FINAL GEOTECHNICAL DESIGN REPORT
TIDE MILL 2 BRIDGE NO. 3171 OVER
CRANE MILL BROOK REPLACEMENT
MAINEDOT WIN 19313.00
EDMUNDS TOWNSHIP, MAINE

PREPARED FOR:
TranSystems Corporation
Boston, Massachusetts

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

August 2014
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GeoEnvironmental, Inc.

Engineers and Scientists

VIA E-MAIL

August 20, 2014
File No. 09.0025733.01

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GZA GeoEnvironmental, Inc. (GZA) is pleased to provide you with this Final Geotechnical Design Report prepared for the Crane Mill Brook Bridge replacement project. Our services were provided in accordance with our executed contract dated July 19, 2013, which references the Continuous Services Subcontract Agreement between TranSystems Corporation and GZA dated February 1, 2012 and the attached Limitations included in **Appendix A**.

It has been a pleasure serving you on this project. 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.

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Attachment: Final Geotechnical Design Report, Bridge No. 3171 over Crane Mill Brook

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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) final geotechnical evaluation for the proposed replacement of the Maine Department of Transportation (MaineDOT) Tide Mill 2 Bridge No. 3171 over Crane Mill Brook) located in Edmunds Township, Maine. Our services were provided in accordance with our contract executed on July 19, 2013, and the attached Limitations included in **Appendix A**.

GZA conducted a preliminary geotechnical exploration program in support of the Preliminary Design Report developed by TranSystems Corporation (TranSystems), the results of which were included in GZA's Preliminary Geotechnical Design Basis memorandum dated January 25, 2013. The evaluations and recommendations presented herein supersede the recommendations presented in our January 25, 2013 memorandum. GZA also prepared a summary memorandum containing soil data for scour analysis dated September 11, 2012. The scour analysis was conducted by TranSystems and is not discussed herein.

GZA is providing geotechnical engineering services as a Subconsultant to TranSystems, who is under contract with MaineDOT for final design of the proposed bridge. MaineDOT plans for the bridge contract to be developed as a design-detail package, whereby the package identifies the location and performance requirements of the bridge and the bidder must prepare the sealed, detailed drawings of the bridge structure.

1.1 BACKGROUND

The Crane Mill Brook Bridge carries River Road (U.S. Route 1) over the Crane Mill Brook, as shown on the *Locus Plan*, **Figure 1**. The existing bridge was originally constructed in 1936 and consists of an approximately 125-foot-long, single span, steel through-truss with a concrete deck. Original bridge construction plans indicate the bridge is founded on reinforced concrete spill-through abutments. Each spill-through abutment consists of a bridge seat supported on three concrete columns with footings bearing on bedrock. The existing bridge location is shown on the *Exploration Location Plan*, **Figure 2**.

GZA's understanding of the project is based on the Preliminary Plans (Sheets 1 through 9) provided by TranSystems on June 7, 2013. The replacement bridge is anticipated to consist of a composite arch bridge, or "bridge in a backpack," and will be constructed approximately 45 feet east of the existing bridge. The new bridge will be approximately 60 feet long and 48 feet wide. The bridge superstructure will bear on continuous footings, both of which are planned to have a top of footing elevation of El. 20[1]. The bottom of footing elevations at the south abutment (Abutment 1) and the north abutment (Abutment 2) will be at El. 12 and El. 11, respectively. The proposed bridge location is shown on **Figure 2**.

Geosynthetic reinforced soil (GRS) walls are planned to be used as headwalls, retaining the soil placed above the arches, and wingwalls. MaineDOT GRS walls are a specific subgroup of AASHTO-defined mechanically stabilized earth (MSE) walls that utilize preapproved, large, wet cast blocks, such as Redi-Rock or StoneStrong, with closely spaced geosynthetic reinforcing in the soil mass; and specifically exclude MSE walls with steel strap soil reinforcements and precast concrete panels.

[1] Elevations discussed in this report are in feet and reference the North American Vertical Datum of 1988 (NAVD 88) unless otherwise noted.

We understand the mudline beneath the proposed bridge will remain similar to the existing watercourse, estimated between El. 17 and El. 20. The proposed roadway grades vary between El. 37 and El. 39 at the approach embankments, which is 2 to 4 feet higher than the existing approach embankments. The new bridge alignment will require approach fills up to 19 feet above existing grades at the abutments.

Abutment 1 will be constructed east of the existing south abutment, generally outside of the low-lying area near the brook. Abutment 2 will be constructed southeast of the existing bridge span, generally within the low-lying area of the brook. Approximate existing grades near each substructure are presented in **Table A**.

TABLE A – EXISTING GRADES AT SUBSTRUCTURES	
Substructure	**Approximate Range in Existing Grades (feet, NAVD 88)**
Abutment 1	El. 19 to El. 33
Southeast Wingwall	El. 17 to El. 22
Southwest Wingwall	El. 33 to El. 36
Abutment 2	El. 17 to El. 21
Northeast Wingwall	El. 19 to El. 24
Northwest Wingwall	El. 20 to El. 22

The ordinary high water level ($Q_{1.1}$) is shown at El. 19.47 in the preliminary plans. The 50-year flood (Q_{50}) is shown at El. 23.89, and riprap protection is planned roughly up to this elevation adjacent to the proposed footings and wingwalls.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide geotechnical engineering recommendations for final design of the proposed Crane Mill Brook Bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted a site visit to observe surficial conditions; and reviewed existing bridge plans, and mapped surficial and bedrock geology of the site;
- Coordinated and observed two phases of subsurface explorations consisting of four test borings in the first phase and seven additional test borings, four test probes, and six test pits in the second phase;
- Conducted a laboratory testing program to evaluate engineering properties of the site soils and bedrock;
- Conducted geotechnical engineering analyses to evaluate foundation design for the replacement bridge and wingwalls, embankment global stability and settlement considerations, and seismic design considerations;
- Developed geotechnical engineering recommendations, including foundation alternatives and foundation design recommendations for the preferred bridge abutment and wing wall foundation types, embankment design considerations, seismic design considerations and construction considerations; and
- Prepared this final geotechnical report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a preliminary design exploration program in 2012 consisting of four test borings, and a final design exploration program in 2013 consisting of seven test borings, four test probes and six test pits. The locations and ground surface or bridge deck elevations at the completed explorations were surveyed by MaineDOT. The exploration locations are shown on **Sheet 2**.

Maine Test Boring of Hermon, Maine provided drilling services and coordinated utility clearance and traffic control. The borings conducted for the proposed bridge were drilled using 3-inch or 4-inch driven or spun casing and drive-and-wash drilling techniques. Standard penetration testing (SPT) and split-spoon sampling were performed using were conducted using a safety hammer and a rope-and-cathead system and a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock cores were obtained using NQ2 wire-line coring equipment. Embankment borings and test probes were advanced using 3-inch-diameter solid-flight augers. GZA personnel monitored the drilling work and prepared logs of each boring. Additional details of each program are described below.

2.1 PRELIMINARY EXPLORATIONS

Four test borings (BB-ECB-101 through BB-ECB-104) were completed for the proposed replacement bridge between March 30, 2012 and April 2, 2012. At the time the test borings were drilled, an on-alignment bridge replacement with a shorter span was being considered. Therefore, all of the test borings were completed using a trailer mounter drill rig through holes cored in the existing bridge deck.

Test borings were drilled to depths of approximately 20 to 33 feet below ground surface (bgs) and were terminated approximately 10 to 13.5 feet into bedrock. SPTs were performed continuously through the overburden soils. Logs of each boring are included in **Appendix B**.

The borings were backfilled with ¾-inch crushed stone or soil cuttings, and the bridge deck was patched using Portland cement concrete.

2.2 FINAL EXPLORATIONS

Seven test borings (BB-ECB-201 through BB-ECB-207), four test probes (BB-ECB-P202 through BB-ECB-P205), and six test pits (TP-1 through TP-6) were completed for the proposed replacement bridge, as described below.

2.2.1 Test Borings and Test Probes

The test borings and probes were completed between August 21, 2013 and August 28, 2013 using a track-mounted drill rig. Test boring BB-ECB-205 was advanced through the deck of the existing bridge.

Test borings were drilled to depths of approximately 8 to 31 feet below ground surface. Five of the seven borings were terminated after coring approximately 5.5 to 10 feet into bedrock. The remaining two test borings (BB-ECB-201 and -206) were terminated after encountering auger refusal.

SPT and split-spoon sampling were performed continuously in the upper 10 feet for borings within or adjacent to the brook (BB-ECB- 203 through BB-ECB-205) and at 5-foot typical intervals in overburden soils at greater depths and in other borings. *In-situ* vane shear tests were conducted in cohesive soils in test borings BB-ECB-203 and BB-ECB-204 using a 55x110 mm MaineDOT standard vane. Test probes were advanced to depths of approximately 6.1 to 10.4 feet below ground surface. Each probe was completed to practical refusal using a solid flight auger.

GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**. Test boring and test probe results are summarized in **Table 1**.

2.2.2 Test Pits

Hansom Construction, Inc. of Marshfield, Maine provided excavation services and coordinated utility clearance for the test pit work. The test pits were completed and observed by GZA personnel between August 24, 2013 and August 25, 2013 using a John Deere 80C excavator with a maximum reach of approximately 13.5 feet. Test pit locations were backfilled with the excavated material, placed loosely and tamped down with the excavator in lifts. GZA personnel monitored the excavations and recorded depths to refusal at each of the locations.

Test pits were excavated to depths of approximately 0.5 to 9.5 feet bgs. Each of the test pits encountered refusal. Results of the test pits, including general soil types, are summarized in **Table 1**. Due to the wet conditions at the test pit locations, soil stratification could not be discerned in the test pits. Therefore, logs were not developed.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to complete soil and bedrock testing programs on the BB-ECB-100 series and BB-ECB-200 series borings to assess the gradation and engineering characteristics of the soil and strength of the bedrock. Results of the laboratory testing are included in **Appendix D**.

The testing program consisted of the following:

- Gradation analysis/AASHTO Classification/Frost Classification assessments of 12 soil samples (five from BB-ECB-100 series borings and seven from BB-ECB-200 series borings);
- Hydrometer tests of three soil samples (one from BB-ECB-100 series borings and two from BB-ECB-200 series borings);
- Atterberg Limits tests of seven soil samples (three from BB-ECB-100 series borings and four from BB-ECB-200 series borings),
- Water content assessments on 25 soil samples (eight from BB-ECB-100 series borings and 17 from BB-ECB-200 series borings); and
- Unconfined compression strength and modulus determinations on five representative bedrock core samples (three from BB-ECB-100 series borings and two from BB-ECB-200 series borings).

4.0 SUBSURFACE CONDITIONS

Four soil units were encountered in the test borings overlying bedrock: Topsoil, Fill, Pond Sediments and Glacial Till.

The thicknesses and generalized description are presented in **Table B**, in descending order from existing ground surface.

TABLE B – GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	**Approx. Encountered Thickness (ft)**	**Generalized Description**
Topsoil	2	Soft, dark brown to tan SILT. *Encountered in boring BB-ECB-206*
Fill	0.5 to 10	Loose, brown, fine to coarse SAND, some Gravel, trace to little Silt. Includes 2 to 3 feet of riprap below ground surface at BB-ECB-101 and -104. *Encountered in borings BB-ECB-101 through -104, -201, and -204* • MaineDOT Frost Classification = II
Pond Sediments	3 to 10	Soft to stiff, gray-brown organic SILT, grading with depth to Silty CLAY, trace fine Sand, trace Gravel, with organic fibers. *Encountered in borings BB-ECB-102 through -104, -203, -204, -205, and -207* • MaineDOT Frost Classification = IV Index/strength properties based on laboratory and in-situ testing: • Water Content: 19.0 to 51.4 percent • Liquid limit (LL): 26 to 70 • Plasticity index (PI): 11 to 36 • Field vane strength (Su) of Silty CLAY: 960 to greater than 1,150 pounds per square foot (psf)
Glacial Till	1 to 14	Hard, brown Gravelly SILT, little Sand to medium dense to very dense, gray, fine to coarse SAND, little to some Gravel, little to some Silt. Where both units were encountered, SILT till overlies SAND till. Both units contain cobbles and boulders. • SILT till: MaineDOT Frost Classification = IV, *encountered in borings BB-ECB-103, -201, -203, -204, and -205*. • SAND till: MaineDOT Frost Classification = I to II, *encountered in borings BB-ECB-102 through -104 and -205 through -207*.
Top of Bedrock Elevation	Gabbro or Tuff Breccia, Encountered Top of Rock: Approx. El. -4.4 to El. 20.1	

Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendices B** and **C.** The approximate thickness of each stratum encountered in each boring is summarized in **Table 1 – Summary of Subsurface Explorations**. The encountered top of glacial till and top of bedrock elevations at each exploration is presented on **Sheet 3** and on **Table 1**. The interpreted subsurface conditions are also shown in relation to the bridge proposed abutment and wingwall cross sections on **Sheet 4**.

4.1 BEDROCK

Bedrock was cored in five of the seven test borings and was classified as either Gabbro or Tuff Breccia, as described below.

Gabbro was encountered at boring locations BB-ECB-101, -202, -204, and -207 and was described as hard, fresh to severely weathered, medium grained, and gray. The primary joint set was extremely closely to widely spaced, horizontal to low angle, undulating, rough, open to tight, and discolored to moderately weathered. A secondary joint set was occasionally noted and was widely spaced, high angle to vertical, undulating to planar, rough to smooth, open, and fresh to moderately weathered. Occasional silt deposits were noted on joint surfaces. Oxidation was also noted on the joint surfaces. The Rock Quality Designation (RQD) ranged from 0 to 93 percent, with an average RQD of 59 percent.

Tuff Breccia was encountered at boring locations BB-ECB-102, -103, -104, -203 and -205 and was described as hard, fresh, fine grained, and gray/green with pink garnet inclusions. The primary joint set was very closely to moderately spaced, horizontal to near vertical, undulating, smooth to rough, fresh to discolored, and tight to open. A secondary joint set was noted and was closely to moderately spaced, high angle to vertical, planar, smooth to rough, fresh to slightly weathered, and tight to open. Occasional silt deposits were noted on joint surfaces. Calcite was also occasionally observed at the joints. The RQD ranged from 0 to 90 percent, with an average RQD of 57 percent.

Five laboratory unconfined compressive strength / secant modulus tests were conducted on representative bedrock core samples of Gabbro and Tuff Breccia. The test results are included in **Appendix C** and summarized in **Table C** below. It should be noted that the unconfined compression test on the rock core sample from BB-ECB-203 (17.1 to 17.5 feet bgs) indicated a relative low strength in comparison to the other samples, and the failure for that sample was along a healed fracture. Therefore, the laboratory test result for the sample from BB-ECB-203 is not considered representative of the intact rock mass.

TABLE C – RESULTS OF LABORATORY TESTING ON ROCK

Boring	Run	Depth to Top of Sample (ft)	Elevation of top of Sample (ft)	Unconfined Compressive Strength (ksi)	Modulus (ksi)	Total Unit Weight (pcf)	Rock Type
BB-ECB-101	R1	10.0	19.7	19.01	5.01	179.6	GABBRO
BB-ECB-102	R3	17.0	3.3	27.87	6.92	180.4	TUFF BRECCIA
BB-ECB-104	R1	15.3	14.5	27.29	6.57	169.6	TUFF BRECCIA
BB-ECB-202	R1	1.5	19.6	22.85	6.74	179.6	GABBRO
BB-ECB-203	R3	17.1	1.4	2.06	2.10	176.4	TUFF BRECCIA

4.2 GROUNDWATER

Groundwater was measured in the completed boreholes immediately after drilling at boring locations BB-ECB-201 and BB-ECB-206. Measured groundwater levels were 6.1 feet bgs (El. 32.1) at BB-ECB-201 and 4.3 feet bgs (El. 17.4) at BB-ECB-206. Groundwater levels were obscured in borings BB-ECB-202 through -205 and -207 due to the drive and wash drilling method.

Where feasible, groundwater measurements were obtained in the boreholes completed as part of the 2012 subsurface exploration program, either immediately after drilling or a few hours after drilling. Measured groundwater levels in BB-ECB-102 and BB-ECB-103 were 1.4 feet bgs (El. 18.9) and 1.3 feet bgs (El. 21.3), respectively. At the time of drilling, the brook level was approximately El. 18 to El. 19.

The groundwater observations were made at the times and under the conditions stated in the borings logs. Fluctuations in groundwater and brook levels will occur due to variations in seasonal influences, precipitation amounts, and other factors. Consequently, water levels during and after construction are likely to vary from those encountered at the time the observations noted herein were made.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA conducted geotechnical engineering evaluations in accordance with 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition, with 2013 Interims (herein referred to as LRFD) and the Maine Department of Transportation Bridge Design Guide, 2003 Edition (MaineDOT BDG). The sections that follow describe the evaluations made and the geotechnical basis for evaluation of each element. Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

Approach fills are proposed up to 19 feet above existing grades behind the abutments. The approaches will be retained by GRS wingwalls with a maximum height of about 23 feet, tapering and extending approximately 20 feet behind the abutments. Beyond the wingwalls, earth side slopes are anticipated to have 2 horizontal to 1 vertical (2H:1V), or flatter slopes, with loam and seed surface treatments. Steeper slopes may be utilized near the abutments and wingwalls in conjunction with riprap protection. The new roadway alignment is proposed to be approximately 6 to 19 feet above existing grade on the south side and approximately 19 feet above existing grade on the north side.

5.2.1 Abutment 1 (South) Approach

Subsurface conditions at the south (Abutment 1) approach include granular fill and/or glacial till overlying bedrock. These materials are expected to compress elastically as the new embankment fill is placed. Therefore, post-construction settlements for the south approach are anticipated to be negligible.

The southern approach embankments will be constructed per MaineDOT standard specifications and details using engineered fill placed over fill, glacial till and/or bedrock. In our experience, conventional earthfill embankments and GRS wingwalls constructed over relatively dense overburden soils meet the minimum required safety factors for global stability.

5.2.2 Abutment 2 (North) Approach

Subsurface conditions at the north (Abutment 2) approach include soft to stiff pond sediments, granular fill and dense glacial till overlying bedrock. The estimated pond sediment thickness is approximately 6 to 10 feet in the vicinity of the proposed abutment and gradually decreases with increased distance north of the abutment. Pond sediments were not observed in test boring BB-ECB-206, located approximately 50 feet behind the proposed abutment.

The pond sediments are not considered suitable to support bridge foundations or GRS wingwalls, as described later herein.

Feasible alternatives for construction of the approach embankment include excavation of the pond sediments or construction of the new fill over this layer. If the pond sediments are left in-place beneath the new embankment, several inches of settlement would be anticipated to occur during the next several months following embankment construction. Initial fill placement would also require special care to avoid disturbing this material. It is our understanding that this is not considered to be a desirable alternative by the project team. Therefore, the pond sediments should be removed to expose natural glacial till and replaced with compacted embankment fill prior to construction of the new embankment. As a result, the GRS walls and embankment will be underlain by dense soil or rock, and post-construction settlement and global stability are anticipated to be within tolerable limits.

5.3 EVALUATION OF FOUNDATION TYPE

Cast-in-place concrete footings are proposed to bear at El. 11 to El. 12 to support the proposed bridge abutments. GRS structures will form the sides of the earth fill bridge superstructure and extend as wingwalls into the approach embankments. The wingwall base elevation is proposed to be at El. 14. GZA considered foundation support options based on these bearing levels.

The explorations near Abutment 1 generally encountered bedrock at elevations ranging from El. 14 to El. 20. The exception is probe BB-ECB-P203, just north of the footing limits, where rock is at or below El. 11. Based on these findings, bedrock is anticipated to be present at or above the bearing level for the abutment foundation and the base level of the GRS wingwalls. Therefore, footings bearing on bedrock are the preferred foundation type for the Abutment 1 foundations. GRS wingwalls are also likely to be supported on bedrock but could also bear on undisturbed glacial till depending on the variation in bedrock elevation along the wingwall alignment.

The explorations conducted near Abutment 2 generally encountered up to 10 feet of pond sediments overlying glacial till and bedrock. The top of glacial till elevation is relatively consistent beneath the proposed abutment and wingwalls, between El. 9 and El. 11, and beneath the northeast wingwall where rock was encountered as shallow as El. 14. Therefore, suitable glacial till bearing material is anticipated to be encountered within a few feet of proposed bearing levels. The top of rock ranges from El. -5 to El. 14 across the abutment and wingwalls, as much as 17 feet below footing level. Due to its depth, preparation of a footing subgrade on bedrock would require a cofferdam with a tremie seal. The footing bearing load is not likely to be high enough to warrant the additional expense anticipated to excavate to and prepare a bedrock subgrade. Therefore, footings bearing on glacial till or suitable backfill (i.e., structural fill or concrete fill) placed after removal of unsuitable soil are the preferred foundation type for the Abutment 2 foundations, and GRS walls should also bear on glacial till.

5.4 LOAD AND RESISTANCE FACTORS

AASHTO LRFD factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV) and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength and extreme limit state design.

Recommended LRFD resistance factors for strength limit state design of the abutment spread footing foundations, from LRFD Tables 10.5.5.2.2-1 and 11.5.7-1, are presented in **Table E**.

TABLE E – RESISTANCE FACTORS – STRENGTH LIMIT STATE		
Foundation Resistance Type	**Method/Condition**	**Resistance Factor (ϕ)**
Bearing	Footings on Rock / Theoretical Method in Sand	0.45
Bearing	MSE/GRS Walls	0.65
Sliding	Cast-in-Place Concrete on Rock[1]	0.80
Sliding	Cast-in-Place Concrete on Sand or Tremie Concrete[1]	0.80
Sliding	MSE/GRS Walls	1.0

[1] Sliding resistance factor for concrete on rock or concrete is taken as equal to footing on sand.

Resistance factors for service and extreme limit state design should be taken as 1.0. For service limit design of abutment foundations, GZA has established settlement criteria of ½ inch or less.

5.5 EVALUATION OF FOUNDATIONS

5.5.1 Footing Bearing on Bedrock – Abutment 1

Bedrock at the south abutment underlies medium dense fill material. Footings will be founded on intact bedrock. Therefore, foundation design is controlled by the engineering properties of the bedrock.

GZA developed design engineering parameters for the bedrock mass based on evaluation of the rock types, RQD, and unconfined compression strength data from the test borings, which are presented below:

- RQD Design Value = 25 (ranged from 0 to 93 in borings)
- Design Unconfined Compressive Strength ($\sigma_{u,r}$) = 19 ksi corresponding to the lowest laboratory test value excluding boring BB-ECB-203, for which the sample failed along a healed joint
- Rock Mass Rating (RMR) = 42 (Fair Rock Quality)
- Semi-empirical rock quality constants, m=0.27, s=0.000065 (by interpolation)

The RMR-based approach outlined in LRFD was used to calculate the nominal and factored bearing resistances for spread footings bearing on intact bedrock. Recommended nominal, factored and service bearing resistance values for footings bearing on rock are presented in **Section 6.3**.

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 generally anticipated to experience ½ inch or less of elastic settlement.

5.5.2 Footing Bearing Resistance on Glacial Till – Abutment 2

The subsurface profile below the foundation bearing level at the north abutment consists of pond sediments overlying dense to very dense glacial till and bedrock. The proposed footing level is El. 11. This will require excavation and replacement of existing pond sediments with compacted structural fill or concrete fill. Over-excavation of approximately 2 feet is anticipated to remove the pond sediments, which were encountered as deep as approximately El. 9.

After subgrade preparation, footings will bear on compacted structural fill, concrete fill and/or dense to very dense glacial till. Therefore, foundation design is controlled by the engineering properties of the structural fill and glacial till, which are summarized below:

- Total Unit Weight = 125 pounds per cubic foot
- Representative SPT N-value = 20 blows per foot
- Representative Internal Friction Angle (ϕ') = 34 degrees (for soil anticipated in zone-of-influence of footing)
- Representative Sliding Friction Angle (ϕ_s') = 32 degrees (for soil anticipated at footing-soil interface, reduced from internal friction angle due to potential silty nature of interface material)

Bearing resistance recommendations have been developed for footings bearing on glacial till or compacted structural fill following removal of unsuitable soils based on these parameters and are presented in **Section 6.4** of this report. Preparation of the natural glacial till subgrade and placement of concrete fill or structural fill beneath the footing are critical to performance of the soil-supported foundation; recommended procedures and materials are presented in **Section 7.3**.

Service limit state design was conducted based on an allowable abutment foundation settlement of ½ inch. Foundation settlement is expected to occur elastically as the loads are applied.

5.5.3 Frost and Scour Protection

Soils anticipated to be present at the abutments will consist of either existing fill or new backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1150, and with medium-moisture content (20 percent) soils, due to the water level associated with the brook, the estimated depth of frost penetration is 5 feet. Foundations bearing on soil should be embedded at or below the depth of frost penetration. Spread footings bearing on bedrock require no minimum depth of embedment.

Footing embedment should also meet the following MaineDOT BDG requirements related to scour and streambed considerations:

- BDG 2.3.11.1 Paragraph 2 requires that footings for bridges be constructed a minimum of 2 feet below the design flood scour elevation;
- BDG 2.3.11.1 Paragraph 5 requires that the bottom of spread footings on soils for non-spill through abutments be located a minimum of 6 feet below the lowest streambed elevation in the immediate vicinity of the bridge; and
- BDG 5.3.1.4 states that footings shall be constructed 2 feet below the maximum calculated depth of scour (can be interpreted as scour due the check flood).

We understand that BDG 2.3.11.1 Paragraph 5 controls the design bottom of footing elevation at Abutment 2 (El. 11).

5.6 SEISMIC DESIGN CONSIDERATIONS

The new abutments will be supported on spread footings bearing on bedrock (Abutment 1) or in dense soils (Abutment 2). Existing pond deposits will be removed beneath and adjacent to new foundations before foundation construction. Therefore, the subsurface profile considered for seismic design includes the proposed approach fills (including backfill behind and beneath abutments), natural glacial till and bedrock.

Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value from the soil materials encountered in the borings and assumed properties for new fills. LRFD allows the assumption that rock within the upper 100 feet of the profile has an N-value equal to 100. However, the SPT N-value used to determine the site class was conservatively evaluated by including only the blow counts and thickness of soil above the rock, reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet. On this basis, the SPT N-value fell between 15 and 50 blows per foot. Therefore, the bridge should be assigned to Site Class D. Recommended seismic design parameters are presented in **Section 6.2**.

The test boring data indicate that the fill and glacial till encountered at the site are sufficiently dense and the pond sediments are cohesive so that the potential for liquefaction is very low.

