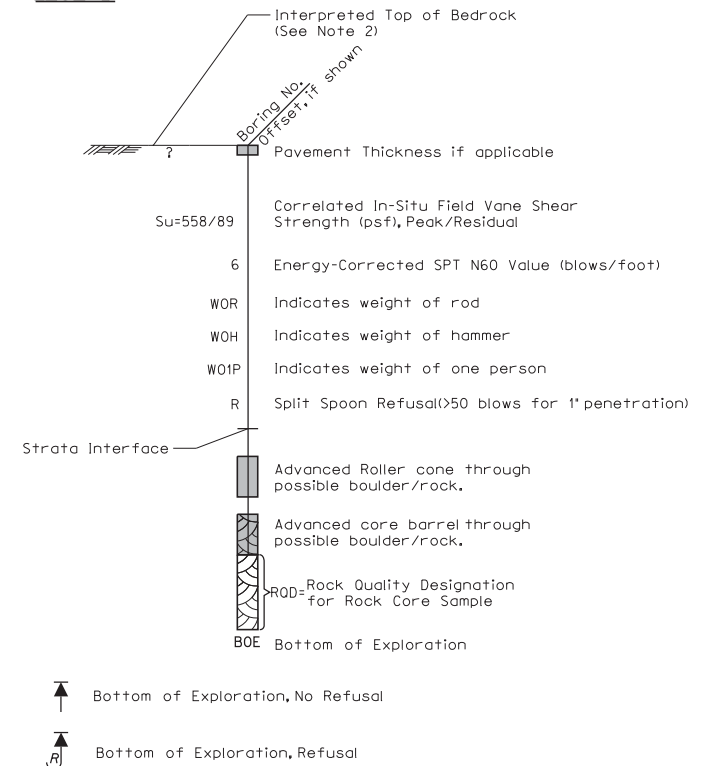
ELEVATION, FEET (NAVD 88)



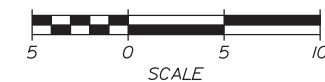
NOTES

- 1) The as drilled locations of the test borings were estimated by taping to existing bridge structural elements and should be considered approximate.
- 2) This generalized interpretive subsurface cross section is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

LEGEND



PROFILE



THOMPSONS BRIDGE NO. 2848 OVER DECKER COVE
MAINEDOT WIN 18748.00
SOUTHPORT, ME

INTERPRETIVE SUBSURFACE CROSS SECTION

PREPARED BY: **GZA GeoEnvironmental, Inc.**
Engineers and Scientists
www.gza.com

PREPARED FOR: STANTEC

PROJ MGR: NVW REVIEWED BY: ARB CHECKED BY: CLS FIG
DESIGNED BY: NVW DRAWN BY: NVW SCALE: AS SHOWN
DATE: 2/21/19 PROJECT NO. 09.0025910.01 REVISION NO. 0 SHEET NO. 4 OF 1

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-1874(800)

THOMPSONS BRIDGE
DECKER COVE
LINCOLN COUNTY
SOUTHPORT

INTERPRETIVE SUBSURFACE
CROSS SECTION

STATE OF MAINE
Andrew R. Blaisdell
10957
PROFESSIONAL ENGINEER

SIGNATURE
10957
P.E. NUMBER
02/21/2019
DATE

BRIDGE NO. 2848
WIN
18748.00
BRIDGE PLANS

SHEET NUMBER
8
OF 43



APPENDIX A – LIMITATIONS



GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the contract documents, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.
3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.
4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.
6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.



7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.
8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.
9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Cost Estimates

11. Unless otherwise stated, our cost estimates are only for comparative and general planning purposes. These estimates may involve approximate quantity evaluations. Note that these quantity estimates are not intended to be sufficiently accurate to develop construction bids, or to predict the actual cost of work addressed in this Report. Further, since we have no control over either when the work will take place or the labor and material costs required to plan and execute the anticipated work, our cost estimates were made by relying on our experience, the experience of others, and other sources of readily available information. Actual costs may vary over time and could be significantly more, or less, than stated in the Report.

Additional Services

12. GZA recommends that we be retained to provide services during any future: site observations, design, implementation activities, construction and/or property development/redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our design concepts and opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design; and iv) assess the consequences of changes in technologies and/or regulations.



APPENDIX B – GZA BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM	
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES		
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	
		GC	Clayey gravels, gravel-sand-clay mixtures.		
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	
		(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):				Desired Rock 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				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 mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))	
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
 Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-101

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	13.0 (est.)	Auger ID/OD:	-
Operator:	M. Porter / B. Enos	Datum:	NAVD 88	Sampler:	SS, R
Logged By:	N. Williams (GZA)	Rig Type:	ATV Mobile Drill Series 2	Hammer Wt./Fall:	140/30
Date Start/Finish:	05-24-16/05-24-16	Drilling Method:	SSA/Drive & Wash	Core Barrel:	NX
Boring Location:	N124365, E1500398	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in.) Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
0									12.6	-ASPHALT-		
	1D	24/12	1.0 - 3.0	15-13-8-7	21	21				Brown/black, dry, medium dense, fine to coarse SAND, little Gravel, little Silt. -FILL- (SM)	G#1 A-1-b, SM	
5	2D	24/10	5.0 - 7.0	1-3-3-40	6	6				Brown/black, dry, loose, fine to medium SAND, little Gravel, trace Silt. -FILL- (SP)		
									76			
									70			
									52			
10	3D	24/3	10.0 - 12.0	9-40-52-11	92	92	39			Tan, wet, very dense, gravelly fine to coarse SAND, trace Silt. Full diameter rock pieces (possible cobbles) in spoon tip. -FILL- (SP)		
									49			
									56			
									107	Possible wood from 13.0'-14.8' bgs. See Remark 2.		
									250			
15	R1	62/43	16.3 - 21.5	RQD = 42%					-3.3	Advanced roller cone to 16.1' bgs, spun casing to 16.3' bgs and set up to core.		
									-4.4	R1: 16.3'-17.4': Apparent cobbles and brick fragments in core recovery. Apparent Top of Rock at 17.4' bgs.	q _p =1650 ksf	
									-17.4	R1: 17.4'-21.5': Hard, slightly weathered, medium grained, blue/grey, SCHIST. Joints are close, horizontal to moderately dipping, planar, rough, discolored, open. Rock Mass Quality = Poor Recovery = 69% Rock Core Times (min/ft): 0.75, 0.75, 1.0, 1.0, 1.15		
20	R2	60/60	21.5 - 26.5	RQD = 65%					-8.5	R2: Hard, slightly weathered, medium grained, blue/grey, SCHIST. Joints are close to moderately spaced, horizontal to moderately dipping, planar, rough, discolored, open. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min/ft): 1.75, 1.0, 1.5, 1.25, 1.25		
25												

Remarks:

- Advanced SSA to 5.0' bgs.
- Drive & Wash HW casing to 14.8' bgs. Casing behavior like driving into wood, from 13.0'-14.8' bgs.
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-101

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	13.0 (est.)	Auger ID/OD:	-
Operator:	M. Porter / B. Enos	Datum:	NAVD 88	Sampler:	SS, R
Logged By:	N. Williams (GZA)	Rig Type:	ATV Mobile Drill Series 2	Hammer Wt./Fall:	140/30
Date Start/Finish:	05-24-16/05-24-16	Drilling Method:	SSA/Drive & Wash	Core Barrel:	NX
Boring Location:	N124365, E1500398	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25												
	R3	54/54	26.5 - 31.0	RQD = 76%								
30												
35												
40												
45												
50												

Remarks:

- Advanced SSA to 5.0' bgs.
- Drive & Wash HW casing to 14.8' bgs. Casing behavior like driving into wood, from 13.0'-14.8' bgs.
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.

Driller: New England Boring Contractors	Elevation (ft.): -14.2 (est.)	Auger ID/OD: -
Operator: M. Porter / B. Enos	Datum: NAVD 88	Sampler: SS, R
Logged By: N. Williams (GZA)	Rig Type: ATV Mobile Drill Series 2	Hammer Wt./Fall: 140/30
Date Start/Finish: 05-27-16/05-27-16	Drilling Method: Drive & Wash/Spin	Core Barrel: NX
Boring Location: N124429, E1500381	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plasticity Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/3	0.0 - 2.0	WOH-WOH-WOH-WOH	0	0	PUSH			Dark grey, very loose, fine to coarse SAND, some Silt, little Gravel, shell fragments. -RIVER BOTTOM DEPOSIT- (SM)	G#2 A-2-4, SM WC=46.9	
	2D	24/12	2.0 - 4.0	WOH-WOH-WOH-WOH	0	0				Dark brown, wet, very soft, organic SILT, some fine Sand, shell fragments. -RIVER BOTTOM DEPOSIT- (OL)	LL=NP PL=NV WC=62.9 OC=4.5	
5	3D	24/5	4.0 - 6.0	WOH-WOH-WOH-WO1P	0	0				Dark grey, wet, very soft, organic SILT, little fine to coarse Sand, trace Gravel, shell and timber fragments. -RIVER BOTTOM DEPOSIT- (OL)		
	4D	24/24	6.0 - 8.0	WOR-WOIP-1-1	1	1			-21.2	Top 12": Dark grey, wet, very soft, organic SILT, little fine to coarse Sand, trace Gravel, shell and timber fragments. -RIVER BOTTOM DEPOSIT- (OL)	LL=37 PL=21 PI=16 WC=39.2	
10	5D	24/24	8.0 - 10.0	WOR-WOR-WOR-WOIP	0	0			-7.0	Bottom 12": grey, wet, medium stiff to stiff, Silty CLAY. PP = 0.5 tsf -MARINE DEPOSIT- (CL) Grey, wet, soft to medium stiff, Silty CLAY. -MARINE DEPOSIT- (CL)	WC=36.9	
	6D	24/24	12.0 - 14.0	WOR-WOR-WOR-WOR	0	0			-28.2	Grey, wet, soft to medium stiff, Silty CLAY. -MARINE DEPOSIT- (CL)		
15										Increased casing resistance at 14.0' bgs, probable granular soil.		
	7D	17/8	17.0 - 18.4	8-6-70/5"	12	12	14		-32.2	Grey, wet, medium dense, fine to coarse SAND, some Silt, little Gravel. Possible cobble/boulder fragments or rock in spoon recovery. -GLACIAL TILL- (SM)	G#7 A-2-4, SM	
20									-18.0	Increased roller cone resistance from 18.0'-21.0' bgs, possible decomposed/weathered rock or cobbles/boulders. Advanced casing to 21.0' bgs and started to core. -DECOMPOSED ROCK-		
	R1	60/60	21.0 - 26.0	RQD = 78%					-35.2			
									-36.1	R1: 21.0'-21.9': Hard, fresh, coarse to very coarse grained, pink/tan/grey, PEGMATITE. Joints are moderately spaced, moderately dipping, fresh to discolored, planar, rough, open.	q _p =1090 ksf	
25									-21.9	R1: 21.9'-26.0': Hard, fresh, medium grained, blue/white/grey, SCHIST. Joints are close to wide, moderately		

Remarks:

- Mudline level at 27.2' below bridge deck.
- Drive & Wash NW casing to 21.0' bgs prior to coring. Drive & Wash HW casing 18.4' bgs. Advanced roller cone to 21.0' bgs ahead of casing.
- PP indicates Pocket Penetrometer used on split spoon recovery; pocket penetrometer unconfined compression strength reported in tons per square foot (tsf). OC in Lab Test Results indicates Organic Content (%).
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
 Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-103

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-11.2 (est.)	Auger ID/OD:	-
Operator:	M. Porter / B. Enos	Datum:	NAVD 88	Sampler:	SS, R
Logged By:	N. Williams (GZA)	Rig Type:	ATV Mobile Drill Series 2	Hammer Wt./Fall:	140/30
Date Start/Finish:	05-25-16/05-26-16	Drilling Method:	Drive & Wash/Spin	Core Barrel:	NX
Boring Location:	N124458, E1500373	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RCD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)				
0	1D	24/0	0.0 - 2.0	WOR-WOR-WO1P-WOH	WO1P		PUSH			No recovery. Minimal resistance during roller cone advancement. Fine sand and shells in wash water return from 2.0'-5.0' bgs.		
5	2D	24/11	5.0 - 7.0	WOR-WOR-WOR-WOR	WOR					Dark brown, wet, soft, organic SILT, little fine Sand, shell and timber fragments. -RIVER BOTTOM DEPOSIT- (OL)	WC=71.3 OC=6.4	
10	3D	24/16	10.0 - 12.0	6-2-1-2	3	3	18			Grey, medium stiff to stiff, Silty CLAY, trace fine Sand. PP = 0.5 tsf -MARINE DEPOSIT- (CL)		
15	4D	24/24	15.0 - 17.0	WOR-WOR-WOH-WOH	WOR		9			Grey, wet, soft, Silty CLAY, trace fine Sand. -MARINE DEPOSIT- (CL)		
20	5D	24/10	20.0 - 22.0	8-5-6-9	11	11	27			Blow counts increased at 18.2' bgs during casing advancement; probable granular material. Grey, wet, medium dense, silty, fine to coarse SAND, trace Gravel. -GLACIAL TILL- (SM) Roller cone advancement experienced increased resistance from 22.0'-25.0' bgs.		

Remarks:

- Mudline level at 24.2' below bridge deck. Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.
- Drive & Wash HW casing to 26.0' bgs. Advanced roller cone and spun NW casing to 26.8' bgs before coring. NW casing advanced to 43.7' bgs after 9D and prior to coring.
- OC in Lab Test Results indicates Organic Content (%).

Driller: New England Boring Contractors	Elevation (ft.): -11.2 (est.)	Auger ID/OD: -
Operator: M. Porter / B. Enos	Datum: NAVD 88	Sampler: SS, R
Logged By: N. Williams (GZA)	Rig Type: ATV Mobile Drill Series 2	Hammer Wt./Fall: 140/30
Date Start/Finish: 05-25-16/05-26-16	Drilling Method: Drive & Wash/Spin	Core Barrel: NX
Boring Location: N124458, E1500373	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	6D	11/6	25.0 - 25.9	13-60/5"	R					Brown, wet, medium dense to dense, fine to coarse SAND, trace Silt, trace Gravel. -GLACIAL TILL- (SP)		
	R1	60/17	26.8 - 31.8	RQD = 0%								
30										R1: 2"-6" Pegmatite and Phyllite rock pieces. Possible cobbles/boulders/decomposed rock. Roller cone advancement from 26.8'-31.8' bgs experienced intermittent higher and lower resistance levels, sand in wash water return. -GLACIAL TILL-		
	R2	62/0	31.8 - 37.0	RQD = 0%								
35										R2: No recovery. Roller cone advancement from 31.8'-37.0' bgs similar to 26.8'-31.8' bgs, grey fine sand in wash water return.		
	7D	24/0	37.0 - 39.0	7-7-11-8	18	18						
40										No recovery. Roller cone advancement from 37.0'-42.0' bgs similar to previous.		
	8D	0/0	42.0 - 42.0	50/0"								
	R3	17/7	42.0 - 43.4	RQD = 0%	R					Roller cone experienced increased resistance at 42.0' bgs. blue and white angular rock fragments in wash water return. Attempted sample 8D; no recovery. Set up to core.		
	9D	4/4	43.4 - 43.7	125/4"	R							
45	R4	60/35	44.0 - 49.0	RQD = 0%						R3: Hard, fresh, grey, PHYLLITE, fractured into gravel, pieces range from .75"-2.0" in size. Rock Core Times (min/ft): 1.5, 2.25 -DECOMPOSED ROCK- 9D: grey, GRAVEL. Slate fragments. -DECOMPOSED ROCK- R4: Moderately hard to hard, slightly weathered, medium grained, grey, PHYLLITE fractured into gravel; pieces range in size from 0.25" to 2.0". -DECOMPOSED ROCK- Very soft to soft, fine grained, SCHIST. -DECOMPOSED ROCK-		
50	R5	60/50	49.0 - 54.0	RQD = 39%						Rock Mass Quality = Very Poor		

Remarks:

- Mudline level at 24.2' below bridge deck. Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.
- Drive & Wash HW casing to 26.0' bgs. Advanced roller cone and spun NW casing to 26.8' bgs before coring. NW casing advanced to 43.7' bgs after 9D and prior to coring.
- OC in Lab Test Results indicates Organic Content (%).

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-103

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-11.2 (est.)	Auger ID/OD:	-
Operator:	M. Porter / B. Enos	Datum:	NAVD 88	Sampler:	SS, R
Logged By:	N. Williams (GZA)	Rig Type:	ATV Mobile Drill Series 2	Hammer Wt./Fall:	140/30
Date Start/Finish:	05-25-16/05-26-16	Drilling Method:	Drive & Wash/Spin	Core Barrel:	NX
Boring Location:	N124458, E1500373	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions:
 D = Split Spoon Sample R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 MD = Unsuccessful Split Spoon Sample attempt SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 U = Thin Wall Tube Sample HSA = Hollow Stem Auger N-uncorrected = Raw field SPT N-value LL = Liquid Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone Hammer Efficiency Factor = Annual Calibration Value PL = Plastic Limit
 V = Insitu Vane Shear Test WOR = weight of 140lb. hammer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50										Rock Core Times (min/ft): 3.5, 1.0, 1.75, 1.75, 1.75 R5: 49.0'-49.8': Decomposed Rock.		
										R5: 49.8'-54.0': Very soft to soft, highly to moderately weathered, medium grained, grey, SCHIST. Joints are very close to close, moderately dipping, undulating, rough, decomposed, open. Sand infilling. Rock Mass Quality = Poor Recovery = 83%		
55	R6	60/60	54.0 - 59.0	RQD = 40%						R6: 54.0'-56.8': Very soft to medium soft, completely to moderately weathered, medium grained, grey, SCHIST. Joints are close, moderately dipping, planar, rough, decomposed, open, with sand infilling.		
										R6: 56.8'-59.0': Hard, slightly weathered, coarse grained, pink/tan/grey, PEGMATITE. Joints are close, moderately dipping, planar, rough, discolored, open. Rock Mass Quality = Poor Recovery = 100%		
60	R7	60/60	59.0 - 64.0	RQD = 100%						R7: Hard, fresh, coarse grained, pink/tan/grey, PEGMATITE. Joints are close to wide, moderately dipping to high angle, undulating, rough, discolored, open, some sand infilling. Rock Mass Quality = Excellent Recovery = 100%		
										Rock Core Times (min/ft): 2.0, 2.25, 2.25, 2.75, 3.5		
65										Bottom of Exploration at 64.00 feet below ground surface.		
70												
75												

Remarks:

- Mudline level at 24.2' below bridge deck. Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.
- Drive & Wash HW casing to 26.0' bgs. Advanced roller cone and spun NW casing to 26.8' bgs before coring. NW casing advanced to 43.7' bgs after 9D and prior to coring.
- OC in Lab Test Results indicates Organic Content (%).

Driller: New England Boring Contractors	Elevation (ft.): -9.0 (est.)	Auger ID/OD: -
Operator: M. Porter / B. Enos	Datum: NAVD 88	Sampler: SS, R
Logged By: N. Williams (GZA)	Rig Type: ATV Mobile Drill Series 2	Hammer Wt./Fall: 140/30
Date Start/Finish: 05-25-16/05-25-16	Drilling Method: Drive & Wash	Core Barrel: NX
Boring Location: N124495, E1500362	Casing ID/OD: 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in.) Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/2	0.0 - 2.0	WOR-WOR-WOR-WOH	0	0	WOH			Black/grey, wet, very soft, sandy organic SILT, shell fragments. -RIVER BOTTOM DEPOSIT- (OL) grey wash water return contained wood, brick, shells, fine sand.	WC=39.4	
5	2D	24/6	5.0 - 7.0	WOH-1-WOR-2	1	1				Dark grey, wet, very loose, fine to coarse SAND, some Silt, little Gravel, shell and wood fragments. -RIVER BOTTOM DEPOSIT- (SM)	G#10 A-2-4, SM WC=46.1	
									-16.0	Casing resistance increased at 7.0' bgs, probable granular material.		
10	3D	24/6	10.0 - 12.0	7-7-13-13	20	20	36			Brown, wet, medium dense, fine to coarse SAND, some Gravel, little Silt. -GLACIAL TILL- (SM)	G#11 A-1-b, SM	
15	4D	24/5	15.0 - 17.0	10-33-14-15	47	47	10			Brown, wet, dense, fine to coarse SAND, some Silt, some Gravel. -GLACIAL TILL- (SM)	G#12 A-2-4, SM	
										Increased resistance at 18.5' bgs. Apparent Top of Rock. Advanced casing and roller cone to 19.0' bgs and started to core.		
20	R1	33/31	19.0 - 21.8	RQD = 61%					-27.5	R1: Hard, fresh, fine grained, grey, SCHIST. Joints are close to moderately spaced, low angle, undulating, rough, discolored, partially open. Rock Mass Quality = Fair Recovery = 94% Rock Core Times (min/ft): 3.0, 1.75, 1.75 R2: 21.8'-23.1': Hard, fresh, coarse grained, grey SCHIST. Joints are close, moderately dipping, planar, rough, discolored, partially open.	q _p =1850 ksf	
	R2	26/26	21.8 - 24.0	RQD = 63%								
25	R3	54/54	24.0 - 28.5	RQD = 65%					-32.1	R2: 23.1'-24.0': Hard, slightly weathered, aphanitic, blue, PHYLLITE. Joints are very close to close, high angle, planar, smooth, fresh, partially open.		

Remarks:

- Mudline level at 22.0' below bridge deck elevation.
- Drive & Wash NW casing to 18.0' bgs.
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine


Boring No.: BB-SDC-104

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-9.0 (est.)	Auger ID/OD:	-
Operator:	M. Porter / B. Enos	Datum:	NAVD 88	Sampler:	SS, R
Logged By:	N. Williams (GZA)	Rig Type:	ATV Mobile Drill Series 2	Hammer Wt./Fall:	140/30
Date Start/Finish:	05-25-16/05-25-16	Drilling Method:	Drive & Wash	Core Barrel:	NX
Boring Location:	N124495, E1500362	Casing ID/OD:	3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25									-35.0	 <p>Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min/ft): 1.75, 2.0, 2.25, 1.75 R3: 24.0'-26.0': Hard, fresh, aphanitic, blue, PHYLLITE. Joints are very close to close, high angle to vertical, planar, rough discolored, open.</p> <p>R3: 24.0'-29.5': Hard, fresh, fine grained, SCHIST. Joints are close to moderately spaced, moderately dipping, planar, rough, discolored, open. Some silt infilling.</p>		
30									-38.5		<p>Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min/ft): 2.0, 2.75</p> <p>Bottom of Exploration at 29.50 feet below ground surface.</p>	
35												
40												
45												
50												

Remarks:

- Mudline level at 22.0' below bridge deck elevation.
- Drive & Wash NW casing to 18.0' bgs.
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.

Driller: New England Boring Contractors	Elevation (ft.): 13.0 (est.)	Auger ID/OD: -
Operator: M. Porter / B. Enos	Datum: NAVD 88	Sampler: SS, R
Logged By: N. Williams (GZA)	Rig Type: ATV Mobile Drill Series 2	Hammer Wt./Fall: 140/30
Date Start/Finish: 05-24-16/05-24-16	Drilling Method: SSA/Drive & Wash/Spin	Core Barrel: NX
Boring Location: N124561, E1500343	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plasticity Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
5	1D	24/10	1.0 - 3.0	9-6-7-7	13	13				Red-brown, dry, medium dense, fine to coarse SAND, little Gravel, trace Silt. -FILL- (SW-SM)	G#13 A-1-b, SW-SM	
	2D	24/6	5.0 - 7.0	2-1-2-4	3	3	7			Red-brown, dry, very loose, fine to medium SAND, trace Gravel, trace Silt. -FILL- (SP-SM)	G#14 A-3, SP-SM	
							10					
10							16					
							20					
	3D	24/8	10.0 - 12.0	9-8-16-84	24	24	15			White/grey/tan, wet, medium dense, GRAVEL, some fine to medium Sand, trace Silt, angular Gravel, several full diameter pieces (probable cobbles/boulders). -FILL (POSSIBLE ROCK FILL)- (GP)		
15							32					
							32					
							79					
20							50					
							42					
	4D	24/5	15.0 - 17.0	9-3-2-3	5	5	40			Black and grey, wet, loose, GRAVEL, little fine to medium grained Sand. -FILL- (GP)		
25							18					
							26					
	R1	60/60	19.6 - 24.6	RQD = 90%					-5.9	Casing refusal at 18.9' bgs. Advanced roller cone to approximately 19.2' bgs, spun NW casing to 19.6' bgs and started to core. Apparent Top of Rock at 18.9' bgs.	q _p =1740 ksf	
25							36					
									-8.1	R1: 19.6'-21.1': Hard, fresh, coarse to very coarse grained, pink/tan/grey, PEGMATITE. Joints are close to moderately spaced, moderately dipping, planar, rough, discolored, open.		
	R2	60/60	24.6 - 29.6	RQD = 88%						R1: 21.1'-24.6': Hard, fresh, fine to medium grained, grey and white, SCHIST. Joints are moderately to widely spaced, low angle, planar, rough, fresh to discolored, open. Rock Mass Quality = Good Recovery = 100% Rock Core Times (min/ft): 2.0, 1.5, 1.5, 1.5, 1.5	q _p =1360 ksf	

Remarks:

- Advanced SSA through asphalt 5" thick, continued with SSA to 5.0' bgs.
- Drove HW casing to 14.2' bgs. Spun NW casing to 19.6' prior to coring.
- Approximate boring locations were determined by taping from existing bridge structural elements. Coordinates were estimated to the nearest foot using measured offset distances in the plans. Ground surface/mudline elevations were estimated based on an assumed bridge deck and approach road elevation of El. 13'.

