FINAL GEOTECHNICAL DESIGN REPORT
BRIDGE NO. 5069 OVER FIFTEEN MILE STREAM
MAINE DOT WIN 18231.00
BENTON, MAINE

PREPARED FOR:
Erdman Anthony
Rochester, New York

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

January 2014
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GeoEnvironmental, Inc.

Engineers and Scientists

January 15, 2014
File No. 09.0025730.00

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Re: Final Geotechnical Design Report
Bridge No. 5069 Over Fifteen Mile Stream
Benton, Maine

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Dear Frank:

We are pleased to provide this Final Geotechnical Design Report for MaineDOT Bridge No. 5069 over Fifteen Mile Stream in Benton, Maine. Our work was completed in accordance with the Subconsultant Agreement between Erdman Anthony and GZA GeoEnvironmental, Inc. (GZA) dated April 4, 2012, which incorporates GZA's proposal No. 09.P000114.12, dated March 1, 2012, and the attached Limitations included in **Appendix A** of this report.

It has been a pleasure serving the Erdman Anthony team 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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1.0 INTRODUCTION

This report presents the results of GZA's geotechnical evaluation for the proposed replacement of Maine Department of Transportation (MaineDOT) Bridge No. 5069 over Fifteen Mile Stream. Our services were provided in accordance with the Subconsultant Agreement between Erdman Anthony and GZA GeoEnvironmental, Inc. (GZA) dated April 4, 2012, which incorporates GZA's proposal No. 09.P000114.12, dated March 1, 2012, and the attached Limitations included in **Appendix A**.

GZA previously prepared a preliminary geotechnical design basis memorandum for the subject project, the results of which were presented in a memorandum dated June 20, 2012. The evaluations and recommendations presented in this report supersede the preliminary findings presented in the memorandum.

1.1 BACKGROUND

Bridge No. 5069 carries State Route 139 (Unity Road) over Fifteen Mile Stream in Benton, Maine as shown on **Figure 1, Locus Plan**. The existing bridge was constructed in 1961 and consists of a two-span bridge supported by full-height cantilever abutments and a center pier. Each span is approximately 38 feet in length. The abutments and center pier appear to be supported on spread footing foundations bearing on soil.

Erdman Anthony's preliminary plans show that the replacement bridge will consist of a 120-foot-long single-span bridge, extending from approximately Sta. 106+81 to Sta. 108+01, the location of which is shown on **Figure 2, Boring Location Plan.** The proposed bridge will be longer than the existing bridge, with the proposed abutments moved approximately 22 feet beyond (uphill from) the existing abutments.

The new bridge is proposed to consist of an integral abutment bridge supported by driven steel H-piles. We understand that the anticipated thermal deformation of the bridge superstructure is approximately 0.7 inches, which would result in approximately 0.35 inches of pile head translation at each abutment.

The centerline of the proposed bridge will be shifted about 23 feet south of the current bridge alignment. The proposed bridge alignment will require approach fills up to 13 feet above existing grades along the right shoulder of the roadway, and typical fill heights of 1 to 4 feet above the existing road grade.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide final geotechnical engineering recommendations for the proposed Bridge No. 5069 replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions and reviewed mapped surficial and bedrock geology of the site;
- Coordinated and observed four test borings;
- Conducted a laboratory testing program to evaluate engineering properties of the site soil;

- Conducted geotechnical engineering analyses to evaluate foundation design for the replacement bridge, embankment design considerations, and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven H-piles, lateral earth pressures and seismic design parameters; and
- Prepared this final report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

GZA completed a subsurface investigation program consisting of four (4) test borings, including two (2) test borings (BB-BB-101 and BB-BB-104) to assess the existing approach fill and two (2) test borings (BB-BB-102 and BB-BB-103) for the proposed abutment foundations. The as-drilled locations were approximated, based on taped measurements from site features, and ground surface elevations of borings were interpolated from contour plans provided by Erdman Anthony. The approximate as-drilled boring locations are shown on **Figure 2.** Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 88).

Maine Test Boring of Hermon, Maine provided drilling services and coordinated utility clearance for the project. All of the test borings were completed using a track-mounted drill rig outside of the existing travelway to enhance safety and eliminate the need for traffic control and lane closures. Drilling was completed on March 17 and 18, 2012.

Test borings were drilled to depths of 9.1 to 35 feet below ground surface (bgs). Borings drilled for approach fills were terminated at 12 feet bgs or upon split spoon refusal, and borings drilled for abutment foundations were terminated approximately 10 feet into bedrock. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix B**.