5.7 LATERAL EARTH PRESSURES

We anticipate that the lateral thrust imparted to the abutment footings/stem walls from the bridge arches will limit translation and rotation of the walls. The thrust may be large enough to move the stem walls in toward the backfill. Therefore, at a minimum, the walls should be designed to resist at-rest earth pressures. If the walls are anticipated to deflect into the backfill, the reinforcing design should be checked using passive earth pressures. The material properties will be controlled by the backfill material, which is anticipated to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.5** of this report.

Lateral earth pressure evaluations for abutments and wing walls are based on the BDG and are summarized below:

- Footing-supported stem walls should be designed using at-rest earth pressures, but if wall deflection is anticipated into the backfill, the design should be checked using passive earth pressures.
- Passive resistance in front of the abutment footing should be ignored for sliding and eccentricity evaluations.
- Soil Type 4 (Granular Borrow Underwater Backfill Material or Granular Borrow; MaineDOT Standard Specifications Section 703.19) was used to develop earth pressure coefficients in accordance with Table 3-3 of the BDG.
- Live load horizontal surcharge pressures were evaluated in accordance with Table 3-4 of the BDG.
- Seismic active earth pressure was not considered based on the seismic site class and corresponding Seismic Design Category of the proposed bridge.

Design lateral earth pressure recommendations were developed based on the evaluations summarized above are presented in **Section 6.5** of this report.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment slopes with inclinations of 2H:1V or flatter should be provided with loam and seed for permanent erosion protection. For slopes steeper than 2H:1V, riprap slope protection should be used in accordance with MaineDOT Standard Detail 610(03), Plain Riprap Slope at Structures.

TranSystems will be responsible for selection of scour countermeasures to be employed in front of the new abutments and wingwalls. If riprap slopes are selected, they should be constructed in accordance with MaineDOT Standard Detail 610(03).

Excavated soil may be suitable for reuse in construction of the proposed embankments. Construction considerations regarding material reuse are presented in **Section 7.4**.

6.2 SEISMIC DESIGN

The United States Geological Survey on-line interactive tool, "Seismic Design Parameters Maps," 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. These results are summarized in **Table F**.

TABLE F – SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.127 g
SDs (Period = 0.2 sec)	0.247 g
SD1 (Period = 1.0 sec)	0.094 g

6.3 SPREAD FOOTING DESIGN – ABUTMENT 1 (BEARING ON BEDROCK)

- Abutment 1 should be supported on spread footing foundations bearing on intact bedrock. Footings designed to bear on intact bedrock should be designed for a nominal bearing resistance, q_n, of 151 ksf. At the strength limit state, footings may be designed for a maximum factored bearing resistance of 68 ksf. A bearing resistance of 68 ksf should be used for service limit state design.
- Subfooting concrete fill may be placed to provide a level surface for placement of footing reinforcing steel. The bedrock surface should be free of loose soil and rock at the time of concrete placement for subfooting concrete or the footing. Bearing surface preparation should be completed in accordance with **Section 7.3**.
- The top of bedrock at the Abutment 1 ranges from approximately El. 20 to El. 11. The design bearing level is El. 12 for this abutment. We therefore expect that the bedrock bearing surface will be encountered primarily above the design bearing level. However, after removal of loose soil and rock, the prepared surface may be lower in some areas. It

is important to note that the top of rock is not known for the entire foundation area until it is exposed. Some construction-phase engineering should be anticipated to address the encountered conditions. Considerations for rock removal are addressed in **Section 7.3**.

- Concrete used for subfootings and footings should consist of Class A Concrete, in accordance with MaineDOT BDG guidelines and MaineDOT Standard Specifications Section 502.05 – Composition and Proportioning.
- The base resistance against sliding is based on NAVFAC DM7.02-63, Table 1, which indicates the sliding resistance coefficient (tan δ) is 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 recommended factored sliding resistance coefficient is 0.56 for the Strength Limit State.
- Anchoring, doweling, benching or other means of improving sliding resistance are recommended at locations where the prepared bedrock surface is steeper than 4 horizontal to 1 vertical (4H:1V) in any direction. Based on available boring data, the bedrock slope at Abutment 1 is not expected to exceed 4H:1V following excavation to El. 12.
- Spread footings founded on bedrock should be checked for eccentricity in accordance with LRFD Section 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 farther than 0.45 B from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.
- Since the footings will be founded on bedrock, there is no minimum embedment required for frost protection per BDG Article 5.2.1. However, the footings should be constructed at least 2 feet below the maximum calculated scour depth per BDG Article 5.3.1.4. For footings bearing on rock, the 2 feet of cover may be provided by riprap scour protection.

6.4 SPREAD FOOTING DESIGN – ABUTMENT 2 (BEARING ON SOIL)

- Abutment 2 should be supported on spread footing foundations bearing directly on undisturbed glacial till or on compacted structural fill or concrete fill, placed after removal of unsuitable soils, over undisturbed glacial till. Factored strength limit bearing resistance and service limit bearing resistance (corresponding to ½ inch of foundation settlement) are plotted as a function of effective footing width on **Figure 5**. The structural designer may use the figure to check that the bearing pressure for each strength and service load condition is less than or equal to the bearing pressure for the effective foundation width (accounting for eccentricity). If the proposed bearing pressure plots at or beneath the respective curves for each load case and effective footing width, the foundation design is shown to have met the following conditions:
  - Strength: Factored bearing resistance is greater than the factored bearing pressure; and
  - Service: Anticipated settlement is ½ inch or less.
- The footing should bear on compacted structural fill or concrete fill placed over undisturbed glacial till or directly on undisturbed glacial till. Concrete, structural fill or fill concrete should be placed in-the-dry. Bearing surface preparation should be completed in accordance with **Section 7.3**.

- If bedrock is encountered beneath a portion of the footing, there is a potential for approximately ½ inch of abrupt differential settlement (i.e., over a distance of approximately 10 feet) between the soil-supported and rock-supported portions of the footing, as the rock may be nearly incompressible. If this is considered unsuitable, overexcavation of 12 inches of rock and replacement with compacted structural fill should be performed where rock is encountered at subgrade.
- The base resistance against sliding was evaluated in accordance with LRFD Article 10.6.3.4 using $\phi_s' = \delta = 32$ degrees. Nominal sliding resistance for footings on glacial till is equal to the vertical force multiplied by the sliding resistance coefficient (tan δ), which is equal to 0.62. The recommended factored sliding resistance coefficient is 0.50.
- Concrete used below the footing bearing elevation should consist of Class Fill Concrete, in accordance with MaineDOT BDG guidelines and MaineDOT Standard Specifications Section 502.05 – Composition and Proportioning (Item 502.56).
- Footings founded on structural fill, concrete fill or glacial till should be checked for eccentricity per LRFD 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 1/3 B from the centerline of the footing, where B is the principal dimension of the footing perpendicular to the axis of rotation.
- Passive resistance of soil backfill at the toe of abutment footings and wingwalls should be neglected for resistance to sliding and overturning.
- In accordance with Table 5.1 of the BDG, we recommend that the footing be embedded at least 5 feet below exterior grade for frost protection.

6.5 ABUTMENT DESIGN

- Backfill behind new abutments 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
  - Coefficient of At-Rest Earth Pressure, K_o = 0.47 (abutments, minimum earth pressure)
  - Coefficient of Passive Earth Pressure, K_p = 3.25 (abutments, maximum earth pressure)
- As a minimum, at-rest earth pressure should be assumed to act on the back of the wall to react against the arch thrust. If movement is anticipated with at-rest earth pressure, passive pressure may be engaged. The abutment design should consider the more conservative of at-rest or passive conditions.
- The base resistance against sliding should be evaluated as recommended in **Sections 6.3** and **6.4**.
- At-rest earth pressure and base sliding may be considered in combination to resist arch thrust toward the backfill. If deflection is still anticipated into the backfill, a maximum resisting earth pressure equal to one-third of the design passive resistance should be used in combination with full sliding resistance.

- Live load surcharge should be applied as a uniform vertical 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.
- Foundation drainage should be provided in accordance with Section 5.4.1.4 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments. The drains should outlet through a series of 4-inch diameter weep holes, spaced approximately 10-feet center-to-center.

6.6 GRS WINGWALL DESIGN

The bridge superstructure and wingwalls will consist of vendor- or specialty-firm-designed GRS walls. As mentioned previously, MaineDOT GRS walls are a specific subgroup of AASHTO-defined MSE walls that utilize preapproved, large, wet cast blocks, such as Redi-Rock or StoneStrong, with closely spaced geosynthetic reinforcing in the soil mass; and specifically exclude MSE walls with steel strap soil reinforcements and precast concrete panels. Recommended GRS wall design parameters are listed below:

- GRS walls should be designed and constructed in compliance with the requirements of Special Provision 635, Geosynthetic Reinforced Soil Wall.
- GRS wall design should be in accordance with the requirements of Section 11.10 of LRFD and Section 5.6.5.4 of the BDG.
- Pond sediments encountered at the anticipated bearing elevation should be over excavated to expose dense/very stiff glacial till or bedrock. The unsuitable material should be removed from within the entire plan limits of the reinforcement straps. This is anticipated to be necessary only for construction of the northeast and northwest wingwalls, which are located in low-lying terrain adjacent to the brook limits.
- The GRS walls will be underlain by dense granular soils (compacted structural fill or glacial till) over bedrock or directly on bedrock, and global stability of a similar height embankment underlain by pond sediments was shown to be acceptable (**Section 5.2.2.2**). Therefore, global stability of the GRS walls is judged to be suitable.
- A maximum GRS wall width of approximately 16 feet is anticipated for the wingwalls. Bearing resistance is anticipated to be controlled by the service condition within the range of GRS widths for the project. We recommend factored and service bearing resistance values of 8.5 and 6.5 ksf, respectively, for either natural, undisturbed glacial till or compacted structural fill placed after removal of pond sediments. This recommended service bearing pressure corresponds to a maximum settlement of ½ inch. Greater settlements are generally suitable for the GRS wingwalls, but the bearing resistance values presented above are anticipated to be significantly higher than the bearing pressure from the GRS walls.
- The bottom of the concrete leveling pad placed beneath the GRS wall facing should be embedded at least 5 feet below finish grade for frost protection.
- To reduce the potential for brook water flow to run behind the GRS facing, we recommend that either the concrete leveling pad be sufficiently thick to extend from the glacial till/bedrock bearing surface to the bottom of the facing, or suitable well-graded compacted structural fill (refer to **Section 7.3**) should be placed between the glacial till subgrade and bottom of the leveling pad.

- Fill in the reinforcement backfill zone should be specified as Gravel Borrow (MaineDOT Specification 703.20). The fill in the reinforcement zone shall meet the electrochemical and plasticity requirements in Section 636.049.
- A minimum uniform vertical surcharge load equal to the equivalent fill height (H_{eq}) value in accordance with **Section 6.5** should be applied on the roadway surface above the wall. Lateral surcharge resulting from this load should be calculated and provided by the wall designer.
- GRS wall systems should be capable of sustaining post-construction differential settlements up to 1V:200H in accordance with Section 11.10.4.1 of LRFD.
- A continuous construction joint should be provided where the GRS blocks transition in support conditions from the bridge superstructure to natural/structural foundation material.
- Backfill within the unreinforced zone should meet the requirements of either Granular Borrow for Underwater Backfill or Granular Borrow (MaineDOT 703.19).
- The GRS wall design submittal should provide drainage details to control potential hydrostatic pressure to affect the wall.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 APPROACH EMBANKMENT CONSTRUCTION

Where present, the pond sediments should be fully removed beneath approach embankments to expose natural glacial till, and replaced with compacted granular borrow. In the event of persistent seepage it may be desirable to place crushed stone wrapped in geotextile separation fabric to establish a firm base for compaction and allow dewatering. Backfill details should consider the potential for crushed stone to act as a conduit for brook flow behind/toward the foundations. If necessary, the use of flowable fill would limit this potential.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavation to construct the abutment foundations at the planned bearing levels will extend up to 11 feet below ground surface and up to 11 feet below brook level. At Abutment 1, the excavation will be primarily in rock, and at Abutment 2, the excavation will be primarily in pond sediments.

A sloped, open-cut excavation may be feasible at both abutments. To limit dewatering associated with this alternative, it may be desirable to divert brook flow temporarily by creating a temporary dam and carrying the flow past the excavation in a pipe that discharges downstream from the excavation work.

Depending on the surface water (brook) and/or groundwater conditions at the time of construction, we anticipate that cantilever or braced sheeting systems may be necessary to support temporary foundation excavations at Abutments 1 and 2. Installation of sheet piles may be difficult due to the presence of cobbles/boulders in the glacial till deposit. If sheets are driven below the bearing surface, which is likely at Abutment 2 due to the anticipated deeper excavation requirement and thicker soil, we recommend they be left in place below the bottom of foundation excavation to limit the potential for settlement (after removal).

Temporary dewatering will likely be required to control groundwater inflow in excavations regardless of the preferred excavation method(s). It is anticipated that the inflow of groundwater and infiltration to excavations can be handled by open pumping from sumps installed in sumps at the bottom of excavations. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. Dewatering discharge should be managed in accordance with the contractor's Stormwater Prevention Plan and MaineDOT Best Management Practices.

The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater should comply with all local, State, and federal regulations.

7.3 FOUNDATION SUBGRADE PREPARATION

It is anticipated that the footing excavation for Abutment 1 may be completed in-the-wet within a braced cofferdam, but dewatering will be feasible to allow visual confirmation of the condition of the bedrock bearing surface. A combination of standard excavation equipment, hydraulic hoe-ramming equipment, and/or air lifting may be needed to remove the overburden and fractured/weathered rock. All soil and loose, decomposed, highly weathered and fractured bedrock should be removed from the bearing surface prior to placement of subfooting or footing concrete. The bedrock bearing surface should be prepared as recommended in **Section 6.2**.

Bedrock is anticipated to be encountered as high as 10 feet above the proposed foundation bearing level beneath a portion of Abutment 1. Therefore, we anticipate that drilling and blasting will be required to excavate bedrock for the abutment footings. Blast design should focus on limiting damage to the bedrock surface at the foundation bearing level. Blast design should also consider the presence of the existing bridge immediately west of the required excavation area. We recommend vibration criteria of 2 inches per second or less at the bridge foundation/abutment nearest to the blast area.

It is recommended that the footing excavation and foundation construction for Abutment 2 be completed in-the-dry. Excavation to foundation subgrade should be performed by means that limit subgrade disturbance. Final excavations should be made using a smooth-edge bucket to limit disturbance to the natural glacial till bearing soil. If the subgrade becomes disturbed by construction activities, the disturbed material should be excavated and replaced as specified below. Foundation subgrade should be observed by the geotechnical engineer prior to placement of concrete to confirm that the exposed material is suitable.

It is anticipated that the top of glacial till will be 2 to 3 feet below the current design bearing level beneath most of the Abutment 2. Crushed stone placed in this area could act as a conduit for brook flow beneath the foundations. Therefore, we recommend that the backfill placed between the glacial till subgrade and bottom of footing level consist of compacted structural fill or Class Fill concrete (MaineDOT Item 502.56). Structural fill used beneath footings should be a crushed sand and gravel material meeting the requirements MaineDOT Section 703.20, Gravel Borrow. Gravel Borrow placed beneath foundation should achieve the compaction requirements presented in Section 304.04 of the BDG.

7.4 REUSE OF ON-SITE MATERIALS

Based on the test boring results, the pond sediment material at the abutments contains more than 10 percent passing the No. 200 sieve. Therefore, the excavated material does not meet the MaineDOT requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill

and is unsuitable for reuse as structural backfill. Pond sediments may be suitable for use as Common Borrow outside of the main core of the embankment (i.e., outside of a 1.5H:1V line extending from the edge of the finished shoulder to existing ground) if it can be moisture-conditioned and compacted to a stable condition.

Granular material excavated from the existing approach embankments may meet the MaineDOT requirements for Granular Borrow and/or Granular Borrow for Underwater Backfill. If the contractor wishes to reuse excavated granular material as embankment fill, 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.

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TABLE

Table 1 - Summary of Subsurface Explorations
Crane Mill Brook Bridge
Edmunds Township, Maine

Exploration Designation	Ground Surface Elev. [1,2]	Northing [2]	Easting [2]	Approximate Thickness of Strata Encountered [3]				Top of Glacial Till		Top of Rock [5,6]		Total Exploration Depth (ft)
				Fill	Pond Sediments	Glacial Till	Bedrock	Depth (ft)	Elev. [1]	Depth (ft)	Elev. [1]	
BB-ECB-101	29.7	364109	2481512	9.9	NE	NE	10.0	NE	NE	9.9	19.8	19.9
BB-ECB-102	20.3	364134	2481519	4.5	3.0	4.3	10.2	7.5	12.8	11.8	8.5	22.0
BB-ECB-103	22.6	364197	2481484	0.7	5.8	13.2	13.5	6.5	16.1	19.7	2.9	33.2
BB-ECB-104	29.8	364221	2481491	9	4	0.7	10.0	13	16.8	13.7	16.1	23.7
BB-ECB-201	38.9	363942	2481605	3.5	NE	4.2	0 (R)	3.5	35.4	> 7.7	< 31.2	7.7
BB-ECB-202	21.1	364110	2481564	NE	NE	NE	10.0	NE	NE	1.0	20.1	11.0
BB-ECB-203	18.5	364155	2481579	NE	8.5	2.5	9.5	8.5	10.0	11.0	7.5	20.5
BB-ECB-204	19.0	364163	2481538	0.5	7.6	14.7	5.4	8.1	10.9	22.8	-3.8	28.2
BB-ECB-205	19.4	364175	2481507	NE	10.0	13.8	7.2	10.0	9.4	23.8	-4.4	31.0
BB-ECB-206	21.7	364212	2481521	NE	NE	6.5	0 (R)	2.0	19.7	> 8.5	< 13.2	8.5
BB-ECB-207	19.1	364160	2481550	NE	9.8	8	5.5	9.8	9.3	17.8	1.3	23.3
BB-ECB-P202	35.9	364088	2481535	Sand, Gravel and Silt (see Remark 4)						> 10.4	< 25.5	10.4
BB-ECB-P203	17.2	364111	2481587	Mud, Gravel, Cobbles and Boulders (see Remark 4)						> 6.1	< 11.1	6.1
BB-ECB-P204	18.4	364158	2481563	Clay and Silt (see Remark 4)						> 9.6	< 8.8	9.6
BB-ECB-P205	21.9	364177	2481586	Silt (see Remark 4)						> 9.2	< 12.7	9.2
TP-1	21.2	364107	2481566	See BB-ECB-202 (see Remark 4)						1	20.2	1
TP-2	16.7	364098	2481611	Sand, Gravel and Silt (see Remark 4)						0.5	16.2	0.5
TP-3	17.6	364096	2481603	Sand, Gravel and Silt (see Remark 4)						1-2.5	16.6-15.1	2.5
TP-4	17.5	364110	2481589	Mud, Gravel, Cobbles and Boulders (see Remark 4)						3-3.5	14.5-14.0	3.5
TP-5	18.3	364163	2481581	Cobbles, Boulders and Rock Fragments (see Remark 4)						4-7	14.3-11.3	7
TP-6	18.7	364165	2481586	Cobbles, Boulders and Rock Fragments (see Remark 4)						> 9.5	< 9.2	9.5

Notes:

1) Elevations are in feet and reference the North American Vertical Datum of 1988 (NAVD 88). Coordinates are in feet and reference the North American Datum of 1983 (NAD 83), Maine State Plane 2000 Coordinate System.

2) As-completed exploration locations and elevations were surveyed by a MaineDOT survey crew and provided to GZA.

3) ">" indicates the boring was terminated at the depth indicated into the stratum, "NE" indicates stratum not encountered. "R" indicates refusal.

4) Generalized conditions are provided for probes and test pits; detailed stratification not described due to lack of sampling, wet conditions, and/or no discernible layering.

5) Top of rock corresponds to:

a) top of cored rock at BB-ECB-100 series borings and BB-ECB-200 series borings, except BB-ECB-201 and -206 (not cored);

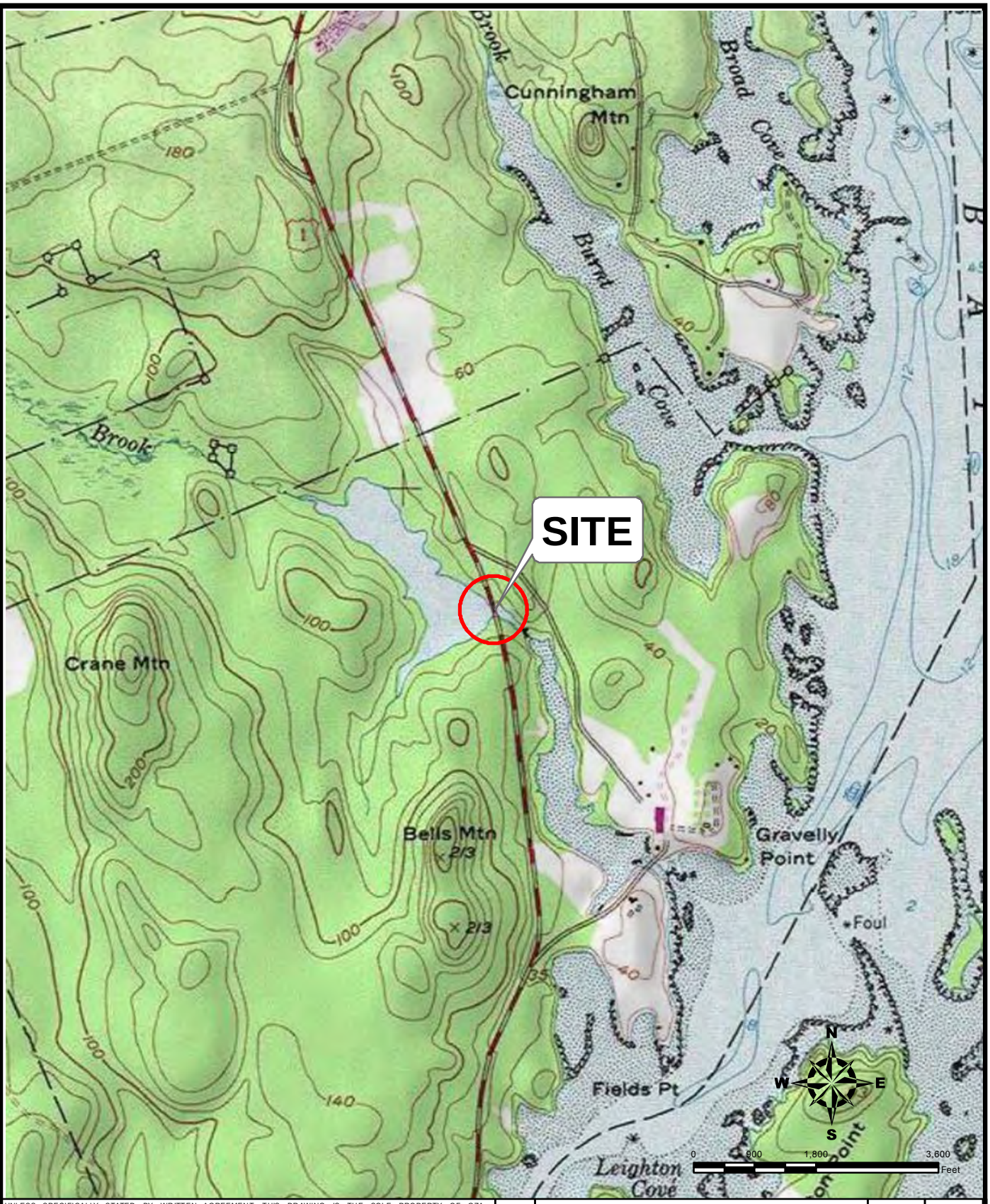
b) exposed rock confirmed in test pit excavations (range in values indicates variation within test pit); or

c) practical refusal of augers for BB-ECB-P200 series probes, boring BB-ECB-201 and -206, and test pit TP-6, and may consist of cobbles, boulders or bedrock.

6) Range in values for test pits indicates variable rock depth; deeper rock elevation was typically closer to the brook.

7) See boring logs in Appendices B and C for additional details.

FIGURES



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NO.	ISSUE / DESCRIPTION	BY	DATE

BRIDGE # 3171 OVER CRANE MILL BROOK
MAINE DOT PIN 19313.00
EDMUNDS TOWNSHIP, MAINE

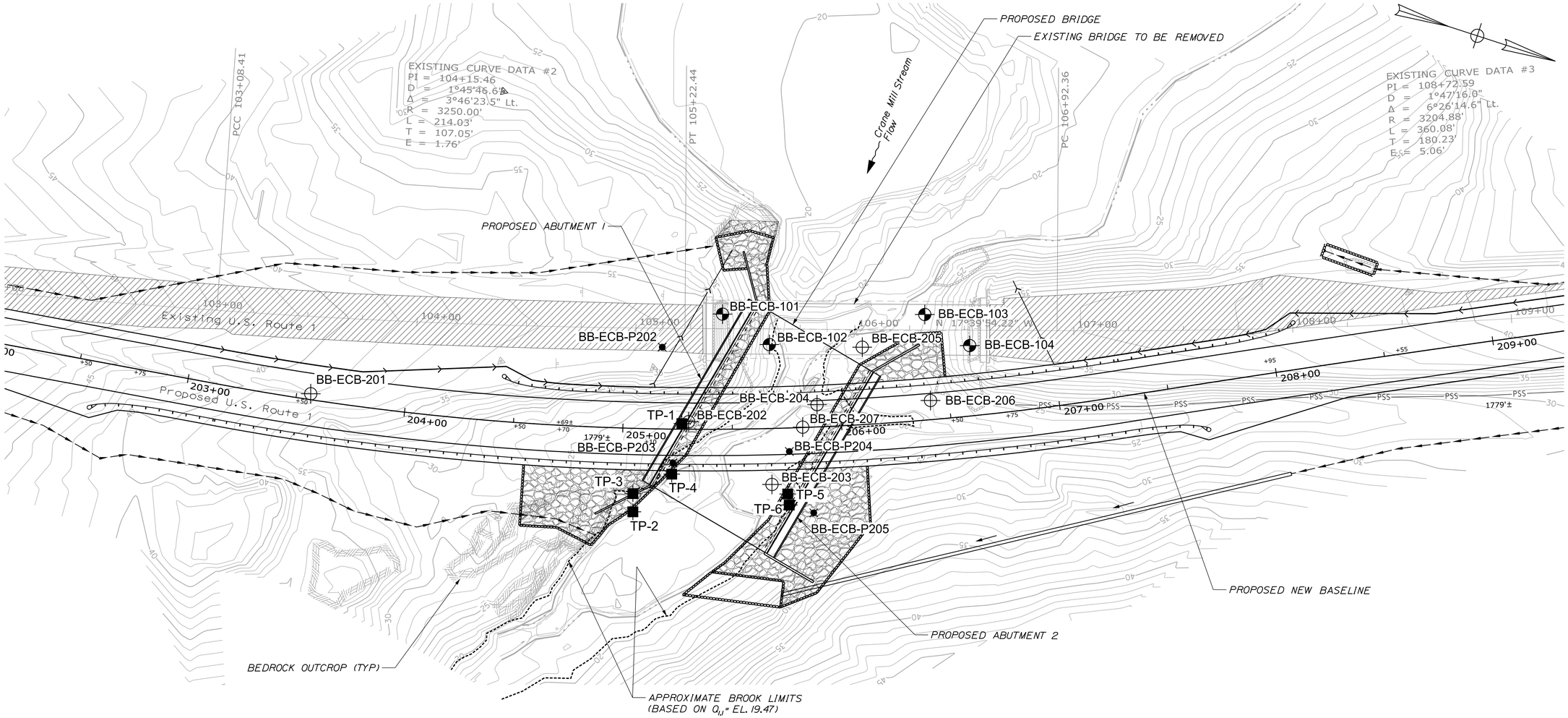
PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
TRANSYSTEMS

LOCUS PLAN

PROJ MGR: ARB	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 1
DESIGNED BY: EJB	DRAWN BY: JAP	SCALE: 1 in = 2,000 ft	
DATE: 06/27/2012	PROJECT NO. 09.0025733.00	REVISION NO.	SHEET NO.