Driller: New England Boring Contractors	Elevation (ft.): 4.7 (est.)	Auger ID/OD:
Operator: Brad Enos	Datum: NAVD 88	Sampler: Split Spoon
Logged By: B. Woodman (GZA)	Rig Type: ATV-Mounted Mobile Drill B-23	Hammer Wt./Fall: 140/30
Date Start/Finish: 12/27/18-12/27/18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N124376, E1500407	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.869 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0										-RIPRAP-		
2.5	1D	24/6	2.2 - 4.2	22-17-30-26	47	68		2.5		Brown/tan, moist, very dense, Gravelly SAND. (Sample 1D taken using a 3" spoon.) -FILL- (GP)		
5								-1.2		Practical refusal during advancement of 4" casing at 5.9'. Apparent bedrock at 5.9'. Advance roller cone to 6.7', and set up to core.		
	R1	26/14	6.7 - 8.9	RQD = 0%				-2.6		R1: 6.7'-7.3': Hard, fresh, coarse to very coarse grained, pink/tan/black, PEGMATITE. Joints are extremely close to very close, low angle, undulating, rough, discolored, open.	q _p =1057 ksf	
	R2	36/24	8.9 - 11.9	RQD = 22%				-7.3		R1: 7.3'-8.9': Hard, slightly weathered, fine grained, blue/grey, SCHIST. Joints are extremely close, low angle to moderately dipping, undulating, rough, discolored, fractured into gravel.		
10								-7.2		Rock Mass Quality = Very Poor Recovery = 54% Rock Core Times (min:sec): 6.7-7.7' (1:50), 7.7-8.7' (2:30)		
	R3	60/60	11.9 - 16.9	RQD = 100%				-11.9		R2: Hard, fresh to slightly weathered, medium grained, grey/blue, SCHIST. Joints are extremely close to close, moderately dipping, undulating, rough, fresh to discolored, open, trace silt infilling. Rock Mass Quality = Very Poor Recovery = 67% Rock Core Times (min:sec): 8.9-9.9' (1:23), 9.9-10.9' (1:40), 10.9-11.9' (1:56)		
										R3: Hard, fresh, coarse to very coarse grained, pink/tan/grey, PEGMATITE. Joints are close to moderately spaced, low angle, undulating, rough, fresh, partially open. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 11.9-12.9' (0:56), 12.9-13.9' (1:19), 13.9-14.9' (1:17), 14.9-15.9' (1:17), 15.9-16.9' (1:22)		
25												

Remarks:

- Mudline level at 8.3' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Automatic Hammer: NEBC Drill Rig No. 23, calculated hammer efficiency factor = 0.869.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-201

PIN: 018748.00

Driller: New England Boring Contractors	Elevation (ft.): 4.7 (est.)	Auger ID/OD:
Operator: Brad Enos	Datum: NAVD 88	Sampler: Split Spoon
Logged By: B. Woodman (GZA)	Rig Type: ATV-Mounted Mobile Drill B-23	Hammer Wt./Fall: 140/30
Date Start/Finish: 12/27/18-12/27/18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N124376, E1500407	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.869 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25									-20.4		Bottom of Exploration at 25.10 feet below ground surface.	
30												
35												
40												
45												
50												

Remarks:

- Mudline level at 8.3' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Automatic Hammer: NEBC Drill Rig No. 23, calculated hammer efficiency factor = 0.869.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-202

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-11.6 (est.)	Auger ID/OD:	--
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	N. Williams (GZA)	Rig Type:	Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall:	140/30
Date Start/Finish:	11/13/18-11/14/18	Drilling Method:	Drive & Wash	Core Barrel:	NQ
Boring Location:	N124454, E1500352	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions:
 R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows ((6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
0	1D	24/8	0.0 - 2.0	WOR-WOR-WOR-WOR	WOR		PUSH		Dark brown, wet, very loose, Clayey fine to medium SAND, trace Gravel, organics, shells. -RIVER BOTTOM DEPOSIT- (SC)	G#25 A-4, SC WC=77.2	
5	2D V1 V2	24/8 5/5 5/5	5.0 - 7.0 5.6 - 6.0 6.6 - 7.0	PUSH S _u =357/112 psf S _u =402/89 psf	--				Dark brown, wet, soft, organic SILT, little fine Sand, little Clay, shells, wood. -RIVER BOTTOM DEPOSIT- (OH) 55 x 110 mm In Situ Vane Shear Tests: V1: 8/2.5 ft-lbs V2: 9/2 ft-lbs	LL=80 PL=40 PI=40 WC=72.8	
10	3D	24/24	10.0 - 12.0	WOH-WOH-WOH-2	WOH				Grey, wet, soft, Silty CLAY, trace fine Sand. -MARINE DEPOSIT- (CL)	WC=33.8	
15	4D V3 V4	24/24 5/5 5/5	15.0 - 17.0 15.6 - 16.0 16.6 - 17.0	PUSH S _u =201/45 psf S _u =223/45 psf	--				Grey, wet, soft, Silty CLAY, trace fine Sand. -MARINE DEPOSIT- (CL) 55 x 110 mm In Situ Vane Shear Tests: V3: 4.5/1 ft-lbs V4: 5/1 ft-lbs	WC=36.1	
20	5D	24/14	20.0 - 22.0	WOH-WOH-7-4	7	7	18		20.0'-21.0': Grey, wet, soft, Silty CLAY, trace fine Sand. -MARINE DEPOSIT- (CL) 21.0'-22.0': Grey, wet, Silty fine to medium SAND, trace coarse Sand. -GLACIAL TILL- (SM)		
25							40				

Remarks:

- Mudline level at 24.6' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Coarse sand in wash return at 20.0'.
- Attempted to push vane at 20.0', could not advance vane.

Driller: New England Boring Contractors	Elevation (ft.): -11.6 (est.)	Auger ID/OD: --
Operator: Brad Enos	Datum: NAVD 88	Sampler: Split Spoon
Logged By: N. Williams (GZA)	Rig Type: Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall: 140/30
Date Start/Finish: 11/13/18-11/14/18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N124454, E1500352	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger N-uncorrected = Raw field SPT N-value LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone Hammer Efficiency Factor = Annual Calibration Value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOR = weight of 140lb. hammer N₆₀ = SPT N-uncorrected corrected for hammer efficiency PI = Plasticity Index
 V = Insitu Vane Shear Test WO1P = Weight of one person G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	6D	24/4	25.0 - 27.0	50-34-5-6	39	39					Grey, wet, dense, Silty fine to medium SAND. -GLACIAL TILL- (SM)	
30	7D	24/10	30.0 - 32.0	14-9-11-11	20	20					Grey, wet, medium dense, Silty fine to medium SAND, trace Gravel. -GLACIAL TILL- (SM)	
35	8D	24/10	35.0 - 37.0	10-11-10-13	21	21					Grey, wet, medium dense, fine to medium SAND, some Silt. -GLACIAL TILL- (SM)	
40	9D	20/8	40.0 - 41.7	24-13-46-75/2"	59	59	RC				Tan, wet, very dense, Sandy GRAVEL, little Silt. -DECOMPOSED ROCK- Intermittent increases in resistance during roller cone advancement from 41.7'-45.0'.	
45	10D	8/7	45.0 - 45.7	33-75/2"	R						Tan, wet, very dense, Sandy GRAVEL, little Silt. -DECOMPOSED ROCK-	
50	12D	5/5	49.0 - 49.4	125/5"	R						Tan, wet, very dense, Sandy GRAVEL, little Silt. -DECOMPOSED ROCK-	

Remarks:

- Mudline level at 24.6' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Coarse sand in wash return at 20.0'.
- Attempted to push vane at 20.0', could not advance vane.

Driller: New England Boring Contractors	Elevation (ft.): -11.6 (est.)	Auger ID/OD: --
Operator: Brad Enos	Datum: NAVD 88	Sampler: Split Spoon
Logged By: N. Williams (GZA)	Rig Type: Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall: 140/30
Date Start/Finish: 11/13/18-11/14/18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N124454, E1500352	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50										Advanced roller cone to 52.1' where increase in roller cone resistance observed.		
	13D R1	1/0 23/11	52.1 - 52.2 52.5 - 54.4	130/1" RQD = 25%	R		NQ2	-64.1 -64.3		No recovery. Advanced roller cone and 3" casing to 52.5'. Set up to core.		
	R2	24/24	54.4 - 56.4	RQD = 0%						R1: Top 2": Moderately weathered Gravel pieces of PEGMATITE.		
55										R1: Bottom 9": Hard, moderately weathered, fine grained, grey, SCHIST. Joints are closely spaced, moderately dipping, rough, disintegrated. Sand infilling. Low recovery makes it difficult to see joint spacing. Rock Mass Quality = Very Poor Recovery = 48% Rock Core Times (min:sec): 52.5-53.5' (1:15), 53.5-54.4' (2:45)		
	R3	12/12	56.4 - 57.4	RQD = 0%								
	R4	37/20	57.4 - 60.5	RQD = 0%								
60										R2: Hard, slightly weathered, fine grained, grey, PHYLLITE. Joints are extremely close to close, high angle, planar, rough, discolored, open, clay infilling from 55.4'-56.2'. Highly fractured into gravel pieces. Rock Mass Quality = Very Poor Recovery = 100% Rock Core Times (min:sec): 54.4-55.4' (1:43), 55.4-56.4' (4:02)		
	R5	25/25	60.5 - 62.6	RQD = 30%				-72.1				
	R6	36/36	62.6 - 65.6	RQD = 67%				-74.2		R3: Medium hard, moderately weathered, fine grained, grey, SCHIST. Joints are extremely close to very close, high angle, planar / undulating, rough, discolored / disintegrated, moderately wide (gravel pieces). Rock Mass Quality = Very Poor Recovery = 100%		
65										R4: Soft, moderately to highly weathered, fine grained, grey, SCHIST. Joints are extremely close to close, moderately dipping, undulating, rough, decomposed / disintegrated, moderately wide. Rock Mass Quality = Very Poor Recovery = 54% Rock Core Times (min:sec): 56.4-57.4' (2:32)		
	R7	58/58	65.6 - 70.4	RQD = 83%				-77.2				
										R5: Hard, slightly weathered, coarse grained, grey/pink/orange, PEGMATITE. Joints are extremely close to close, high angle, planar, rough, discolored, partially open to open. Moderately weathered rock from 62.3'-62.6'. Rock Mass Quality = Poor Recovery = 100% Rock Core Times (min:sec): 57.4-58.4' (2:57), 58.4-59.4' (1:45), 59.4-60.4 (2:10)		
70												
	R8	58/58	70.4 - 75.2	RQD = 100%								
										R6: Hard, slightly weathered, medium grained, grey, SCHIST. Joints are very close to moderately spaced,		
75												

Remarks:

- Mudline level at 24.6' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Coarse sand in wash return at 20.0'.
- Attempted to push vane at 20.0', could not advance vane.

Driller: New England Boring Contractors	Elevation (ft.): -11.6 (est.)	Auger ID/OD: --
Operator: Brad Enos	Datum: NAVD 88	Sampler: Split Spoon
Logged By: N. Williams (GZA)	Rig Type: Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall: 140/30
Date Start/Finish: 11/13/18-11/14/18	Drilling Method: Drive & Wash	Core Barrel: NQ
Boring Location: N124454, E1500352	Casing ID/OD: 4/4.5", 3/3.5"	Water Level*: Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
75									-86.8		moderately dipping, planar, rough, fresh to discolored, partially open to moderately wide, with silt infilling. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec): 62.6-63.6' (1:15), 63.6-64.6' (1:31), 64.6-65.6' (1:15)	
											R7: Hard, slightly weathered, coarse grained, grey/pink/orange, PEGMATITE. Joints are very close to moderately spaced, moderately dipping, planar, rough, discolored, partially open to open, with sand infilling. Rock Mass Quality = Good Recovery = 100% Rock Core Times (min:sec): 65.6-66.6' (1:07), 66.6-67.6' (2:39), 67.6-68.6' (3:22), 68.6-69.6' (3:21), 69.6-70.4' (4:54)	
											R8: Very hard, fresh, coarse grained, grey/pink/orange, PEGMATITE. Joints are close to moderately spaced, low angle, undulating, rough, fresh, open. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 70.4-71.4' (3:09), 71.4-72.4' (3:45), 72.4-73.4' (4:48), 73.4-74.4' (5:58), 74.4-75.2' (5:08)	
											Bottom of Exploration at 75.20 feet below ground surface.	
100												

Remarks:

- Mudline level at 24.6' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.
- Coarse sand in wash return at 20.0'.
- Attempted to push vane at 20.0', could not advance vane.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-203

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-16.7 (est.)	Auger ID/OD:	--
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	B. Woodman (GZA)	Rig Type:	Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall:	140/30
Date Start/Finish:	11/14/18-11/15/18	Drilling Method:	Drive & Wash	Core Barrel:	NQ
Boring Location:	N124475, E1500402	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
0	1D	24/2	0.0 - 2.0	WOR-WOR-WOR-WOR	WOR		PUSH		Dark brown, wet, very loose, Clayey fine to medium SAND, trace Gravel, with organics, shells. -RIVER BOTTOM DEPOSIT- (SC)	G#25 A-4, SC WC=77.2	
5	2D V1 V2	24/20 5/5 5/5	5.0 - 7.0 5.6 - 6.0 6.6 - 7.0	PUSH S _u =558/89 psf S _u =553/125 psf	--				Dark brown, wet, very soft, organic CLAY, little fine Sand. Wood fragments in spoon. -RIVER BOTTOM DEPOSIT- (OH) 55 x 110 mm In Situ Vane Shear Tests: V1: 12.4/2.0 ft-lbs V2: 12.4/2.75 ft-lbs	LL=73 PL=33 PI=40 WC=66.3	
10	3D V3 MV	24/24 5/5 0/0	10.0 - 12.0 10.6 - 11.0 11.0 - 11.0	PUSH S _u =961/27 psf	--				Gravel pieces in wash return. Grey, wet, very soft, Silty CLAY. -MARINE DEPOSIT- (CL) 65 x 130 mm In Situ Vane Shear Tests: V3: 35.0/1.0 ft-lbs Unable to advance vane after V3.	LL=33 PL=19 PI=14 WC=34.3	
15	4D MV	24/18 0/0	15.0 - 17.0 15.0 - 15.0	3-1-2-11	3	3	52		Increase in resistance during roller cone advancement from 14.0'-15.0'. 15.0'-16.0': Grey, wet, Silty CLAY. -MARINE DEPOSIT- (CL) Attempted vane at 15.0'. Could not advance vane. 16.0'-17.0': Grey, wet, fine to coarse SAND, some Silt, little Gravel.		
20	5D	24/8	20.0 - 22.0	63-17-16-10	35	35	R/C		Grey, wet, dense, Silty fine to coarse SAND, some Gravel. -GLACIAL TILL- (SM)		
25											

Remarks:

1. Mudline level at 29.2' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine

Boring No.: BB-SDC-203

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-16.7 (est.)	Auger ID/OD:	--
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	B. Woodman (GZA)	Rig Type:	Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall:	140/30
Date Start/Finish:	11/14/18-11/15/18	Drilling Method:	Drive & Wash	Core Barrel:	NQ
Boring Location:	N124475, E1500402	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50										(1:21), 47.1-48.1' (2:07), 48.1-49.1' (1:43), 49.1-49.9' (1:45) R2: Very soft, highly weathered, medium grained SCHIST. Joints are extremely close to close, low angle undulating, rough, decomposed, open. Sand infilling. Rock Mass Quality = Very Poor Recovery = 53% Rock Core Times (min:sec): 49.9-50.9' (0:38), 50.9-51.9' (2:21), 51.9-52.9' (2:20), 52.9-53.9' (1:13), 53.9-54.9' (1:37), 54.9-55.9' (2:00)		
55	R3	58/28	55.9 - 60.7	RQD = 0%						R3: Very soft, highly weathered, medium grained SCHIST. Joints are extremely close to close, low angle to moderately dipping, undulating, rough, decomposed, open. Sand infilling. Rock Mass Quality = Very Poor Recovery = 48% Rock Core Times (min:sec): 55.9-56.9' (1:14), 56.9-57.9' (0:53), 57.9-58.9' (1:30), 58.9-59.9' (2:31), 59.9-60.7' (2:30)		
60	R4	54/12	60.7 - 65.2	RQD = 0%				-77.4		R4: Hard, slightly weathered, coarse grained, grey/blue/pink, PEGMATITE. Joints are extremely close to close, moderately dipping to high angle, undulating, rough, discolored, open. Top 6" fractured into Gravel. Rock Mass Quality = Very Poor Recovery = 22% Rock Core Times (min:sec): 60.7-61.7' (1:00), 61.7-62.7' (1:58), 62.7-63.7' (1:47), 63.7-64.7' (0:53), 64.7-65.2' (1:20)		
65	R5	54/54	65.2 - 69.7	RQD = 69%						R5: Hard, fresh, coarse grained, grey/ blue/pink, PEGMATITE. Joints are close to moderately spaced, moderately dipping, undulating / planar, rough, fresh to discolored, partially open. Rock Mass Quality = Fair Recovery = 100% Rock Core Times (min:sec): 65.2-66.2' (3:05), 66.2-67.2' (2:51), 67.2-68.2' (4:10), 68.2-69.2' (4:16)		
70	R6	54/54	69.7 - 74.2	RQD = 78%						R6: Hard, fresh, coarse grained, grey/ blue/pink, PEGMATITE. Joints are close to moderately spaced, low angle, undulating, rough, fresh to discolored, partially open. Rock Mass Quality = Good Recovery = 100% Rock Core Times (min:sec): 69.7-70.7' (2:08), 70.7-71.7' (5:22), 71.7-72.7' (8:10), 72.7-73.7' (3:49), 73.7-74.2' (3:29)		
75	R7	36/36	74.2 - 77.2	RQD = 100%						R7: Hard, fresh, coarse grained, grey/ blue/pink, PEGMATITE. Joints are close to moderately spaced, low		

Remarks:

1. Mudline level at 29.2' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Thompson Bridge #2848
Over Decker Cove

Location: Southport, Maine


Boring No.: BB-SDC-203

PIN: 018748.00

Driller:	New England Boring Contractors	Elevation (ft.)	-16.7 (est.)	Auger ID/OD:	--
Operator:	Brad Enos	Datum:	NAVD 88	Sampler:	Split Spoon
Logged By:	B. Woodman (GZA)	Rig Type:	Barge-Mounted Mobile Drill B-23	Hammer Wt./Fall:	140/30
Date Start/Finish:	11/14/18-11/15/18	Drilling Method:	Drive & Wash	Core Barrel:	NQ
Boring Location:	N124475, E1500402	Casing ID/OD:	4/4.5", 3/3.5"	Water Level*:	Tidal

Hammer Efficiency Factor: 0.6 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) W_C = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger N-uncorrected = Raw field SPT N-value LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone Hammer Efficiency Factor = Annual Calibration Value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOR = weight of rods N₆₀ = SPT N-uncorrected corrected for hammer efficiency PI = Plasticity Index
 V = Insitu Vane Shear Test WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
75									-94.5		angle, undulating, rough, fresh to discolored, partially open. Rock Mass Quality = Excellent Recovery = 100% Rock Core Times (min:sec): 74.2-75.2' (7:11), 75.2-76.2' (6:41), 76.2-77.2' (9:32)	
80											Bottom of Exploration at 77.80 feet below ground surface.	
85												
90												
95												
100												

Remarks:

1. Mudline level at 29.2' below bridge deck. Ground surface/mudline elevations were estimated based on bridge deck elevation of El. 13' from previous MaineDOT survey. Boring coordinates were estimated based on tape ties to existing elements of the bridge and should be considered approximate.



APPENDIX C – ROCK CORE PHOTOGRAPHS



**Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-101	R1	16.3 - 21.5	43	69	15	42	ROCK/BRICK FRAGMENTS, SCHIST	1
BB-SDC-101	R2	21.5 - 26.5	60	100	39	65	SCHIST	1,2



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-101	R3	26.5 - 31.0	54	100	41	76	SCHIST	1



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-102	R1	21.0 - 26.0	60	100	47	78	PEGMATITE, SCHIST	1
BB-SDC-102	R2	26.0 - 31.0	60	100	43	72	PEGMATITE, SCHIST	2

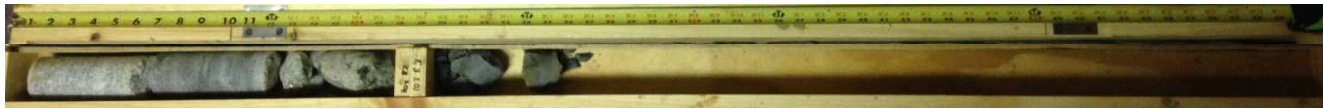


Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-103	R1	26.8 - 31.8	17	28	---	---	BOULDER/COBBLE PIECES	1
BB-SDC-103	R2	31.8 - 37.0	0	0	---	---	NA	--
BB-SDC-103	R3	42.0 - 43.4	0	0	---	---	NA	--



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
2. Top photo is dry, bottom photo is wet.



**Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-103	R4	44.0 - 49.0	35	58	0	0	PHYLLITE, SCHIST	1
BB-SDC-103	R5	49.0 - 54.0	50	83	23	39	SCHIST	1,2
BB-SDC-103	R6	54.0 - 59.0	60	100	24	40	SCHIST, PEGMATITE	2,3
BB-SDC-103	R7	59.0 - 64.0	60	100	60	100	PEGMATITE	4

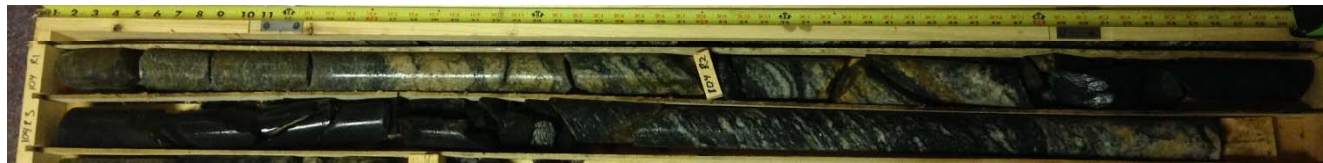


- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
 2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-104	R1	19.0 - 21.8	31	94	20	61	SCHIST	1
BB-SDC-104	R2	21.8 - 24.0	26	100	17	63	SCHIST, PHYLLITE	1
BB-SDC-104	R3	24.0 - 28.5	54	100	35	65	PHYLLITE, SCHIST	2



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 3=Bottom.
 2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-105	R1	19.6 - 24.6	60	100	54	90	PEGMATITE, SCHIST	1
BB-SDC-105	R2	24.6 - 29.6	60	100	53	88	SCHIST, PEGMATITE	2



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-201	R1	6.7 - 8.9	14	54	0	0	PEGMATITE, SCHIST	1
BB-SDC-201	R2	8.9 - 16.9	24	67	8	22	SCHIST	1
BB-SDC-201	R3	11.9 - 16.9	60	100	60	100	PEGMATITE	1,2



- Notes:
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



**Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-202	R1	52.5 - 54.4	11	48	6	25	PEGMATITE, SCHIST	1
BB-SDC-202	R2	54.4 - 56.4	24	100	0	0	PHYLLITE	1
BB-SDC-202	R3	56.4 - 57.4	12	100	0	0	SCHIST	1
BB-SDC-202	R4	57.4 - 60.5	20	54	0	0	SCHIST	2
BB-SDC-202	R5	60.5 - 62.6	25	100	8	30	PEGMATITE	2
BB-SDC-202	R6	62.6 - 65.6	36	100	24	67	SCHIST	3
BB-SDC-202	R7	65.6 - 70.4	58	100	48	83	PEGMATITE	4



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Photo is wet.



**Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs**

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-202	R8	70.4 - 75.2	58	100	58	100	PEGMATITE	1
BB-SDC-203	R1	45.1 - 49.9	29	51	9	16	SCHIST	2
BB-SDC-203	R2	49.9 - 55.9	38	53	0	0	SCHIST	2,3
BB-SDC-203	R3	55.9 - 60.7	28	48	0	0	SCHIST	3
BB-SDC-203	R4	60.7 - 65.2	12	22	0	0	PEGMATITE	3
BB-SDC-203	R5	65.2 - 69.7	54	100	37	69	PEGMATITE	4

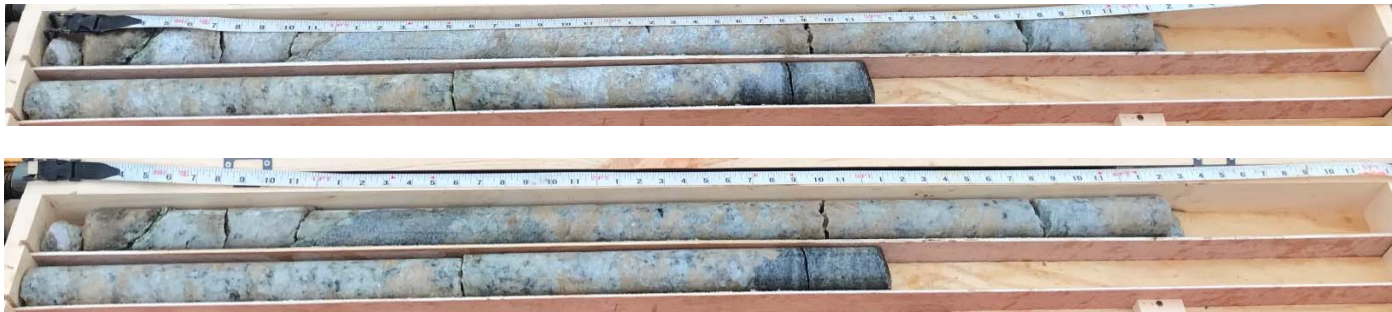


- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-203	R6	69.7 - 74.2	54	100	42	78	PEGMATITE	1
BB-SDC-203	R7	74.2 - 77.2	36	100	36	100	PEGMATITE	2



- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



Thompson Bridge No. 2848 over Decker Cove
MaineDOT WIN 18748.00, Southport, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-SDC-204	R1	10.8 - 15.8	60	100	57	95	PEGMATITE	1
BB-SDC-204	R2	15.8 - 20.8	60	100	54	90	PEGMATITE, SCHIST	2



Notes: 1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
2. Top photo is dry, bottom photo is wet.