The abutment foundation borings (BB-BB-102 and BB-BB-103) were drilled using 3- and 4-inch spun casing and drive and wash drilling techniques. Borings completed for the approach fills (BB-BB-101 and BB-BB-104) were drilled with hollow stem augers. Standard penetration testing and split spoon sampling were performed continuously in the fill materials and at 5-foot typical intervals at greater depths using a 24-inch-long, 1-3/8-inch inside diameter sampler. Bedrock cores were obtained using NQ2 coring equipment. The borings were generally backfilled with ¾-inch crushed stone or soil cuttings.

3.0 LABORATORY TESTING

GZA retained Thielsch Engineering's Geotechnical Laboratory (formerly GZA's laboratory) in Hopkinton, Massachusetts to complete a soils testing program to assess the gradation and engineering characteristics of the soil. The testing program consisted of gradation analysis/AASHTO Classification/Frost Classification assessments of seven soil samples, water content of nine soil samples and Atterberg Limits of one soil sample. Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available literature, surficial geologic units mapped in the Bridge No. 5069 area include Glacial Till deposits overlying bedrock. Bedrock outcrops are also mapped in the vicinity of the bridge. Bedrock at the site is mapped as the Vassalboro Formation bedrock unit. The Vassalboro Formation is described as typically a massive, bluish-gray meta-sandstone, with local areas of quartzite, and containing numerous calcareous beds.

4.2 SUBSURFACE PROFILE

Five soil units: Fill, Recent Alluvium, Sand and Gravel, Marine Deposit, and Glacial Till, were encountered above bedrock in the test borings. The encountered thicknesses and generalized descriptions are presented in the following table, in descending order from ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile based on the test boring results is presented as **Figure 3, Interpretive Subsurface Profile**.

Soil Unit	Approx. Encountered Thickness (ft)	Generalized Description
Fill	2 to 14	Loose, brown, fine to coarse SAND, little to some Gravel, trace to some Silt. • MaineDOT Frost Classification = 0-II. • Encountered in borings BB-BB-101, -102 and -104. • Possible rock fill encountered in BB-BB-102, 6 to 13.2 feet bgs.
Recent Alluvium	2	Soft, brown/gray, SILT, some fine to coarse Sand, with organics. • Encountered at the ground surface in boring BB-BB-103.
Sand and Gravel	8	Loose to medium dense, gray to olive, fine to medium Sandy SILT, some Gravel to fine to coarse SAND, some to trace Gravel, little to trace Silt. • MaineDOT Frost Classification = IV. • Encountered beneath fill in boring BB-BB-103. • Possible foundation backfill.
Marine Deposit	2 to 7	Stiff to very stiff, olive to gray, Clayey SILT, little to some fine to medium Sand, trace Gravel. • Encountered in borings BB-BB-101, -102 and -104.
Glacial Till	4 to 5	Very dense, gray to brown, fine to coarse SAND, with varying amounts of Gravel and Silt. • MaineDOT Frost Classification = II. • Encountered in borings BB-BB-102, -103 and -104.
Top of Bedrock Elevation	El. 107 to El. 108 where cored at proposed abutments.	

4.2.1 Bedrock

Bedrock encountered in the borings consisted of Metasandstone and was generally described as hard, fresh, medium grained and gray with calcite stringers. Joints were generally close to moderately spaced, low to high angle, planar, rough, fresh to discolored and tight with occasional oxidation staining. The Rock Quality Designation ranged from 83 to 100 percent.

Considering that several outcrops are visible adjacent to the east approach, GZA interpreted the split spoon refusal encountered in boring BB-BB-104 to represent possible bedrock.

4.2.2 Groundwater

Groundwater was measured in the completed boreholes either immediately after drilling or a few hours after drilling was complete. There was no free water observed in borings BB-BB-101 and BB-BB-104, and measured groundwater levels in BB-BB-102 and BB-BB-103 were 14.2 and 1.0 feet bgs respectively. The measured levels correspond to approximately El. 118.5 to El. 119.5, and were at or slightly above the water level in the stream at the time of the measurements.

The groundwater observations were made at the times and under the conditions stated in the boring logs. Groundwater levels fluctuate due to season, precipitation, infiltration and construction activity in the area. Therefore, groundwater levels during and after construction are likely to vary from those encountered at the time of the test borings.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (herein known as AASHTO) and the MaineDOT Bridge Design Guide, 2003 Edition (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations developed by GZA for the project are attached in **Appendix D** of this report.

5.2 APPROACH EMBANKMENTS

On the west approach, borings BB-BB-101 and BB-BB -102 were drilled through 5 to 6 feet of loose sand and gravel embankment fill; rock fill (BB-BB-102 only); clayey silt marine deposits; and very dense glacial till. The rock fill was approximately 7 feet thick in BB-BB-102, and extended to the top of the marine deposit. On the east approach, boring BB-BB-104 was drilled through approximately 2 feet of loose sand and gravel embankment fill; 3 feet of stiff clayey silt, and very dense glacial till.