© 2012 - GZA GeoEnvironmental, Inc. P:\09 Jobs\0025700s\09.0025733.00\Figures-CAD\GZA\draft 25733 Figure 1 Locus Map 062712.mxd, 6/27/2012, 3:17:20 PM, jennifer.nisani



NOTES:

1) BASE MAP DEVELOPED FROM ELECTRONIC FILES PROVIDED BY TRANSYSTEMS ON JUNE 20, AND JUNE 23, 2014 (FILES INCLUDED CONTOURS-042913-2D.DGN, WETLANDS.DGN, ALIGNMENT.DGN, TOPO.DGN, AND REPLAN.DWG).

2) AS-COMPLETED EXPLORATION LOCATIONS AND GROUND SURFACE OR BRIDGE DECK ELEVATIONS AND LIMITS OF BEDROCK OUTCROPS WERE SURVEYED BY MAINE DOT AND PROVIDED TO GZA.

3) THE 100-SERIES BORINGS WERE PERFORMED BY MAINE TEST BORING OF HERMON, MAINE BETWEEN APRIL 20, 2012 AND MAY 2, 2012 AND OBSERVED BY GZA PERSONNEL.

4) THE 200-SERIES BORINGS AND PROBES WERE PERFORMED BY MAINE TEST BORING OF HERMON, MAINE BETWEEN AUGUST 21, 2013 AND AUGUST 28, 2013 AND OBSERVED BY GZA PERSONNEL.

5) REFER TO SHEET NUMBERS 24 THROUGH 26 FOR BORING LOGS.

6) THE TEST PITS WERE PERFORMED BY HANSCOM EXCAVATING BETWEEN AUGUST 24, 2013 AND AUGUST 25, 2013 AND OBSERVED BY GZA PERSONNEL.

7) UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHER, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

LEGEND

BB-ECB-201	LOCATION AND DESIGNATION OF DESIGN PHASE TEST BORING (DRILLED BY MAINE TEST BORING FOR GZA)
BB-ECB-104	LOCATION AND DESIGNATION OF PRELIMINARY PHASE TEST BORING (DRILLED BY MAINE TEST BORING FOR GZA)
BB-ECB-P201	LOCATION AND DESIGNATION OF AUGER PROBE (DRILLED BY MAINE TEST BORING FOR GZA)
TP-6	LOCATION AND DESIGNATION OF TEST PIT (PERFORMED BY HANSCOM EXCAVATING FOR GZA)

PLAN

25 0 25 50

Scale of Feet

FIGURE 2

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BH-1931(300)X

WIN 19313.00

BRIDGE NO. 3171

GEOTECH PLANS

		BY	DATE
PROJ. MANAGER	L. TIMBERLAKE		
DESIGN-DETAILED	ETL	ETL	
CHECKED-REVIEWED	ARB	CLS	
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

TIDE MILL 2 BRIDGE
CRANE MILL STREAM
EDMUNDS TWP. WASHINGTON

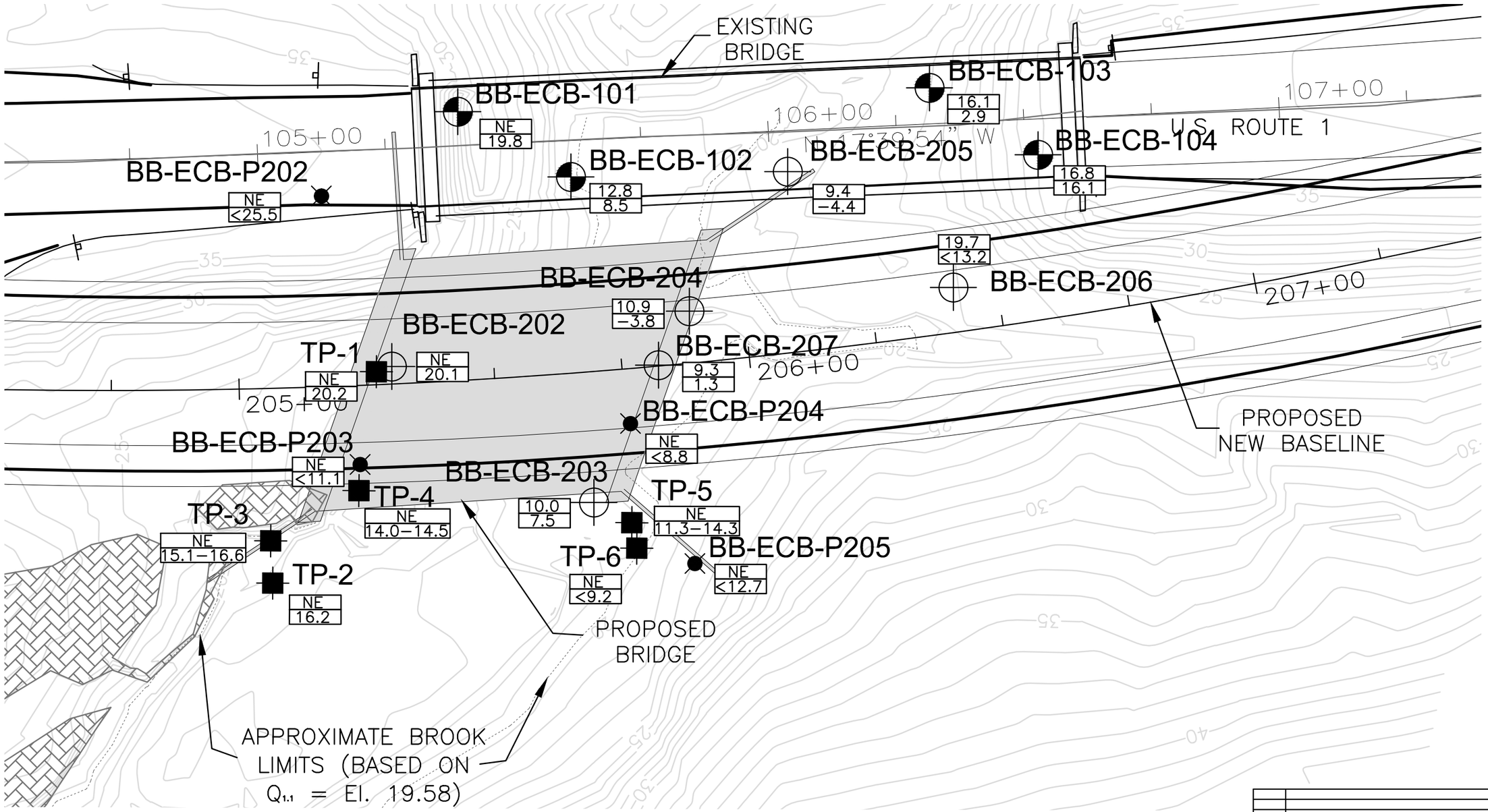
EXPLORATION LOCATION PLAN

SHEET NUMBER

22

OF 49

Filename: ...\GEOTECH\MSTA\022_PLAN_01.dgn Division: GEOTECH Username: D. LANDI Date:8/1/2014



NOTES:

1) SEE NOTES AND LEGEND ON EXPLORATION LOCATION PLAN.

2) REFUSAL DEPTHS IN PROBES AND TEST PITS MAY BE COBBLES/BOULDERS OR BEDROCK.

LEGEND

16.8
16.1

APPROXIMATE TOP OF GLACIAL TILL ELEVATION (FT, NAVD 88)
APPROXIMATE TOP OF ROCK ELEVATION (FT, NAVD 88)
(NE INDICATES STRATUM NOT ENCOUNTERED. < INDICATES REFUSAL ELEVATION, ROCK MAY BE BELOW REFUSAL ELEVATION)

BEDROCK OUTCROPS (LIMITS BASED ON MAINE DOT SURVEY)

PROPOSED BRIDGE

N

0 10 20 40
SCALE IN FEET

NO.	ISSUE/DESCRIPTION	BY	DATE

UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

BRIDGE # 3171 OVER CRANE MILL BROOK
MAINE DOT PIN 19313.00
EDMUNDS TOWNSHIP, MAINE

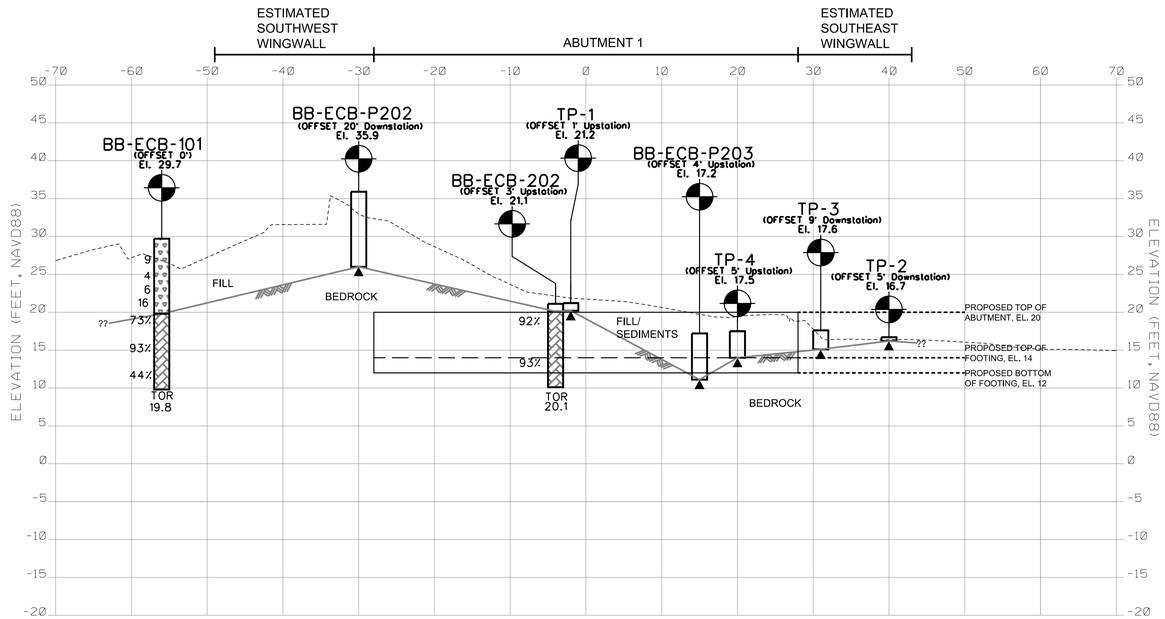
GLACIAL TILL AND BEDROCK SUMMARY PLAN

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

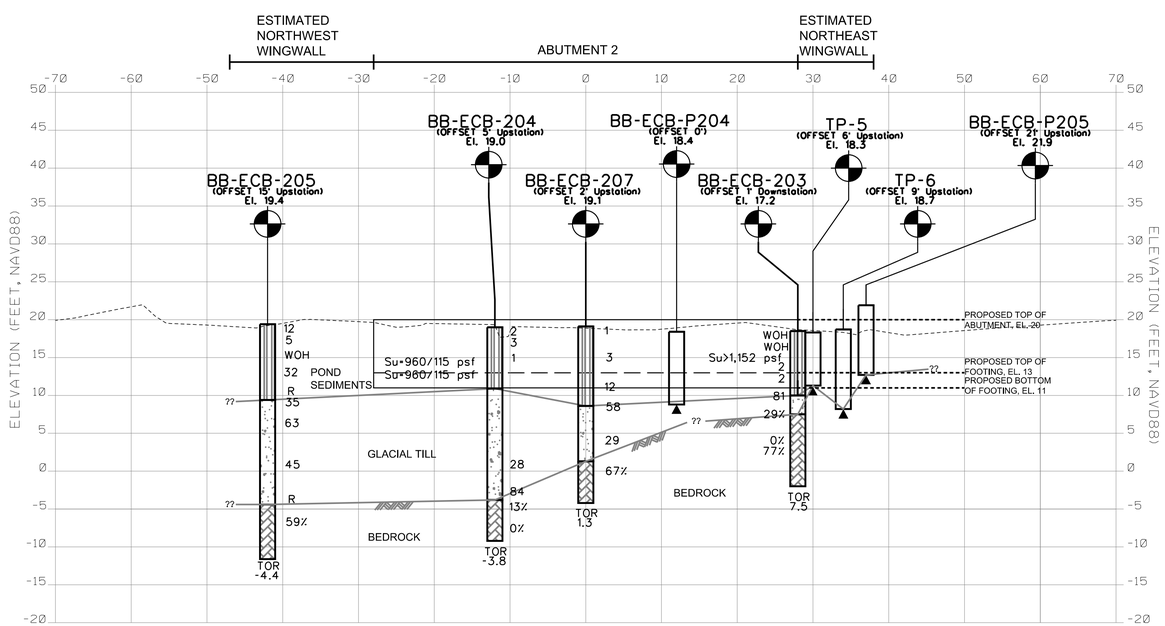
PREPARED FOR:
TRANSYSTEMS

PROJ MGR: ARB	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 3
DESIGNED BY: ETL	DRAWN BY: ETL	SCALE: AS SHOWN	
DATE: MAY 2014	PROJECT NO. 09.0025733.01	REVISION NO.	SHEET NO. 1 OF 1

© 2011 – GZA GeoEnvironmental, Inc. GZA-P:\09 Jobs\0025700s\09.0025733.01\Figures-CAD\GZA\25733.01_EDMUNDS Boring Location Plan.dwg [FIGURE 3] May 08, 2014 – 1:47pm eric.baron



205+23



205+84

BORING STICK HATCH LEGEND:

- FILL
- POND SEDIMENTS
- GLACIAL TILL
- CORED BEDROCK

LEGEND:

BB-ECB-207 (OFFSET 2' Upstation) El. 19.1
- EXPLORATION DESIGNATION
- OFFSET FROM BASELINE
- GROUND SURFACE ELEVATION AT EXPLORATION LOCATION

30 — ENERGY CORRECTED STANDARD PENETRATION TEST (SPT) N
WOH — WEIGHT OF HAMMER
Su=960/115 psf
IN-SITU FIELD VANE SHEAR STRENGTH (PEAK/RESIDUAL): ">" INDICATES VANE DID NOT ROTATE
R — REFUSAL
30% — RQD OF CORE RUN
ROCK CORE
DRILLING/EXCAVATION TERMINATED WITHOUT ROCK CORE
TOR 1.3 — TOP OF CORED BEDROCK (FT-NAVD 1988)

NOTES:

1) BASELINE PROFILE DEVELOPED FROM ELECTRONIC FILE PROVIDED BY TRANSYSTEMS ON DECEMBER 3, 2013 ENTITLED "Geo Coordination.dgn" AND JUNE 20, 2014 ENTITLED "PROFILE.DGN." STATIONING CORRESPONDS TO PROPOSED BASELINE.

2) 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.

3) UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA. ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

FIGURE 4

VERTICAL SCALE IN FEET
0 5 10 20
0 5 10 20
HORIZONTAL SCALE IN FEET

PREPARED BY: GZA

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1931(300)X

WIN 19313.00

BRIDGE NO. 3171

GEOTECH PLANS

PROJ. MANAGER	L. TIMBERLAKE	BY	DATE
DESIGN-DETAILED	ETL	ETL	
CHECKED-REVIEWED	ARB	CLS	
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

TIDE MILL 2 BRIDGE
CRANE MILL STREAM
EDMUNDS WASHINGTON

INTERPRETIVE SUBSURFACE CROSS SECTIONS

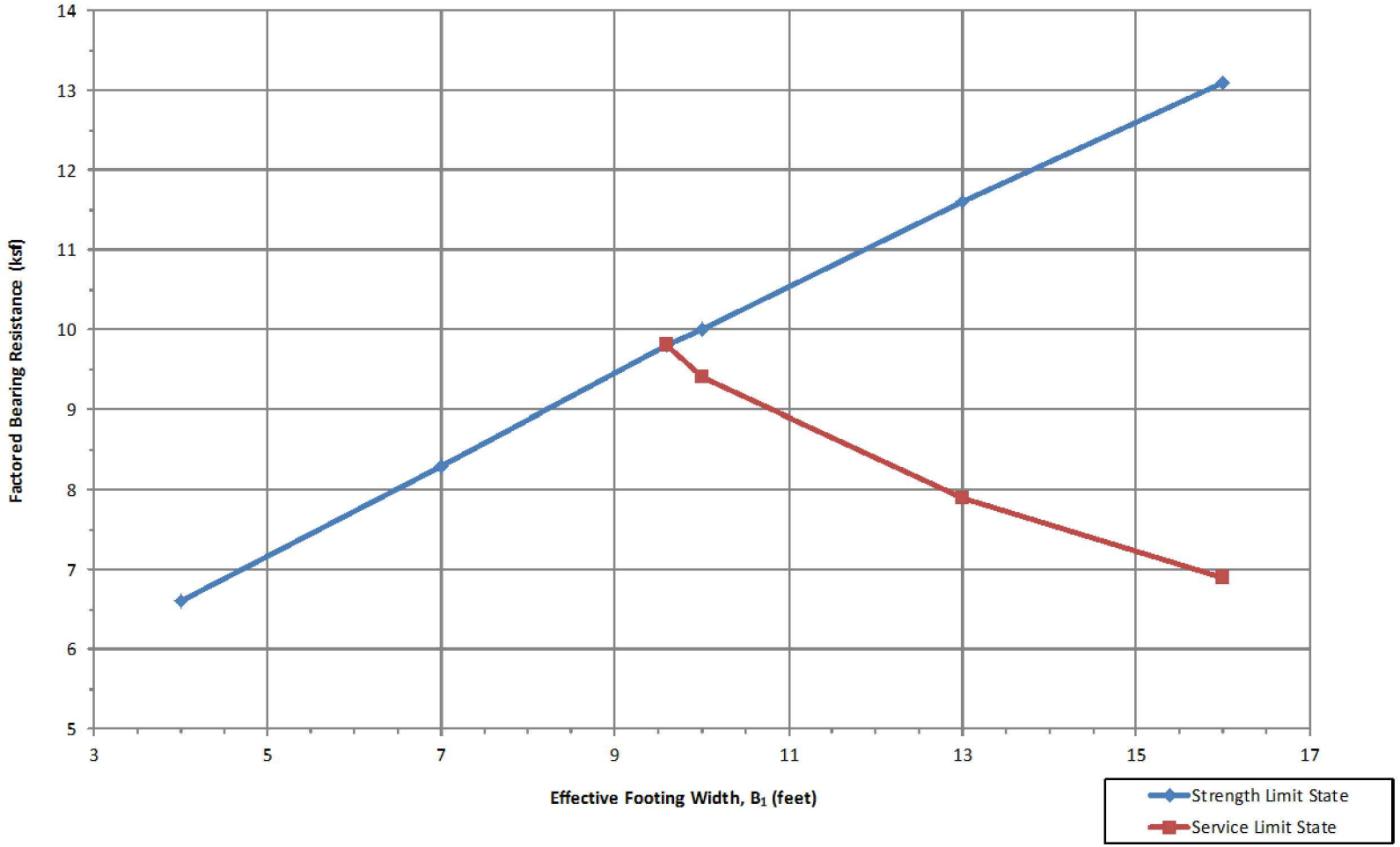
SHEET NUMBER

23

OF 49

Filename: ...\MSTA\023_PROFILE_01.dgn Division: GEOTECH Username: D. LANDI Date:7/28/2014

© 2014– GZA GeoEnvironmental, Inc. GZA–P:\09 Jobs\0025700s\09.0025733.01\Figures–CAD\GZA\09.0025733.01 – Edmunds Final Design.dwg [8.5x11–LAND] June 02, 2014 – 2:09pm evan.lonstein



NOTES:

1. THE STRUCTURAL DESIGNER SHOULD DETERMINE THE BEARING RESISTANCE FOR THE GOVERNING STRENGTH AND SERVICE LIMIT LOAD CASES USING THE CORRESPONDING EFFECTIVE FOOTING WIDTHS.
2. SEE CALCULATIONS IN APPENDIX E FOR DEVELOPMENT OF BEARING RESISTANCE VALUES.

NO.	ISSUE/DESCRIPTION	BY	DATE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists, 477 CONGRESS STREET, SUITE 700, PORTLAND, MAINE 04101, (207) 879-9190

PREPARED FOR: TRANSYSTEMS CORPORATION

BRIDGE # 3171 OVER CRANE MILL BROOK
MAINE DOT PIN 19313.00

EDMUNDS TOWNSHIP, MAINE

BEARING RESISTANCE ON GLACIAL TILL

PROJ MGR: ARB	DATE MAY 2014	FIGURE 5
DESIGNED BY: JLB		
REVIEWED BY: CLS	PROJECT NO. 09.0025733.01	
DRAWN BY: ARB		
CHECKED BY: ARB	REVISION NO.	SHEET NO.
SCALE: N/A		

APPENDIX A

LIMITATIONS

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.

Additional Services

11. 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

BB-ECB-100 SERIES TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

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

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

PIN	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: Tide Mill Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-101
PIN: 19313

Driller:	Brad Enos	**Elevation (ft.)**	29.7	**Auger ID/OD:**	
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	CME-45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5-2-12/5-2-12	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NQ2
Boring Location:	N 364109 E 2481512	**Casing ID/OD:**	4" and 3"	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							Spun		RIPRAP	
								27.7	2.0	
	1D	24/0	2.0 - 4.0	5-5-4-3	9	9	10		No Recovery. Drill cuttings and action indicate brown sand and gravel.	
							6			
	2D	24/1	4.0 - 6.0	3-2-2-2	4	4	9		Loose, brown, GRAVEL, little Sand. -FILL-	
5							10			
	3D	24/3	6.0 - 8.0	2-3-3-8	6	6	10		Loose, brown-gray, medium to coarse Sandy GRAVEL, trace Silt, piece of Gravel stuck in tip. -FILL-	
							13			
	4D	23/7	8.0 - 9.9	7-8-8-50/5"	16	16	13	21.7	8.0 Medium dense, brown-gray, fine to coarse SAND, some Gravel, some Silt with brown organic silt clusters. -FILL-	
	R1	48/47	9.9 - 13.9	RQD = 73%			NQ2	19.8	9.9 Hard, fresh, medium grained, gray, GABBRO. Joints are close to moderately spaced, low angle, undulating, rough, fresh, tight to partially open, oxidation on surfaces. Core Times (min/ft): 2.75, 3.25, 3.0, 3.0 Rock Mass Quality: Fair	q_p=19.0KSI
10										
	R2	54/53	13.9 - 18.4	RQD = 93%					Hard, fresh, medium grained, gray, GABBRO. Primary joints are close to moderately spaced, low angle, undulating, rough, fresh, tight, oxidation on surfaces. Secondary joints are widely spaced, high angle, undulating, rough, fresh to discolored, partially open, oxidation on joints. Core Times (min/ft): 3.5, 3.25, 3.0, 3.25 Rock Mass Quality: Excellent	
15										
	R3	18/17	18.4 - 19.9	RQD = 44%					Hard, fresh, medium grained, gray, GABBRO. Joints are close to moderately spaced, low angle, undulating, rough, fresh, tight, oxidation on surfaces. Core Times (min/ft): 2.75 Rock Mass Quality: Poor	
20								9.8	19.9	
25									**Bottom of Exploration at 19.90 feet below ground surface.**	

Remarks:

1. Boring drilled through bridge deck. 8" thick concrete deck, 5.9 feet from top of deck to ground surface.
2. Bridge deck patched with quick set portland cement after drilling.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tide Mill Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-102
PIN: 19313

Driller:	Brad Enos	**Elevation (ft.)**	20.3	**Auger ID/OD:**	
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	CME-45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	4-30-12/5-1-12	**Drilling Method:**	Driven Casing	**Core Barrel:**	NQ2
Boring Location:	N 364134 E 2481519	**Casing ID/OD:**	4" and 3"	**Water Level*:**	1.4
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							48			
							54			
	1D	24/0	2.0 - 4.0	2-5-4-5	9	9	74		No Recovery. Drill cuttings and action indicate gravel and concrete. -FILL-	
							43			
	2D	24/5	4.5 - 6.5	1-9-2-12	11	11	42	15.8	4.5	
5							51		Medium stiff to stiff, olive-brown and gray, SILT, some Sand, little Gravel, with organic fibers. -POND SEDIMENTS-	
	3D	24/0	6.5 - 8.5	6-10-19-20	29	29	47		No Recovery. Drill cuttings and action indicate similar to 2D with increasing gravel and sand below 7.5'.	
							38	12.8	7.5	
							47		Drill cuttings and action indicate sand, gravel and silt, possible Glacial Till.	
	R1	10/5	9.8 - 10.6				R	10.5	9.8	
10	4D	14/8	10.6 - 11.8	19-36-50/2"			56	10.1	Cored through cobble. 10.2	
	R2	18/18	11.8 - 13.3	RQD = 28%			NQ2	8.5	Very dense, gray, Sandy GRAVEL, little Silt. -GLACIAL TILL- 11.8	
									Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are very close to close, low angle to moderately dipping, undulating, rough, discolored, partially open. Secondary joints are close, high angle to near vertical, undulating, rough, discolored, partially open with silt infillings. Core Times (min/ft): 4.75, 4.5 Rock Mass Quality: Poor	
	R3	56/54	13.3 - 18.0	RQD = 64%						
15									Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are very close to moderately spaced, low angle, undulating, rough, fresh to discolored, partially open with silt infilling. Secondary joints are close to moderately spaced, high angle to near vertical, undulating, rough, discolored, partially open. Core Times (min/ft): 3.75, 3.25, 3.0, 2.75, 2.75 Rock Mass Quality: Fair	q_p=27.9 KSI
	R4	48/45	18.0 - 22.0	RQD = 77%						
20									Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are very close to moderately spaced, low angle to near horizontal, undulating, rough, fresh to discolored, partially open to open with open 1/4" weathered seam at 20.6'. Secondary joint is near vertical, undulating, rough, discolored, partially open. Core Times (min/ft): 2.75, 3.25, 3.25, 3.0 Rock Mass Quality: Good	
								-1.7	22.0	
25									**Bottom of Exploration at 22.00 feet below ground surface.**	