APPENDIX D – LABORATORY TEST RESULTS



State of Maine - Department of Transportation
Laboratory Testing Summary Sheet

**Thompson Bridge
 over Decker Cove**

MDOT Project Number:

GZA Project Number: 09.0025887.00

Town(s): Southport, ME

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Lab Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-SDC-101		1D	1.0-3.0	1					SM	A-1-b	II
BB-SDC-102		1D	0.0-2.0	2		46.9			SM	A-2-4	III
BB-SDC-102		2D	2.0-4.0	3	4.48	62.9	NP	NV			
BB-SDC-102		4D	7.0-8.0	4		32.2	37	21			
BB-SDC-102		5D	8.0-10.0	5		39.2					
BB-SDC-102		6D	12.0-14.0	6		36.9					
BB-SDC-102		7D	17.0-18.4	7					SM	A-2-4	III
BB-SDC-103		2D	5.0-7.0	8	6.35	71.3					
BB-SDC-104		1D	0.0-2.0	9		39.4					
BB-SDC-104		2D	5.0-7.0	10		46.1			SM	A-2-4	III
BB-SDC-104		3D	10.0-12.0	11					SM	A-1-b	III
BB-SDC-104		4D	15.0-17.0	12					SM	A-2-4	III
BB-SDC-105		1D	1.0-3.0	13					SW-SM	A-1-b	I
BB-SDC-105		2D	5.0-7.0	14					SP-SM	A-3	I

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MDOT and Corps of Engineers Classification Systems.

- GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)
- WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98
- LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98
- PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

ORGANIC CONTENT DATA SHEET

Project Name: Thompson Bridge Tested By RR Date 08.01.16
over Decker Cove Calculated By RR Date 08.02.16
 Project No. 09.0025910.00 Reviewed By MJC Date 08.03.16

Lab No.	Boring	Sample	Depth	Cup #	Wt. Cup (gms)	Wt. Wet Soil & Cup (gms)	Wt. Dry Soil & Cup (gms)	Wt. Ash & Cup (gms) 440°C	Water Content %	Ash %	Organic %
L3	BB-SDC-102	2D	2-4	Y	114.52	193.89	193.48	189.94	0.52	95.52	4.48
L-8	BB-SDC-103	2D	5-7	Z	133.25	217.39	216.45	211.17	1.13	93.654	6.35

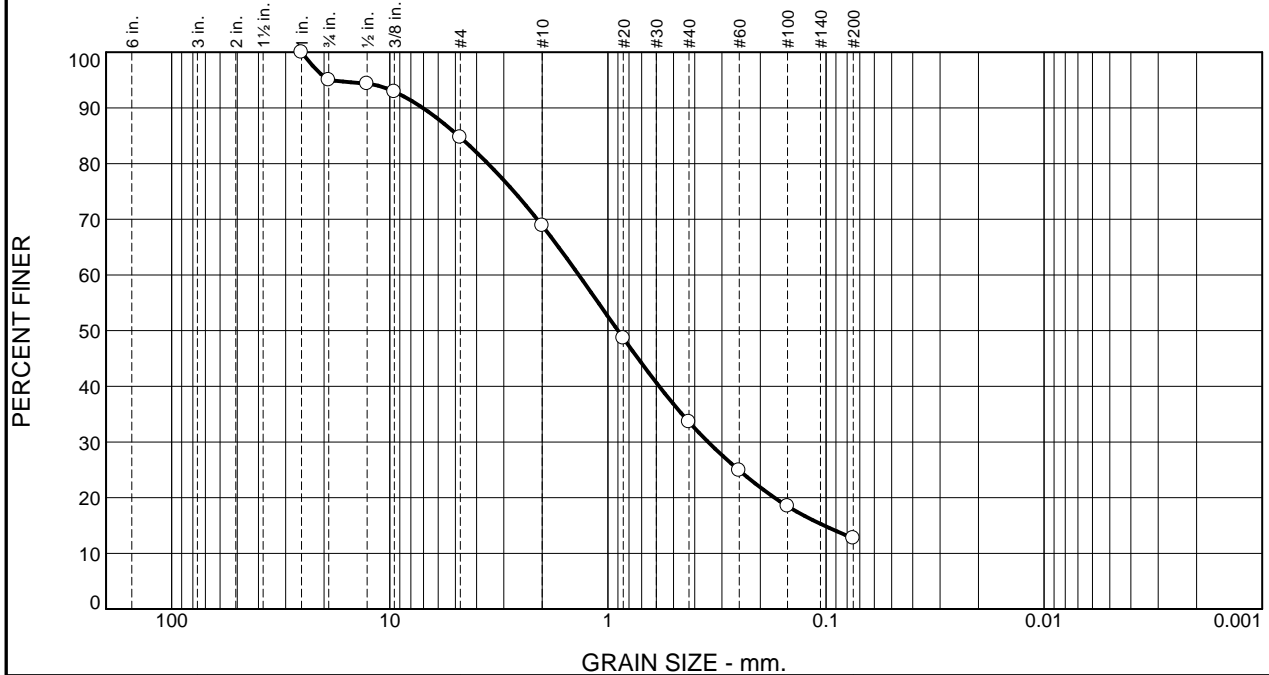
Moisture Content = $100 \times (\text{wt.wet soil\&cup} - \text{wt.dry soil\&cup}) / (\text{wt.dry soil\&cup} - \text{wt.cup})$

Ash % = $100 \times (\text{wt.ash\&cup} - \text{wt.cup}) / (\text{wt.dry soil\&cup} - \text{wt.cup})$

Organic % = $100 - \text{Ash \%}$



Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.0	10.3	15.8	35.3	20.9	12.7	

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	95.0		
.5"	94.3		
.375"	92.9		
#4	84.7		
#10	68.9		
#20	48.7		
#40	33.6		
#60	24.9		
#100	18.5		
#200	12.7		

* (no specification provided)

Material Description

Dark Brown SAND, little Gravel, little Silt (SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 7.0565 D₈₅= 4.8438 D₆₀= 1.3606
D₅₀= 0.8991 D₃₀= 0.3472 D₁₅= 0.1021
D₁₀= C_u= C_c=

Remarks

Date Received: 07.22.16 Date Tested: 07.26.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 1.0-3.0
Sample Number: BB-SDC-101 1D

Date Sampled: 07.21.16

Thielsch Engineering Inc.

Cranston, RI

Client: GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
Thompson Bridge over Decker Cove

Project No: 09.0025910.00

Figure L-1

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.4	10.2	17.5	19.1	16.7	19.0	10.1

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1"	100.0		
0.75"	92.6		
.5"	92.6		
.375"	92.6		
#4	82.4		
#10	64.9		
#20	55.0		
#40	45.8		
#60	39.4		
#100	34.5		
#200	29.1		
0.0445 mm.	26.0		
0.0317 mm.	24.5		
0.0204 mm.	21.9		
0.0119 mm.	18.9		
0.0085 mm.	17.4		
0.0070 mm.	15.4		
0.0060 mm.	13.9		
0.0039 mm.	11.3		
0.0031 mm.	10.8		
0.0013 mm.	9.4		

* (no specification provided)

Material Description

Dark Grey Brown SAND, some Silt, little Gravel

Atterberg Limits (ASTM D 4318)

PL= _____ LL= _____ PI= _____

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 7.2542 D₈₅= 5.3947 D₆₀= 1.3618
D₅₀= 0.5772 D₃₀= 0.0854 D₁₅= 0.0068
D₁₀= 0.0019 C_u= 705.86 C_c= 2.77

Remarks

Date Received: 07.22.16 Date Tested: 08.02.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 0.0-2.0
Sample Number: BB-SDC-102 1D

Date Sampled: 07.21.16

Thielsch Engineering Inc.

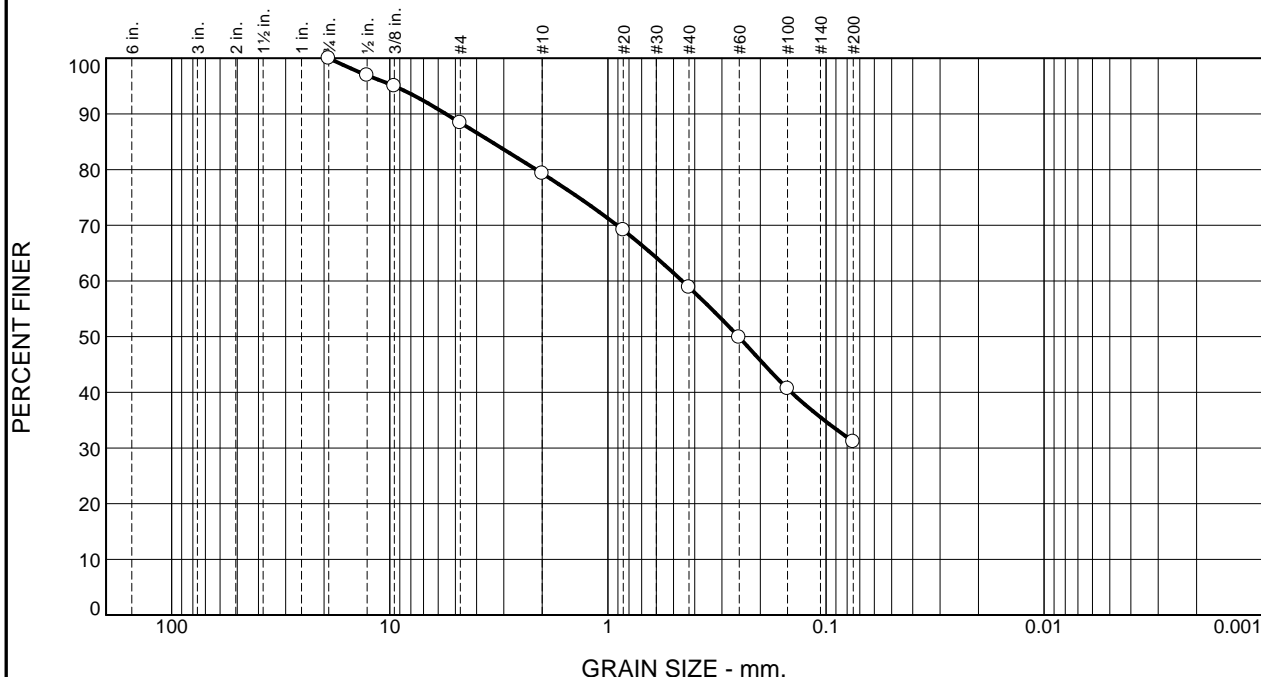
Cranston, RI

Client: GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
Thompson Bridge over Decker Cove

Project No: 09.0025910.00

Figure L-2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.6	9.1	20.4	27.8	31.1	

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
.5"	96.9		
.375"	95.0		
#4	88.4		
#10	79.3		
#20	69.1		
#40	58.9		
#60	49.9		
#100	40.6		
#200	31.1		

* (no specification provided)

Material Description

Gray SAND, some Silt, little Gravel (SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 5.5306 D₈₅= 3.4245 D₆₀= 0.4565
D₅₀= 0.2519 D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 07.22.16 Date Tested: 07.26.16
Tested By: RR
Checked By: Matthew Colman, P.E.
Title: Laboratory Manager

Source of Sample: Borings Depth: 17.0-18.4
Sample Number: BB-SDC-102 7D

Date Sampled: 07.21.16

Thielsch Engineering Inc.

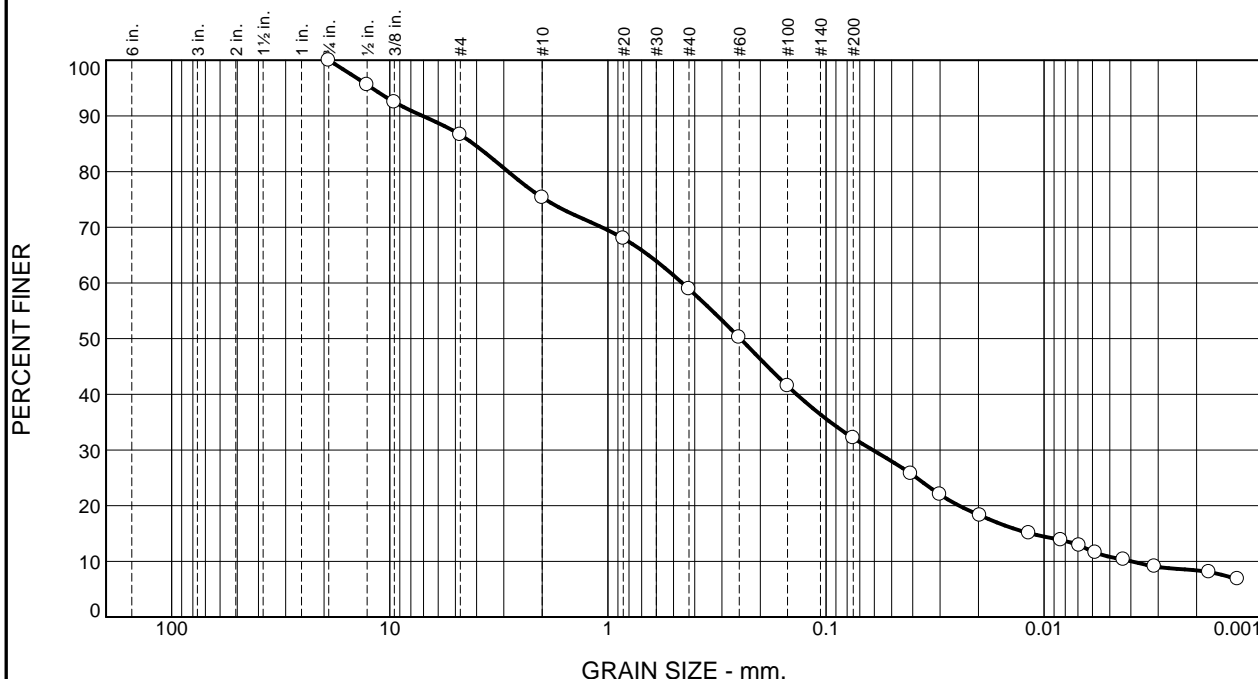
Cranston, RI

Client: GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
Thompson Bridge over Decker Cove

Project No: 09.0025910.00

Figure L-7

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	13.4	11.3	16.4	26.7	23.8	8.4

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
.5"	95.6		
.375"	92.5		
#4	86.6		
#10	75.3		
#20	68.0		
#40	58.9		
#60	50.2		
#100	41.5		
#200	32.2		
0.0409 mm.	25.8		
0.0300 mm.	22.0		
0.0197 mm.	18.3		
0.0117 mm.	15.1		
0.0084 mm.	13.8		
0.0069 mm.	12.9		
0.0058 mm.	11.6		
0.0043 mm.	10.3		
0.0031 mm.	9.1		
0.0018 mm.	8.1		
0.0013 mm.	6.8		

* (no specification provided)

Material Description

Dark Gray SAND, some Silt, little Gravel (SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-2-4(0)

Coefficients

D₉₀= 7.0716 D₈₅= 4.1356 D₆₀= 0.4562
D₅₀= 0.2470 D₃₀= 0.0610 D₁₅= 0.0115
D₁₀= 0.0040 C_u= 114.44 C_c= 2.05

Remarks

Date Received: 07.22.16 Date Tested: 07.27.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 5.0-7.0
Sample Number: BB-SDC-104 2D

Date Sampled: 07.21.16

Thielsch Engineering Inc.

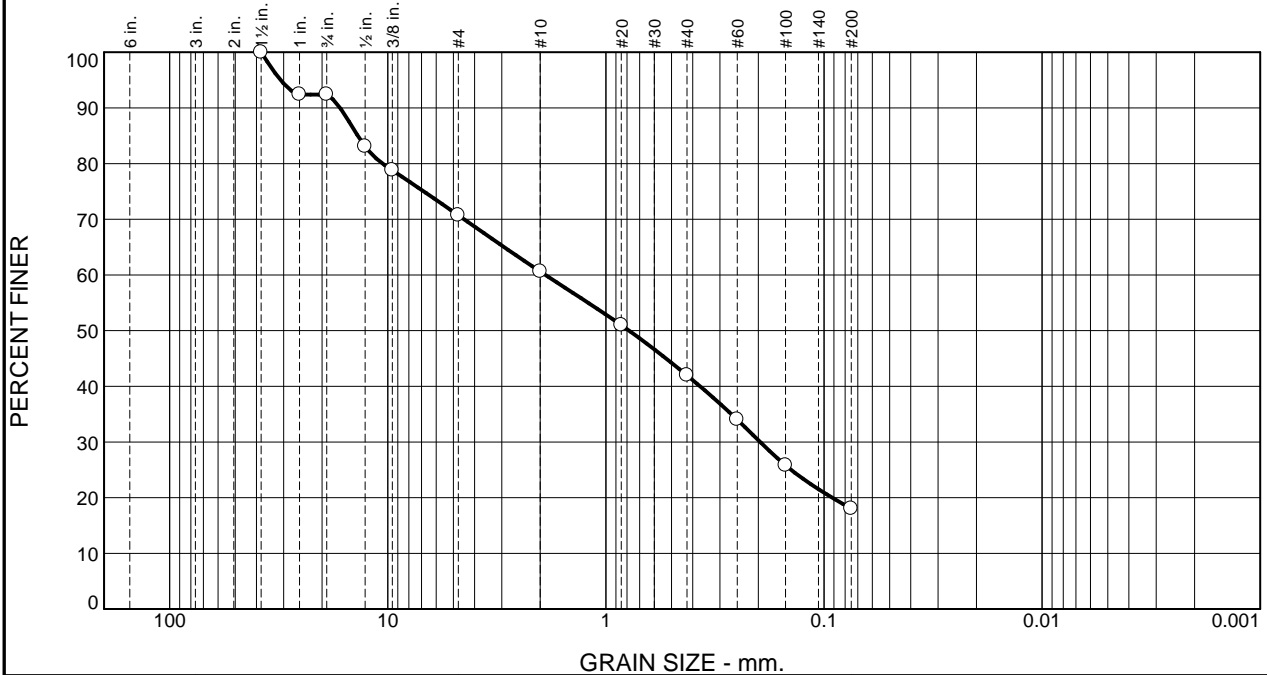
Cranston, RI

Client: GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
Thompson Bridge over Decker Cove

Project No: 09.0025910.00

Figure L-10

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.6	21.7	10.1	18.6	24.0	18.0	

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5"	100.0		
1"	92.4		
0.75"	92.4		
.5"	83.1		
.375"	78.8		
#4	70.7		
#10	60.6		
#20	51.0		
#40	42.0		
#60	34.0		
#100	25.8		
#200	18.0		

* (no specification provided)

Material Description

Brown SAND, some gravel, little Silt (SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 16.5470 D₈₅= 13.7289 D₆₀= 1.8942
D₅₀= 0.7835 D₃₀= 0.1961 D₁₅=
D₁₀= C_u= C_c=

Remarks

Date Received: 07.22.16 Date Tested: 07.26.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 10.0-12.0
Sample Number: BB-SDC-104 3D

Date Sampled: 07.21.16

Thielsch Engineering Inc. Cranston, RI	Client: GZA GeoEnvironmental, Inc. Project: Maine Dept of Transportation Thompson Bridge over Decker Cove Project No: 09.0025910.00
Figure L-11	

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.0	6.4	26.6	39.2	8.8	

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
.5"	91.3		
.375"	88.7		
#4	81.0		
#10	74.6		
#20	66.4		
#40	48.0		
#60	25.9		
#100	13.7		
#200	8.8		

* (no specification provided)

Material Description

Red Brown SAND, little Gravel, trace Silt (SW-SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SW-SM AASHTO (M 145)= A-1-b

Coefficients

D₉₀= 11.2323 D₈₅= 6.6759 D₆₀= 0.6217
D₅₀= 0.4484 D₃₀= 0.2781 D₁₅= 0.1635
D₁₀= 0.1007 C_u= 6.18 C_c= 1.24

Remarks

Date Received: 07.22.16 Date Tested: 07.26.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 1.0-3.0
Sample Number: BB-SDC-105 1D

Date Sampled: 07.21.16

Thielsch Engineering Inc. Cranston, RI	Client: GZA GeoEnvironmental, Inc. Project: Maine Dept of Transportation Thompson Bridge over Decker Cove Project No: 09.0025910.00
Figure L-13	

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.0	4.3	20.3	60.1	9.3	

TEST RESULTS (D422)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.5"	100.0		
.375"	96.8		
#4	94.0		
#10	89.7		
#20	84.0		
#40	69.4		
#60	38.6		
#100	15.9		
#200	9.3		

* (no specification provided)

Material Description

Red Brown SAND, trace Silt, trace Gravel (SP-SM)

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= SP-SM AASHTO (M 145)= A-3

Coefficients

D₉₀= 2.1121 D₈₅= 0.9638 D₆₀= 0.3545
D₅₀= 0.3008 D₃₀= 0.2137 D₁₅= 0.1452
D₁₀= 0.1025 C_u= 3.46 C_c= 1.26

Remarks

Date Received: 07.22.16 Date Tested: 07.26.16

Tested By: RR

Checked By: Matthew Colman, P.E.

Title: Laboratory Manager

Source of Sample: Borings Depth: 5.0-7.0
Sample Number: BB-SDC-105 2D

Date Sampled: 07.21.16

Thielsch Engineering Inc.