Grading will require a 2 to 3 foot cut into the existing embankment fill on the west approach to reach the bottom of the proposed pavement subgrade. Pavement subgrade soils are anticipated to consist of sand and gravel embankment fill on the west. On the east the pavement subgrade occurs at or above existing grade, so a combination of sand and gravel embankment fill and clayey silt marine deposits are anticipated at proposed pavement subgrade level. Ground preparation for the pavement section should include proof-compaction of loose sand and gravel fill where exposed. Clayey silt marine deposits do not require densification.

The proposed embankment will be widened to the south of the existing approach with a maximum fill height of approximately 14 feet. The maximum side slope angles are anticipated to be 2 horizontal to 1 vertical (2H:1V), or flatter, with loam and seed surface treatments. Based on our experience with similar conditions, it is GZA's opinion that settlement under the new approach fills

will occur elastically during fill placement and that post-construction settlement of the widened embankments will be less than 2 inches.

The embankments will be constructed per MaineDOT standard specifications and details using engineered fill and controlled methods over existing stiff silt and medium dense sand and gravel. In our experience, conventional earthfill embankments constructed over relatively dense/stiff overburden soils meet the minimum required safety factors for global stability.

5.3 SEISMIC DESIGN CONSIDERATIONS

The new abutments will bear on steel HP-section piles driven to bearing in dense soils or on bedrock.

The subsurface profile at Bridge No. 5069 for seismic design includes the proposed and existing approach fills (including backfill behind abutments), existing marine deposits, 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 values for proposed fill above (i.e., below the proposed roadway surface). 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, and the bridge is assigned to Site Class D.

The available subsurface data indicates that the natural materials encountered at the site are sufficiently dense that the potential for liquefaction is very low.

5.4 EVALUATION OF FOUNDATIONS

5.4.1 Foundation Type Assessment

GZA presented two feasible foundation alternatives in our June 20, 2012 memorandum: spread footings founded on glacial till (full height abutment option) or pile supported foundations (integral abutment option). Based on constructability and cost considerations, Erdman Anthony selected an integral abutment bridge supported on steel H-piles as discussed previously. Design considerations are presented below.

5.4.2 Pile Design Considerations and Load and Resistance Factors

Evaluations were conducted for axial compressive geotechnical resistance of the piles. The geotechnical static resistance of piles (side friction and end bearing) in glacial till was calculated using the Meyerhof (SPT) method in accordance with AASHTO Article 10.7. The results indicate the piles will gain support through primarily through end bearing in glacial till or on bedrock, with approximately 10 percent or less of the resistance derived from skin friction in overburden soil. Based on our experience with similar soils, we anticipate that the piles will be driven into or near bedrock to achieve the required resistance. The geotechnical static resistance was also used as an input in wave equation analyses conducted to assess the pile drivability. Axial tensile geotechnical (uplift) resistance was not evaluated because the integral abutment configuration will not impose uplift loading on the piles. Since the piles will gain support in primarily dense granular soil and/or bedrock, there is no reduction for group interaction in axial compression.

By utilizing end bearing steel H-piles, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

Piles should be designed at the strength limit state considering the structural resistance of the piles and a resistance factor of 0.50, per LRFD Section 10.7.3.2.3 for hard driving condition; the geotechnical resistance of the piles; and the potential loss of lateral support due to scour at the design flood event. In GZA's experience for end bearing piles on bedrock, the structural resistance or drivability resistance will control the geotechnical static resistance of the pile.

The pile driving criteria are expected to be established based on dynamic pile testing with signal matching analysis. The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.

GZA considered the potential for downdrag loading for piles supporting the abutments. It is GZA's opinion that the stiff marine deposit soils will be relatively incompressible under the pressure imposed by the new fill at the abutment locations. Therefore, downdrag loading was not included in the pile design.

Pile design recommendations are presented in **Section 6.4** of this report.

AASHTO LRFD load 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 AASHTO Table 3.4.1-2 for strength and extreme limit state design. A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck.

5.4.3 Pile Type and Loading Data

The abutments are planned to be supported on ASTM A572, Grade 50 (fy=50 ksi) steel H-piles. Each abutment will include a single row of five, HP14x89 piles. The maximum factored axial load for the strength condition provided by Erdman Anthony is 350 kips per pile. Considering the resistance factor of 0.65 for drivability, the required nominal pile resistance is approximately 539 kips.