Remarks:

1. Boring drilled through bridge deck. 8" thick concrete deck, 15.4 feet from top of deck to ground surface.
2. Bridge deck patched with quick set portland cement after drilling.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tide Mill Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-103
PIN: 19313

Driller:	Brad Enos	**Elevation (ft.)**	22.6	**Auger ID/OD:**	
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	CME-45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5-2-12/5-2-12	**Drilling Method:**	Driven Casing	**Core Barrel:**	NQ2
Boring Location:	N 364197 E 2481484	**Casing ID/OD:**	4" and 3"	**Water Level*:**	1.25
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.0 - 2.0	2-4-1-2	5	5	18	21.9	Top 8": Black and gray GRAVEL, trace fine to coarse Sand, crushed concrete and asphalt, moist. -FILL-	
							32		0.7	
	2D	24/5	2.0 - 4.0	2-2-2-3	4	4	25		Bottom: Brown-olive, organic SILT, little fine to meduim Sand, wet. -POND SEDIMENTS-	
							12		Medium stiff to stiff, olive-gray, Silty CLAY, little fine Sand, trace Gravel, organic fibers within, wet. -POND SEDIMENTS-	
	3D	24/24	4.0 - 6.0	1-1-3-2	4	4	11		Medium stiff to stiff, gray, CLAY and SILT, little fine Sand, trace organic fibers, wet. -POND SEDIMENTS-	G#1 A-4, CL WC=19.0 LL=26 PL=15 PI=11
5				PP=1.25 ksf			13			
	4D	24/16	6.0 - 8.0	3-3-7-9	10	10	12	16.1	Top 6" Same as 3D	
							17		6.5 Bottom: Loose, gray, Silty fine to coarse SAND, some Gravel. -GLACIAL TILL-	
	5D	24/18	8.0 - 10.0	7-10-11-17	21	21	28		Very stiff, gray, Silty CLAY, some fine Sand, trace Gravel, wet. -GLACIAL TILL-	A-4, CL WC=23.3 LL=40 PI=20
				PP=3.5 ksf			52			
10	6D	24/13	10.0 - 12.0	15-19-25-25	44	44	96		Hard, gray, Clayey SILT, some fine Sand, wet. -GLACIAL TILL-	WC=22.5
				PP=3.25 ksf			88			
	7D	24/7	12.0 - 14.0	12-12-19-26	31	31	36	10.6	12.0 Dense, gray, fine to medium SAND, little Gravel, little Silt, wet. -GLACIAL TILL-	G#4 A-2-4, SM WC=12.9
							52			
	8D	24/8	14.0 - 16.0	32-7-18-23	25	25	68		Medium dense, gray, fine to coarse SAND, some Gravel, little Silt, wet. -GLACIAL TILL-	G#5 A-1-b, SP-SM WC=7.8
15							48			
	9D	24/9	16.0 - 18.0	31-15-12-7	27	27	37		Medium dense, gray, fine to coarse SAND, some Gravel, little Silt, wet. -GLACIAL TILL-	
							44			
	10D	14/9	18.5 - 19.7	11-7-50/2"			38		Medium dense, gray, fine to coarse SAND, some Gravel, little Silt, wet. -GLACIAL TILL-	
	R1	36/36	19.7 - 22.7	RQD = 28%			NQ2	2.9	19.7	
20									Hard, fresh, fine grained, gray and pink, TUFF BRECCIA. Joints are very close to moderately spaced, moderately dipping to high angle, planar to undulating, rough, discolored, partially open, oxidation staining. Core Times (min/ft): 2.25, 3.0, 3.25 Rock Mass Quality: Poor	
	R2	51/50	22.7 - 27.0	RQD = 42%					Hard, fresh, fine grained, gray and pink, TUFF BRECCIA. Joints are very close to moderately spaced, moderately dipping to high angle, planar to undulating, rough, discolored, partially open, oxidation staining. Core Times (min/ft): 2.75, 2.75, 3.0, 3.5 Rock Mass Quality: Poor	
25										

Remarks:

1. Boring drilled through bridge deck. 8" thick concrete deck, 12.8 feet from top of deck to ground surface.
2. Bridge deck patched with quick set Portland cement after drilling.
3. PP indicates pocket penetrometer used to estimate shear strength on split spoon sample; results are listed below blow counts for the sample tested.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tide Mill Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-103
PIN: 19313

Driller:	Brad Enos	**Elevation (ft.)**	22.6	**Auger ID/OD:**	
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	CME-45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5-2-12/5-2-12	**Drilling Method:**	Driven Casing	**Core Barrel:**	NQ2
Boring Location:	N 364197 E 2481484	**Casing ID/OD:**	4" and 3"	**Water Level*:**	1.25
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	R3	38/36	27.0 - 30.2	RQD = 38%						Hard, fresh, fine grained, gray and pink, TUFF BRECCIA. Primary joints are very close to close, low angle, undulating, rough, discolored, partially open. Secondary joints are close, high angle, undulating, rough, discolored, partially open, oxidation staining. Core Times (min/ft): 2.5, 2.75, 2.5 Rock Mass Quality: Poor	
30	R4	36/36	30.2 - 33.2	RQD = 25%						Hard, fresh, fine grained, gray and pink, TUFF BRECCIA. Joints are close to very close, moderately dipping to high angle, undulating, rough, discolored, partially open to open, with 1/8" open joint at 32.4' with sand and silt infilling Core Times (min/ft): 3.0, 2.75, 3.25 Rock Mass Quality: Very Poor	
								-10.6		33.2 **Bottom of Exploration at 33.20 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

1. Boring drilled through bridge deck. 8" thick concrete deck, 12.8 feet from top of deck to ground surface.
2. Bridge deck patched with quick set Portland cement after drilling.
3. PP indicates pocket penetrometer used to estimate shear strength on split spoon sample; results are listed below blow counts for the sample tested.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Tide Mill Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-104
PIN: 19313

Driller:	Brad Enos	**Elevation (ft.)**	29.8	**Auger ID/OD:**	
Operator:	Brad Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	CME-45	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	5-1-12/5-1-12	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NQ2
Boring Location:	N 364221 E 2481491	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							Spun		Drilling action indicates cobbles/boulders from 0 to 2.8 feet. -RIPRAP-	
								27.0	2.8	
	1D	24/7	3.0 - 5.0	4-4-6-9	10	10	15		Loose, brown, fine to coarse SAND and GRAVEL, fine to coarse SAND , little Silt, gravel piece in tip, wet. -FILL-	G#6 A-1-a, SM WC=8.5
							20			
5	2D	24/10	5.0 - 7.0	7-5-5-4	10	10	22		Loose, brown, fine to coarse SAND, some Gravel, little Silt wet. -FILL-	
							27			
	3D	24/5	7.0 - 9.0	4-2-1-3	3	3	11		Very loose, brown to gray, fine to coarse SAND, some Gravel, little Silt, wet. -FILL-	G#7 A-1-b, SM
							2			
								20.8	9.0	
	4D	24/20	9.0 - 11.0	4-1-1-2	2	2	12		Soft to medium stiff, gray-olive, organic SILT, little Sand, trace Gravel, wet. -POND SEDIMENTS-	A-7-6, OL LL=43 PI=19 WC=37.4 WC=17.6
10				PP=1.0 ksf			18			
								18.8	11.0	
	5D	24/16	11.0 - 13.0	2-4-4-11	8	8	20	18.1	Top 8": Brown-gray, fine to medium SAND, trace Silt, with occasional organic silt clods.	
							19		11.7	
								16.8	Bottom: Gray, Sandy SILT, little Gravel, wet. -POND SEDIMENTS-	
	6D	8/6	13.0 - 13.7	23-50/2"			NQ2		13.0	
	R1	60/60	13.7 - 18.7	RQD = 90%				16.1	Gray, fine to medium SAND, some Gravel, little Silt, increasing gravel content with depth, wet. -GLACIAL TILL-	
15									13.7	
									Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are moderately spaced, low angle, undulating, rough, fresh to discolored, tight. Secondary joints are very close to moderately spaced, high angle, undulating, rough, discolored, tight, oxidation staining. Core Times: 2.75, 3.25, 3.0, 2.5, 2.75 Rock Mass Quality: Good	q_p=27.3 KSI
	R2	52/52	18.7 - 23.0	RQD = 90%						
20									Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are moderately spaced, low angle, undulating, rough, fresh to discolored, tight. Secondary joints are very close to moderately spaced, high angle, undulating, rough, discolored, tight, oxidation staining. Core Times: 2.75, 2.5, 2.75, 3.0, 2.0 Rock Mass Quality: Good	
	R4	8/7	23.0 - 23.7	RQD = 88%				6.1	Hard, fresh, fine grained, gray, TUFF BRECCIA. Single Piece. Core Times: 2.75 Rock Mass Quality: Good	
25									23.7	
									Bottom of Exploration at 23.70 feet below ground surface.	

Remarks:

1. Boring drilled through bridge deck. 8" thick concrete deck, 5.9 feet from top of deck to ground surface.
2. Bridge deck patched with quick set Portland cement after drilling.
3. PP indicates pocket penetrometer used to estimate shear strength on split spoon sample; results are listed below blow counts for the sample tested.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

APPENDIX C

BB-ECB-200 SERIES TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

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

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Sample Container Labeling Requirements:

PIN	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: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-201
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	38.9	**Auger ID/OD:**	
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/21/13 - 08/21/13	**Drilling Method:**	Auger (SSA)	**Core Barrel:**	
Boring Location:	N 363942, E 2481605	**Casing ID/OD:**	3"	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.0 - 2.0	2-2-3-4	5	5				Leaf litter, some grass at surface. Brown, dry, loose, well graded SAND, some Gravel, trace Silt. -FILL-	
								35.4		3.5	
5	2D	24/8	5.0 - 7.0	1-47-24-39	71	71				Brown, moist/wet, hard, Silty CLAY, some Gravel, little Sand, with organics (small roots). Wet at 5.6'. -GLACIAL TILL-	G#1 A-7, CL WC=30.5
								31.2		Auger refusal at 7.7'	
										7.7	
										Bottom of Exploration at 7.70 feet below ground surface.	
10											
15											
20											
25											

Remarks:

1. Boring location on road shoulder/highway embankment.
2. Water at 6.8' (measured prior to auger refusal).
3. Prior to filling-in boring, water at 6.1'/open to 6.2'.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-202
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	21.1	**Auger ID/OD:**	
Operator:	Rich Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/26/13 - 08/26/13	**Drilling Method:**	Driven Casing	**Core Barrel:**	NX
Boring Location:	N 364110, E 2481564	**Casing ID/OD:**	3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0										Advanced roller bit through topsoil to 1.0' and started coring.	
								20.1		1.0	
	R1	60/55	1.0 - 6.0	RQD = 92%						Top of Bedrock at 1.0' (El 20.1'). Hard, fresh, medium grained GABBRO. Joints are widely spaced, horizontal, rough, open, moderate weathering/ oxidation on surfaces. No joints from 1.0' to 5.0'. Rock Mass Quality = Excellent Rock Core Times (min/ft): 7.0, 4.5, 2.75, 1.75, 1.75	
5											
	R2	60/60	6.0 - 11.0	RQD = 93%						Hard, fresh, medium grained GABBRO. Joints are moderately spaced, horizontal, rough, open, moderate weathering/oxidation of surfaces. Rock Mass Quality = Excellent Rock Core Times (min/ft): 2.0, 2.25, 2.0, 2.0, 2.0	
10											
								10.1		11.0	
										Bottom of Exploration at 11.00 feet below ground surface.	
15											
20											
25											

Remarks:

1. Bedrock surface at approximately 1.0' bgs.
2. Bedrock surface exposed in test pit TP-1 excavated by Hanscom Construction, Inc.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-203
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	18.5	**Auger ID/OD:**	
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/21/13 - 08/22/13	**Drilling Method:**	Driven Casing	**Core Barrel:**	NX
Boring Location:	N 364155, E 2481579	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/2	0.0 - 2.0	WOH/12"-1/12"			PUSH		Brown, moist/wet, very soft SILT, with organics (small twigs, roots), trace Gravel. -POND SEDIMENTS-	
	2D	24/16	2.0 - 4.0	WOH/24"					Gray-Brown, wet, soft, organic SILT, trace organics (fine roots, wood fibers), mottling. -POND SEDIMENTS-	A-7, OH LL=70 PL=34 PI=36 WC=48.8
	3D	24/18	4.0 - 6.0	Push thru vane	---		1		Gray-Brown, wet, stiff, organic SILT, slightly plastic, trace fine Sand, with organics (small pieces of wood, grass, twigs). -POND SEDIMENTS-	WC=43.9
5	V1		4.6 - 5.0	Su=1152+/0 psf			1		55x110mm vane raw torque readings: V1: 300+/0 in-lbs	
	4D	24/22	6.0 - 8.0	1-1-1-2	2	2	2		4D similar to 3D.	WC=46.4
							3		8.0'-8.5': Similar to 3D.	
	5D	24/17	8.0 - 10.0	WOH-37-44-45	81	81	43	10.0	8.5'-9.5': Gray, wet, very dense, Gravelly SILT, some Sand. -GLACIAL TILL- (8.5)	
							51	8.5	10.0'-11.0': Apparent cobble. (10.0)	
10	R1	50/35	10.0 - 14.2	RQD = 29%			NX	7.5	Top of Bedrock at 11.0' (El 6.2'). (11.0) Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are very closely to closely spaced, horizontal to low angle, undulating, rough, slightly weathered, tight to open, Silt infillings, Calcite contacts. Secondary joints are closely spaced, high angle to vertical, planar, rough, slightly weathered, tight to open, calcite infillings. Rock Mass Quality = Poor Rock Core Times (min/ft): 2.5, 1.5 1.75, 2.0	
15	R2	15/17	14.2 - 15.5	RQD = 0%					Hard, fresh, fine grained, gray, TUFF BRECCIA. Primary joints are closely spaced, horizontal, rough, undulating, slightly weathered, open, Silt infilling/ coating. Secondary joints are closely spaced, vertical, planar, rough, slightly weathered, Calcite infillings. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 1.25, 0.5/3"	
	R3	60/55	15.5 - 20.5	RQD = 77%					Hard, fresh, fine-grained, gray and green, TUFF BRECCIA. Primary joints are closely to moderately spaced, horizontal to moderately dipping, undulating, rough, fresh to slightly weathered, tight to open. Secondary joints are close to moderately spaced, high angle to vertical, planar, rough, fresh, tight to open. Rock Mass Quality = Good Rock Core Times (min/ft): 1.0, 1.25, 1.5, 2.0, 2.25	
20								-2.0	(20.5) **Bottom of Exploration at 20.50 feet below ground surface.**	
25										

Remarks:

1. Boring located on very soft ground.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-204
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	19.0	**Auger ID/OD:**	
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/22/13 - 08/23/13	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NX
Boring Location:	N 364163, E 2481538	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.0 - 2.0	WOH-WOH-2-3	2	2	PUSH	18.5	Top 6": Brown, wet, well graded SAND, little Silt. -FILL- (0.5)	
	2D	24/19	2.0 - 4.0	2-1-2-2	3	3			Bottom 5": Gray, moist, soft to medium stiff, Silty CLAY, trace organics (fine roots, grass). -POND SEDIMENTS- Top 5": Similar to 1D. Bottom 14": Brown, moist, soft to medium stiff, organic SILT, with organics.	WC=46.2
	3D	24/11	4.0 - 6.0	1/12"-WOH/12"	1	1	1		Brown, moist, soft, organic SILT, with organics. -POND SEDIMENTS-	A-7, OH LL=57 PL=33 PI=24 WC=50.1 G#9
5							1			
	4D	24/23	6.0 - 8.0	Push thru vane	---		1		Gray-Brown, wet, medium stiff to stiff, Silty CLAY, some fine to medium Sand, trace organics (wood chips/ fibers). -POND SEDIMENTS-	A-7, CL LL=46 PL=25 PI=21 WC=45.9
	V1		6.6 - 7.0	Su=960/115 psf						
	V2		7.6 - 8.0	Su=960/115 psf			3			
	R1	36/22	8.1 - 11.1				NX	10.9	55x110mm vane raw torque readings: V1: 250/30 in-lbs V2: 250/30 in-lbs (8.1) Cored through apparent cobbles and boulders. -GLACIAL TILL-	
10										
	R2	60/7	11.1 - 16.1							
15							SPIN			
	5D	24/8	18.0 - 20.0	12-13-15-17	28	28			Gray, wet, medium dense, Gravelly SILT, little Sand. -GLACIAL TILL-	
20										
	6D	15/10	21.5 - 22.8	8-30-54/3"	84	84		-2.0	(21.0) Cobble. (21.5) Gray, wet, very dense, Gravelly SILT, little Sand.	
	R3	32/32	23.5 - 26.2	RQD = 13%			NX	-2.5 / -3.8	(22.8) Top of Bedrock at 22.8' (El -3.8'). Roller cone to 23.5' and start coring. Hard, moderately to severely weathered, gray GABBRO. Primary joints are extremely closely to very closely	
25										

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-204
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	19.0	**Auger ID/OD:**	
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/22/13 - 08/23/13	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NX
Boring Location:	N 364163, E 2481538	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										spaced, low angle, undulating, rough, discolored to weathered, open, silt and sand infillings. Secondary joints are high angle to vertical, undulating to planar, rough, open, moderately weathered. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 3.5, 2.0, 1.25	
	R4	24/24	26.2 - 28.2	RQD = 0%							
								-9.2		Hard, moderately to severely weathered, gray GABBRO. Recovery comprised of Rock pieces (weathered) through 27.5'. Rocks weathered on nearly all surfaces through 27.5'. No discernible joints. Rock Mass Quality = Very Poor Rock Core Times (min/ft): 2.25, 1.25	
30										28.2	
										Bottom of Exploration at 28.20 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-205
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	19.4	**Auger ID/OD:**	
Operator:	Rich Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/27/13 - 08/27/13	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NX
Boring Location:	N 364175, E 2481507	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/21	0.0 - 2.0	3-5-7-7	12	12	14		Olive-Gray, moist, stiff, Clayey SILT. -POND SEDIMENTS-	
							18			
	2D	24/10	2.0 - 4.0	WOR-3-2-3	5	5	5		Gray, moist, medium stiff to stiff, Silty CLAY, trace Gravel, mottling. -POND SEDIMENTS-	WC=24.9
							3			
	3D	24/8	4.0 - 6.0	WOR/12"-WOH/12"			2		Gray-brown, wet, soft, Silty CLAY, trace fine Sand, with organics (wood, grass fibers). -POND SEDIMENTS-	A-7, CL LL=42 PL=22 PI=20 WC=38.8
5							WOH			
	4D	24/24	6.2 - 8.2	4-18-14-16	32	32			Gray, wet, stiff to very stiff, Silty CLAY, little fine Sand. -POND SEDIMENTS-	
	5D	16/16	8.7 - 10.0	1-3-4/4"					Gray, wet, soft, SILT & CLAY, trace fine Sand, trace Gravel, Sand increasing with depth. -POND SEDIMENTS-	WC=28.5
							20			
10	6D	24/19	10.0 - 12.0	6-10-25-36	35	35	39	9.4	Top 12": Gray, wet, very stiff, Clayey SILT, trace Gravel. Bottom 7": Gray, wet, fine to medium SAND, some Silt, little Gravel. -GLACIAL TILL- (10.0)	G#11 A-2-4, SM WC=12.2
							24			
							33	7.4		
	7D	24/13	13.0 - 15.0	20-39-24-33	63	63	55	6.7	(12.0) Roller cone through Cobble. (12.7)	G#12 A-1-a, SM WC=8.0
							50		Gray, wet, very dense, fine to coarse SAND, some Gravel, little Silt. -GLACIAL TILL-	
15							45			
							51			
							31			
	8D	24/9	18.5 - 20.5	33-30-15-69	45	45	62		Gray, wet, dense, fine to coarse SAND, some Silt, little Gravel. -GLACIAL TILL-	
							41			
20							39			
							38			
							RC			
	9D	9/3	23.0 - 23.8	14-100/3"				-4.4	Gray, wet, very dense, Gravelly SILT, little Sand. -GLACIAL TILL- (23.8)	
25									Top of Bedrock at 23.8' (El -4.4'). Roller cone to 26.0'	

Remarks:

1. Boring completed through deck of bridge. Concrete deck is 8" thick. Top of deck to ground surface 16.9'.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-205
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	19.4	**Auger ID/OD:**	
Operator:	Rich Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/27/13 - 08/27/13	**Drilling Method:**	Driven/Spun Casing	**Core Barrel:**	NX
Boring Location:	N 364175, E 2481507	**Casing ID/OD:**	4" and 3"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NX			and start coring.	
	R1	60/60	26.0 - 31.0	RQD = 59%						Hard, fresh, fine grained, gray TUFF BRECCIA. Primary joints are moderately spaced, low angle, smooth to rough, very slightly weathered, Silt on surfaces. Secondary joints are closely spaced, high angle, smooth, fresh, tight, with Calcite veins. Rock Mass Quality = Fair Rock Core Times (min/ft): 1.75, 3.0, 2.0, 2.0, 2.0	
30								-11.6		31.0 **Bottom of Exploration at 31.00 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

1. Boring completed through deck of bridge. Concrete deck is 8" thick. Top of deck to ground surface 16.9'.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-206
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	21.7	**Auger ID/OD:**	
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/23/13 - 08/23/13	**Drilling Method:**	Auger (SSA)	**Core Barrel:**	
Boring Location:	N 364212, E 2481521	**Casing ID/OD:**	3"	**Water Level*:**	4.3'
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/11	0.0 - 2.0	1-2-2-4	4	4				Dark brown, damp, soft, SILT, transitioning to olive/tan Silt, trace Clay, with mottling. -TOPSOIL-	
								19.7		2.0	
5	2D	24/13	5.0 - 7.0	8-7-8-15	15	15				Brown/Gray, medium dense, fine to coarse SAND, some Silt, some Gravel. -GLACIAL TILL- Apparent cobble at 7.2'.	G#13 A-2-4, SM WC=8.9
								13.2		Auger refusal at 8.5'	
										8.5	
10										**Bottom of Exploration at 8.50 feet below ground surface.**	
15											
20											
25											

Remarks:

1. Encountered cobble at 7.2' while advancing auger.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Crane Mill Brook Bridge
Location: Edmunds Township, Maine

Boring No.: BB-ECB-207
PIN: 19313.00

Driller:	Maine Test Boring	**Elevation (ft.)**	19.1	**Auger ID/OD:**	
Operator:	Rich Leonard	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Brad Tirone	**Rig Type:**	Mobile Drill B-53	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	08/28/13 - 08/28/13	**Drilling Method:**	Driven Casing	**Core Barrel:**	NX
Boring Location:	N 364160, E 2481550	**Casing ID/OD:**	4"	**Water Level*:**	
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	1/12"-1-1	1	1	PUSH		Gray-Brown, moist, soft, organic SILT, trace organics (wood fibers, roots). -POND SEDIMENTS-	WC=42.3
	2D	24/20	4.0 - 6.0	2-1-2-1	3	3			Gray-Brown, wet, soft, organic SILT, plastic, with trace organics.	WC=51.4
5										
	3D	24/24	8.0 - 10.0	2-3-9-13	12	12			Gray-Brown, wet, stiff, Clayey SILT. Gravel in tip of spoon.	
							40			
10								9.3	9.8	
	4D	24/5	10.5 - 12.5	44-31-27-50	58	58	14	8.6	Roller cone through Cobble. 10.5	G#16 A-1-b, GM WC=6.7
							10		Gray, wet, very dense, Sandy GRAVEL, little Silt. -GLACIAL TILL-	
							21			
							52			
							42			
15	5D	24/12	15.0 - 17.0	11-16-13-15	29	29	20		Gray, wet, medium dense, Gravelly fine to coarse SAND, little Silt. -GLACIAL TILL-	G#17 A-1-b, SM WC=7.4
							28			
							29			
							48	1.3	17.8	
	R1	52/36	19.0 - 23.3	RQD = 67%			NX		Top of Bedrock at 17.8' (El 1.3'). Roller cone to 19.0' and start coring. Hard, fresh, medium grained GABBRO. Primary joints are moderately to widely spaced, low angle, undulating, rough, tight, light Silt on surfaces. Secondary joints are widely spaced, high angle to vertical, fresh, smooth, with thin Calcite vein. Rock Mass Quality = Fair Rock Core Times (min/ft): 3.0, 2.5, 2.5, 2.5, 1.0	
20										
								-4.2	23.3	
25									**Bottom of Exploration at 23.30 feet below ground surface.**	

Remarks:

1. RQD=67% for R1 resulted from not recovering 16" of core; recovered core was 2 pieces longer than 1.0' and a 9.3" piece.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

APPENDIX D

LABORATORY TESTING RESULTS

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Bridge #3171 Over Crane Mill Brook

MDOT Project Number: 19313

Town(s): Edmunds Twp.

GZA Project Number: 09.0025733.00

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Reference Number	Organic %	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-ECB-103		3D	4-6			19.0	26	11	CL	A-4	IV
BB-ECB-103		5D	8-10			23.3	40	20	CL	A-4	IV
BB-ECB-103		6D	10-12			22.5					
BB-ECB-103		7D	12-14			12.9			SM	A-2-4	II
BB-ECB-103		8D	14-16			7.8			SP-SM	A-1-b	II
BB-ECB-104		1D	3-5			8.5			SM	A-1-a	II
BB-ECB-104		3D	7-9						SM	A-1-b	II
BB-ECB-104		4D	9-11		4.6	37.4	43	19	OL	A-7-6	IV
BB-ECB-104		5D	11.7-13			17.6					

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.