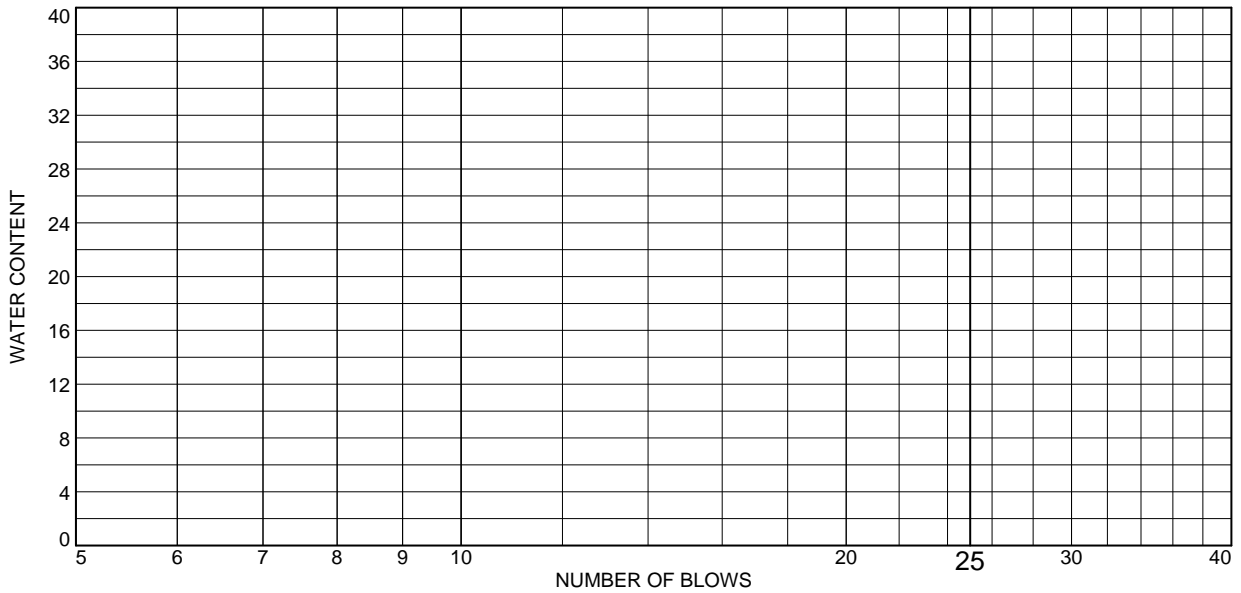
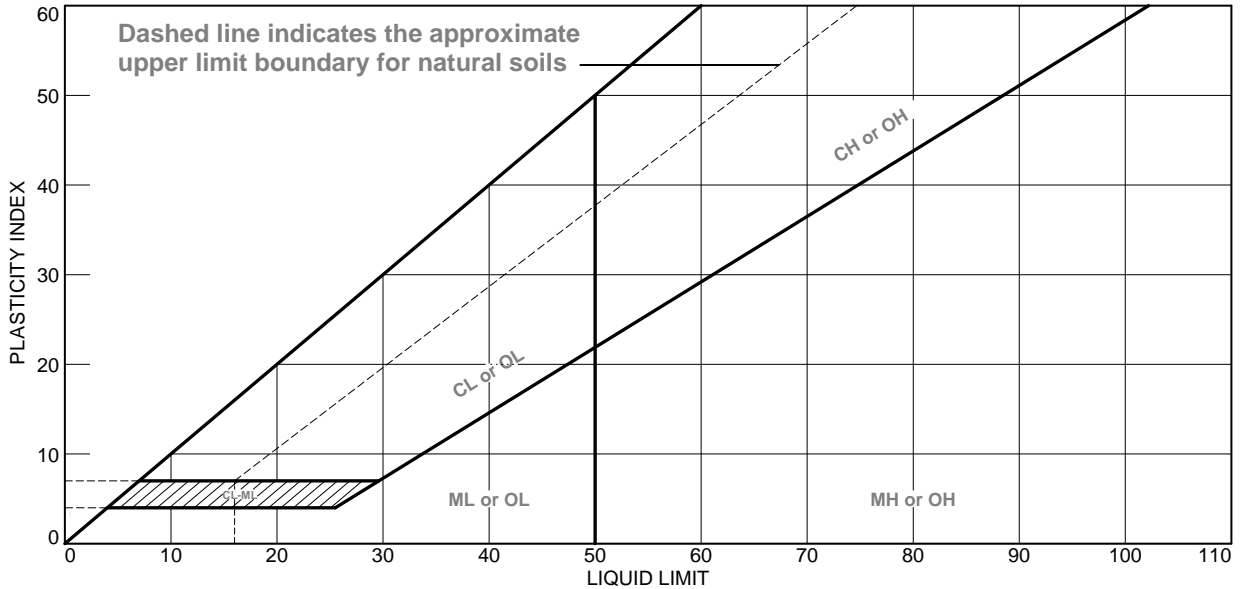
Cranston, RI

Client: GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
Thompson Bridge over Decker Cove

Project No: 09.0025910.00

Figure L-14

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Dark Brown Organic Silt, some Sand (OL)	NV	NP	NP			

Project No. 09.0025910.00 **Client:** GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
 Thompson Bridge over Decker Cove
Source of Sample: Borings **Depth:** 2.0-4.0
Sample Number: BB-SDC-102 2D

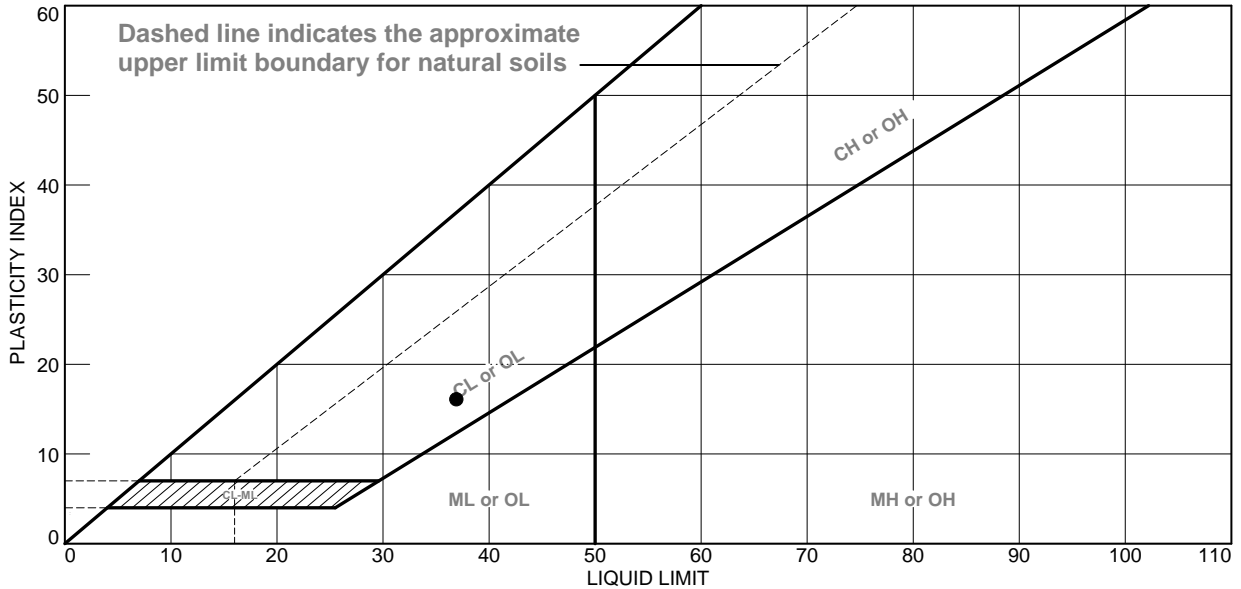
Thielsch Engineering Inc.
Cranston, RI

Remarks:

Figure L-3

Tested By: RR **Checked By:** Matthew Colman, P.E.

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Grey Silty Clay (CL)	37	21	16			

Project No. 09.0025910.00 **Client:** GZA GeoEnvironmental, Inc.
Project: Maine Dept of Transportation
 Thompson Bridge over Decker Cove
Source of Sample: Borings **Depth:** 7.0-8.0
Sample Number: BB-SDC-102 4D

Thielsch Engineering Inc.
 Cranston, RI

Remarks:

Figure L-4

Tested By: RR **Checked By:** Matthew Colman, P.E.

LABORATORY TESTING DATA SHEET



Project Name Thompson Bridge over Decker Cove Location Southport, ME

Reviewed By _____

Project No. 09.0025910.00 Assigned By A. Blaisdell

Project Manager A. Blaisdell Report Date 8/3/2016

Date Reviewed 8/3/2016

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data						Compression Tests								Rock Formation or Description or Remarks	
				Moh's Hardness	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	(3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	σ PSI	I _{S50} PSI		
BB-SDC-101	R1	17.5-17.9	15		1.988	4.582	165.4				U	11,469	0.49		1.92	0.11			
BB-SDC-102	R1	23.4-23.8	16		1.994	4.593	170.3				U	7,551	0.22		4.70	0.08			Sample failed along apparent foliation
		23.8	16		1.994	1.040	168.5				PLA							795	
		23.9-24.1	16		1.994	2.008	169.5				PLD							1157	Sample failed along apparent foliation
BB-SDC-102	R2	28.7-29.1	17		1.994	4.591	163.3				U	6,321	0.22		2.47	0.11			
BB-SDC-104	R2	24.1	18		1.992	0.766	175.4				PLA							3346	
		24.1-24.3	18		1.992	2.022	175.2				PLD							1562	
BB-SDC-104	R1	20.3-20.7	19		1.992	4.601	164.1				U	12,835	0.26		4.66	0.01			Sample failed along apparent foliation
BB-SDC-105	R1	20.1-20.5	20		1.990	4.585	162.2				U	12,063	0.25		3.98	0.11			
BB-SDC-105	R2	22.7-23.1	21		1.990	4.635	171.1				U	9,459	0.19		3.75	0.13			

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

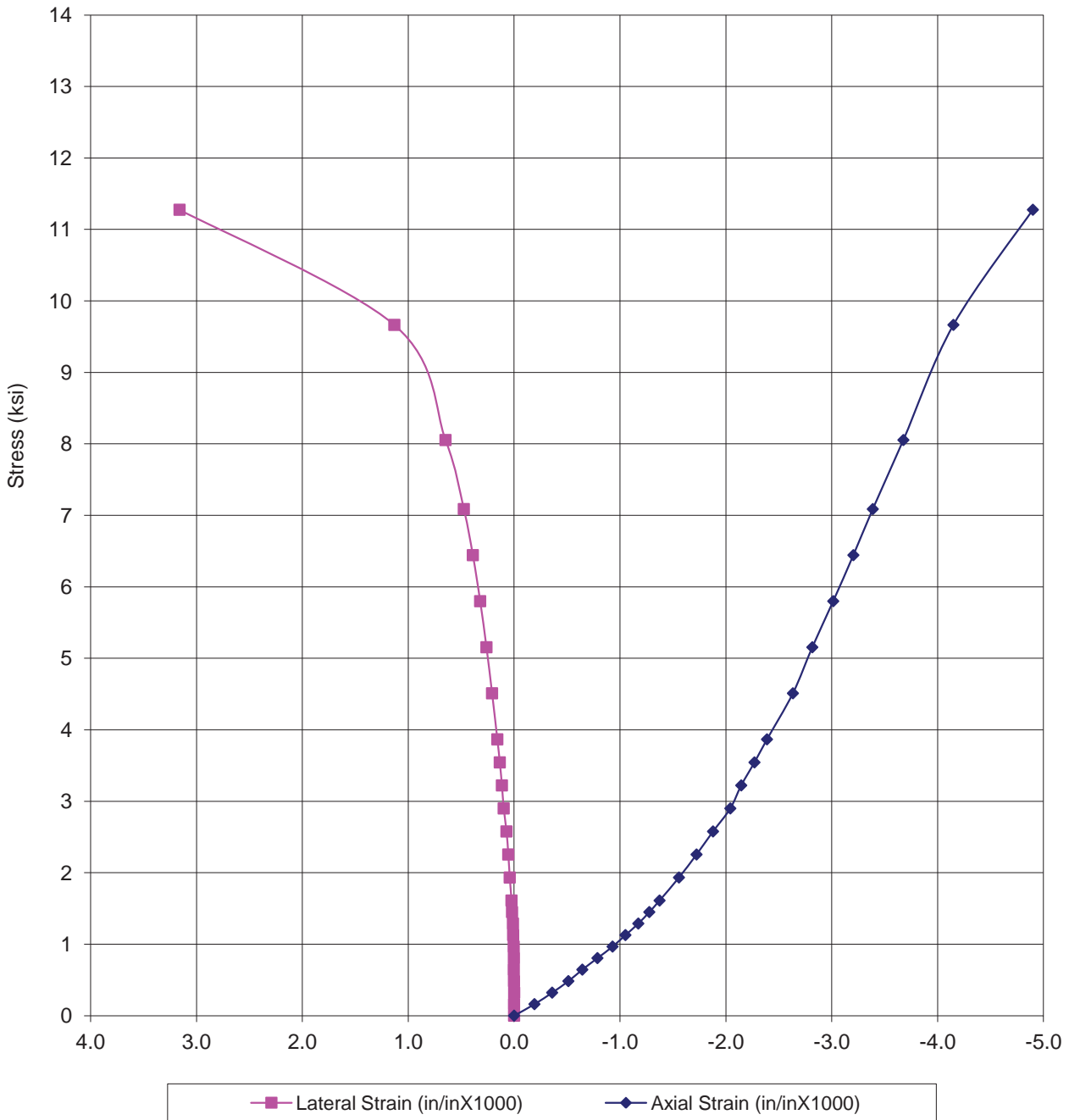
(3) P=Petrographic PLD=Point Load (diametrical),
 PLA= Point Load (Axial) ST= Splitting Tensile
 U= Unconfined Compressive Strength
 (4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
 (6) Represents Confining Stress on Triaxial Tests
 (7) Represents Secant Modulus at 50% of Total Failure Stress
 (8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress



195 Frances Avenue
 Cranston, RI 02910
 401-467-6454

Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-101
 Sample No. R1
 Depth: 17.5-17.9'

File No. 09.0025910.00
 Date: 07/29/16
 Test No. U 15



**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-101</u>	<u>R1</u>	<u>17.5'– 17.9'</u>



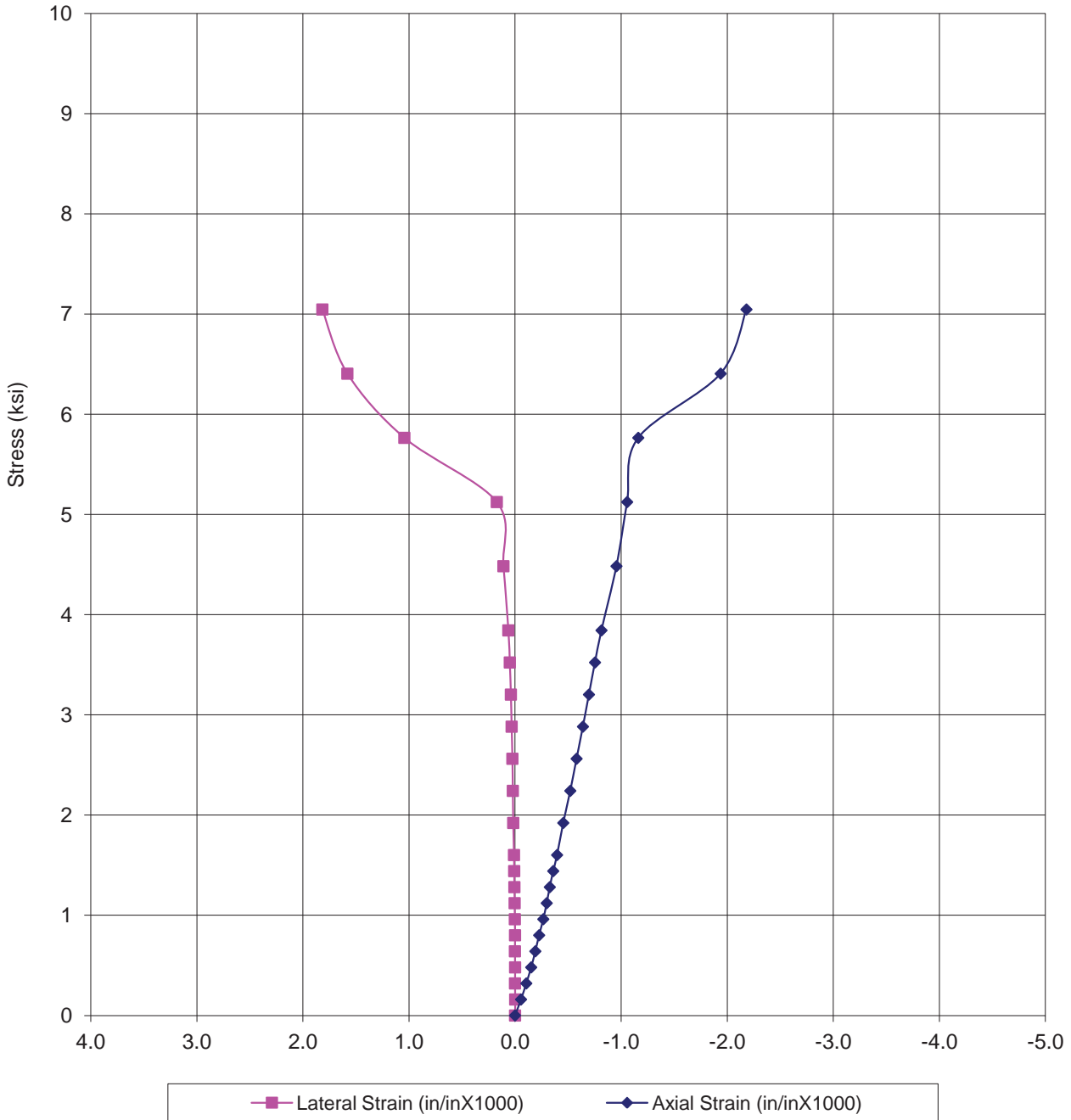
**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-101</u>	<u>R1</u>	<u>17.5' - 17.9'</u>



Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-102
 Sample No. R1
 Depth: 23.4-23.8'

File No. 09.0025910.00
 Date: 07/29/16
 Test No. U 16



**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-102</u>	<u>R1</u>	<u>23.4' - 23.8'</u>



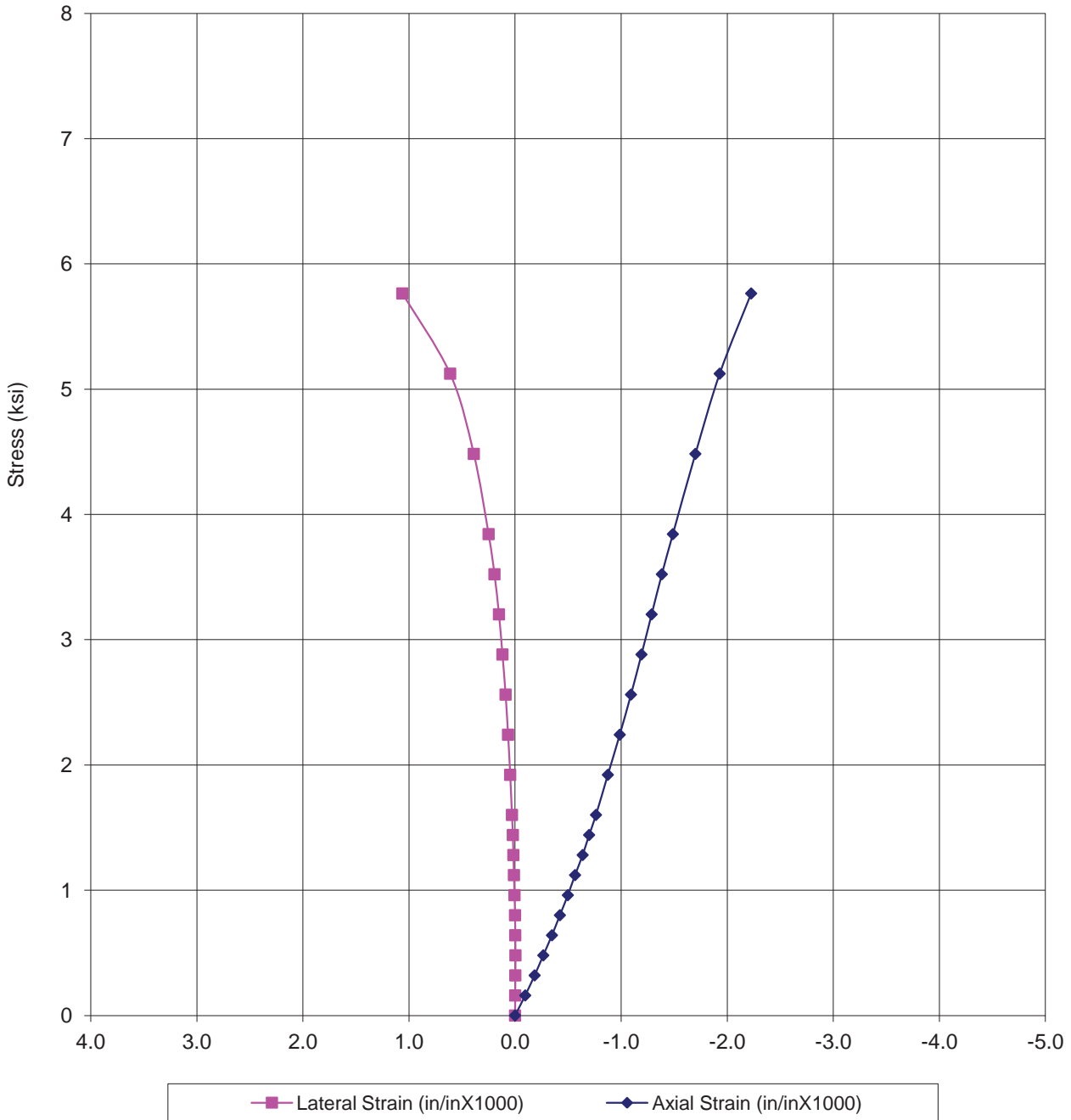
**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-102</u>	<u>R1</u>	<u>23.4' - 23.8'</u>



Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-102
 Sample No. R2
 Depth: 28.7-29.1'

File No. 09.0025910.00
 Date: 07/29/16
 Test No. U 17



Thompson Bridge over Decker Cove
Southport, ME

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-102</u>	<u>R2</u>	<u>28.7'– 29.1'</u>



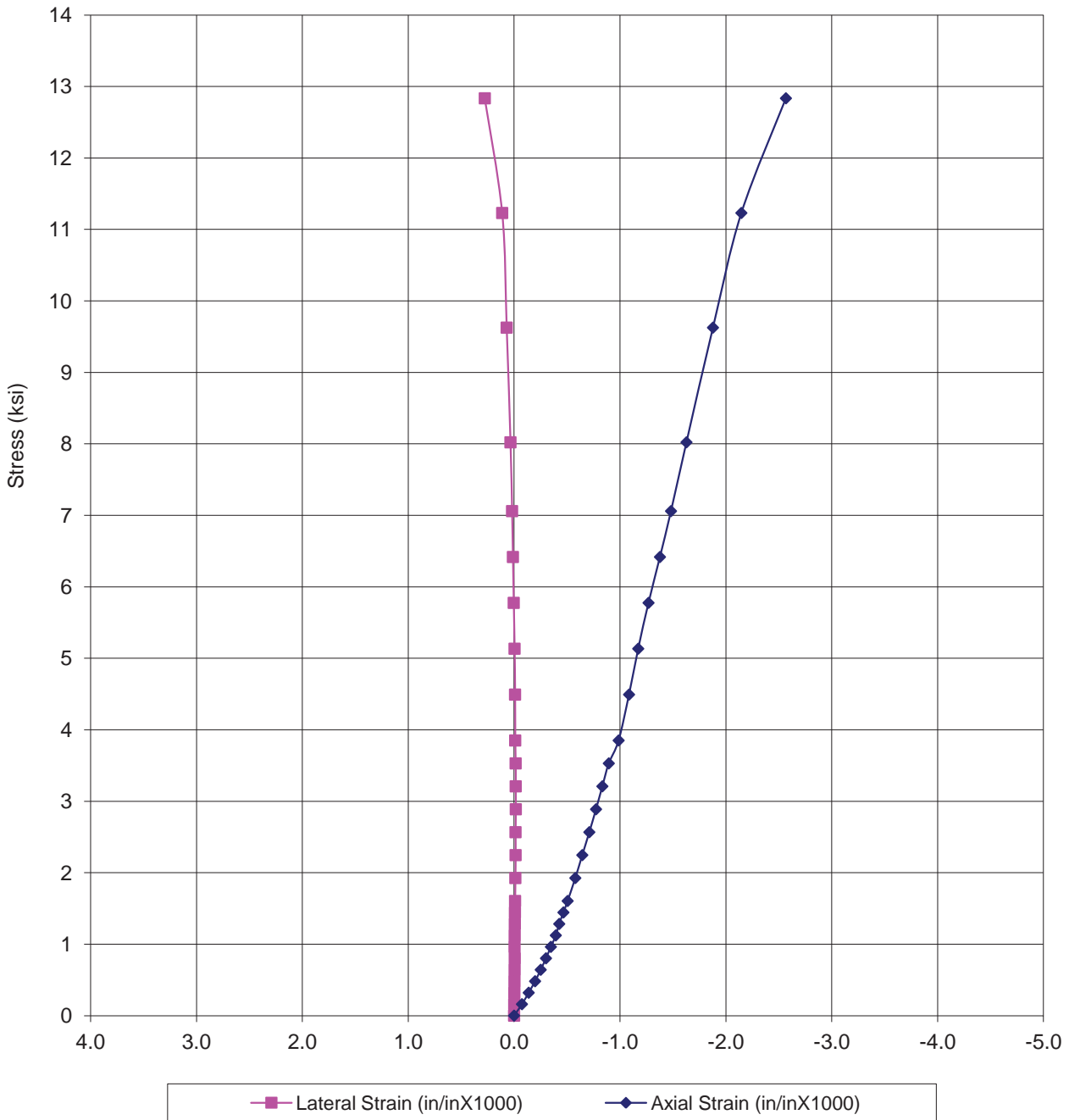
**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-102</u>	<u>R2</u>	<u>28.7' – 29.1'</u>



Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-104
 Sample No. R1
 Depth: 20.3-20.7'

File No. 09.0025910.00
 Date: 07/29/16
 Test No. U 19



Thompson Bridge over Decker Cove
Southport, ME

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-104</u>	<u>R1</u>	<u>20.3' - 20.7'</u>



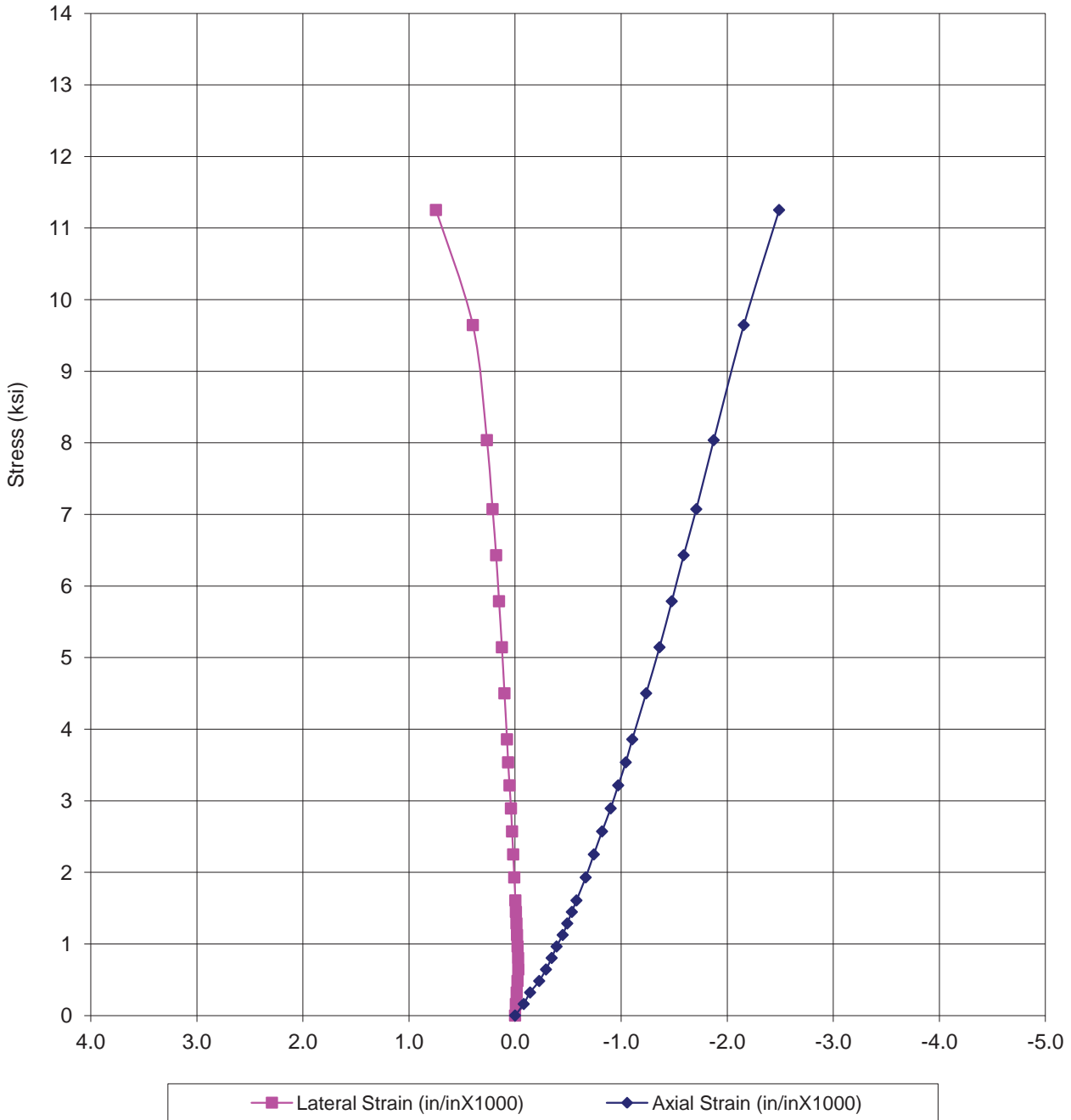
**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-104</u>	<u>R1</u>	<u>20.3' - 20.7'</u>



Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-105
 Sample No. R1
 Depth: 20.1-20.5'

File No. 09.0025910.00
 Date: 07/29/16
 Test No. U 20



**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-105</u>	<u>R1</u>	<u>20.1'– 20.5'</u>



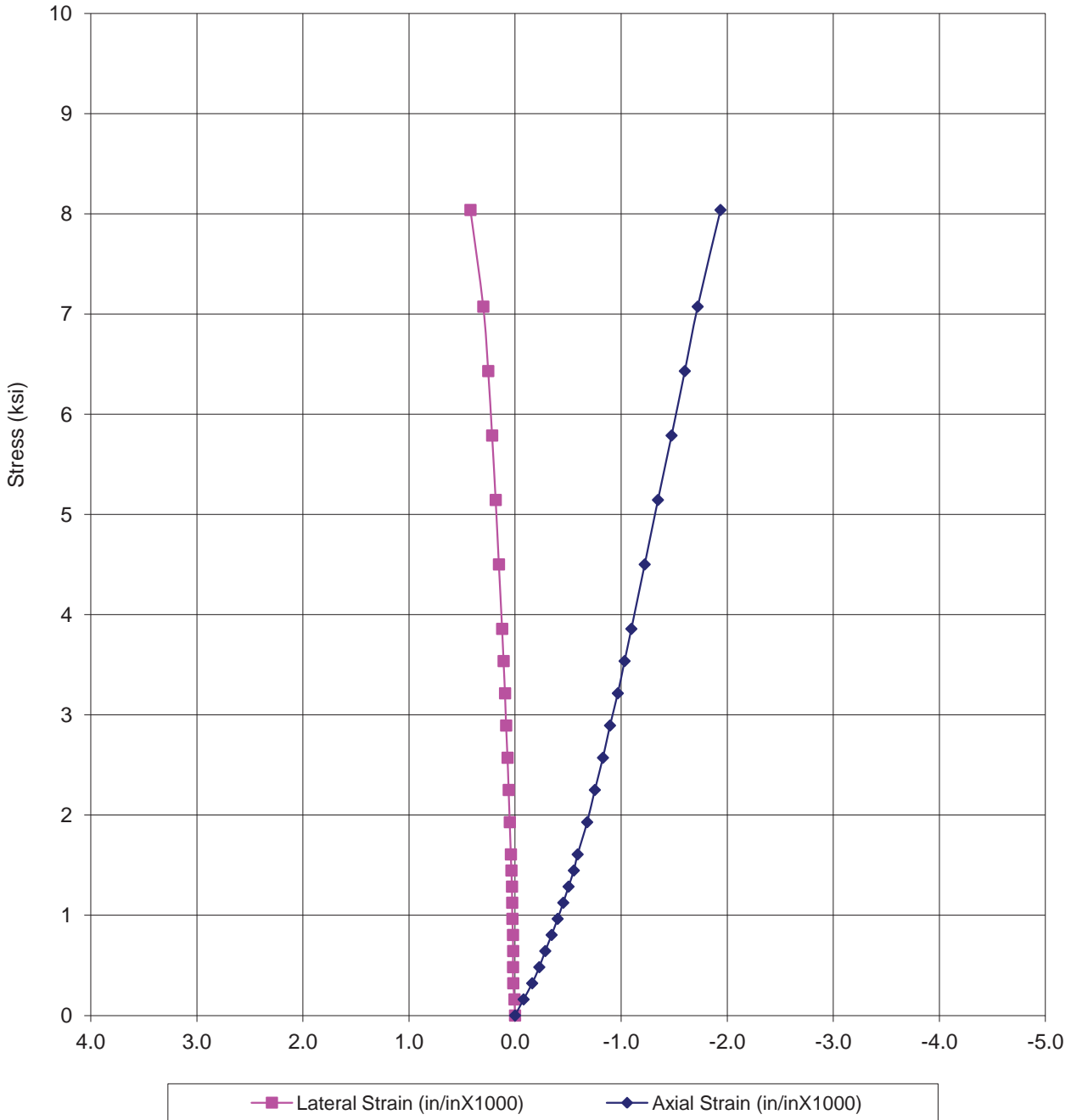
**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-105</u>	<u>R1</u>	<u>20.1'– 20.5'</u>



Thompson Bridge over Decker Cove Southport, ME



Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-105
 Sample No. R2
 Depth: 22.7-23.1'

File No. 09.0025910.00
 Date: 08/01/16
 Test No. U 21



Thompson Bridge over Decker Cove
Southport, ME

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-105</u>	<u>R2</u>	<u>22.7' - 23.1'</u>



**Thompson Bridge over Decker Cove
Southport, ME**

GZA Project # 09.0025910.00

BORING NO.	SAMPLE NO.	DEPTH
<u>BB-SDC-105</u>	<u>R2</u>	<u>22.7' - 23.1'</u>





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 Cranston RI, 02910
 Phone: (401)-467-6454
 Fax: (401)-467-2398
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Let's Build a Solid Foundation

Client Information:
 GZA GeoEnvironmental
 Portland, ME
 PM: N. Williams
 Assigned By: NVW
 Collected By: NVW

Project Information:
Thompson Bridge over Decker Cove
Southport, ME
 GZA Project Number: 09.0025910.01
 Summary Page: 1 of 1
 Report Date: 01.15.2019

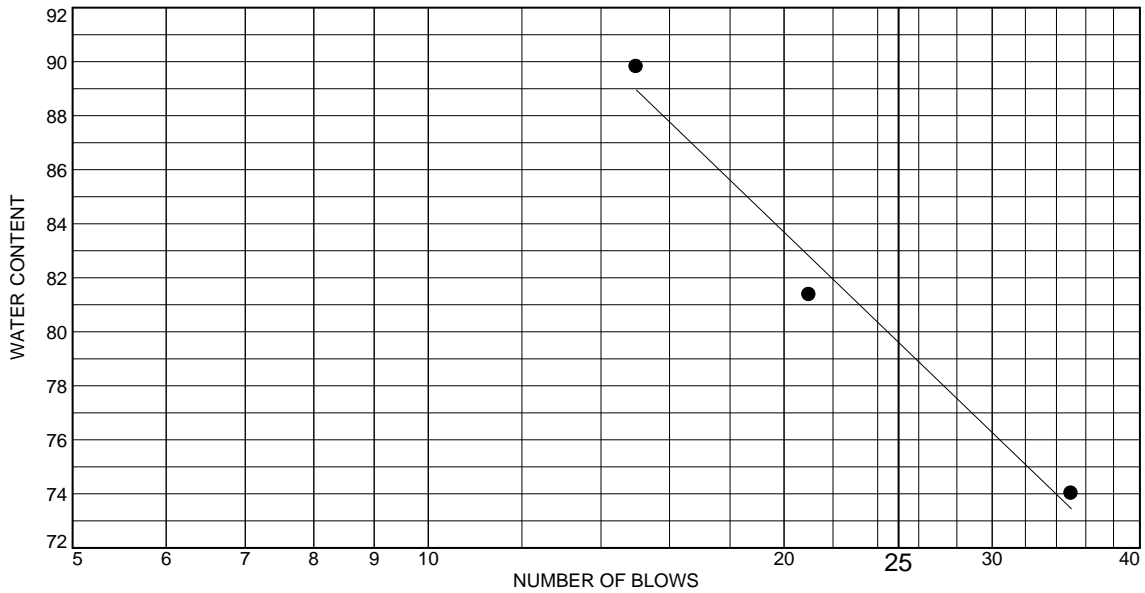
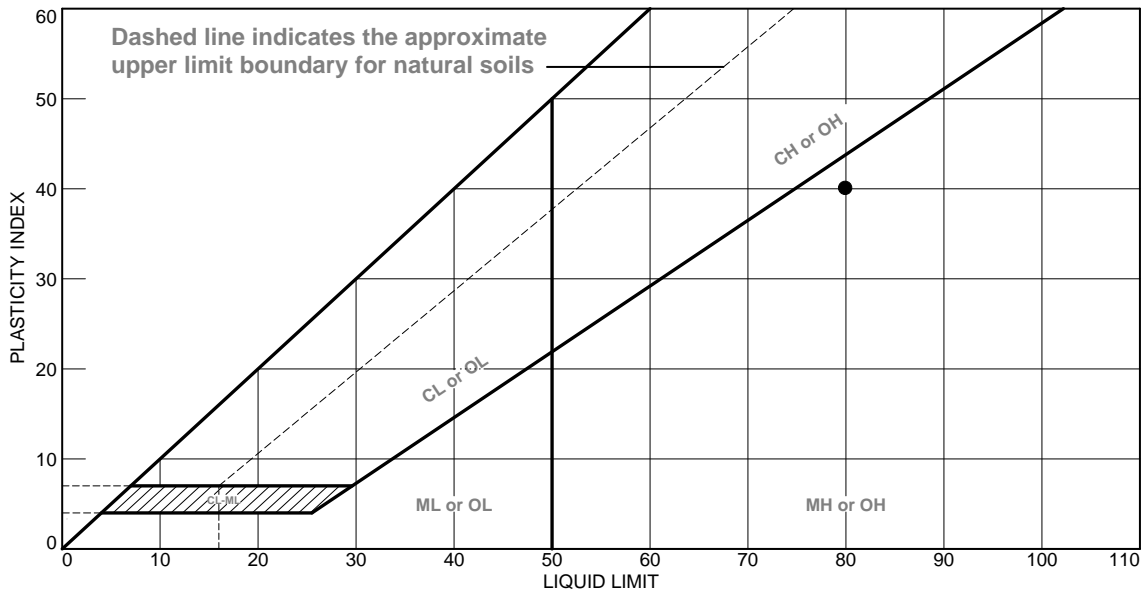
LABORATORY TESTING DATA SHEET

Boring	Sample No.	Depth (ft)	Laboratory No.	Identification Tests								Proctor / CBR / Permeability Tests							Laboratory Log and Soil Description	
				As Received Water Content %	LL %	PL %	Gravel %	Sand %	Fines %	Org. %	G _s	Dry unit wt. pcf	Test Water Content %	γ _d MAX (pcf) W _{opt} (%)	γ _d MAX (pcf) W _{opt} (%) (Corr.)	Target Test Setup as % of Proctor	CBR @ 0.1"	CBR @ 0.2"		Permeability cm/sec
				D2216	D4318	D6913			D2874	D854	D1557									
BB-SDC-202	2D	5-7	S-22	72.8	80	40													Dark Gray organic silt	
BB-SDC-202	3D	10-12	S-23	33.8																
BB-SDC-202	4D	15-17	S-24	36.1																
BB-SDC-203 & -202 Composite	1D	0-2	S-25	77.2			2.2	53.8	44.0										Brown clayey fine to medium SAND trace fine Gravel	
BB-SDC-203	2D	5-7	S-26	66.3	73	40													Dark Brown organic silt	
BB-SDC-203	3D	10-12	S-27	34.3	33	19													Gray lean clay	
BB-SDC-203	7D	30-32	S-28				21.5	34.3	44.2										Gray clayey fine to coarse SAND, some fine Gravel	

Reviewed By SKW

01.16.2019

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Dark Gray organic silt	80	40	40			OH

Project No. 09.0025910.01 **Client:** GZA GeoEnvironmental
Project: Thompson Bridge over Decker Cove
 Southport, ME
Source of Sample: Borings **Depth:** 5-7'
Sample Number: BB-SDC-202 / 2D

Thielsch Engineering Inc.

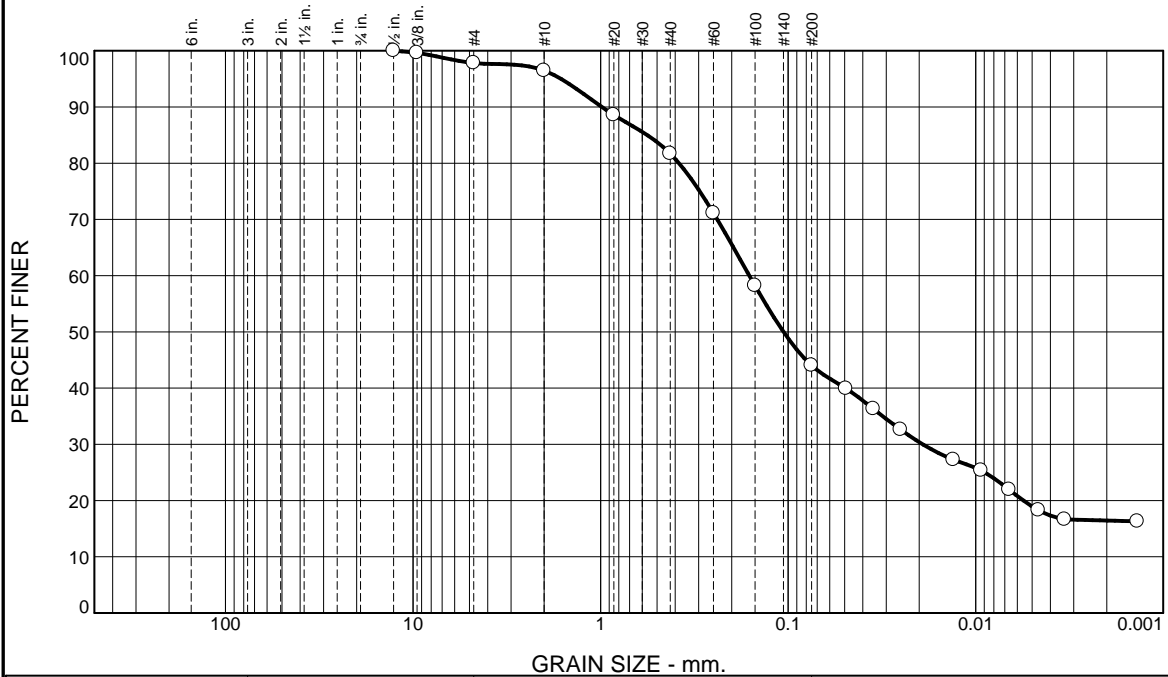
Cranston, RI

Remarks:

Figure L-22

Tested By: GP _____ **Checked By:** sa _____

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.2	1.3	14.8	37.7	27.6	16.4

Test Results (D7928 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.5"	100.0		
0.375"	99.6		
#4	97.8		
#10	96.5		
#20	88.6		
#40	81.7		
#60	71.1		
#100	58.2		
#200	44.0		
0.0491 mm.	39.9		
0.0351 mm.	36.3		
0.0251 mm.	32.6		
0.0131 mm.	27.3		
0.0093 mm.	25.3		
0.0066 mm.	22.0		
0.0046 mm.	18.3		
0.0034 mm.	16.6		
0.0014 mm.	16.3		

* (no specification provided)

Material Description

Brown clayey fine to medium SAND trace fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 0.9844 D₈₅= 0.5635 D₆₀= 0.1610
D₅₀= 0.1058 D₃₀= 0.0191 D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/8".

Date Received: 01.11.19 Date Tested: 01.15.19

Tested By: MN

Checked By: Steven Accetta

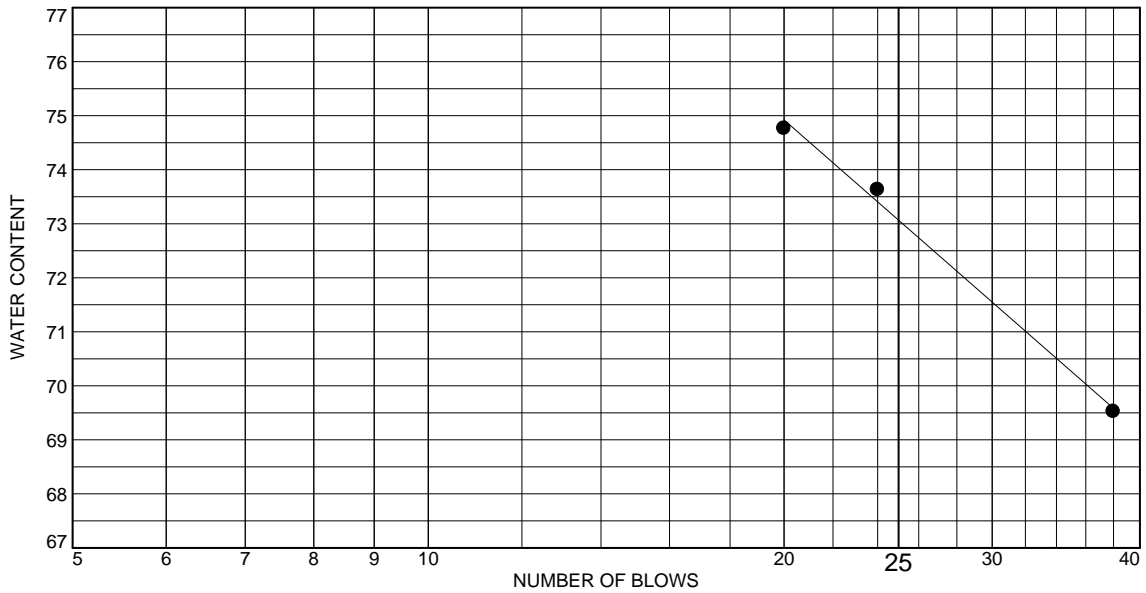
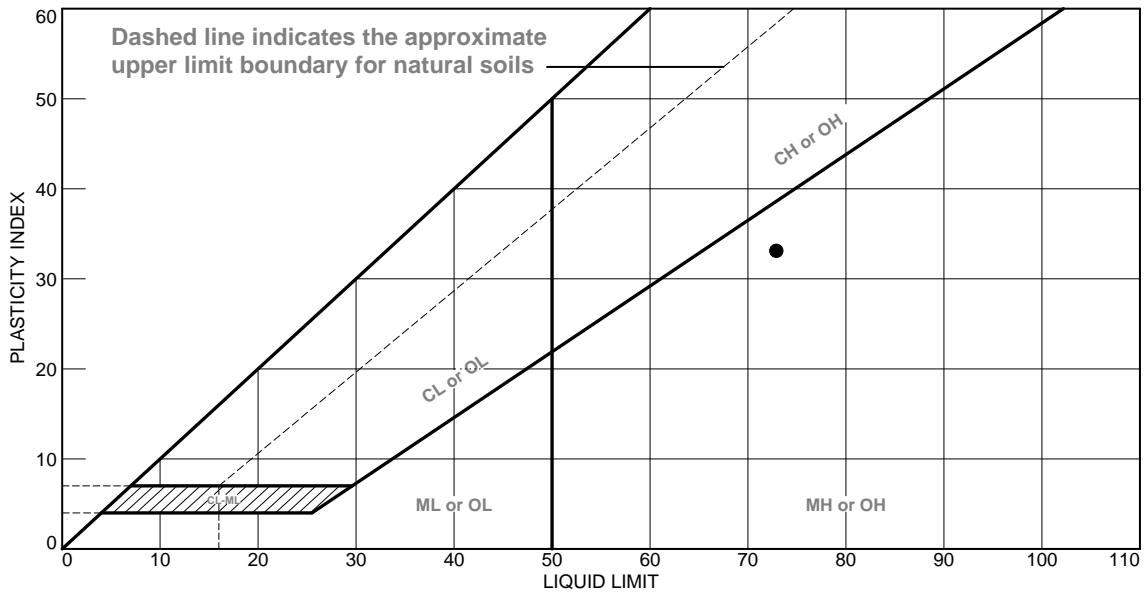
Title: Laboratory Coordinator

Source of Sample: Borings Depth: 0-2'
Sample Number: BB-SDC-203&202 / 1D

Date Sampled:

Thielsch Engineering Inc.	<p>Client: GZA GeoEnvironmental</p> <p>Project: Thompson Bridge over Decker Cove Southport, ME</p> <p>Project No: 09.0025910.01</p>
Cranston, RI	Figure S-25

LIQUID AND PLASTIC LIMITS TEST REPORT

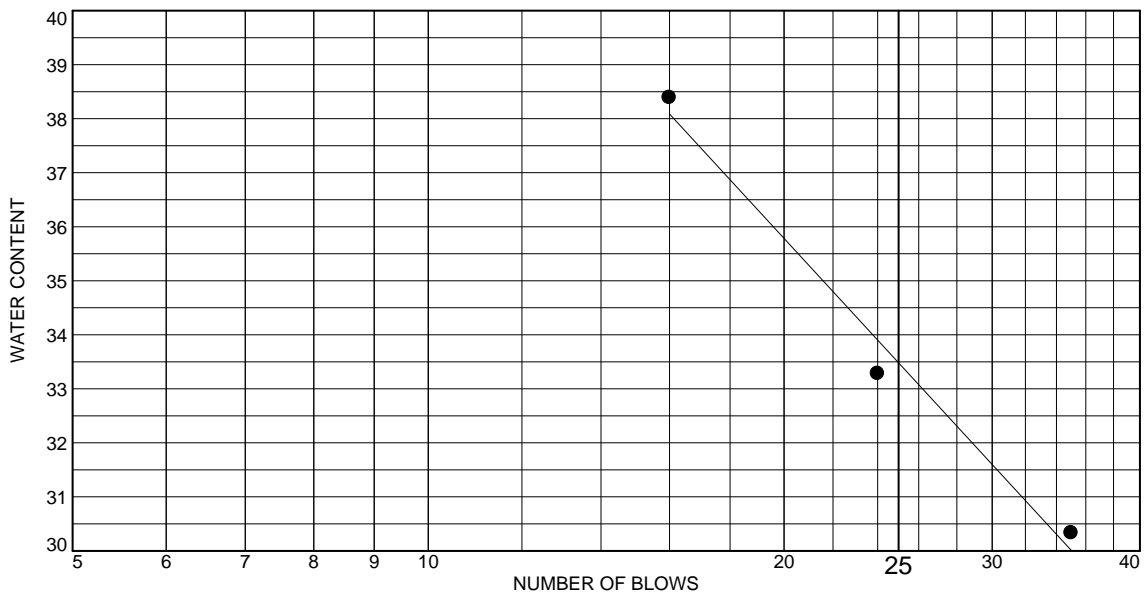
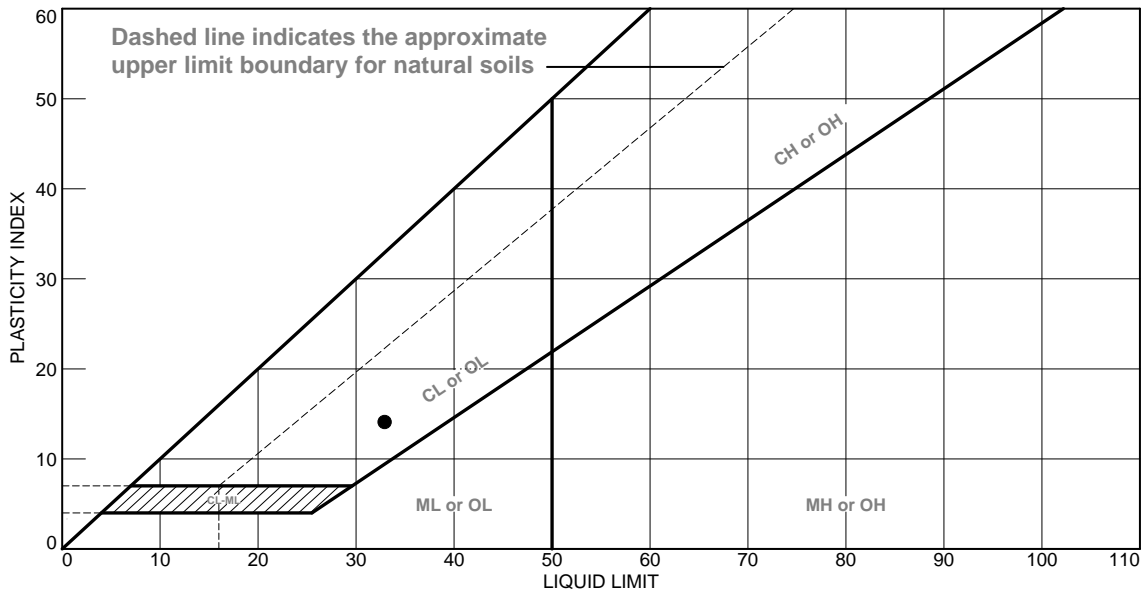


MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Dark Brown organic silt	73	40	33			OH

Project No. 09.0025910.01 Client: GZA GeoEnvironmental Project: Thompson Bridge over Decker Cove Southport, ME Source of Sample: Borings Depth: 5-7' Sample Number: BB-SDC-203 / 2D	Remarks:
Thielsch Engineering Inc. Cranston, RI	
Figure L-26	

Tested By: GP/RR _____ **Checked By:** sa _____

LIQUID AND PLASTIC LIMITS TEST REPORT



MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
● Gray lean clay	33	19	14			CL

Project No. 09.0025910.01 **Client:** GZA GeoEnvironmental
Project: Thompson Bridge over Decker Cove
 Southport, ME
Source of Sample: Borings **Depth:** 10-12'
Sample Number: BB-SDC-203 / 3D

Thielsch Engineering Inc.