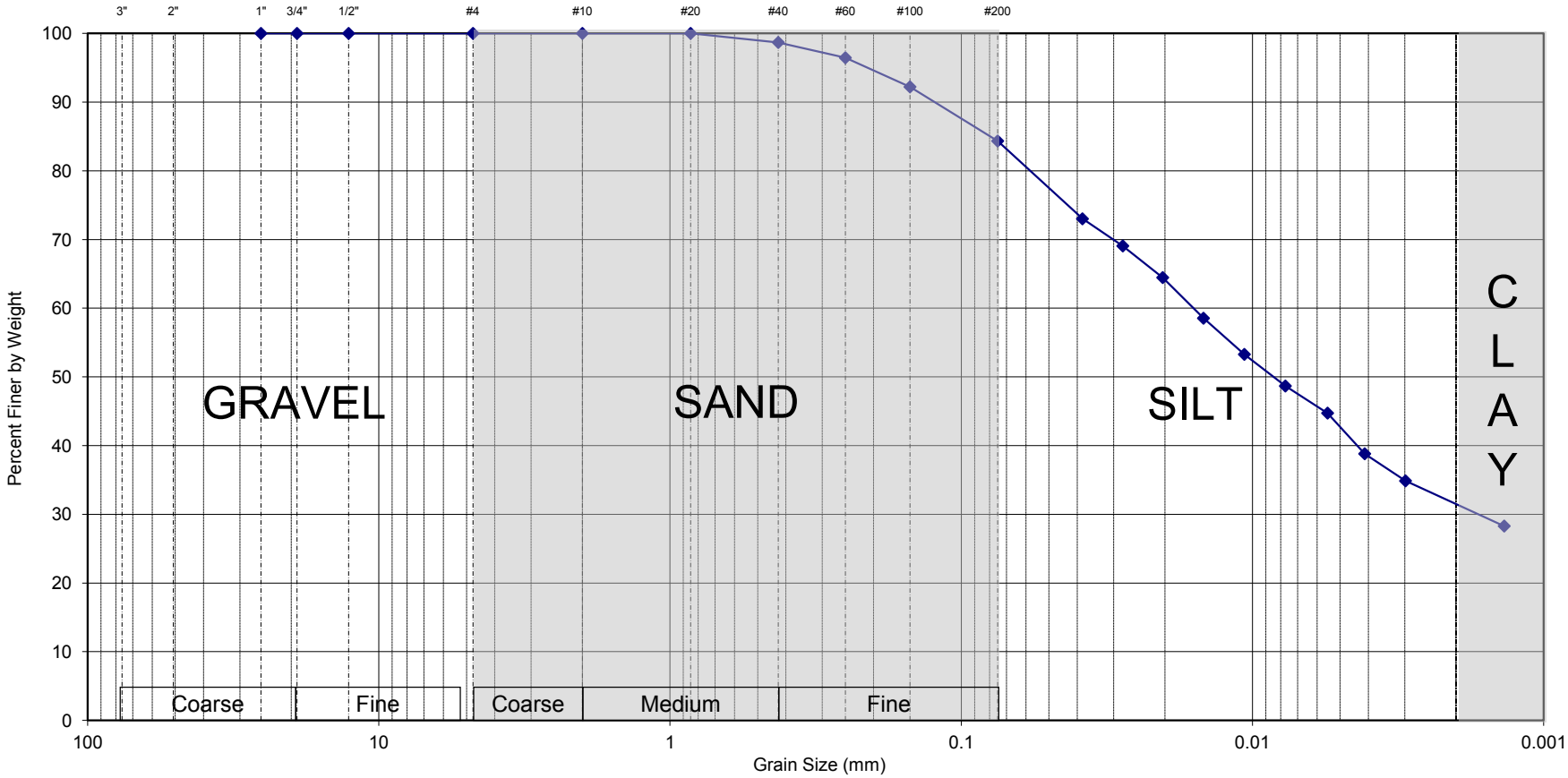
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
0.0%	15.7%	84.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
1	BB-ECB-103	3D	4-6'	Brown CLAY & SILT, little fine Sand (CL)	19.0	26	15	11

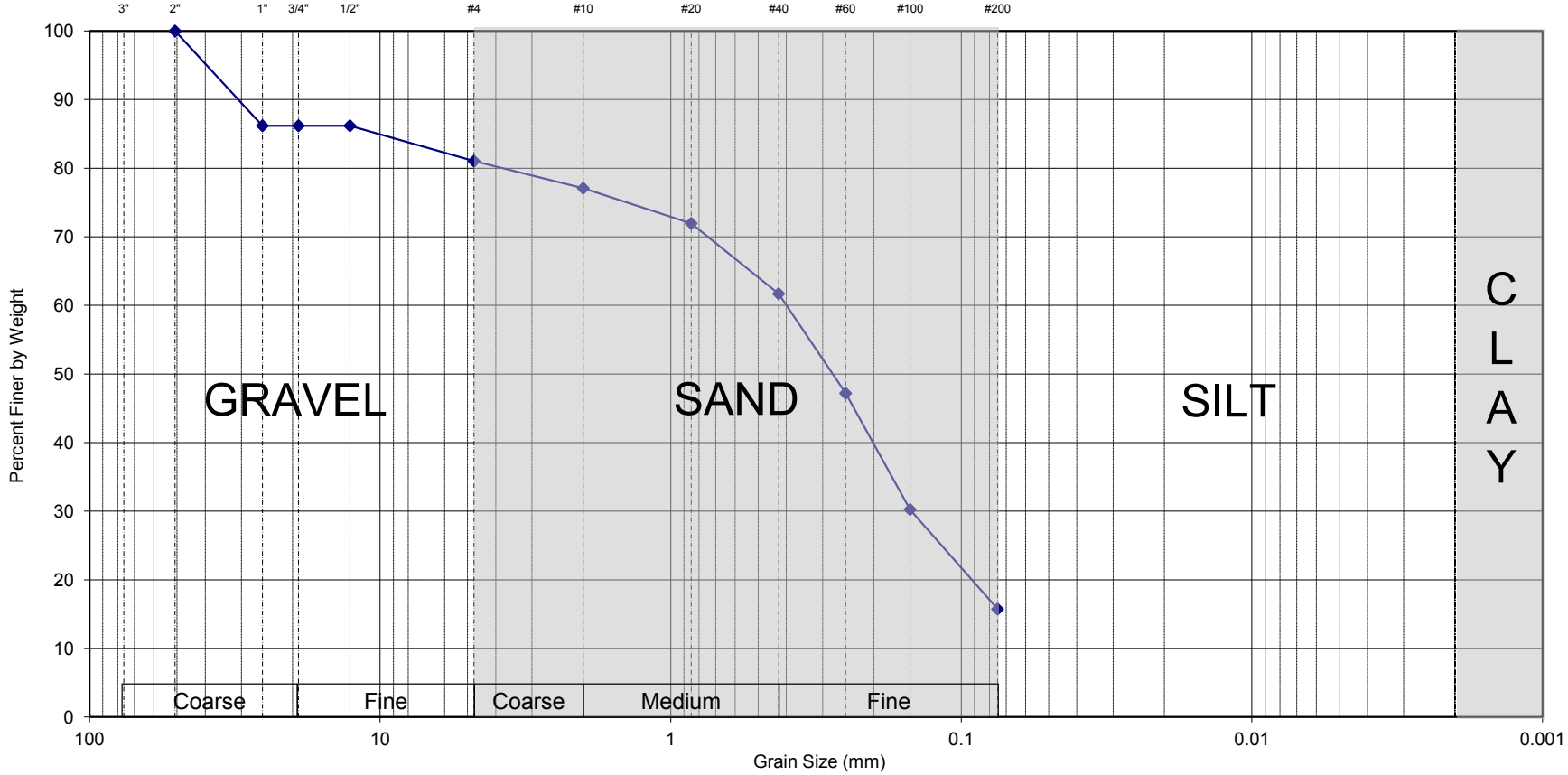
Sieve Size	% Passing
¾"	100.0
½"	100.0
#4	100.0
#10	100.0
#20	100.0
#40	98.7
#60	96.5
#100	92.2
#200	84.3

THIELSCH ENGINEERING

CTS-74-12-0003.07
Tide Mill Bridge
Edmunds Township, ME
GZA File # 09.0025733.00

Tested by: AS Date: 6/13/12
Reviewed by: MBP Date: 6/14/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
19.0%	65.3%	15.7%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
4	BB-ECB-103	7D	12-14'	Gray f-m SAND, little f-c Gravel, little Silt (SM)	12.9			

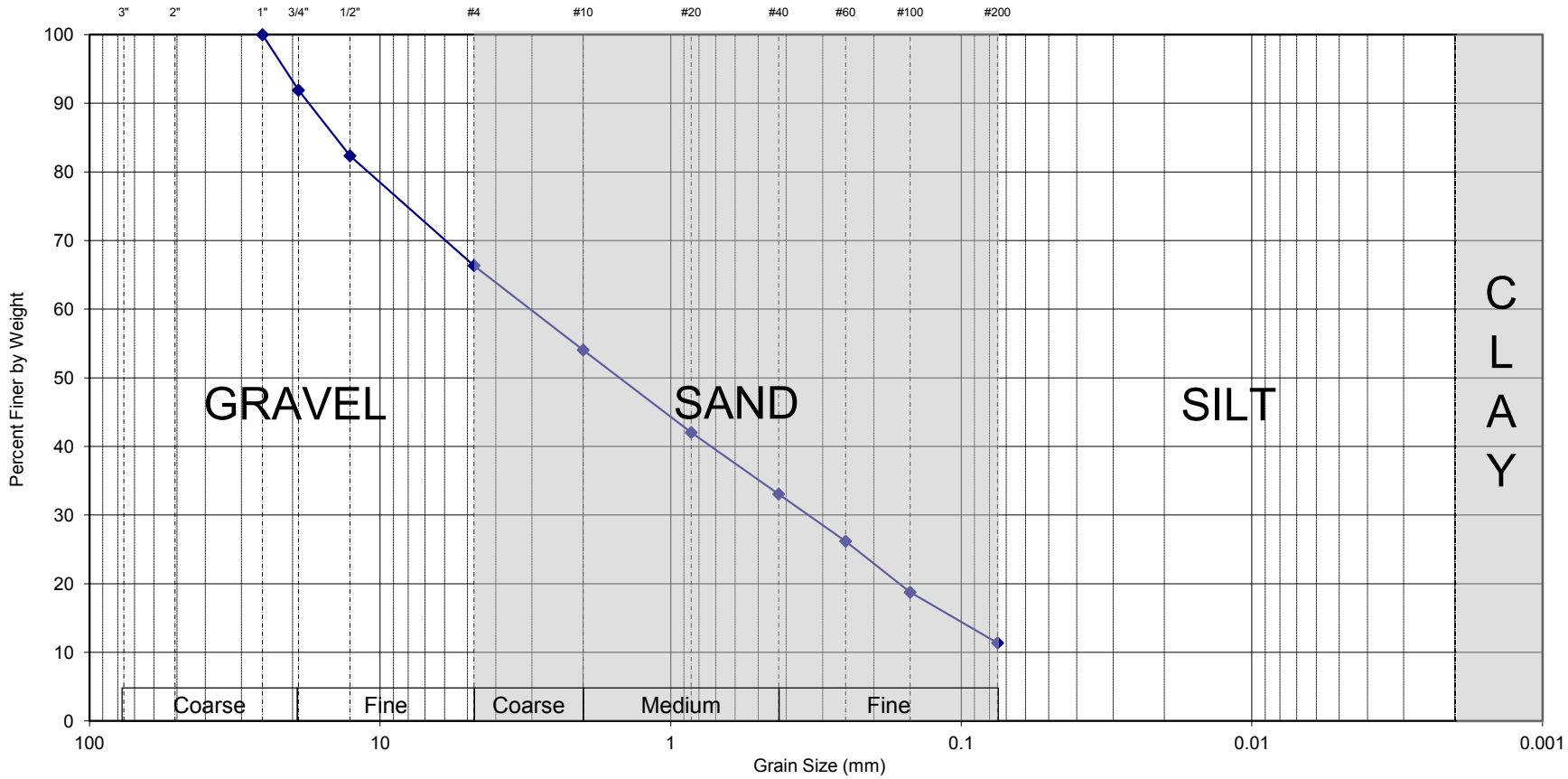
Sieve Size	% Passing
¾"	86.2
½"	86.2
#4	81.0
#10	77.1
#20	72.0
#40	61.7
#60	47.2
#100	30.3
#200	15.7

THIELSCH ENGINEERING

CTS-74-12-0003.15
Tide Mill Bridge
Edmunds Township, ME
GZA File # 09.0025733.00

Tested by: PEC Date: 6/14/12
Reviewed by: MBP Date: 6/14/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
33.7%	55.0%	11.4%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
5	BB-ECB-103	8D	14-16'	Gray f-c SAND, some f-c Gravel, little Silt (SP-SM)	7.8			

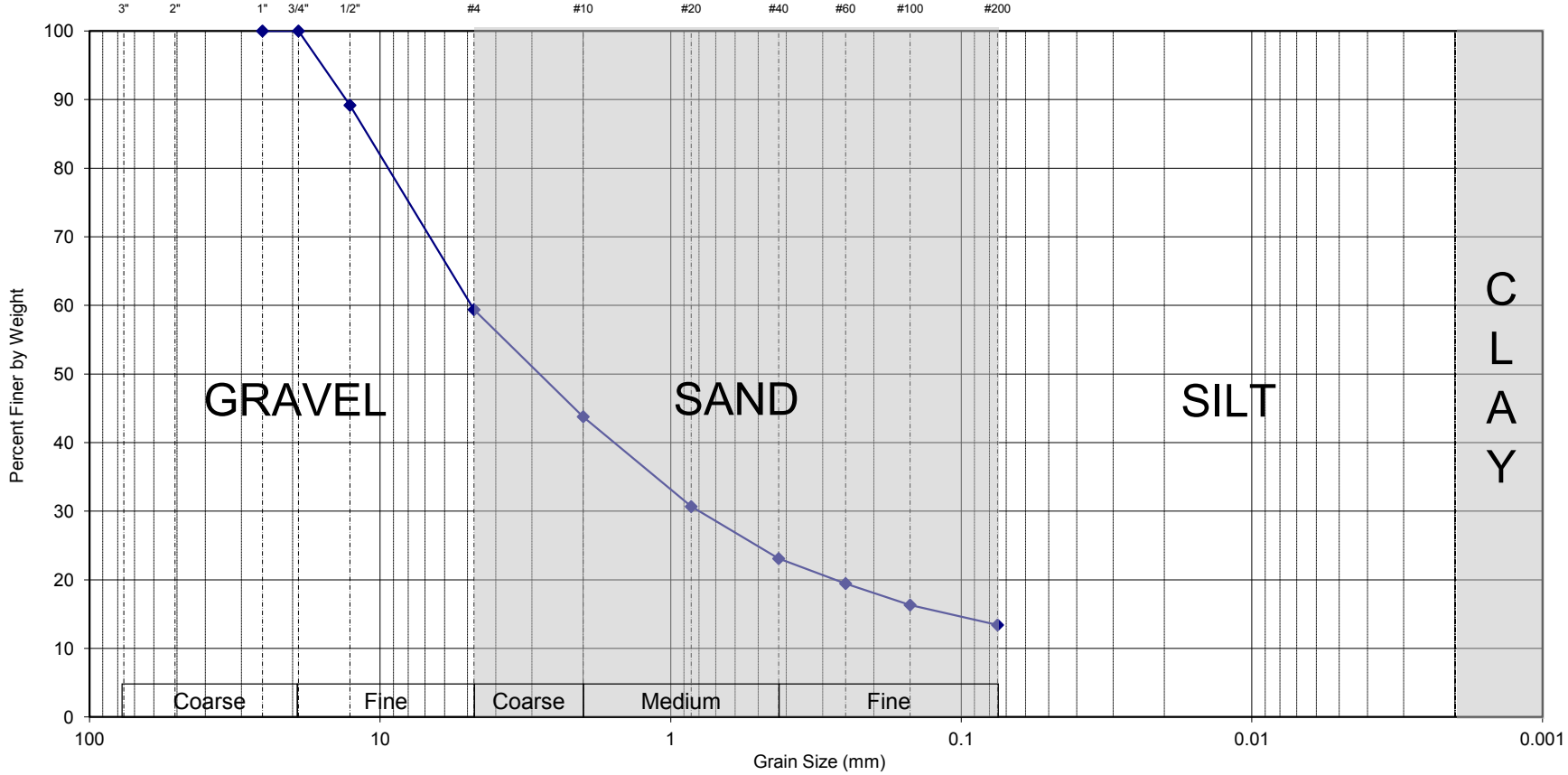
Sieve Size	% Passing
¾"	91.9
½"	82.4
#4	66.3
#10	54.0
#20	42.0
#40	33.1
#60	26.2
#100	18.8
#200	11.4

THIELSCH ENGINEERING

CTS-74-12-0003.15
Tide Mill Bridge
Edmunds Township, ME
GZA File # 09.0025733.00

Tested by: PEC Date: 6/14/12
Reviewed by: MBP Date: 6/14/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
40.6%	46.0%	13.4%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
6	BB-ECB-104	1D	3-5'	Brown f-c SAND and fine GRAVEL, little Silt (SM)	8.5			

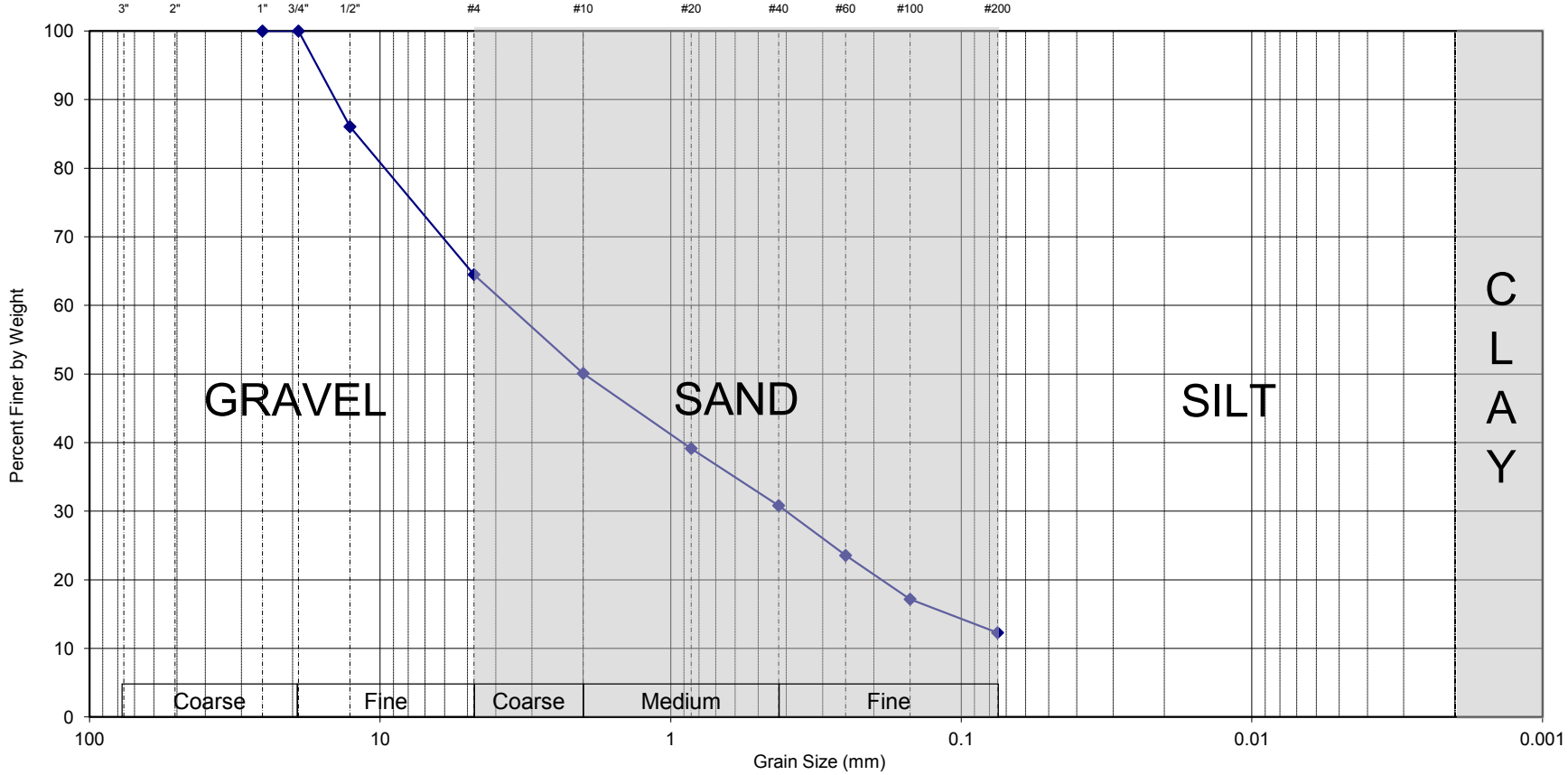
Sieve Size	% Passing
¾"	100.0
½"	89.2
#4	59.4
#10	43.8
#20	30.7
#40	23.1
#60	19.5
#100	16.3
#200	13.4

THIELSCH ENGINEERING

CTS-74-12-0003.15
Tide Mill Bridge
Edmunds Township, ME
GZA File # 09.0025733.00

Tested by: PEC Date: 6/14/12
Reviewed by: MBP Date: 6/14/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
35.5%	52.2%	12.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
7	BB-ECB-104	3D	7-9'	Brown to Gray f-c SAND and fine GRAVEL, little Silt (SM)				

Sieve Size	% Passing
¾"	100.0
½"	86.1
#4	64.5
#10	50.1
#20	39.1
#40	30.8
#60	23.6
#100	17.2
#200	12.3

THIELSCH ENGINEERING

CTS-74-12-0003.15
Tide Mill Bridge
Edmunds Township, ME
GZA File # 09.0025733.00

Tested by: PEC Date: 6/14/12
Reviewed by: MBP Date: 6/14/12

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Crane Mill Brook Bridge

MDOT Project Number:

GZA Project Number: 09.0025733.01

Town(s): Edmunds Township, 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-ECB-201		2D	5-7	1		30.5			CL	A-7	IV
BB-ECB-203		2D	2-4	2		48.8	70	36	OH	A-7	IV
BB-ECB-203		3D	4-6	3		43.9					
BB-ECB-203		4D	6-8	4		46.4					
BB-ECB-204		2D	2-4	5		46.2					
BB-ECB-204		3D	4-6	6		50.1	57	24	OH	A-7	IV
BB-ECB-204		4D	6-8	7		45.9	46	21	CL	A-7	IV
BB-ECB-205		2D	2-4	8		24.9					
BB-ECB-205		3D	4-6	9		38.8	42	20	CL	A-7	IV
BB-ECB-205		5D	8.7-10	10		28.5					
BB-ECB-205		6D	10-12	11		12.2			SM	A-2-4	II
BB-ECB-205		7D	13-15	12		8.0			SM	A-1-a	II
BB-ECB-206		2D	5-7	13		8.9			SM	A-2-4	II
BB-ECB-207		1D	0-2	14		42.3					
BB-ECB-207		2D	4-6	15		51.4					
BB-ECB-207		4D	10.5-12.5	16		6.7			GM	A-1-b	I
BB-ECB-207		5D	15-17	17		7.4			SM	A-1-b	II

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)

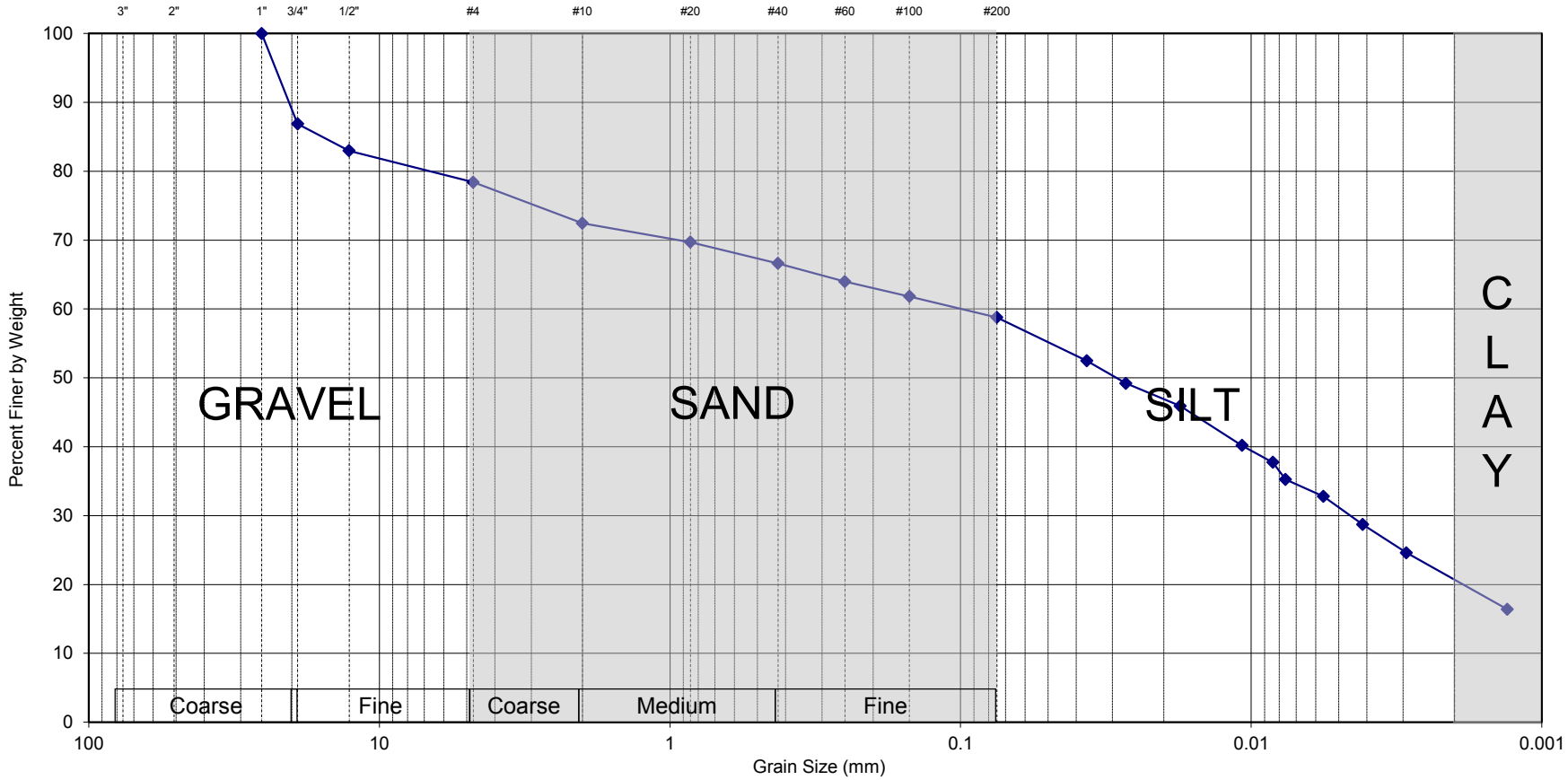
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
21.6%	19.6%	58.8%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
1	BB-ECB-201	2D	5-7'	Brown Silty Clay, some f-c Gravel, little f-c Sand (CL)	30.5			