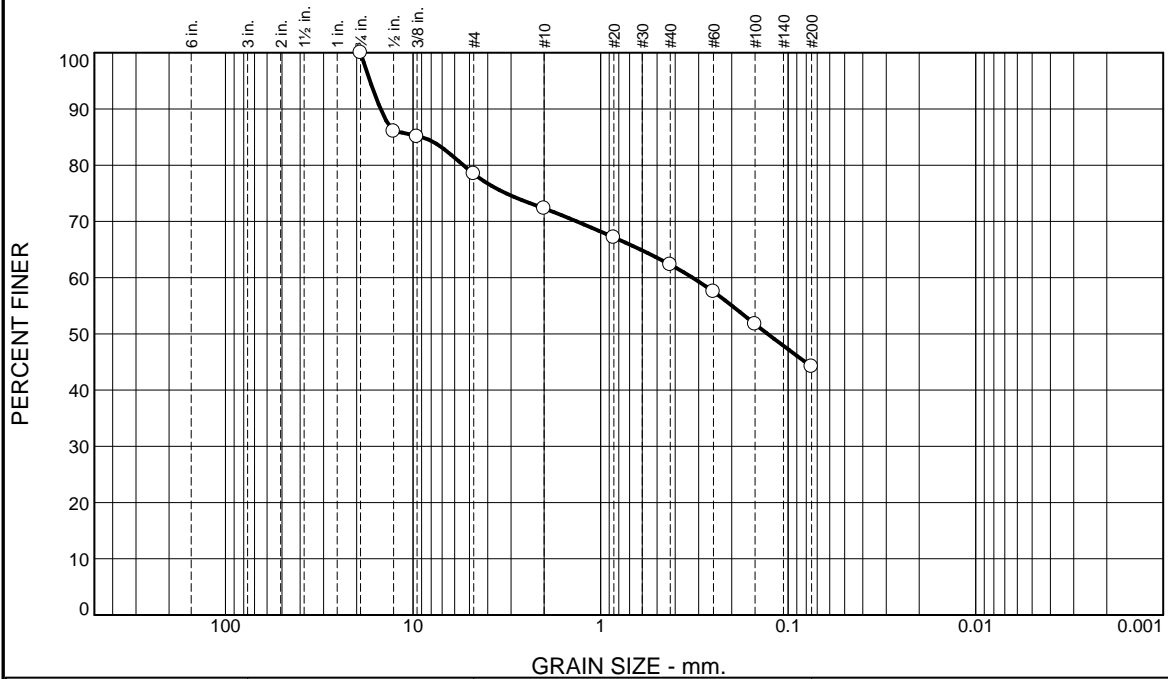
Cranston, RI

Remarks:

Figure L-27

Tested By: MN _____ **Checked By:** RR _____

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	21.5	6.2	10.0	18.1	44.2	

Test Results (D6913 & ASTM D 1140)			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
0.75"	100.0		
0.5"	86.0		
0.375"	85.1		
#4	78.5		
#10	72.3		
#20	67.2		
#40	62.3		
#60	57.5		
#100	51.7		
#200	44.2		

* (no specification provided)

Material Description

Gray clayey fine to coarse SAND, some fine Gravel

Atterberg Limits (ASTM D 4318)

PL= NP LL= NV PI= NP

Classification

USCS (D 2487)= SM AASHTO (M 145)= A-4(0)

Coefficients

D₉₀= 14.8985 D₈₅= 9.0588 D₆₀= 0.3232
D₅₀= 0.1285 D₃₀= D₁₅=
D₁₀= C_u= C_c=

Remarks

Sample visually classified as plastic. Sample rolled to 1/8".

Date Received: 01.11.19 Date Tested: 01.15.19

Tested By: MN

Checked By: Steven Accetta

Title: Laboratory Coordinator

Source of Sample: Borings Depth: 30-32'
Sample Number: BB-SDC-203 / 7D

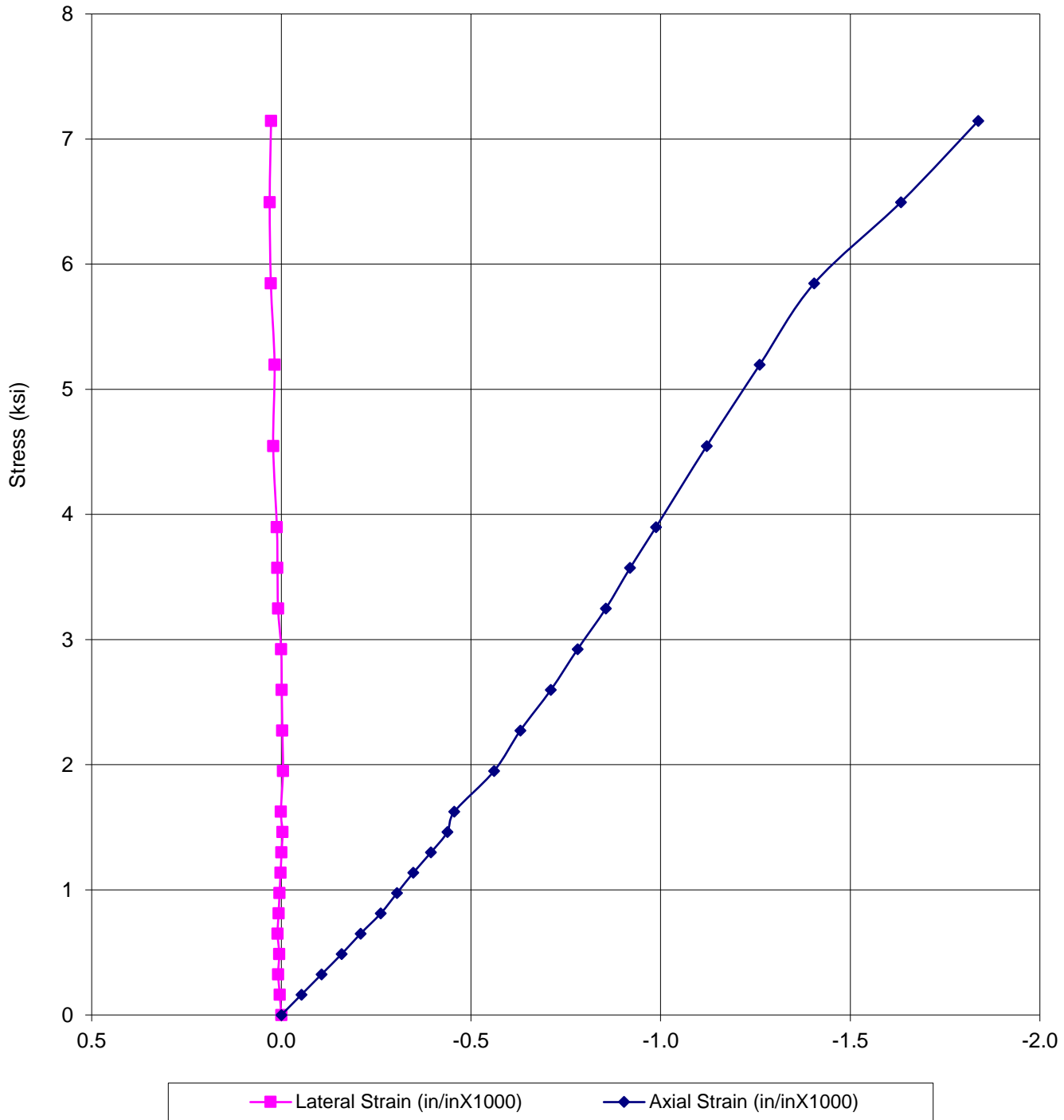
Date Sampled:

Thielsch Engineering Inc.	Client: GZA GeoEnvironmental
Cranston, RI	Project: Thompson Bridge over Decker Cove Southport, ME
	Project No: 09.0025910.01 Figure S-28

LABORATORY TESTING DATA SHEET

Boring No.	Sample No.	Depth (ft)	Laboratory No.	Specimen Data						Compressive Strength Tests								Rock Formation or Description or Remarks	
				Mohs Hardness	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	σ _t PSI	I _{s50} PSI	(8) s _c PSI		
BB-SDC-201	R2	10.9-11.4	R-29		1.980	4.601	181.4					7340	0.184	3.89	0.01				SCHIST: fresh break
BB-SDC-204	R1	11.1-12.3	R-30		1.984	4.765	164.3					5689	0.285	1.84	0.24				PEGMATITE: fresh break
(1) Volume Determined By Measuring Dimensions				Notes	(3) PLD=Point Load (diametrical),						Notes	(5) Strain at Peak Deviator Stress							
(2) Determined by Measuring Dimensions and Weight of Saturated Sample					PLA= Point Load (Axial) ST= Splitting Tensile							(6) Represents Secant Modulus at 50% of Total Failure Stress							
					U= Unconfined Compressive Strength							(7) Represents Secant Poisson's Ratio at 50% of Total Failure Stress							
					(4) Taken at Peak Deviator Stress							(8) Estimated UCS from Table 1 of ASTM D5731 for NX cores (Is x 24)							

**Thompson Bridge over Decker Cove
Southport, ME**



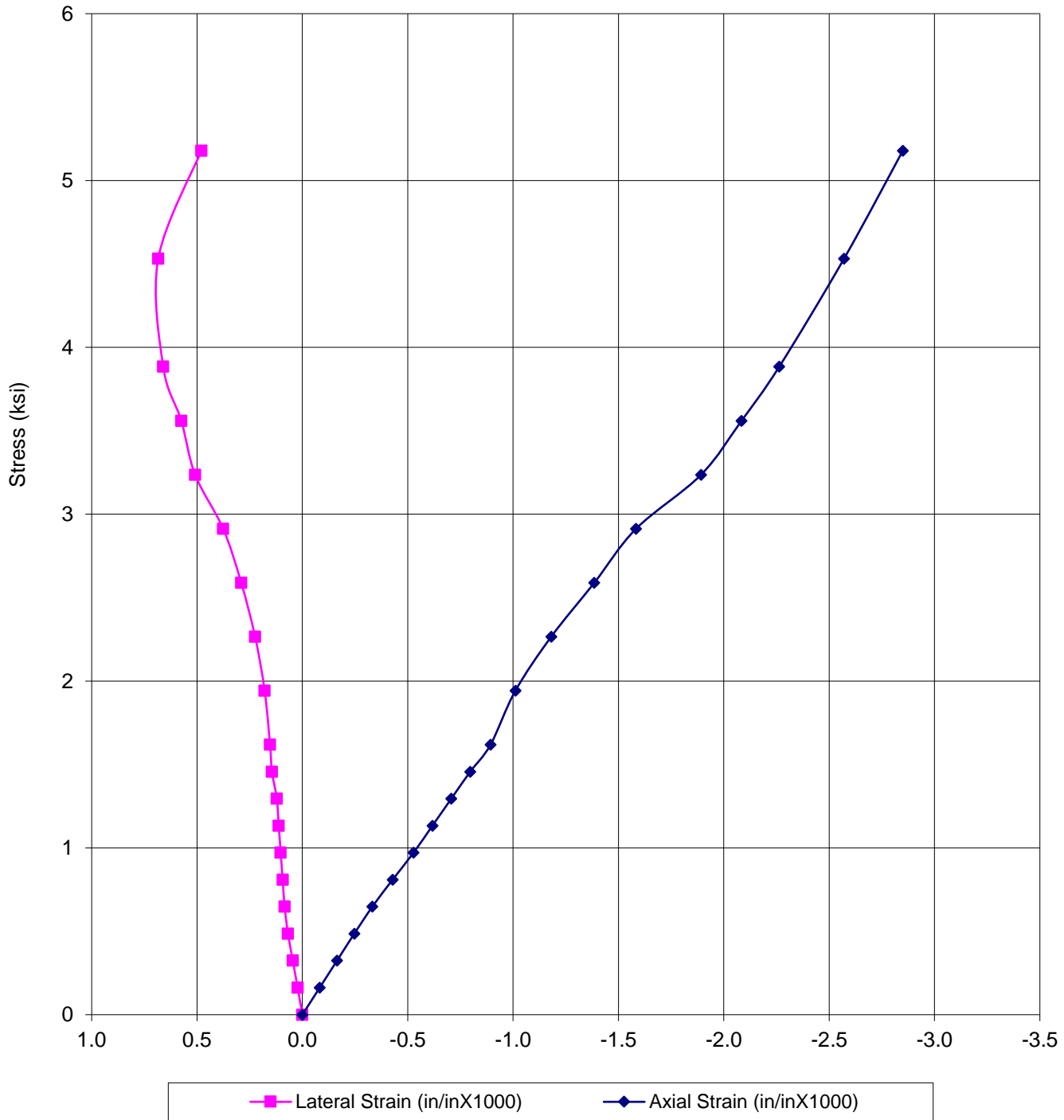
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-201
 Sample No. R2
 Depth: 10.9 - 11.4

File No. 09.0025910.01
 Date: 1.15.19
 Test No. R-29



Thompson Bridge over Decker Cove
Southport, ME



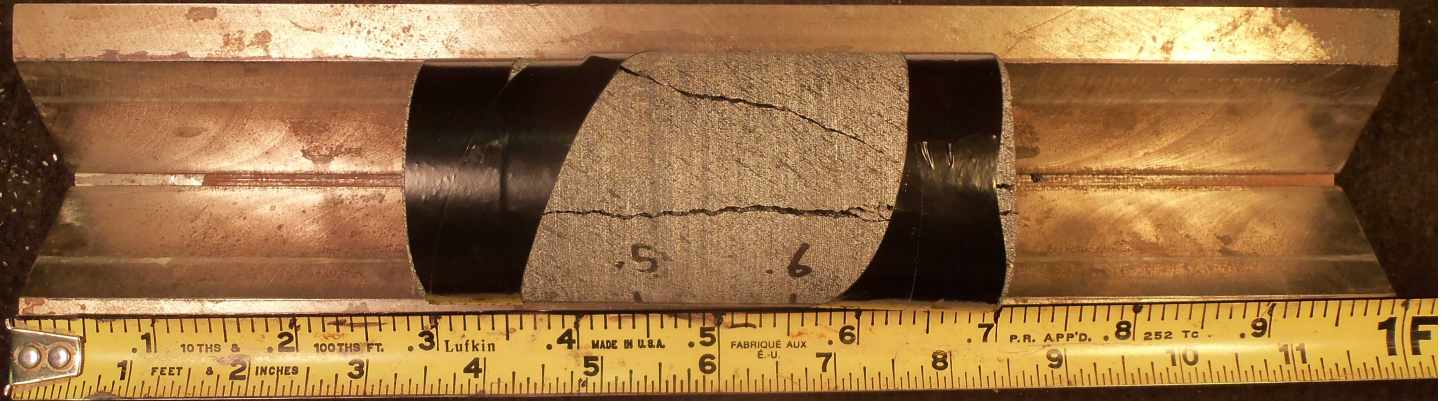
Rock Unconfined Compression Testing - ASTM D7012

Boring No. BB-SDC-204
Sample No. R1
Depth: 11.2-11.6

File No. 09.0025910.01
Date: 1.15.19
Test No. R-30



Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-201</u>	<u>R2</u>	<u>19.2-19.7'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-201</u>	<u>R2</u>	<u>19.2-19.7'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-201</u>	<u>R2</u>	<u>19.2-19.7'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-201</u>	<u>R2</u>	<u>19.2-19.7'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01

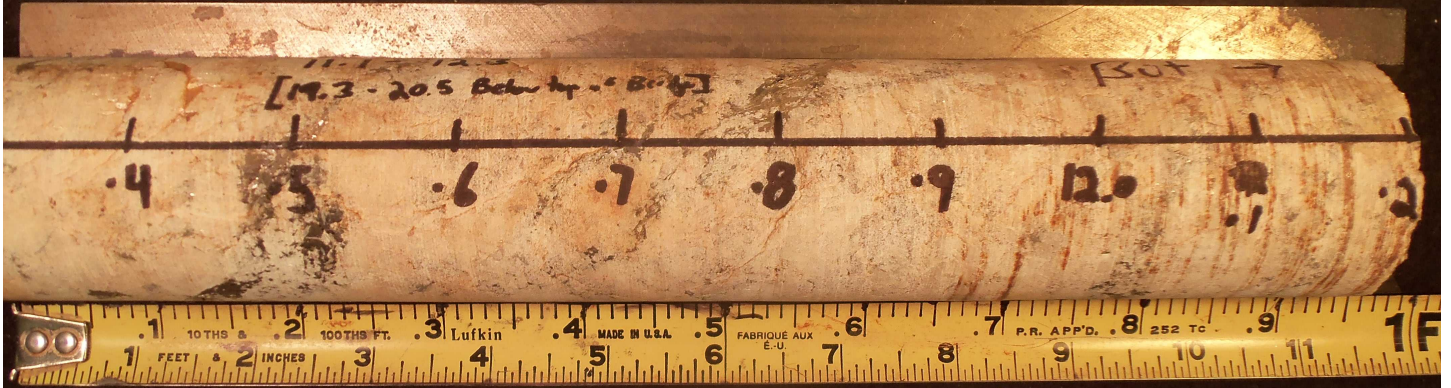
11.1 - 12.3
[19.3 - 20.5 Below top of Bridge]

1
2
3
4
5
6
7
8
9



Boring No.	Sample No.	Depth
<u>BB-SDC-204</u>	<u>R1</u>	<u>11.1-12.3'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



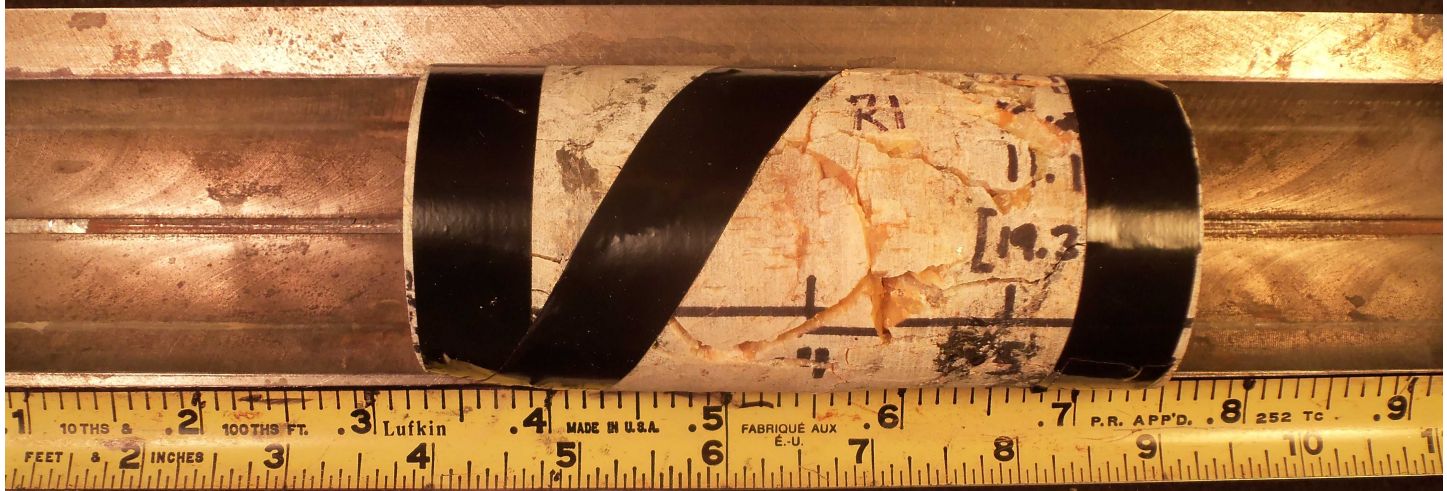
Boring No.	Sample No.	Depth
<u>BB-SDC-204</u>	<u>R1</u>	<u>11.1-12.3'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



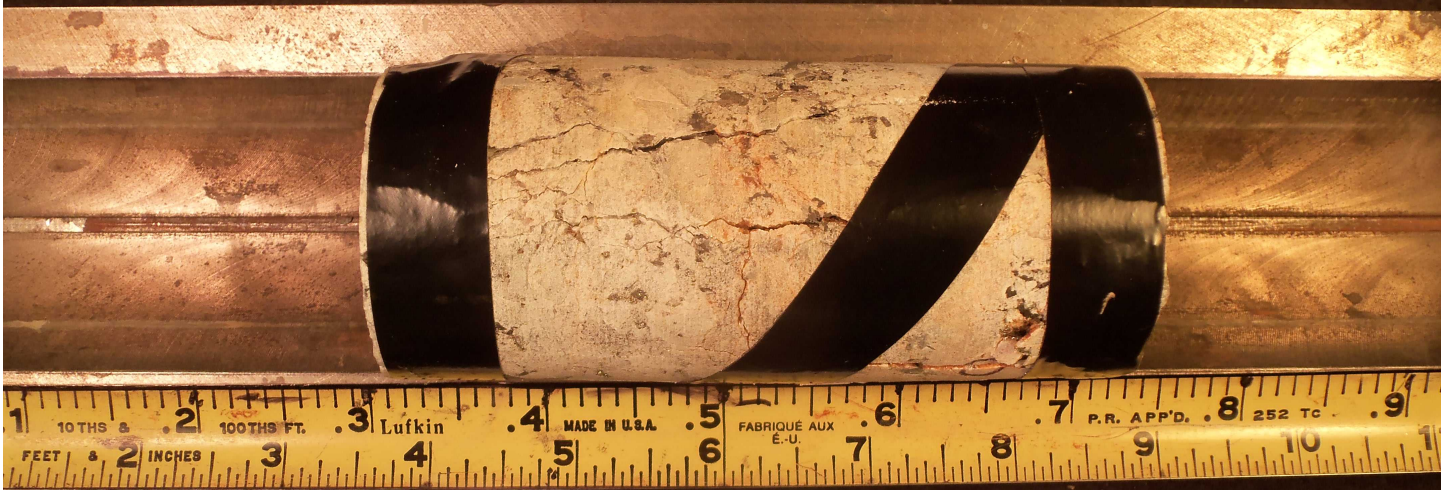
Boring No.	Sample No.	Depth
<u>BB-SDC-204</u>	<u>R1</u>	<u>11.1-12.3'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-204</u>	<u>R1</u>	<u>11.1-12.3'</u>

Thompson Bridge over Decker Cove
Southport, ME
09.0025910.01



Boring No.	Sample No.	Depth
<u>BB-SDC-204</u>	<u>R1</u>	<u>11.1-12.3'</u>



APPENDIX E – CALCULATIONS

SOIL ENGINEERING PROPERTIES



Correlation of SPT N-Values to Internal Friction Angle (ϕ) Using LRFD

Project: Thompson Bridge (Route 238) over Decker Cove
Location: Southport, Maine
Calculated By: NVW **Date:** 1/14/2019
Checked By: ARB **Date:** 1/15/2019

Objective: To estimate the internal friction angle (ϕ) of subsurface strata for the Thompson Bridge (Route 238) Bridge based on Standard Penetration Test (SPT) measurements and empirical correlations using N_{field} , N_{corr} and $(N_1)_{60}$ for use in LRFD analyses.

Method: Correlations between SPT N-values and the soil internal friction angle were considered based on two (2) equations presented in NCHRP Report 651 and in Table 10.4.6.2.4-1 of the AASHTO, 8th Edition (2017). The correlated friction angle using these two (2) correlations are presented in this document. The recommended internal friction angle to be used for design is based on the correlation presented by Peck, Hanson and Thornburn (PHT, 1974) as mentioned in Kulhawy and Mayne (1990) and recommended in NCHRP Report 651 (below).

$$\phi \approx 54 - 27.6034 * e^{-0.014*(N_1)_{60}}$$

Given

Information: SPT measurements and subsurface conditions in borings BB-SDC-103, BB-SDC-202 and BB-SDC-203 performed by New England Boring Contractors between May 25, 2016 and May 26, 2016 and between November 13, 2018 and November 14, 2018 and observed by GZA.

Assumptions: Split spoon samples and SPTs were performed using a rope and cathead. The hammer efficiency of the rope and cathead was taken as 60.0% which was used to calculate N_{60} .

Analysis

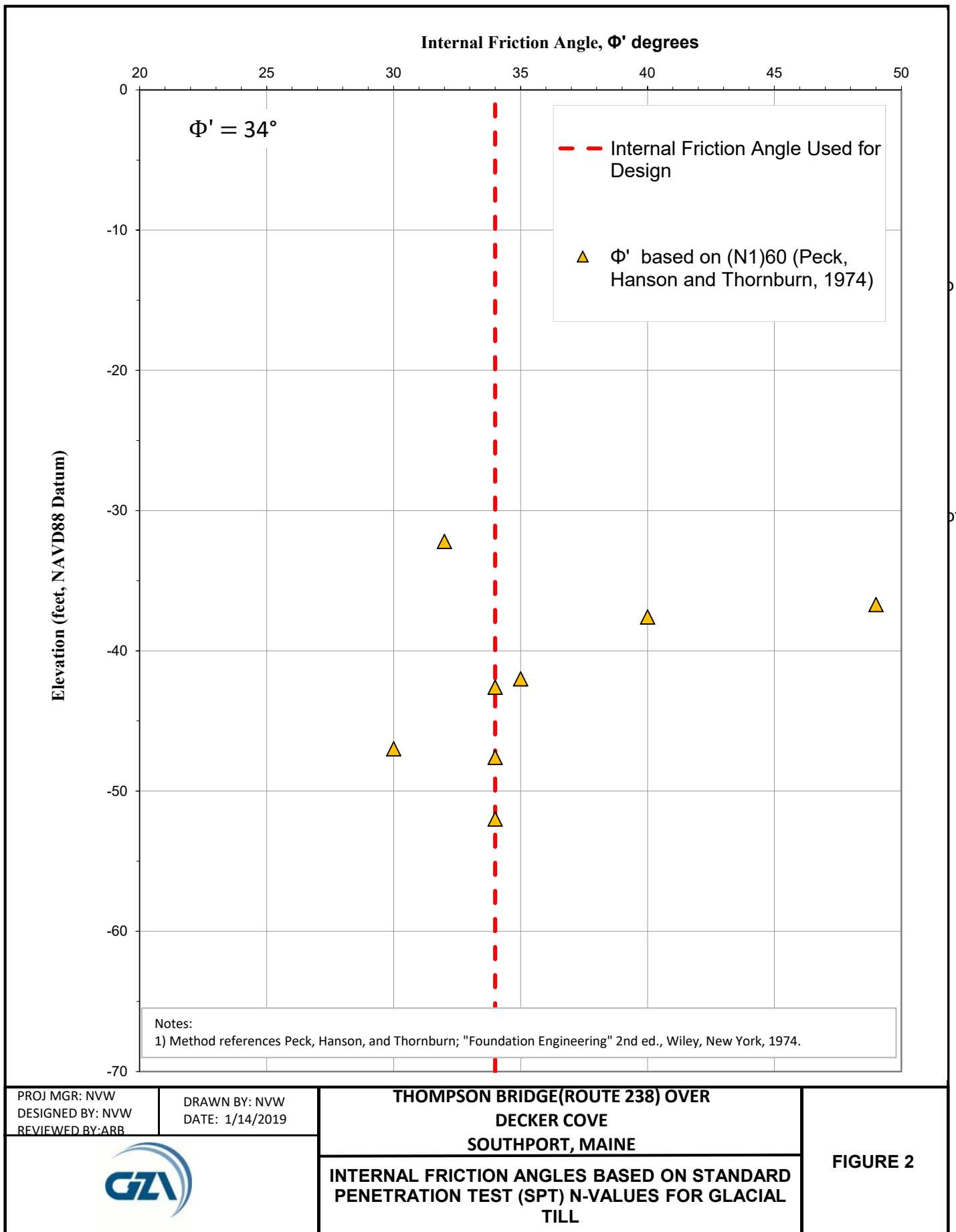
Results:

Layer	Range of SPT $(N_1)_{60}$ - Values	Recomm. ϕ^1 (degrees)
River Bottom	0 to 0	--
Marine Deposit	0 to 35	--
Glacial Till	10 to 100	34
Disintegrated Rock	18 to 100	40

Attachments: 1) Figures
2) Calculations

References:

- 1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017.
- 2) Boring Logs BB-SDC-202, BB-SDC-203, and BB-SDC-103.
- 2) NCHRP Report 651, "LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures," 2010.
- 3) FHWA NHI-06-088, "Soils and Foundations," Volume 1, December 2006.
- 4) Youd, T.L., et. al., "Liquefaction Resistance of Soils: Summary Report From the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils," Journal of Geotechnical and GeoEnvironmental Engineering, October 2001.
- 5) Peck, Hanson, and Thornburn; "Foundation Engineering" 2nd ed., Wiley, New York, 1974
- 6) Kulhawy, F. and Mayne, P (1990). *Manual on Estimation of Soil Properties for Foundation Design*, Report EPRI-EL-6800. Electric Power Research Institute, Palo Alto, CA.



PROJ MGR: NVW
DESIGNED BY: NVW
REVIEWED BY: ARB

DRAWN BY: NVW
DATE: 1/14/2019

**THOMPSON BRIDGE (ROUTE 238) OVER
DECKER COVE
SOUTHPORT, MAINE**



**INTERNAL FRICTION ANGLES BASED ON STANDARD
PENETRATION TEST (SPT) N-VALUES FOR GLACIAL
TILL**

FIGURE 2



Estimation of Horizontal Modulus of Subgrade Reaction (k)

Project: Thompson Bridge (Route 238) over Decker Cove
Location: Southport, Maine
Calculated By: NVW **Date:** 1/14/2019
Checked By: ARB **Date:** 1/15/2019

Objective: To estimate the horizontal modulus of subgrade reaction (k) of subsurface strata for the Thompson Bridge (Route 238) Bridge for use in lateral analyses. K values will be estimated using strata internal friction angles (ϕ).

Method(s): Correlations between SPT N-values and the soil internal friction angle were considered based on two (2) equations presented in NCHRP Report 651 and in Table 10.4.6.2.4-1 of the AASHTO, 8th Edition (2017). The correlated friction angle using these two (2)

Given

Information: SPT measurements and subsurface conditions in borings BB-SDC-103, BB-SDC-202 and BB-SDC-203 performed by New England Boring Contractors between May 25, 2016 and May 26, 2016 and between November 13, 2018 and November 14, 2018 and observed by GZA.

Layer	ϕ (degrees)	Su (Undrained Shear Strength, ksf)
Glacial Till	34	--

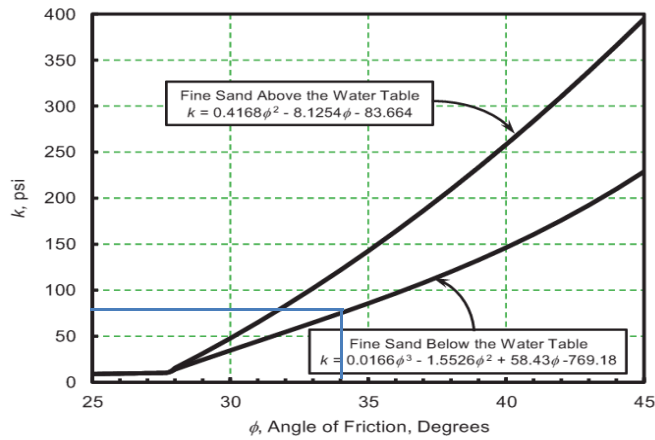


Figure 3-34 Value of k versus Friction Angle for Fine Sand Used in LPile

Assumptions: Split spoon samples and SPTs were performed using a rope and cathead.

Analysis

Results:

Layer	Unit Weight above GWL (pcf)	Unit Weight below GWL (pcf)	Recomm. k above GWL ² (lb/in ³)	Recomm. k below GWL (lb/in ³)
Glacial Till	130	68	120	75

Notes:

- 1) ϕ is based on the Peck, Hanson, & Thornburn (1974) empirical correlation, as mentioned by Kulhawy & Mayne (1990).
- 2) GWL = groundwater level.
- 3) Moduli for stream alluvial are the minimum values recommended in the Lpile Technical Manual.

References: 1) Technical Manual for LPile 2016 by Ensoft, Inc., dated January 14, 2016.

SEISMIC ANALYSIS

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with 2500 ft/sec $< \bar{v}_s < 5000$ ft/sec
C	Very dense soil and soil rock with 1200 ft/sec $< \bar{v}_s < 2500$ ft/sec, or with either $\bar{N} > 50$ blows/ft or $\bar{\tau}_u > 2.0$ ksf
D	Stiff soil with 600 ft/sec $< \bar{v}_s < 1200$ ft/sec, or with either 15 blows/ft $< \bar{N} < 50$ blows/ft or 1.0 ksf $< \bar{\tau}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s < 600$ ft/sec, or with either $\bar{N} < 15$ blows/ft or $\bar{\tau}_u < 1.0$ ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40\%$, and $\bar{\tau}_u < 0.5$ ksf
F	Soils requiring site-specific ground motion response evaluations, such as: <ul style="list-style-type: none"> • Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay, where H = thickness of soil) • Very high plasticity clays ($H > 25$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)
<p>Exceptions:</p> <p>Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site Class E or F should not be assumed unless the authority having jurisdiction determines that Site Class E or F could be present at the site or in the event that Site Class E or F is established by geotechnical data.</p> <p>where:</p> <p>\bar{v}_s = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2</p> <p>\bar{N} = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2</p> <p>$\bar{\tau}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2</p> <p>PI = plasticity index (ASTM D 4318)</p> <p>w = moisture content (ASTM D 2216)</p>	

3.4.2.2—Definitions of Site Class Parameters

The definitions presented below shall be taken to apply to the upper 100 ft of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 100 ft.

The average \bar{v}_s for the site profile shall be taken as:

$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{s,i}}} \quad (3.4.2.2-1)$$

where:

$$\sum_{i=1}^n d_i = \text{thickness of upper soil layers} = 100 \text{ ft}$$

C3.4.2.2

If the site profile is particularly nonuniform, or if the average velocity computed in this manner does not appear reasonable, or if the project involves special design issues, it may be desirable to conduct shear wave velocity measurements. In all evaluations of site classification, the shear wave velocity should be viewed as the fundamental soil property, as this was used when conducting the original studies defining the site categories.

Use of Empirical $v_{s,i}$ Relations: An alternative to applying [Eqs. 3.4.2.2-2](#), [3.4.2.2-3](#), and [3.4.2.2-4](#) to obtain values for \bar{N} , \bar{N}_{cs} , and $\bar{\tau}_u$ is to convert the N values or s_u values into estimated shear wave velocities and then to apply [Eq. 3.4.2.2-1](#). Procedures given in Kramer (1996) can be used for these conversions. The empirical equations identified in Kramer (1996) and in other references can involve significant uncertainty at a specific site, and this

- d_i = thickness of i th soil layer (ft)
- n = total number of distinctive soil layers in the upper 100 ft of the site profile below the bridge foundation
- v_{si} = shear wave velocity of i th soil layer (ft/sec)
- i = any one of the layers between 1 and n

\bar{N} shall be taken as:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (3.4.2.2-2)$$

where:

- N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th soil layer not to exceed 100 ft (blows/ft).

\bar{N}_{oh} shall be taken as:

$$\bar{N}_{oh} = \frac{\sum_{i=1}^m d_i}{\sum_{i=1}^m \frac{d_i}{N_i}} \quad (3.4.2.2-3)$$

where:

- m = total number of cohesionless soil layers in the upper 100 ft of the site profile below the bridge foundation

\bar{s}_u shall be taken as:

$$\bar{s}_u = \frac{\sum_{i=1}^k d_i}{\sum_{i=1}^k \frac{d_i}{s_{ui}}} \quad (3.4.2.2-4)$$

where:

- k = total number of cohesive soil layers in the upper 100 ft of the site profile below the bridge foundation
- s_{ui} = undrained shear strength of i th soil layer not to exceed 5 ksf

3.4.2.3—Site Coefficients

Site coefficients for the peak ground acceleration F_{pgsa} , short-period range F_s , and for the long-period range F_l shall be taken as specified in [Tables 3.4.2.3-1](#) and [3.4.2.3-2](#). Application of these coefficients to determine elastic seismic response coefficients of ground motion shall be as specified in [Article 3.4.1](#).

should be considered during the use of the empirical approach is to calibrate the in-situ velocity measurements when the empirical equations are to be used.

Depth of Motion Determination: For short bridges that involve a limited number of spans, the motion at the abutment will generally be the primary mechanism by which energy is transferred from the ground to the bridge superstructure. If the abutment is backed by an earth approach fill, the site classification should be determined at the base of the approach fill. The potential effects of the approach fill overburden pressure on the shear wave velocity of the soil should be accounted for in the determination of site classification.

For long bridges it may be necessary to determine the site classification at an interior pier. If this pier is supported on spread footings, then the motion computed at the ground surface is appropriate. However, if deep foundations (i.e., driven piles or drilled shafts) are used to support the pier, then the location of the motion will depend on the horizontal stiffness of the soil-cap system relative to the horizontal stiffness of the soil-pile system. If the pile cap is the stiffer of the two, then the motion should be defined at the pile cap. If the pile cap provides little horizontal stiffness or if there is no pile cap (i.e., pile extension), then the controlling motion will likely be at some depth below the ground surface. Typically this will be approximately 4 to 7 pile diameters below the pile cap or where a large change in soil stiffness occurs. The determination of this elevation requires considerable judgment and should be discussed by the geotechnical and bridge engineers.

For cases where the controlling motion is more appropriately specified at depth, site-specific ground response analyses can be conducted to establish ground motions at the point of fixity. This approach or alternatives to this approach should be used only with the Owner's approval.

This guideline would Site Class includes Till and Decomposed Rock, leading to Site Class D (Medium Dense to Dense Soil)

C3.4.2.3

Site Class B (soft rock) is taken to be the reference site category for USGS and IBC ground motion site factors. Site Class B rock is therefore the site condition for which the site factor is 1.0. Site Classes A, C, D, and E have separate sets of site factors for zero-period (F_{pgsa}), the short-period range (F_s), and the long-period range (F_l), as indicated in

Southport Seismic Interpolation for Coefficients

Horizontal Peak ground Acceleration Coefficient – $PGA = 0.069$

Horizontal Response Spectral Accel Coeff for Period of 0.2s – $S_s = 0.140$

Horizontal Response Spectral Accel Coeff for Period of 1.0s – $S_1 = 0.041$

For Class D, values of F_{PGA} and $F_a = 1.6$. $F_v = 2.4$

So

$PGA = 0.069$, $A_s = F_{pga} \times PGA$

$S_s = 0.140$, $S_{DS} = F_a \times S_s$

$S_1 = 0.041$, $S_{D1} = F_v \times S_1$

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F_{pga}	1.6
F_a	1.6
F_v	2.4
A_s (Period = 0.0 sec)	0.110 g
S_{DS} (Period = 0.2 sec)	0.224 g
S_{D1} (Period = 1.0 sec)	0.098 g

Bedrock Bearing Resistance (Abutments)



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Engineers and
 Scientists

Thompson Bridge, Southport
 JOB: 09.0025910.01
 SUBJECT: Bearing Resistance on Bedrock
 SHEET: 1 OF 9
 CALCULATED BY: NVW
 CHECKED BY: ARB
 REVIEWED BY: CLS

Objective

Assess nominal and factored bearing resistance of a foundation on rock based on support in SCHIST/PEGMATITE from borings BB-SDC-101, -102, -104, -105, -201, and -204.

Methodology

Use data from test borings and evaluate the nominal bearing resistance as follows:

1. Bedrock Properties From Test Borings
2. Calculation Of Rock Mass Rating
3. Determine Rock Property Constants s and m
4. Calculate Nominal Bearing Resistance of Bedrock q_n

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD).

Note: AASHTO 8th Edition is now in effect, but the coefficients used in the bedrock bearing evaluations are understood to be correlated relative to the older Hoek and Brown 1988 methodology. Therefore, RMR is used for the evaluation per LRFD 6th Edition rather than GSI per LRFD 8th Edition.

2. Wyllie, Duncan C., "Foundations on Rock", Second edition, 1992.

1. Rock Properties

Bedrock properties were obtained from rock core specimens and logs completed for the Thompson Bridge Project in Southport, ME. Quality of rock was significantly lower in highest rock encountered in BB-SDC-103, -202, and -203 near the middle of the channel, but that rock is not considered representative of the anticipated conditions at spread footing locations and therefore is neglected.

Bedrock Quality

Boring	Depth to Rock (ft)	Depth (ft) Below Top of Rock			Rec (%)	RQD %	Corr. Spacing (in)	Corr. Aperture (in)
		Top		Bottom				
BB-SDC-101	17.4	-1.1	-	4.1	69%	42%	8	0.02-0.1
BB-SDC-101	17.4	4.1	-	9.1	100%	65%	2.5-24	0.02-0.1
BB-SDC-101	17.4	9.1	-	13.6	100%	76%	8	0.01-0.02
BB-SDC-102	21.0	0.0	-	5.0	100%	78%	24	0.02-0.1
BB-SDC-102	21.0	5.0	-	10.0	100%	72%	2.5-80	0.02-0.1
BB-SDC-104	18.5	0.5	-	3.3	92%	61%	2.5-24	0.02-0.1
BB-SDC-104	18.5	3.3	-	5.5	98%	63%	0.75-8	0.01-0.02
BB-SDC-104	18.5	5.5	-	10.0	100%	65%	0.75-24	0.02-0.1
BB-SDC-105	18.9	0.7	-	5.7	100%	90%	2.5-80	0.02-0.1
BB-SDC-105	18.9	5.7	-	10.7	100%	88%	2.5-24	0.02-0.1
BB-SDC-201	5.9	0.8	-	3.0	53%	0%	<0.75-2.5	0.02-0.1
BB-SDC-201	5.9	3.0	-	6.0	67%	22%	<.75	0.02-0.1
BB-SDC-201	5.9	6.0	-	11.0	100%	100%	2.5-24	0.01-0.02
BB-SDC-204	10.2	0.6	-	5.6	100%	95%	2.5-24	0.004-0.1
BB-SDC-204	10.2	5.6	-	10.6	100%	90%	2.5-24	0.01-0.02
					Avg RQD	67%		



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Engineers and
 Scientists

Thompson Bridge, Southport
 JOB: 09.0025910.01
 SUBJECT: Bearing Resistance on Bedrock
 SHEET: 2 OF 9
 CALCULATED BY: NVW
 CHECKED BY: ARB
 REVIEWED BY: CLS

RQD averaged 67% for representative of rock in borings BB-SDC-101, -102, -103, -105, -201, and -204

Bedrock Strength

Boring	Depth to Rock (ft)	LAB				Rock Type
		Depth of Sample into Rock (ft)	UCS (psi)	Modulus (ksi)	Unit Wt (pcf)	
BB-SDC-101	17.4	0.1	11,469	1,920	165.4	SCHIST
BB-SDC-102	21.0	2.4	7,551	4,700	170.3	SCHIST
BB-SDC-102	21.0	7.7	6,321	2,470	163.3	PEGMATITE
BB-SDC-104	18.5	1.8	12,835	4,660	164.1	SCHIST
BB-SDC-105	18.9	1.2	12,063	3,980	162.2	PEGMATITE
BB-SDC-105	18.9	3.8	9,459	3,750	171.1	SCHIST
BB-SDC-201	5.9	13.3	7,340	3,890	181.4	SCHIST
BB-SDC-204	10.2	0.9	5,689	1,840	164.3	PEGMATITE

Select average of measured strengths at BB-SDC-102, BB-SDC-201, and BB-SDC-204, 6,725 psi, covering the range of strengths in the lower half of the laboratory test results.

2. Calculation of Rock Mass Rating (RMR)

From AASHTO LRFD 6th Ed. Table 10.4.6.4-1, determine the RMR.

Parameter 1- Uniaxial Compressive Strength

$\sigma_{u,r} := 6.7 \text{ksi}$ Representative unconfined compressive strength of rock.
 $\sigma_{u,r} = 965 \text{ksf}$

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 4$ for $\sigma_{u,r} = 520 \text{ to } 1080 \text{ksf}$

Parameter 2- Drill Core Quality

Representative RQD: 67%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 13$

Parameter 3- Spacing of Joints

From Boring Logs, generally very close to moderately spaced = <2cm to 60 cm ~ <.75 in to 2 feet



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Thompson Bridge, Southport
JOB: 09.0025910.01
SUBJECT: Bearing Resistance on Bedrock
SHEET: 3 OF 9
CALCULATED BY: NVW
CHECKED BY: ARB
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From AASHTO LRFD Table 10.4.6.4-1

Relative Rating

$$RR_3 := 10$$

Parameter 4- Condition of Joints

From boring logs, hard joint walls and rough surface, with joint separation less than 0.05 in.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 20$

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- Water under moderate pressure

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 4$

Parameter 6-Adjustment for joint orientation

The joint sets are generally moderately dipping and generally rough and open. Therefore the joint orientation is considered Fair.

From AASHTO LRFD Table 10.4.6.4-2

Relative Rating $RR_6 := -7$

Total RMR Rating

$$RMR := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 + RR_6$$

$$RMR = 44$$

From AASHTO LRFD Table 10.4.6.4-3 RMR= 41 to 60 is indicative of Fair Rock Quality



3. Determine Rock Property Constants s and m

Use AASHTO LRFD 6th Ed. Table 10.4.6.4-4 to develop empirical rock property constants

Schist is categorized as rock type D, fine grained igneous (amphibolite, gabbro gneiss, etc.), RMR=47, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 8).

$$m := 0.31$$

$$s := 0.00008$$

4. Calculate Nominal and Factored Bearing Resistance of Bedrock q_n and q_R

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u,r} \cdot \left[1 + \sqrt{m \cdot \left(s \cdot \frac{1}{2} \right) + 1} \right]$$

Where

$$C_{f1} := 1.0$$

From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

$$s = 0.00008$$

For $L/B > 6$, use factor $C_{f1} = 1.0$,

$$m = 0.31$$

For $L/B = 1$, use factor $C_{f1} = 1.12$, therefore,

$$\sigma_{u,r} = 6.7 \cdot \text{ksi}$$

For conservatism, assume long strip, lowest C_{f1} .

Nominal Bearing Resistance

$$q_n := C_{f1} \cdot \sqrt{s} \cdot \sigma_{u,r} \cdot \left[1 + \sqrt{m \cdot \left(s \cdot \frac{1}{2} \right) + 1} \right]$$

$$q_n = 60.2 \cdot \text{ksf}$$

Say 60 ksf

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$$\phi_b := 0.45 \quad \text{Footing on rock}$$

$$q_R := \phi_b \cdot q_n$$

$$q_R = 27.1 \cdot \text{ksf}$$

Say 27 ksf



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➔ Reference:I:\Mathcad\units.xmcd

10-22 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-1 Geomechanics Classification of Rock Masses.

Parameter		Ranges of Values							
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating	15	12	7	4	2	1	0	
2	Drill core quality RQD	90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%			
	Relative Rating	20	17	13	8	3			
3	Spacing of joints	>10 ft.	3–10 ft.	1–3 ft.	2 in.–1 ft.	<2 in.			
	Relative Rating	30	25	20	10	5			
4	Condition of joints	<ul style="list-style-type: none"> • Very rough surfaces • Not continuous • No separation • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock 	<ul style="list-style-type: none"> • Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock 	<ul style="list-style-type: none"> • Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints 	<ul style="list-style-type: none"> • Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints 			
		Relative Rating	25	20	12	6	0		
5	Ground water conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft. tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating	10	7	4	0				



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Table 10.4.6.4-2 Geomechanics Rating Adjustment for Joint Orientations.

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined From Total Ratings.