Sieve Size	% Passing
¾"	86.9
½"	83.0
#4	78.4
#10	72.5
#20	69.7
#40	66.6
#60	64.0
#100	61.8
#200	58.8

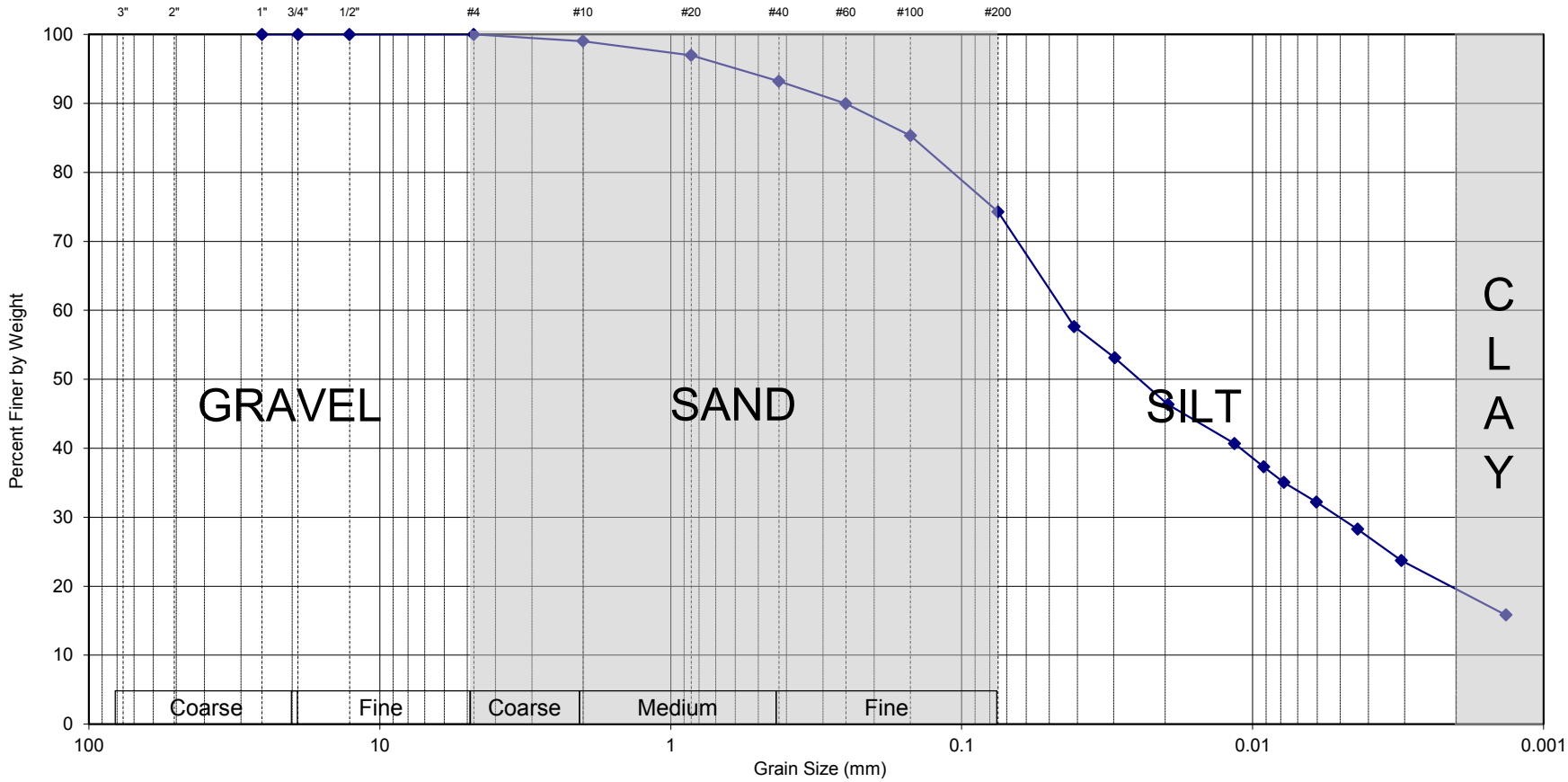
THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/11/13
Reviewed by: MBP Date: 10/15/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
0.0%	25.7%	74.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
7	BB-ECB-204	4D	6-8'	Gray-brown Silty Clay, some f-m Sand (CL)	45.9	46	25	21

Sieve Size	% Passing
¾"	100.0
½"	100.0
#4	100.0
#10	99.0
#20	97.0
#40	93.2
#60	90.0
#100	85.3
#200	74.3

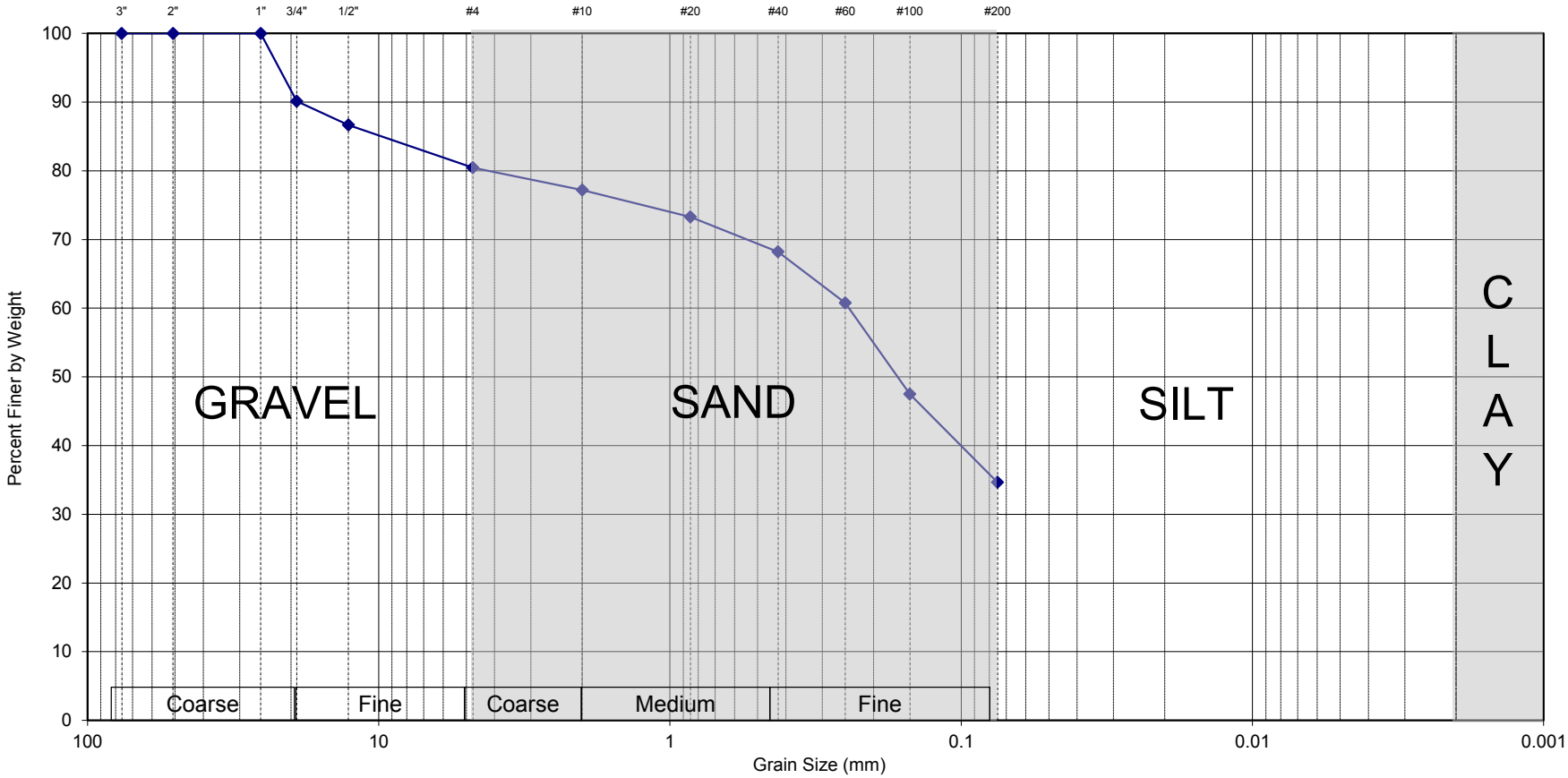
THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/11/13
Reviewed by: MBP Date: 10/15/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
19.6%	45.8%	34.6%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
11	BB-ECB-205	6D	10-12'	Gray f-m SAND, some Silt, little f-c Gravel (SM)	12.2			

Sieve Size	% Passing
¾"	90.1
½"	86.7
#4	80.4
#10	77.2
#20	73.3
#40	68.2
#60	60.8
#100	47.5
#200	34.6

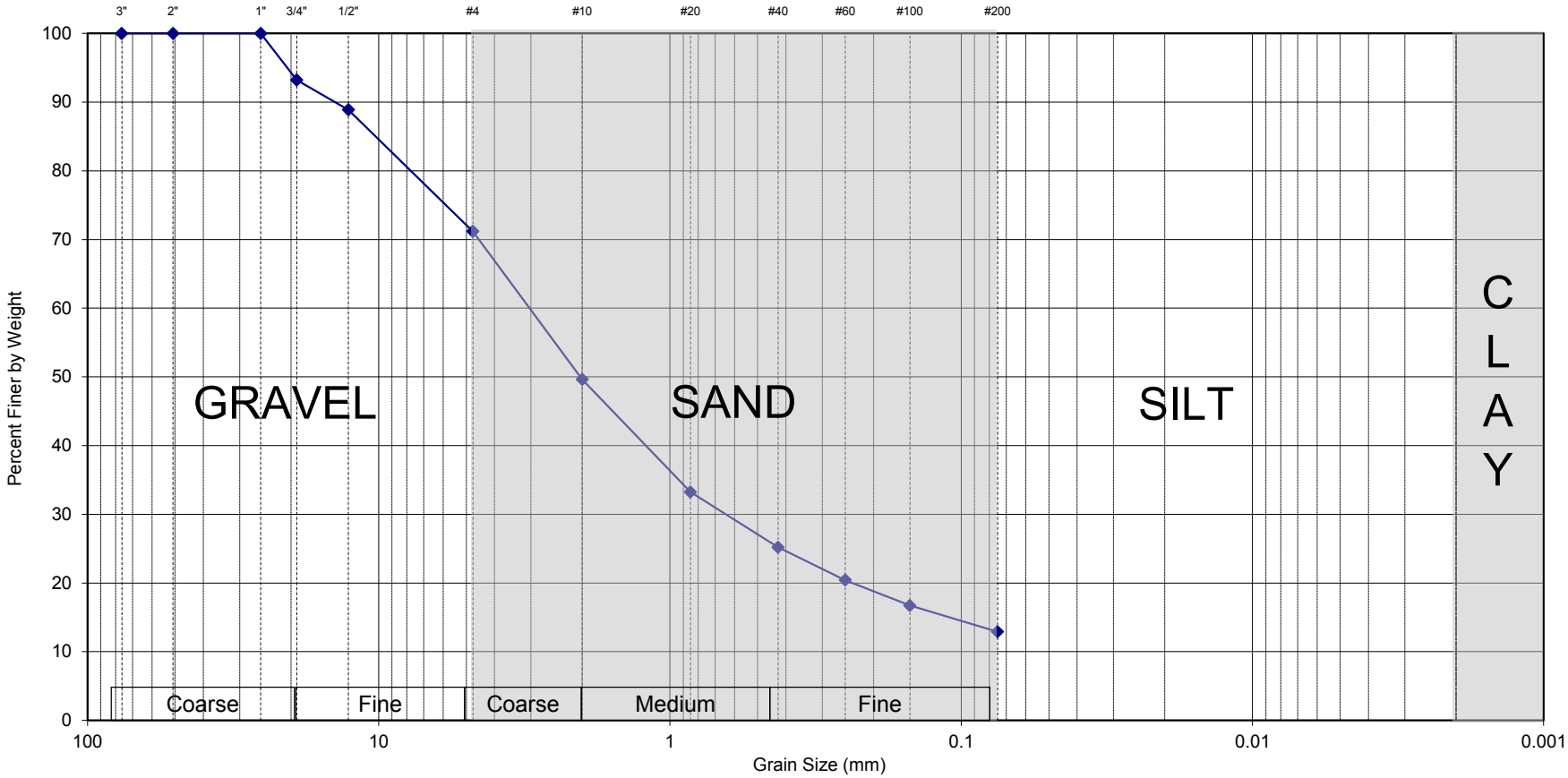
THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/10/13
Reviewed by: MBP Date: 10/11/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
28.8%	58.3%	12.9%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
12	BB-ECB-205	7D	13-15'	Gray f-c SAND, some f-c Gravel, little Silt (SM)	8.0			

Sieve Size	% Passing
¾"	93.2
½"	88.9
#4	71.2
#10	49.6
#20	33.2
#40	25.2
#60	20.4
#100	16.7
#200	12.9

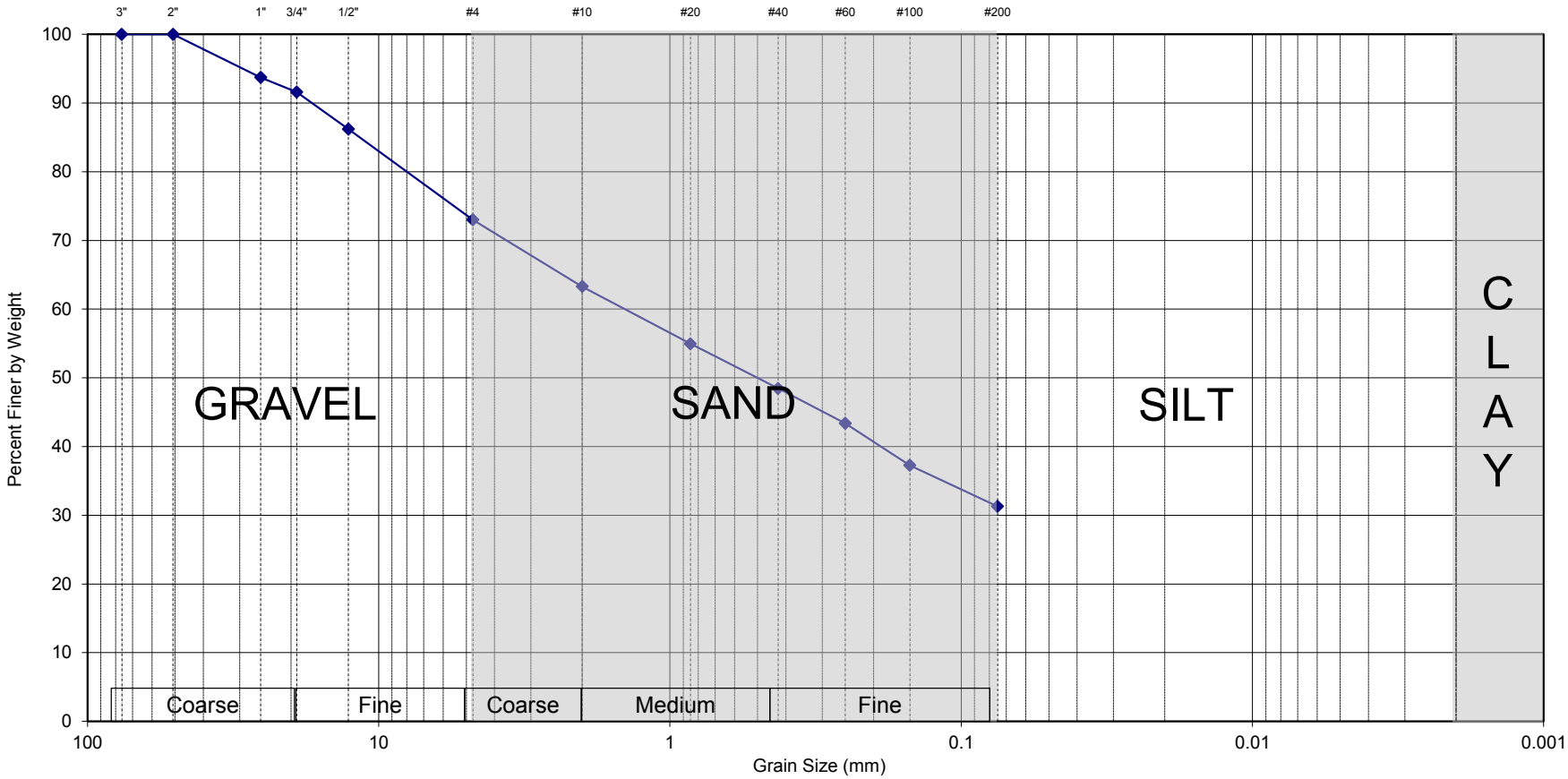
THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/10/13
Reviewed by: MBP Date: 10/11/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
27.0%	41.7%	31.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
13	BB-ECB-206	2D	5-7'	Gray f-c SAND, some Silt, some f-c Gravel (SM)	8.9			

Sieve Size	% Passing
¾"	91.6
½"	86.2
#4	73.0
#10	63.3
#20	54.9
#40	48.4
#60	43.4
#100	37.3
#200	31.3

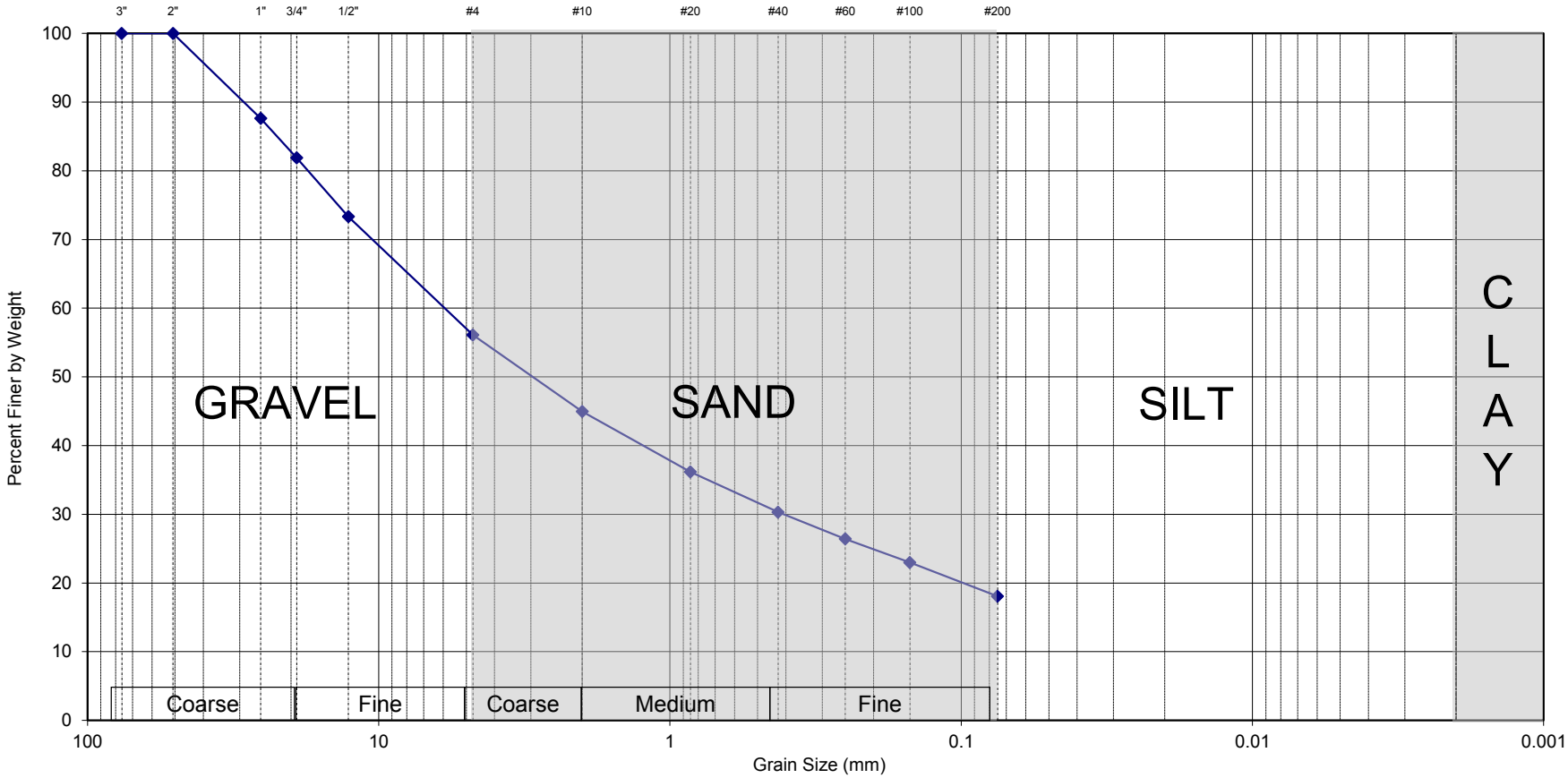
THIELSCH ENGINEERING
195 Frances Ave., Cranston, RI 02910
401-467-6454

CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/10/13
Reviewed by: MBP Date: 10/11/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
43.9%	38.0%	18.1%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
16	BB-ECB-207	4D	10.5-12.5'	Gray Sandy f-c GRAVEL, little Silt (GM)	6.7			

Sieve Size	% Passing
¾"	81.9
½"	73.3
#4	56.1
#10	44.9
#20	36.2
#40	30.3
#60	26.4
#100	23.0
#200	18.1

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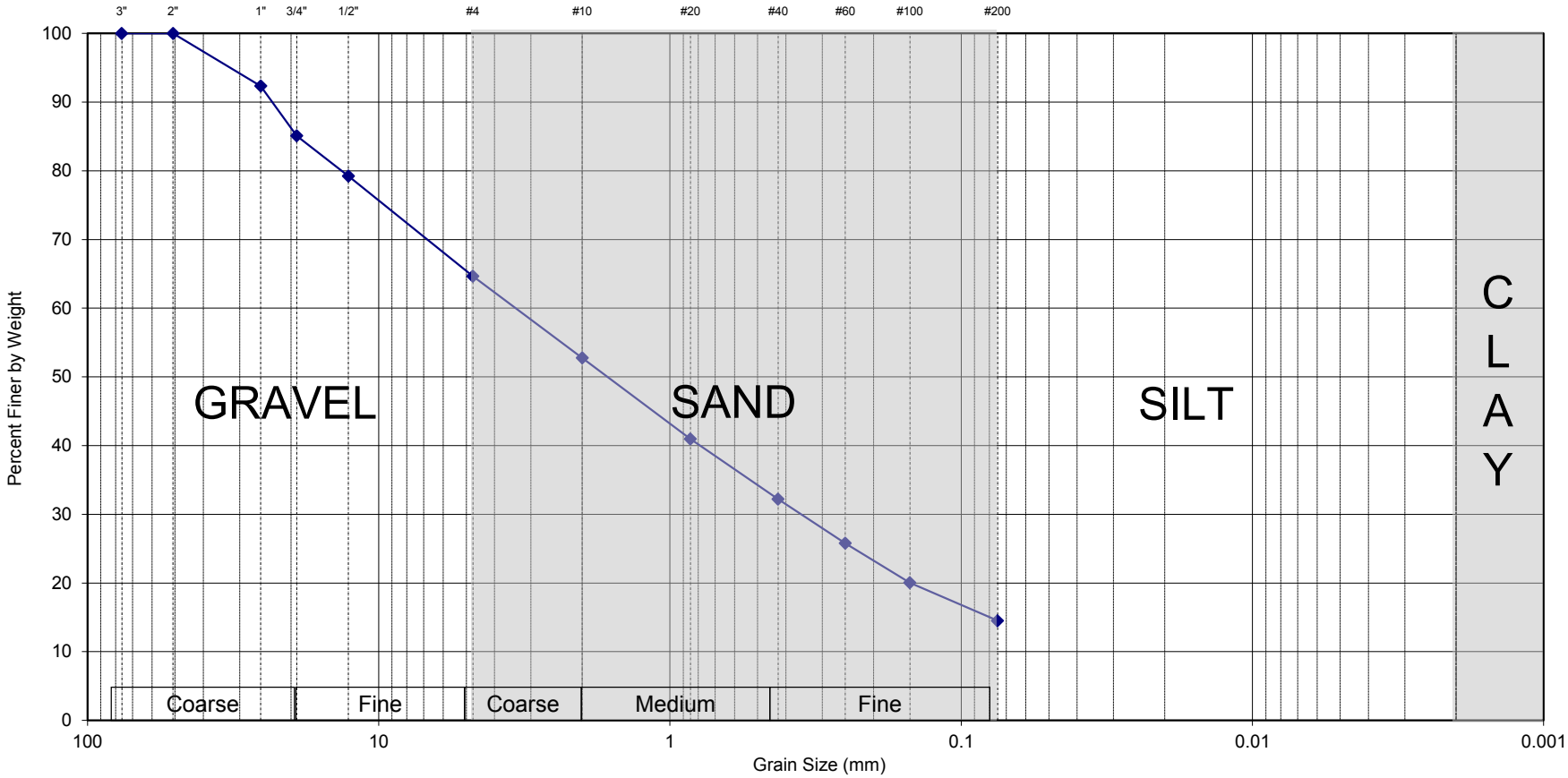
CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/10/13
Reviewed by: MBP Date: 10/11/13

ASTM D422

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
35.3%	50.2%	14.5%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
17	BB-ECB-207	5D	15-17'	Gray Gravelly f-c SAND, little Silt (SM)	7.4			

Sieve Size	% Passing
¾"	85.1
½"	79.2
#4	64.7
#10	52.8
#20	40.9
#40	32.2
#60	25.8
#100	20.0
#200	14.5

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CTS-74-13-0003
Crane Mill Brook Bridge
Edmunds Township, ME
GZA File # 09.0025733.01

Tested by: GG Date: 10/10/13
Reviewed by: MBP Date: 10/11/13

LABORATORY TESTING DATA SHEET

Project Name Tide Mill Bridge
Project No. 09.0025733.00
Project Manager Andrew Blaisdell