RMR Rating	100-81	80-61	60-41	40-21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock



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10-24

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table 10.4.6.4-4 Approximate relationship between rock-mass quality and material constants used in defining nonlinear strength (Hoek and Brown, 1988)

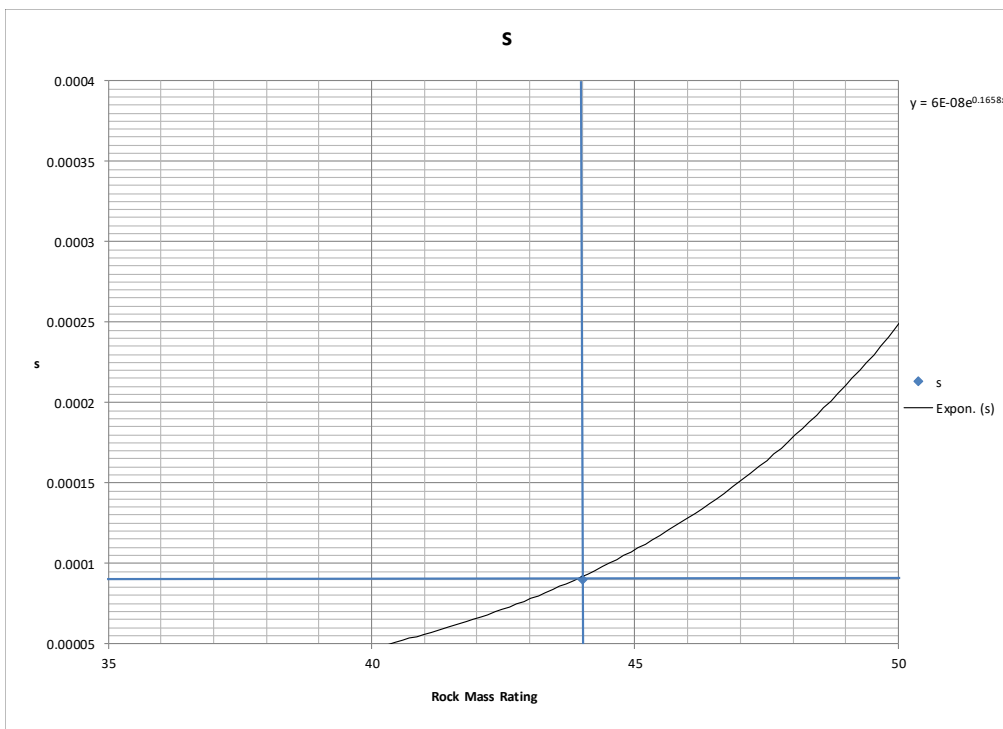
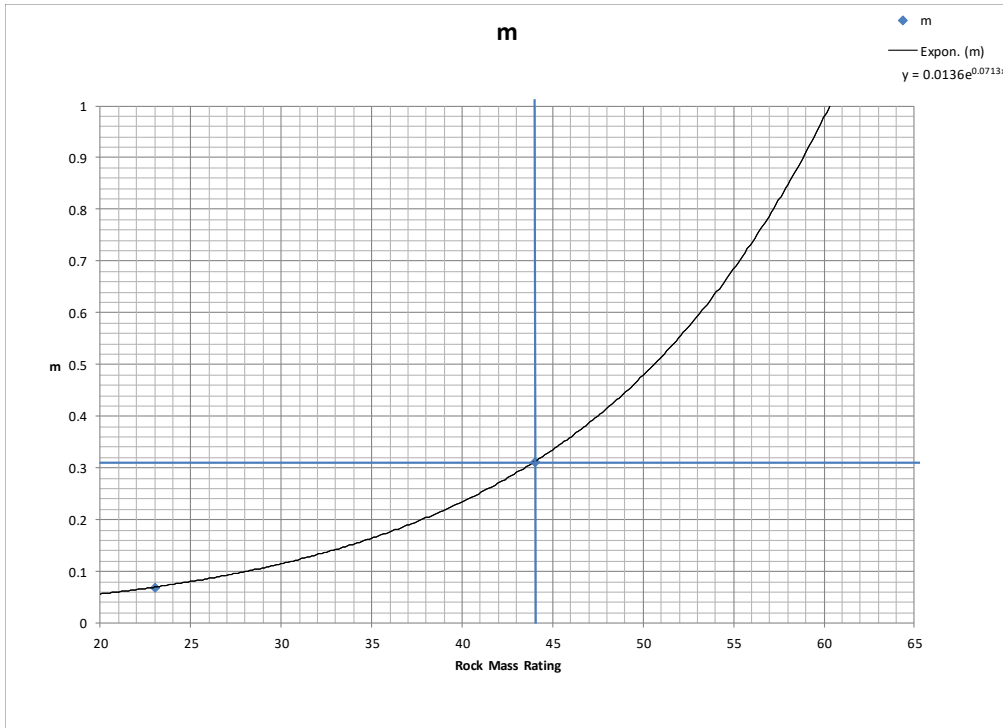
Rock Quality	Constants	Rock Type				
		A = Carbonate rocks with well developed crystal cleavage— <i>dolomite, limestone and marble</i> B = Lithified argillaceous rocks— <i>mudstone, siltstone, shale and slate (normal to cleavage)</i> C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage— <i>sandstone and quartzite</i> D = Fine grained polyminerallic igneous crystalline rocks— <i>andesite, dolerite, diabase and rhyolite</i> E = Coarse grained polyminerallic igneous & metamorphic crystalline rocks— <i>amphibolite, gabbro gneiss, granite, norite, quartz-diorite</i>				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities CSIR rating: <i>RMR = 100</i>	<i>m</i> <i>s</i>	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: <i>RMR = 85</i>	<i>m</i> <i>s</i>	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft. CSIR rating: <i>RMR = 65</i>	<i>m</i> <i>s</i>	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: <i>RMR = 44</i>	<i>m</i> <i>s</i>	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: <i>RMR = 23</i>	<i>m</i> <i>s</i>	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: <i>RMR = 3</i>	<i>m</i> <i>s</i>	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}



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SUBJECT: Bearing Resistance on Bedrock
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LATERAL EARTH PRESSURE



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JOB: 09.0025910.01 Thompsons Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 2
 CALCULATED BY N. Williams 1/30/19
 CHECKED BY A. Blaisdell 2/14/19

Subject: Evaluate lateral earth pressure coefficients

- References:**
1. MaineDOT Bridge Design Guide, Chapter 3 and Chapter 5
 2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)

Input Parameters:

$\beta := 0$ Angle of backfill to the horizontal

$\theta_v := \beta = 0$ Angle of backwall to vertical

$\theta := 90\text{deg}$ Angle of backface of wall to the horizontal

$\phi := 32\text{deg}$ Effective angle of internal friction (*Granular borrow, Soil Type 4, BDG Table 3-3*)

$\delta_f := 19.5\text{deg}$ Interface friction angle between fill and wall (*Formed/precast concrete against Clean Sand, silty sand-gravel mixture, AASHTO Table 3.11.5.3-1*)

$A_s := 0.069$ Peak ground acceleration, g's for Site Class B (footing on rock)

$k_h := 1.2 \cdot A_s = 0.083$ Horizontal earthquake coefficient, 1.2 times the peak ground acceleration for footings on rock (AASHTO 11.6.5.2.1)

$k_v := 0$ Vertical earthquake coefficient assumed to be zero

$$\psi := \text{atan} \left[\frac{k_h}{(1 - k_v)} \right] = 4.7 \cdot \text{deg} \quad \text{Dynamic Angle of Thrust}$$

Earth Pressure on Semi-Integral Abutment/Diaphragm:

Per BDG Section 5.4.3, semi-integral abutments should typically be designed for active or at rest earth pressure over the abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. For a relatively short abutment with footing bearing on rock, at-rest earth pressure coefficient, K_o , is recommended for the abutment. The superstructure backwall (end diaphragm) should typically be designed for full passive pressure. A Coulomb passive earth pressure coefficient, K_p , is recommended. Developing full passive pressure assumes that a ratio of lateral movement to backwall 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 Rankine theory passive earth pressure coefficient.

At-rest Earth Pressure Coefficient

$$K_o := 1 - \sin(\phi) = 0.47$$

$$K_o = 0.47$$



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JOB: 09.0025910.01 Thompsons Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 2 OF 2
 CALCULATED BY N. Williams 1/30/19
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Rankine Passive Earth Pressure Coefficient

$$K_{pr} := \frac{1 + \sin(\phi)}{1 - \sin(\phi)}$$

$$K_{pr} = 3.25$$

Coloumb Passive Earth Pressure Coefficient

$$K_{pc} := \frac{(\sin(\theta - \phi))^2}{\left[(\sin(\theta))^2 \cdot \sin(\theta + \delta_f) \cdot \left[1 - \sqrt{\frac{(\sin(\phi + \delta_f) \cdot \sin(\phi + \beta))}{(\sin(\theta + \delta_f) \cdot \sin(\theta + \beta))}} \right]^2 \right]}$$

$$K_{pc} = 6.73$$

Dynamic Earth Pressure :

Static Coloumb Active Earth Pressure Coefficient

$$\Gamma_{sw} := \left[1 + \sqrt{\frac{\sin(\phi + \delta_f) \cdot \sin(\phi - \beta)}{\sin(\theta - \delta_f) \cdot \sin(\theta + \beta)}} \right]^2 = 2.77$$

$$K_{ac} := \frac{(\sin(\theta + \phi))^2}{\Gamma \cdot \left[(\sin(\theta))^2 \cdot \sin(\theta - \delta_f) \right]}$$

$$K_{ac} = 0.28$$

Dynamic Active Earth Pressure Coefficient:

$$K_{AE} := \frac{\cos(\phi - \theta_v - \psi)^2}{\cos(\psi) \cos(\theta_v)^2 \cdot \cos(\delta_f + \theta_v + \psi) \cdot \left(1 + \sqrt{\frac{\sin(\delta_f + \phi) \sin(\phi - \beta - \psi)}{\cos(\delta_f + \theta_v + \psi) \cos(\beta - \theta_v)}} \right)^2}$$

$$K_{AE} = 0.33$$

Dynamic Active Earth Pressure Coefficient Increment:

$$\Delta K_{AE} := K_{AE} - K_{ac}$$

$$\Delta K_{AE} = 0.05$$

DRIVABILITY ANALYSIS



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JOB: 09.0025910.01 Thompsons Bridge
SUBJECT: Axial Pile Resistance
SHEET: 1 OF 10
CALCULATED BY N. Williams, 1/30/19
REVIEWED BY A. Blaisdell, _____

Objective

Evaluate the axial geotechnical resistance of the center pier piles using a drivability analysis for the Thompsons Bridge Replacement in Southport, ME

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Geotechnical Resistance (Static Analysis)
2. Geotechnical Resistance (Drivability Analysis)
3. Factored Geotechnical Resistance - Strength Limit State
4. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 8th edition, 2017 with interims. (AASHTO LRFD)

Soil Properties

Consider Thompsons Bridge Interpretive Subsurface Cross Section (see Figure 4), subsurface layering and properties relative to pile design are presented in the Soil Engineering Properties calculation and summarized on Sheets 1-3 of this calculation.

Pile Loading

Per the structural engineer, the maximum factored pile load (Strength load case) is 600 kips. Dynamic pile testing/CAPWAP will be complete during construction. Therefore, the required nominal resistance is calculated using a resistance factor of 0.65, resulting in a required nominal resistance of 923 kips. The structural analysis resulted in selecting an HP 18x204.

Structural Properties

HP18x204

Yield Strength of Steel

$$F_y := 50\text{ksi}$$

Area of section

$$A_s := 60.2\text{in}^2$$

Young's Modulus of Steel

$$E_s := 30000\cdot\text{ksi}$$



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JOB: 09.0025910.01 Thompsons Bridge
 SUBJECT: Axial Pile Resistance
 SHEET: 2 OF 10
 CALCULATED BY N. Williams, 1/30/19
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1. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state.

Pile will be installed through an FRP casing installed through the cohesive soils. That will leave approximately 23 feet of till and 8 feet of decomposed rock over competent rock.

Evaluate two conditions:

1. Short Pile: Assume harder rock in decomposed stratum causes abrupt refusal at top of decomposed rock. Consider driving through 23 feet of Till.
2. Long Pile: Assume pile drives through 23 feet of till and 8 feet of decomposed rock, stops with fully plugged tip approximately 8 feet into decomposed rock. Assume decomposed rock behaves like a very dense sand.

Refer to the attached USACE and Nordlund methods used in Apile to estimate side friction resistance during driving. USACE is used for the short pile to give a lower-end estimate to limit the skin friction contribution, and Nordlund is used to give an upper-end estimate to show higher friction/plugging in the till/decomposed rock. The estimated side friction is 100 kips for the short pile USACE analysis and 420 kips for Nordlund long pile analysis. Friction estimates are attached.

Required nominal resistance of 923 kips for design of the center pier based on a maximum factored pile load of 600 kips and a 0.65 resistance factor.

The estimated % skin friction resistance is about 10 and 46% at the required nominal pile resistance.

2. Geotechnical Axial Resistance - Drivability Analysis

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \text{AASHTO Eq. 10.7.8.1}$$

$$f_y := 50 \text{ksi} \quad \text{yield Strength of steel}$$

$$\phi_{da} := 1.0 \quad \text{AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28}$$

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \sigma_{dr} = 45 \cdot \text{ksi} \quad \text{Driving Stress in pile cannot exceed 45 ksi}$$

Short Pile - Drive pile through 23 feet of glacial till to decomposed rock with toe quake representative of very hard driving conditions to assess abrupt take up (0.04in). Model total pile length as 60 feet and embedded length as 23 feet. Model tip end area as pile steel only (not plugged)

Long Pile - Drive pile through 23 feet of glacial till, and 8 feet of decomposed rock with toe quake representative of dense soil driving conditions (toe quake = D/120=0.12 in). Model total pile length as 68 feet and embedded length as 31 feet. Model full tip end area (plugged) and plugged soil in lower 5' of pile.

Drive piles with a Delmag D 36 open-ended diesel hammer with a rated energy of 83,820 ft-lb at the highest fuel setting for both cases. **The proposed hammer is sized to achieve the required nominal pile resistance; not necessarily the maximum drivability resistance for the pile section and profile.**

Allowable stress in piles during driving is $0.9 f_y = 45 \text{ ksi}$

GRLWEAP Output is attached on Sheets 7 through 10.



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 SHEET: 3 OF 10
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Short Pile: $R_{ndr1} := 923 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=37.27 ksi, final penetration resistance=6.9 bpi.

Long Pile: $R_{ndr2} := 923 \text{ kip}$ Required nominal geotechnical resistance, pile driving stress=29.60 ksi, final penetration resistance=7.4 bpi.

3. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

PDA, WEAP and CAPWAP used to establishing driving criteria

$$\phi_{dyn} := 0.65$$

AASHTO Table 10.5.5.2.3-1

Abutment 1: $R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

$$R_{ndr1_factored} = 600 \cdot \text{kip}$$

Abutment 2: $R_{ndr2_factored} := R_{ndr2} \cdot \phi_{dyn}$

$$R_{ndr2_factored} = 600 \cdot \text{kip}$$

4. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{serv_ext} := 1$

From Article 10.5.5.1 and 10.5.5.3

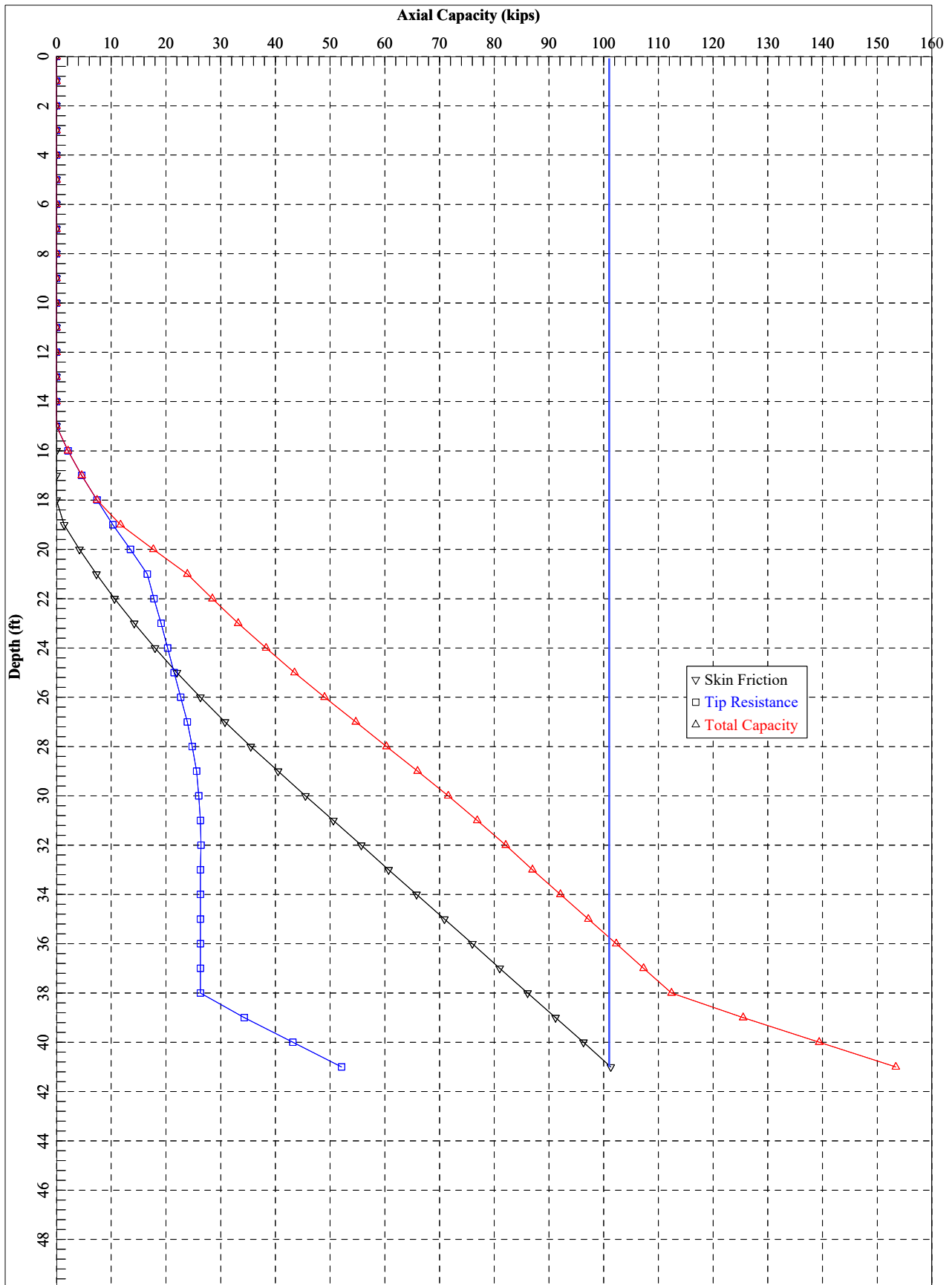
$$R_{ndr1_serv_ext} := R_{ndr1} \cdot \phi_{serv_ext}$$

$$R_{ndr1_serv_ext} = 923 \cdot \text{kip}$$

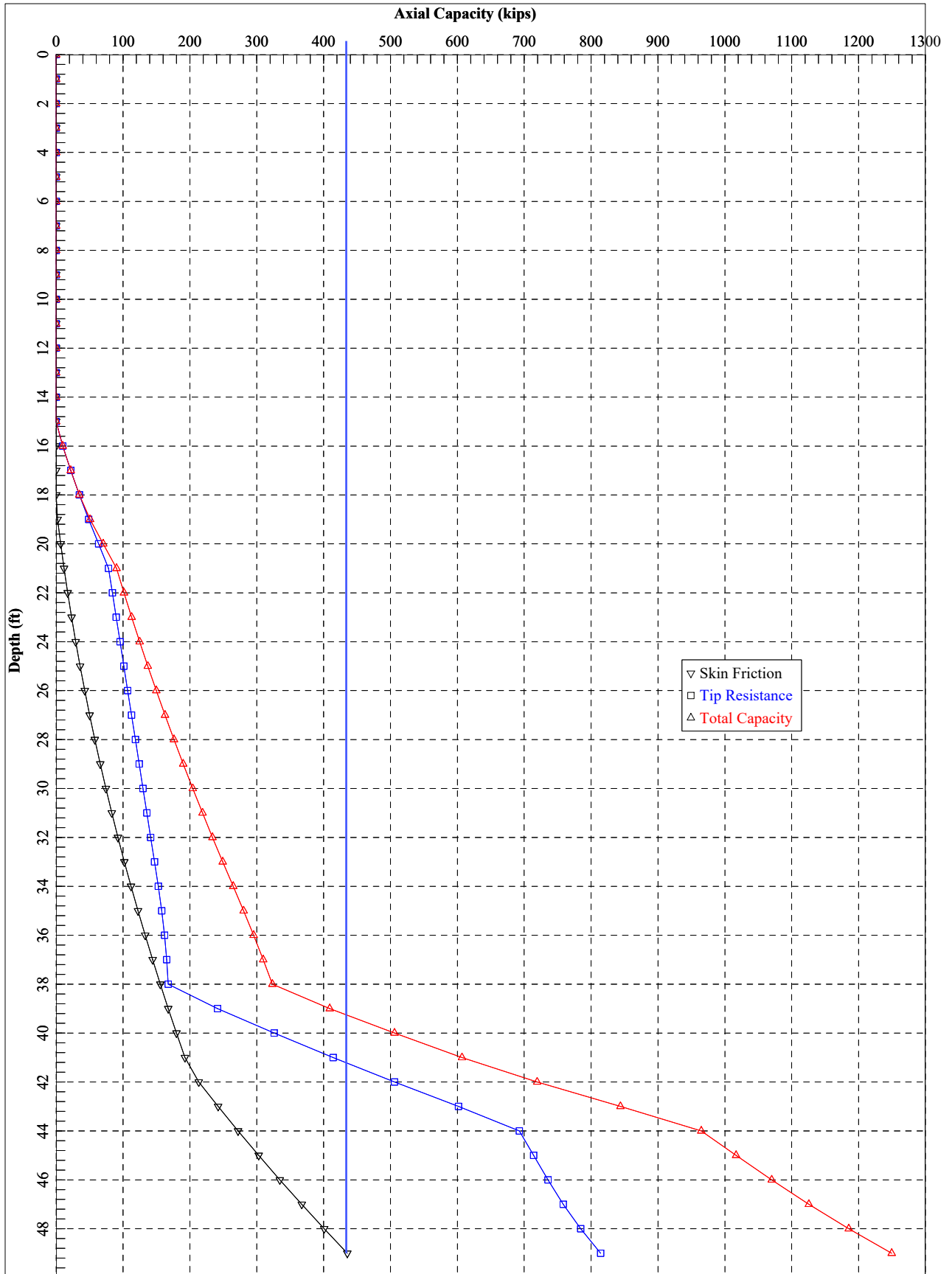
$$R_{ndr2_serv_ext} := R_{ndr2} \cdot \phi_{serv_ext}$$

$$R_{ndr2_serv_ext} = 923 \cdot \text{kip}$$

Short Pile - Unplugged, 18x204
USACE, Side Friction = 100 kips, 10.8% of Nominal



Long Pile, Plugged - 18x204
Nordlund, Side Friction = 420 kips, 45.5% of nominal



Short Pile - 18x204 - Unplugged
Very Hard Driving Conditions

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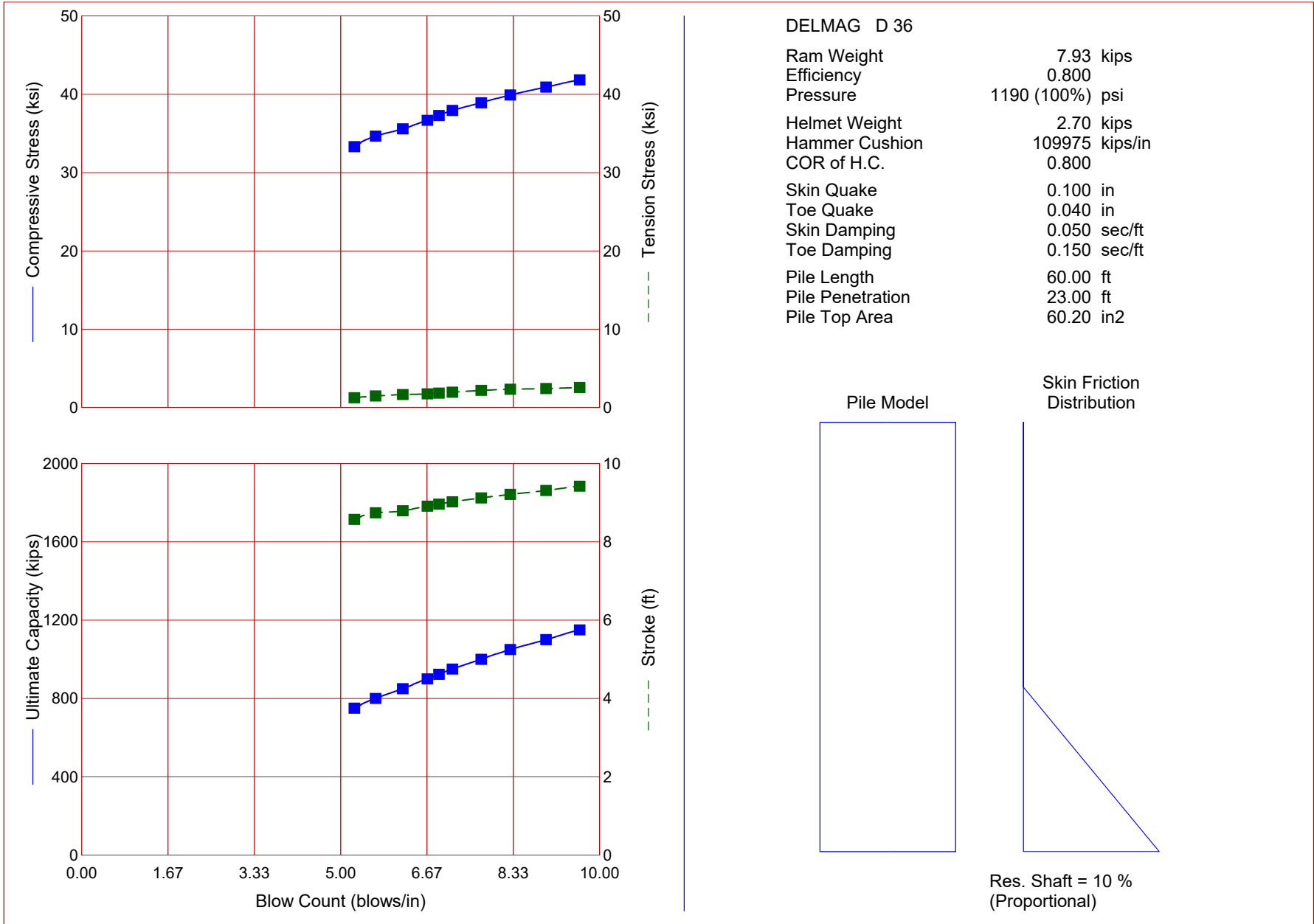
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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
750.0	33.30	1.25	5.3	8.57	35.14
800.0	34.63	1.47	5.7	8.74	35.96
850.0	35.58	1.67	6.2	8.79	36.18
900.0	36.66	1.74	6.7	8.91	36.71
923.0	37.27	1.84	6.9	8.96	36.97
950.0	37.92	1.96	7.2	9.02	37.31
1000.0	38.91	2.19	7.7	9.12	37.74
1050.0	39.90	2.34	8.3	9.21	38.26
1100.0	40.91	2.43	9.0	9.31	38.56
1150.0	41.82	2.56	9.6	9.42	39.07

**Short Pile - 18x204 - Unplugged
Very Hard Driving Conditions**

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Long Pile - 18x204 - Plugged
Dense Driving Conditions

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Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
750.0	28.67	3.25	5.5	8.19	34.26
800.0	28.97	3.16	6.0	8.31	34.66
850.0	29.25	3.04	6.5	8.40	34.87
900.0	29.49	3.14	7.1	8.49	35.21
923.0	29.60	3.22	7.4	8.55	35.32
950.0	29.53	3.28	7.8	8.53	35.13
1000.0	29.69	3.48	8.5	8.61	35.44
1050.0	30.04	3.65	9.2	8.70	35.80
1100.0	30.27	3.81	10.0	8.79	36.19
1150.0	30.42	4.03	10.9	8.88	36.63

**Long Pile - 18x204 - Plugged
Dense Driving Conditions**

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