Location Edmunds Twp., ME
Assigned By E. Baron
Report Date 6/19/2012

Reviewed By
Date Reviewed 6/19/2012

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength KSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ KSI	Is_{50} KSI	Rock Formation or Description or Remarks
BB-ECB -101	R1	10.0-10.4	10		1.978	4.428	179.6			U	19.01	0.33		5.01	0.23			Fresh, medium grained Gray, GABBRO
BB-ECB -102	R3	17.0-17.4	11		1.979	4.565	180.4			U	27.87	0.38		6.92	0.25			Fresh, fine grained Gray TUFF BRECCIA
BB-ECB -104	R1	15.3-15.7	12		1.974	4.705	169.6			U	27.29	0.36		6.57	0.21			Fresh, fine grained Gray TUFF BRECCIA

(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) RST= 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

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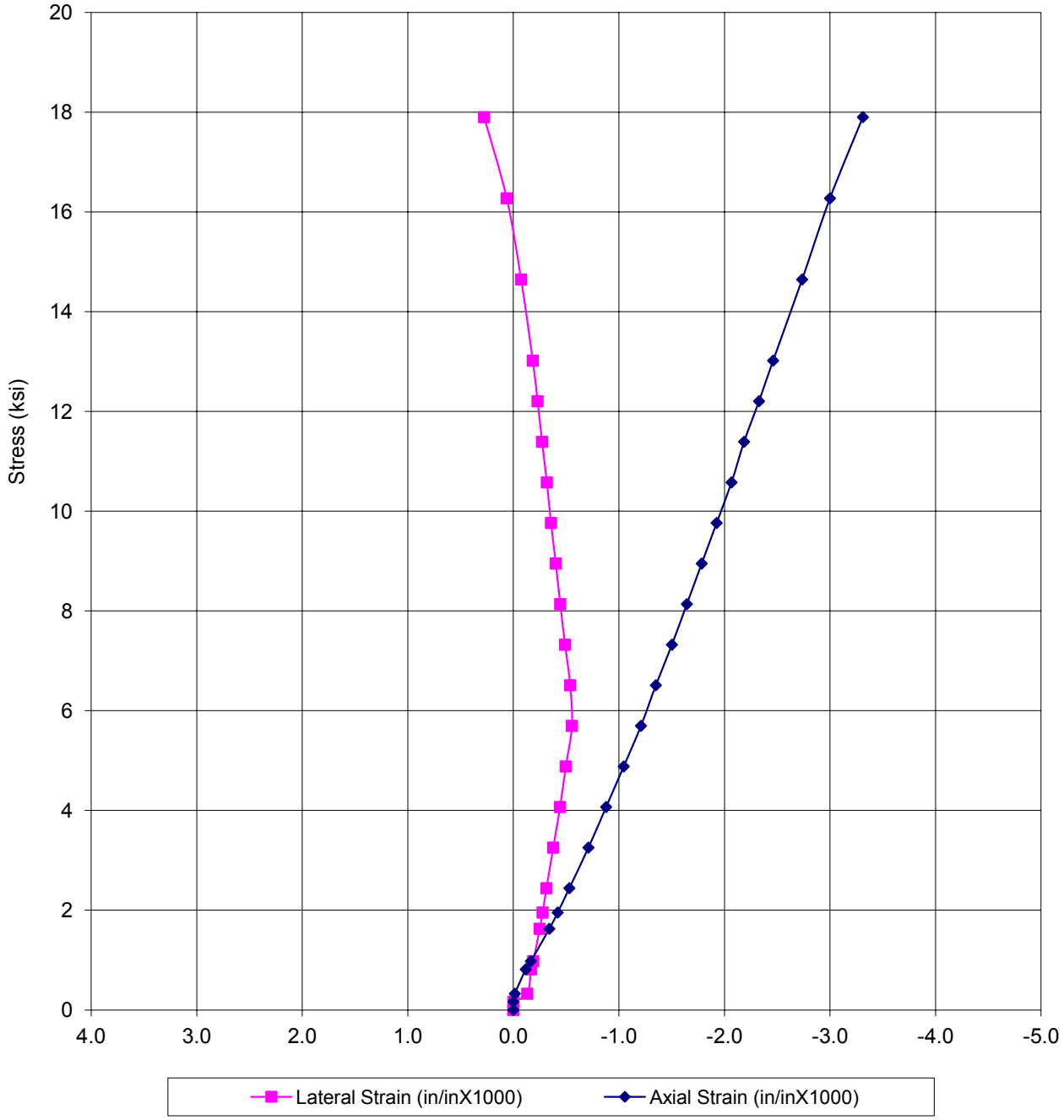
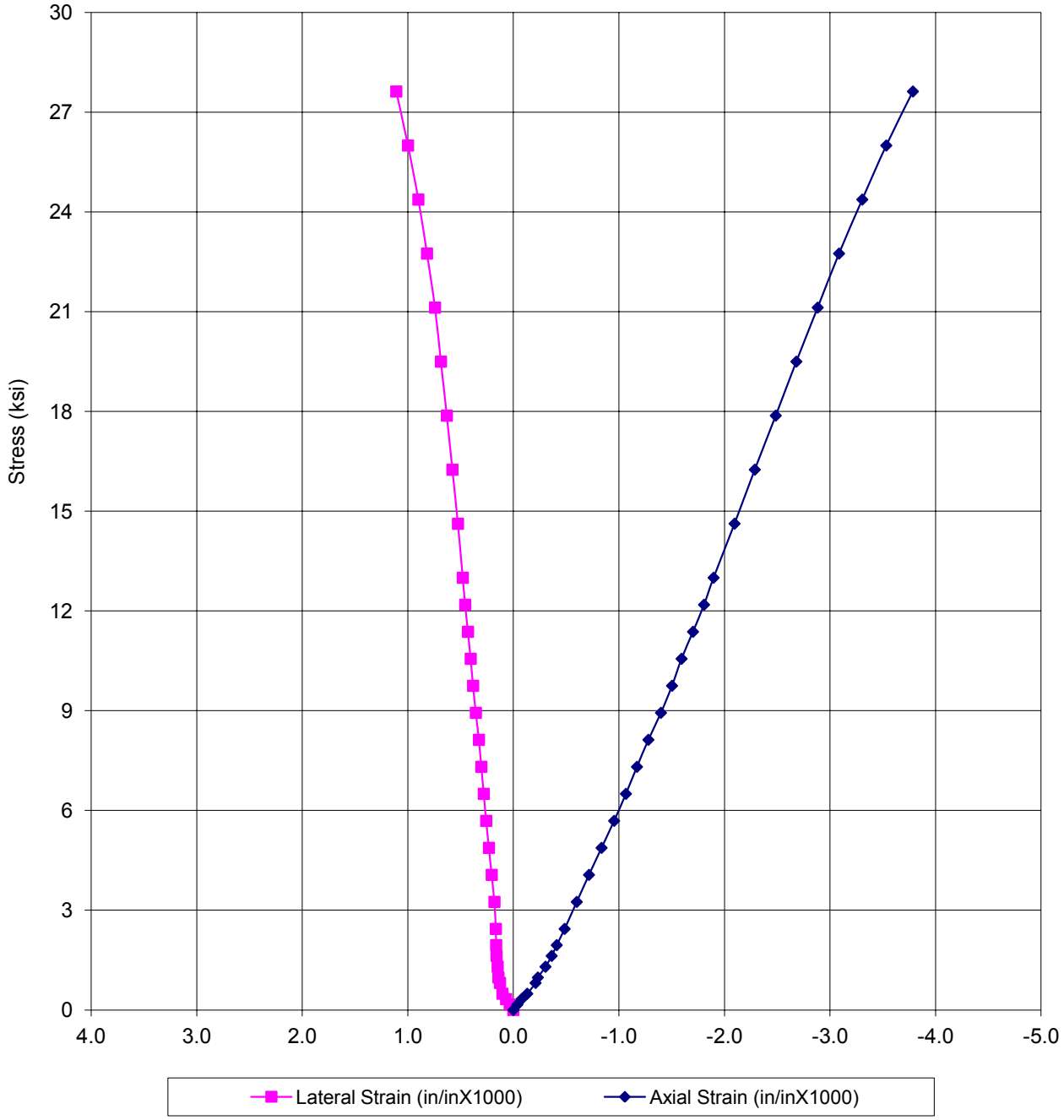
Tide Mill Bridge
Edmunds Township, ME



Rock Unconfined Compression Testing

Boring No.	BB-ECB-101	File No.	09.0025733.00
Sample No.	R1	Date:	05/23/11
Depth:	10.0-10.4'	Test No.	U10

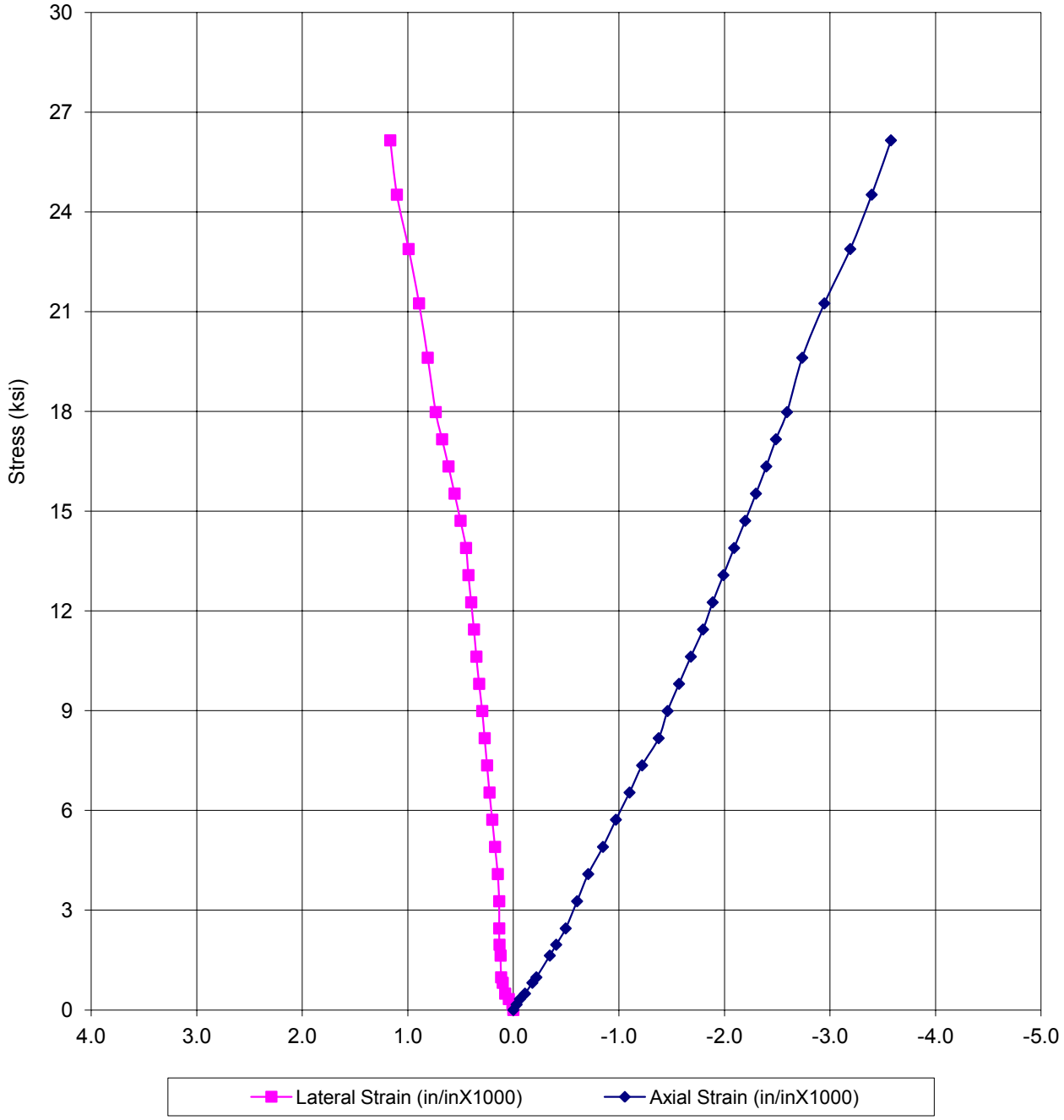
Tide Mill Bridge
Edmunds Township, ME



Rock Unconfined Compression Testing

Boring No.	BB-ECB-102	File No.	09.0025733.00
Sample No.	R3	Date:	06/14/12
Depth:	17.0-17.4'	Test No.	U11

Tide Mill Bridge
Edmunds Township, ME



Rock Unconfined Compression Testing

Boring No.	BB-ECB-104	File No.	09.0025733.00
Sample No.	R1	Date:	06/14/12
Depth:	15.3-15.7'	Test No.	U12

LABORATORY TESTING DATA SHEET

Project Name: Crane Mill Brook Bridge
Project No.: 09.0025733.01
Project Manager: Andrew Blaisdell

Location: Edmunds Township, ME
Assigned By: Michael Devoid
Report Date: 11/19/2013

Reviewed By: Matthew Dalgleish
Date Reviewed: 11/19/2013

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ PSI	Is_{50} PSI	Rock Formation or Description or Remarks
BB-ECB-202	R1	1.5-1.9	18		1.988	4.540	179.6			U	22,854	0.32		6.74				
BB-ECB-203	R3	17.1-17.5	19		1.985	4.516	176.4			U	2,055	0.08		2.10				Sample failed along healed fractures

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

(3) P=Petrographic PLD=Point Load (diametric
PLA= Point Load (Axial) RST= 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

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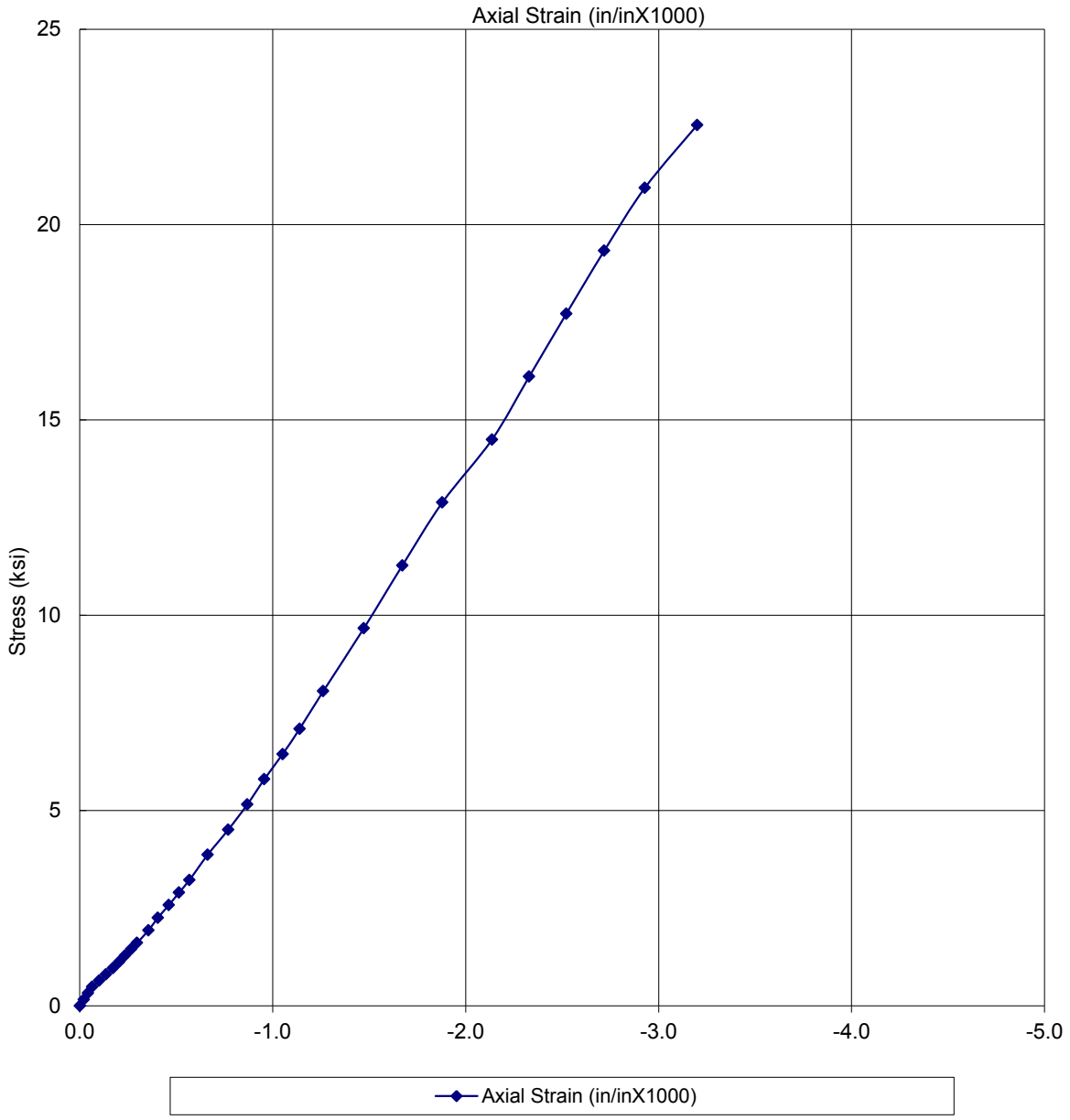
Edmunds Township, ME



Rock Testing

Boring No.	BB-ECB-202	GZA File No.	09.0025733.01
Sample No.	R1	Date:	11/18/2013
Depth:	1.5-1.9'	Test No.	U 18

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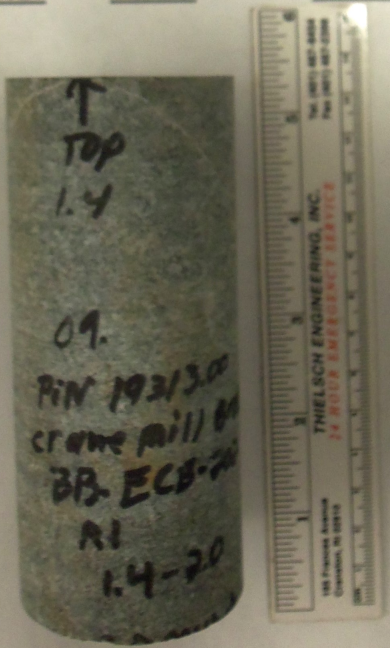
Crane Mill Brook Bridge
Edmunds Township, ME

GZA Project # 09.0025773.01
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-ECB-202	R-1	1.5-1.9'

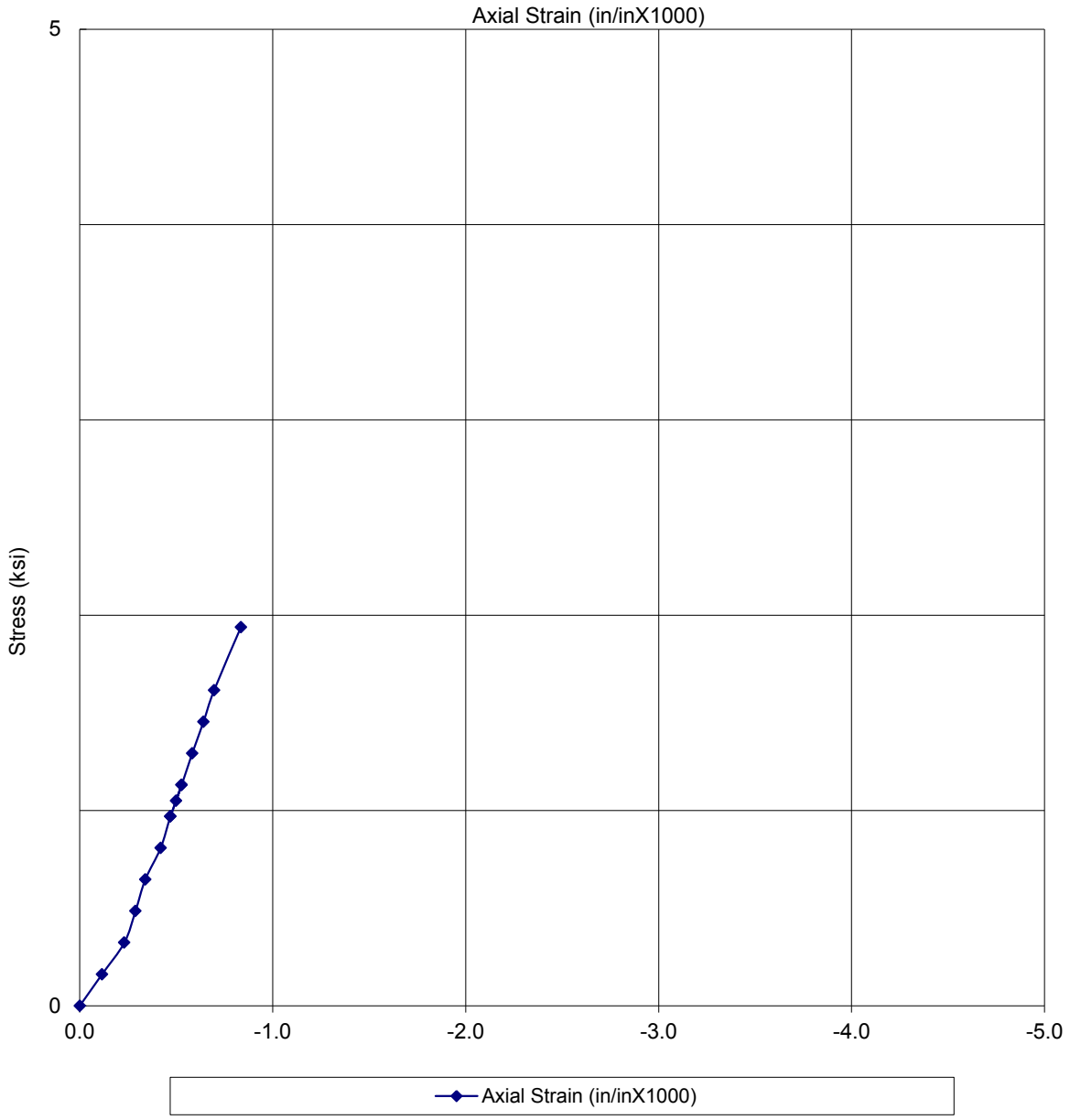


Crane Mill Brook Bridge
Edmunds Township, ME

GZA Project # 09.0025773.01
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-ECB-202	R-1	1.5-1.9'

Crane Mill Brook Bridge
Edmunds Township, ME



Rock Testing

Boring No.	BB-ECB-203	GZA File No.	09.0025733.01
Sample No.	R3	Date:	11/18/2013
Depth:	17.1-17.5'	Test No.	U 19

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Crane Mill Brook Bridge
Edmunds Township, ME

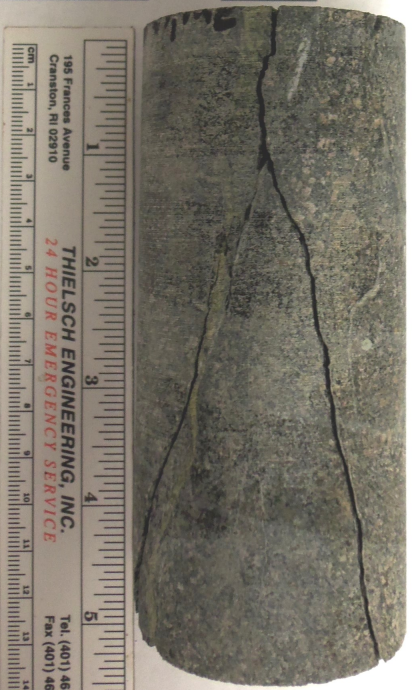
GZA Project # 09.0025773.01
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-ECB-203	R-3	17.1-17.5'

Crane Mill Brook Bridge
Edmunds Township, ME

GZA Project # 09.0025773.01
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-ECB-203	R-3	17.1-17.5'



APPENDIX E

CALCULATIONS

GZA
GeoEnvironmental, Inc
477 Congress Street
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Portland, Maine 04101
207-879-9190
Fax 207-879-0099
http://www.gza.com

Engineers and Scientists

JOB: 09.0025733.01 Edmunds Township
SUBJECT: Rock Bearing Resistance
SHEET: 1 OF 9
CALCULATED BY: ETL 12/5/14
REVIEWED BY: A. Blaisdell 4/15/14

Objective

Assess nominal and factored bearing resistance of a foundation on rock, proposed at Abutment 1 (South Abutment).

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, with 2013 interims. (AASHTO LRFD)

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

1. Rock Properties

Bedrock properties were obtained from rock core classification and strength testing completed for the Edmunds Township, ME Crane Mill Brook Bridge project.

Unconfined Compressive Strength

Five unconfined compressive strength tests were conducted on samples of gabbro (2 tests) and tuff breccia (3 tests), the results of which are presented in the table below. One tuff breccia test resulted in qc=2.06 ksi, but this sample failed along a healed joint, and tests on two other similar samples of tuff breccia ranged from qc=27.29-27.87 ksi. Therefore, sample with qc=2.06 ksi is not considered representative of intact rock and is not used in the evaluation. Two gabbro tests ranged from qc=19.01 to 22.85 ksi.

Design based on the lower bound of four representative laboratory test results. Use qc = σ_{ur}=19.0 ksi for design.

$$\sigma_{u.r} := 19.0\text{ksi}$$

Boring	Run	Depth (ft) Below GS (Core Run)			Length of Core Run (ft)	RQD %	Elev. (ft)		LAB	Rock Type
		Top		Bottom			Top	Bottom	q_c (ksi)	
BB-ECB-101	R1	9.9	-	13.9	4.0	73%	19.8	15.8	19.01	GABBRO
BB-ECB-102	R3	13.3	-	18.0	4.7	64%	7.0	2.3	27.87	TUFF BRECCIA
BB-ECB-104	R1	13.7	-	18.7	5.0	90%	16.1	11.1	27.29	TUFF BRECCIA
BB-ECB-202	R1	1.0	-	6.0	5.0	92%	20.1	15.1	22.85	GABBRO
BB-ECB-203	R3	15.5	-	20.5	5.0	77%	3	-2	2.06	TUFF BRECCIA

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Engineers and Scientists

JOB: 09.0025733.01 Edmunds Township
SUBJECT: Rock Bearing Resistance
SHEET: 2 OF 9
CALCULATED BY: ETL 12/5/14
REVIEWED BY: A. Blaisdell 4/15/14

Rock Quality Designation

The RQD data set from all test borings (see table below) was evaluated statistically including all of the borings for the project and also evaluating borings separately at each foundation location. The mean and standard deviation of the RQD were consistent between all of the foundation locations.

RQD ranges from 0% to 93%, and the weighted average is 56% for the entire site.

Boring	Run	Depth (ft) Below GS (Core Run)			Length of Core Run (ft)	Rec (in)	Rec (%)	RQD (in)	RQD %	Corr. Spacing (in)	Corr. Aperture (in)	Elev. (ft)		Rock Type	Avg RQD, by Boring
		Top		Bottom								Top	Bottom		
BB-ECB-101	R1	9.9	-	13.9	4.0	47.0	98%	35	73%	2.5-24	0.004-0.1	19.8	15.8	GABBRO	78%
BB-ECB-101	R2	13.9	-	18.4	4.5	53.0	98%	50	93%	2.5-80	0.01-0.02	15.8	11.3	GABBRO	
BB-ECB-101	R3	18.4	-	19.9	1.5	17.0	94%	8	44%	2.5-24	0.004-0.01	11.3	9.8	GABBRO	
BB-ECB-102	R2	11.8	-	13.3	1.5	18.0	100%	5	28%	0.75-8	0.01-0.02	8.5	7.0	TUFF BRECCIA	64%
BB-ECB-102	R3	13.3	-	18.0	4.7	54.0	96%	36	64%	0.75-24	0.01-0.02	7.0	2.3	TUFF BRECCIA	
BB-ECB-102	R4	18.0	-	22.0	4.0	45.0	94%	37	77%	0.75-24	0.01-0.1	2.3	-1.7	TUFF BRECCIA	
BB-ECB-103	R1	19.7	-	22.7	3.0	36.0	100%	10	28%	0.75-24	0.01-0.02	2.9	-0.1	TUFF BRECCIA	36%
BB-ECB-103	R2	22.7	-	27.0	4.3	50.0	97%	22	42%	0.75-24	0.01-0.02	-0.1	-4.4	TUFF BRECCIA	
BB-ECB-103	R3	27.0	-	30.2	3.2	36.0	94%	15	38%	0.75-8	0.01-0.02	-4.4	-7.6	TUFF BRECCIA	
BB-ECB-103	R4	30.2	-	33.2	3.0	36.0	100%	9	25%	0.75-8	0.01-0.1	-7.6	-10.6	TUFF BRECCIA	
BB-ECB-104	R1	13.7	-	18.7	5.0	60.0	100%	54	90%	0.75-24	0.004-0.01	16.1	11.1	TUFF BRECCIA	90%
BB-ECB-104	R2	18.7	-	23.0	4.3	52.0	101%	46	90%	0.75-24	0.004-0.01	11.1	6.8	TUFF BRECCIA	
BB-ECB-104	R3	23.0	-	23.7	0.7	7.0	83%	7	88%	2.5-8	-	6.8	6.1	TUFF BRECCIA	
BB-ECB-202	R1	1.0	-	6.0	5.0	55.0	92%	55	92%	24-80	0.02-0.1	20.1	15.1	GABBRO	93%
BB-ECB-202	R2	6.0	-	11.0	5.0	60.0	100%	56	93%	8-24	0.02-0.1	15.1	10.1	GABBRO	
BB-ECB-203	R1	10.0	-	14.2	4.2	35.0	69%	15	29%	0.75-8	0.004-0.1	8.5	4.3	TUFF BRECCIA	48%
BB-ECB-203	R2	14.2	-	15.5	1.3	17.0	109%	0	0%	2.5-8	0.02-0.1	4.3	3.0	TUFF BRECCIA	
BB-ECB-203	R3	15.5	-	20.5	5.0	55.0	92%	46	77%	2.5-24	0.004-0.1	3	-2	TUFF BRECCIA	
BB-ECB-204	R3	23.5	-	26.2	2.7	32.0	99%	4	13%	<0.75-8	0.02-0.1	-4.5	-7.2	GABBRO	4%
BB-ECB-204	R4	26.2	-	28.2	2.0	24.0	100%	0	0%	<0.75-8	0.02-0.1	-7.2	-9.2	GABBRO	
BB-ECB-205	R1	26.0	-	31.0	5.0	60.0	100%	35	59%	8-24	0.004-0.01	-6.6	-11.6	TUFF BRECCIA	59%
BB-ECB-207	R1	19.0	-	23.3	4.3	36.0	70%	35	67%	8-80	0.004-0.01	0.1	-4.2	GABBRO	67%
														Weighted Average RQD:	56%

2. Calculation of Rock Mass Rating (RMR)

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

Parameter 1- Uniaxial Compressive Strength

$\sigma_{u.r} = 19 \cdot ksi$

$\sigma_{u.r} = 2736 \cdot ksf$ Representative unconfined compressive strength of intact rock (see discussion above)

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_1 := 12$ for $\sigma_{u.r}$= 2160 to 4320 ksf

Parameter 2- Drill Core Quality

RQD ranges from 0% to 93%, and the average is 56% for the entire site. However, assume RQD=25-50 to allow for some lower quality rock at bearing level.

Mean RQD = 48%

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_2 := 8$ for RQD = 25 to 50

Parameter 3- Spacing of Joints

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

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_3 := 10$

Parameter 4- Condition of Joints

From boring logs, for Abutment 1, rough surface, with joint seperation less than 0.05 in.

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_4 := 12$

Parameter 5- Ground Water Conditions

Hydrostatic Conditions- Interstitial water

From AASHTO LRFD Table 10.4.6.4-1

Relative Rating $RR_5 := 7$

Adjustment for joint orientation (Parameter 6)

The joint sets are generally low angle and generally rough and tight to partially 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

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

$$RMR = 42$$

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

3. Evaluate Rock Property Constants s and m

From AASHTO LRFD Table 10.4.6.4-4 for Fair Quality Rock Mass

Categorized as rock type D, tuff breccia/gabbro, Fair Rock Quality, using s and m values interpolated from the logarithmic trend of plotted values from AASHTO Table 10.4.6.4-4 (plots on sheet 10).

$m := 0.27$

$s := 0.000065$

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

From Wyllie "Foundations on Rock"

Eq. 5.4 Pg.138

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

Where

$C_{f1} := 1.0$ From Wyllie Table 5.4 Pg. 138 Correction factor for foundation shape for rectangular foundation:

$s = 0.000065$ For L/B>6, use factor C_{fl}=1.0,

$m = 0.27$ For L/B=1, use factor C_{fl}=1.12, therefore,

$\sigma_{u.r} = 19 \cdot ksi$ For conservatism, assume long strip, lowest $C_{fl.}$

Nominal Bearing Resistance

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

$q_n = 151.6 \cdot ksf$ Say 151 ksf

Factored Bearing Resistance

Bearing Resistance Factor is specified in Table 10.5.5.2.2-1

$\phi_b := 0.45$ Footing on rock

$q_R := \phi_b \cdot q_n$

$q_R = 68.2 \cdot ksf$ Say 68 ksf

Reference:I:\Mathcad\units.xmcd

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Engineers and Scientists

JOB: 09.0025733.01 Edmunds Township
SUBJECT: Rock Bearing Resistance
SHEET: 6 OF 9
CALCULATED BY: ETL 12/5/14
REVIEWED BY: A. Blaisdell 4/15/14

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		• Very rough surfaces • Not continuous • No separation • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• 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			

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

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—*dolomite, limestone and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polymineralic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite* E = Coarse grained polymineralic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities CSIR rating: *RMR* = 100	*m* *s*	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: *RMR* = 85	*m* *s*	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: *RMR* = 65	*m* *s*	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: *RMR* = 44	*m* *s*	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: *RMR* = 23	*m* *s*	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: *RMR* = 3	*m* *s*	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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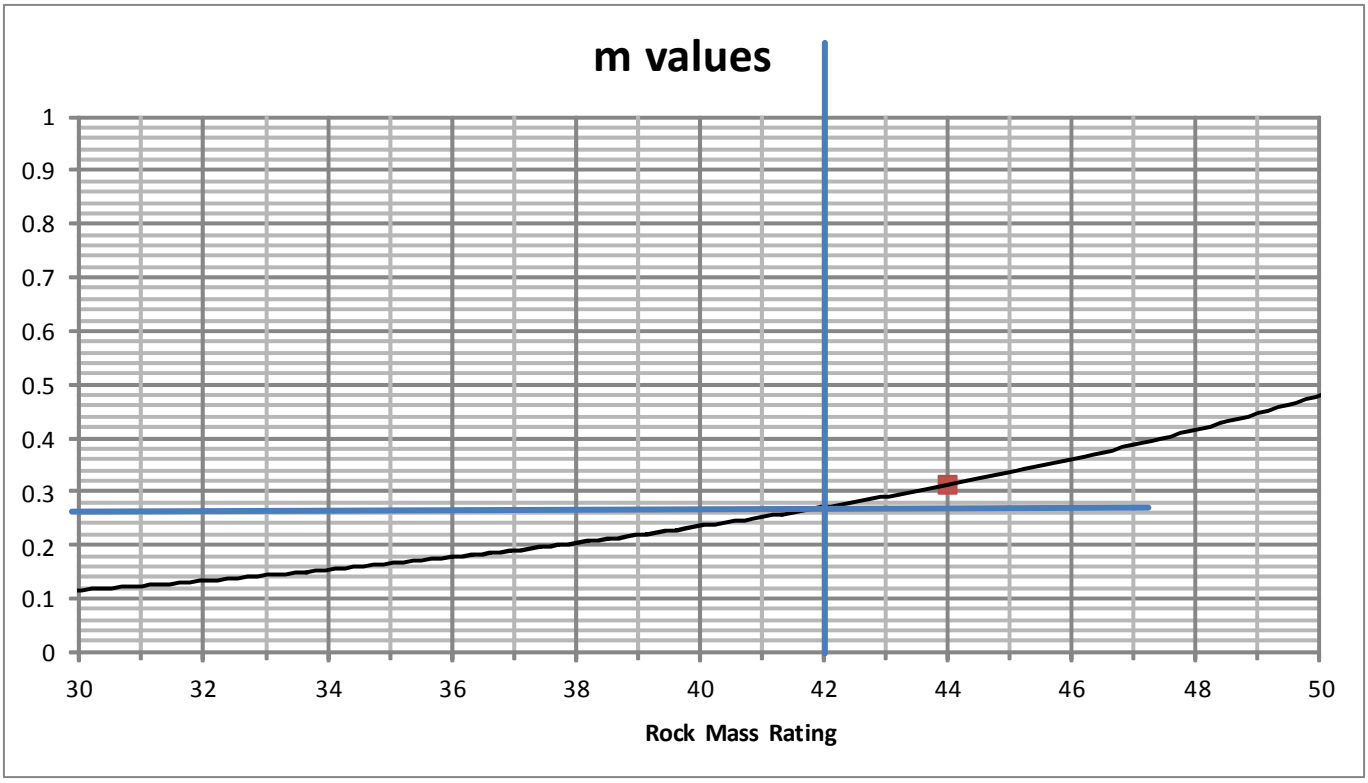
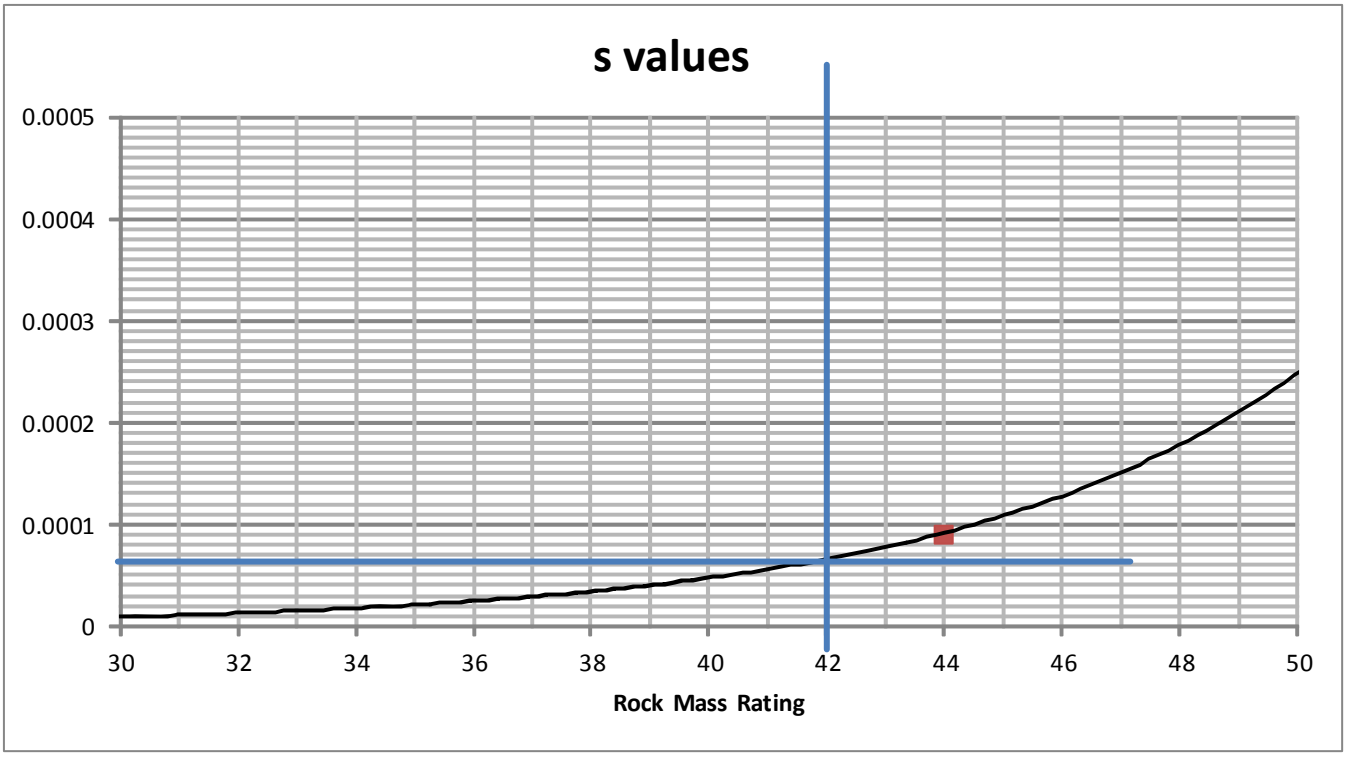
JOB: 09.0025733.01 Edmunds Township
SUBJECT: Rock Bearing Resistance
SHEET: 9 OF 9
CALCULATED BY: ETL 12/5/14
REVIEWED BY: A. Blaisdell 4/15/14





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JOB: 09.0025733.01 Edmunds Township
SUBJECT: Soil Bearing Resistance
SHEET: 1 OF 8
CALCULATED BY J. Baron 1/7/2014
CHECKED BY A. Blaisdell 4/15/14

Objective

Calculate soil bearing resistance for abutment spread footing (North Abutment) and MSE wingwall bearing on granular soils with a friction angle greater than 0 using the Theoretical method (Munfakh et al., 2001) in sand using SPT data. Evaluate strength and service limit bearing resistance for a range of effective footing widths.

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012 (AASHTO LRFD), Articles 10.5.5.2.2 and 10.6.3.1.
2. Terzaghi, Peck & Mesri, Soil Mechanics in Engineering Practice, Third Edition, 1996.

Soil Properties and Geotechnical Inputs

$\phi_f := 34deg$	Friction angle of soil
$\phi_b := 0.45$	Bearing resistance factor as specified in Table 10.5.5.2.2-1 (Theoretical Method, SPT Data, Strength Limit)
$c := 0ksf$	Cohesion, taken as undrained shear strength
$\gamma := 125pcf$	Unit weight of soil above or below the bearing depth of the footing
$N_c := 42.2$	Cohesion term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$N_q := 29.4$	Surcharge term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$N_\gamma := 41.1$	Total unit weight term bearing capacity factor as specified in Table 10.6.3.1.2a-1
$C_{wq}, C_{w\gamma} :=$	Correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2
	Depth to water table at depth of footing (D_f) $C_{wq} := 0.5$ $C_{w\gamma} := 0.5$
$d_q :=$	Correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation as specified in Table 10.6.3.1.2a-4
$s_c, s_\gamma, s_q :=$	Footing shape correction factors as specified in Table 10.6.3.1.2a-2
$S_c := 0.5in$	Allowable settlement
$q_s =$	Service limit bearing resistance for allowable settlement
$N_{60} := 20$	Representative SPT N_{60} value below footing

Load inclination factors are omitted considering modest embedment of footing per C10.6.3.1.2a.

NOTE:

The bearing material consists of very dense glacial till. However, surface water from the brook and/or encountered groundwater has the potential to weaken the upper portion of the bearing soil. In addition, some structural fill may be placed between the top of naturally deposited glacial till and bottom of footing. Therefore, reduced representative strength parameters (friction angle of 34 degrees and SPT N60 of 20 blows per foot) have been selected for design.

Footing Dimensions

$B_1 := \begin{pmatrix} 4 \\ 7 \\ 10 \\ 13 \\ 16 \end{pmatrix} ft$ Range of effective footing widths considered (includes eccentricity)

$L_1 := 56ft$ Length of footing

$D_f := 5ft$ Footing embedment depth (minimum for frost protection)

Strength Limit Design

$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$ Nominal Bearing Resistance Formula

$q_r = \phi_b \; q_n$ Factored Bearing Resistance Formula

Correction Factors

$d_{qtable} := \frac{D_f}{B_1}$ $d_{qtable} = \begin{pmatrix} 1.25 \\ 0.71 \\ 0.5 \\ 0.38 \\ 0.31 \end{pmatrix}$ Using Table 10.6.3.1.2a-4

$d_q := 1$ dq assumed soil above footing less competent that soil below footing.

$s_c := 1 + \left(\frac{B_1}{L_1}\right)\left(\frac{N_q}{N_c}\right)$ $s_c = \begin{pmatrix} 1.05 \\ 1.09 \\ 1.12 \\ 1.16 \\ 1.2 \end{pmatrix}$

$$s_q := 1 + \left(\frac{B_1}{L_1}\tan(\phi_f)\right) \qquad s_q = \begin{pmatrix} 1.05 \\ 1.08 \\ 1.12 \\ 1.16 \\ 1.19 \end{pmatrix}$$

$$s_\gamma := 1 - 0.4\left(\frac{B_1}{L_1}\right) \qquad s_\gamma = \begin{pmatrix} 0.97 \\ 0.95 \\ 0.93 \\ 0.91 \\ 0.89 \end{pmatrix}$$

Bearing Capacity Factors

$$N_{cm} := N_c \cdot s_c \qquad N_{cm} = \begin{pmatrix} 44.3 \\ 45.9 \\ 47.5 \\ 49 \\ 50.6 \end{pmatrix}$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \qquad N_{qm} = \begin{pmatrix} 30.8 \\ 31.9 \\ 32.9 \\ 34 \\ 35.1 \end{pmatrix}$$

$$N_{\gamma m} := N_\gamma \cdot s_\gamma \qquad N_{\gamma m} = \begin{pmatrix} 39.9 \\ 39 \\ 38.2 \\ 37.3 \\ 36.4 \end{pmatrix}$$

Nominal Bearing Resistance

$$q_n := \overrightarrow{\left(c \cdot N_{cm} + \gamma \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma \cdot B_1 \cdot N_{\gamma m} \cdot C_{w\gamma}\right)} \qquad q_n = \begin{pmatrix} 15 \\ 19 \\ 22 \\ 26 \\ 29 \end{pmatrix} \cdot ksf$$

Factored Bearing Resistance - Strength Limit State

$$q_R := \phi_b \cdot q_n \qquad q_R = \begin{pmatrix} 6.6 \\ 8.3 \\ 10 \\ 11.6 \\ 13.1 \end{pmatrix} \cdot ksf \qquad \text{for} \quad B_1 = \begin{pmatrix} 4 \\ 7 \\ 10 \\ 13 \\ 16 \end{pmatrix} \cdot ft$$

Service Limit Design

Evaluate service limit bearing resistance for the specified allowable settlement using the semi-empirical SPT Method by Burland and Burbidge (1985) provided in Terzaghi, Peck & Mesri, 96.

$$S_{cm} := S_c \cdot \frac{1}{1mm} \qquad S_{cm} = 12.7$$

Allowable settlement in millimeters and unitless

$$B_{1m} := B_1 \cdot \frac{1}{1m} \qquad B_{1m} = \begin{pmatrix} 1.2 \\ 2.1 \\ 3 \\ 4 \\ 4.9 \end{pmatrix}$$

Effective footing width in meters and unitless

$$S_{cmr} := S_{cm} \cdot \left[\frac{\left[1.25 \cdot \left(\frac{L_1}{B_1} \right) \right]}{\left(\frac{L_1}{B_1} \right) + 0.25} \right]^2 \qquad S_{cmr} = \begin{pmatrix} 19.2 \\ 18.7 \\ 18.2 \\ 17.7 \\ 17.3 \end{pmatrix}$$

Correction formula for rectangular footings (Terzaghi EQ 50.14)

$$EQ_1 := S_{cm} \cdot \left(\frac{S_{cm}}{S_{cmr}}\right) \qquad EQ_1 = \begin{pmatrix} 8.42 \\ 8.64 \\ 8.87 \\ 9.1 \\ 9.33 \end{pmatrix}$$

$$EQ_2 := \frac{N_{60}^{1.4}}{1.7 \cdot B_{1m}^{0.75}} \qquad EQ_2 = \begin{pmatrix} 34 \\ 22 \\ 17 \\ 14 \\ 12 \end{pmatrix}$$

$$q_{snc} := \overrightarrow{\left(EQ_1 \cdot EQ_2\right)}$$

$q_{snc} =$

	0
0	283.0
1	190.9
2	149.9
3	126.3
4	110.9

Formula results are in kPa (Terzaghi EQ 50.28)

Results represent normally consolidated soil.

$$q_s := 3 \cdot q_{snc} \qquad q_s = \begin{pmatrix} 849 \\ 573 \\ 450 \\ 379 \\ 333 \end{pmatrix}$$

Assumes sand is overconsolidated with average bearing pressure less then preconsolidation pressure, reducing compressibility by a factor of 3 (Terzaghi EQ 50.11b)

$$q_{sm} := q_s \cdot 1kPa \qquad q_{sm} = \begin{pmatrix} 849 \\ 573 \\ 450 \\ 379 \\ 333 \end{pmatrix} \cdot kPa$$

Service limit bearing resistance for allowable settlement (metric units)

$$q_{se} := q_{sm} \qquad q_{se} = \begin{pmatrix} 17.7 \\ 12 \\ 9.4 \\ 7.9 \\ 6.9 \end{pmatrix} \cdot ksf$$

Service limit bearing resistance for allowable settlement (English units)

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Bearing Resistance Chart

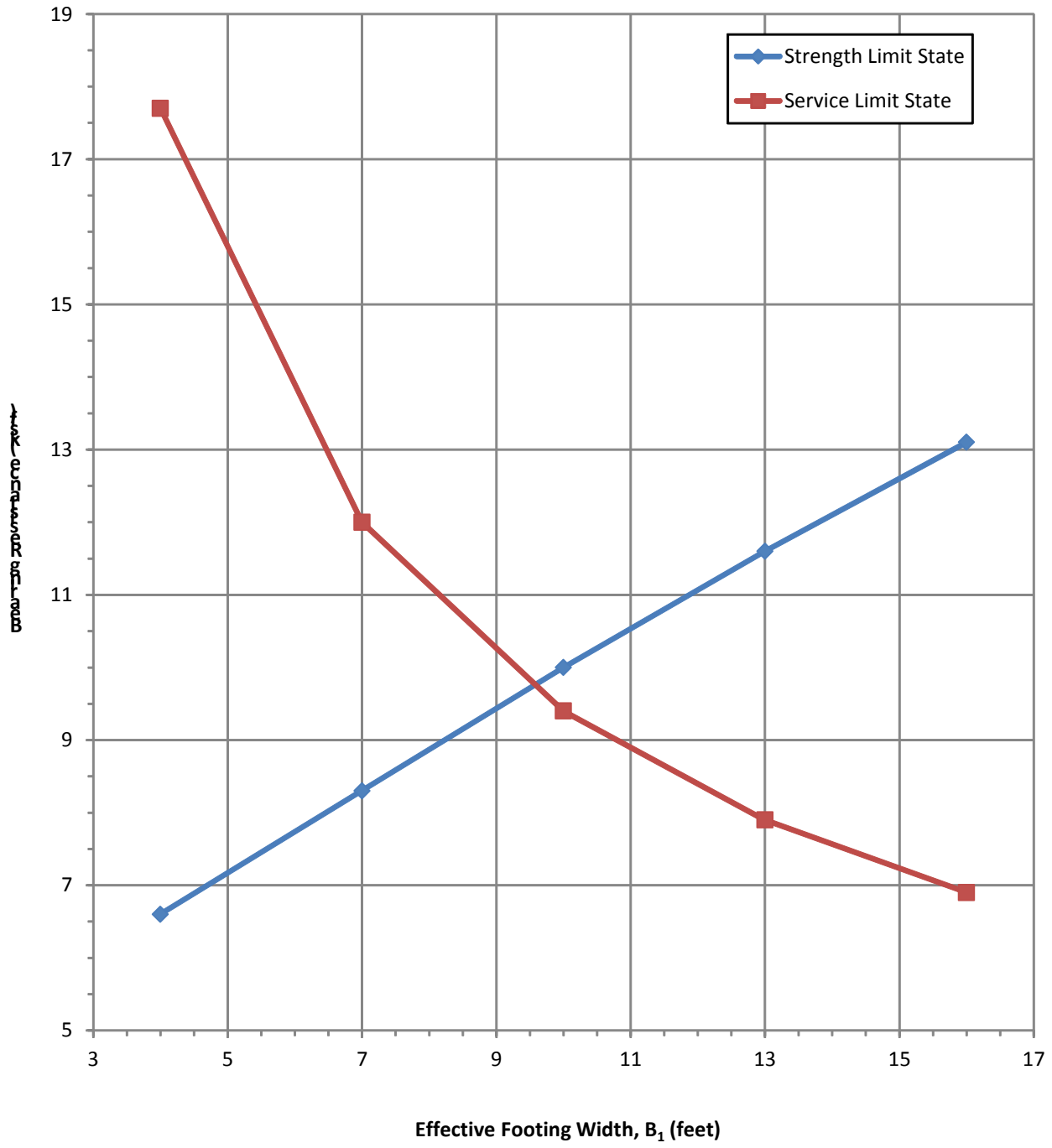

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

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

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

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

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

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

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

Table 10.6.3.1.2a-4—Depth Correction Factor d_q

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

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0.

Linear interpolations may be made for friction angles in between those values shown in Table 10.6.3.1.2a-4.