5.4.4 Wave Equation Analyses

Wave equation analyses were performed to assess pile drivability; results are included in **Appendix D**. The analyses set out to estimate the largest nominal pile resistance that could be achieved with the selected soil model and hammer system, and without exceeding allowable driving stresses in the pile. It has been our practical experience that the wave equation analysis is conservative in estimating driving stresses, and that a larger nominal resistance is frequently achieved in the field during dynamic pile testing.

The analyses used the design soil profile with shallower rock presented for Abutment 2 in **Section 5.4.1**, with an embedded pile length of approximately 15 feet. The profile at Abutment 2 is considered to conservatively represent driving conditions at both abutments considering that it is the shortest anticipated pile length. The evaluation used a Delmag D16-32 open-end diesel pile driving hammer with a manufacturer's rated energy of 40,200 ft-lbs and a fuel setting

corresponding to the maximum chamber pressure of 1,425 psi. The results of the wave equation analyses are summarized below.

WAVE EQUATION ANALYSIS RESULTS					
Embedded Pile Length	**Driving System**	**Nominal Geotechnical Drivability Resistance (kips)**	**Factored Geotechnical Drivability Resistance (kips)**	**Maximum Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
15 feet	Delmag D16-32 (40,200 ft-lb)	575	374	45	9 to 10

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT range of 3 and 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the nominal resistance noted.

Hard driving is anticipated in rock fill layers, glacial till and/or on or near the bedrock surface. Recommendations are provided for tip protection in **Section 6.5** to reduce the potential for driving damage.

5.4.5 Design Subsurface Profiles for Lateral Pile Analysis

The subsurface profile at the Bridge No. 5069 generally consists of loose fill, medium stiff to very stiff marine deposits and very dense glacial till overlying bedrock. Loose to medium dense sand and gravel was encountered at Abutment 2 above the natural soils and is likely foundation backfill material for the existing bridge. It is planned to support the new bridge abutments on driven piles. The following soil profiles are provided for use in evaluating abutment pile design at each abutment.

DESIGN SUBSURFACE PROFILE – ABUTMENT 1 (Boring BB-BB-102)					
Strata Designation	**Approx. Base El. (ft-NAVD 88)**	**Total Unit Weight (pcf)**		**Representative Φ' (°) or S_u (psf) for layer**	
		Lower Bound	**Upper Bound**	**Lower Bound**	**Upper Bound**
Fill / Rock Fill	120	120	135	31º	36º
Marine Deposit	113	115	120	2000 psf	
Glacial Till	109	125	135	36º	40º
Bedrock	--	--	--	--	--

DESIGN SUBSURFACE PROFILE – ABUTMENT 2 (Borings BB-BB-103 and BB-BB-104)					
Strata Designation	**Approx. Base El. (ft-NAVD 88)**	**Total Unit Weight (pcf)**		**Representative Φ' (°) or S_u (psf) for layer**	
		Lower Bound	**Upper Bound**	**Lower Bound**	**Upper Bound**
New Fill	122-118	120	130	32º	36º
Sand and Gravel	114-110	120	125	31º	35º
Glacial Till	110-107	125	135	36º	40º
Bedrock	--		--	--	--

The design parameters and elevations at Abutment 1 are based solely on boring BB-BB-102, considering that it was drilled to the west (embankment side) of Abutment 1. The engineering design parameters at Abutment 2 are based on boring BB-BB-103, but the elevations consider the possibility that rock is shallower at Abutment 2 than boring BB-BB-103, which was drilled to the west (stream side) of Abutment 2. The shallower rock elevation (El. 110) is based on linear interpolation at the centerline of bearing between the top of rock encountered in BB-BB-103 and the refusal elevation in boring BB-BB-104.

The tables present the anticipated lower- and upper-bound values for the LPile input parameters. These are intended to allow Erdman Anthony to assess the probable range of pile performance, and the impact of the input parameters on structural design of the integral abutment piles.

5.4.6 Lateral Pile Resistance

We understand lateral pile analysis will be completed by Erdman Anthony using L-Pile® or FB-Pier® software that use non-linear P-Y curves to model soil reactions. We recommend that the pile analysis results be used to check that the combined axial and bending demand meet LRFD structural requirements. It is anticipated the controlling analysis will be based on the maximum thermal deflection of 0.35 inches, as provided by Erdman Anthony. Since the pile lengths are close to or at the minimum embedment length provided in Table 5-5 of the BDG, it is possible that the piles will not achieve a fixed condition. We recommend that the lateral pile analysis be required to show that the tips meet a pinned condition (no translation), if a fixed condition is not evident.

Geotechnical design parameters were developed for L-Pile® analyses corresponding the design soil engineering properties tabulated in **Section 5.4.5**. The recommended design parameters are presented in **Section 6.5**.

5.4.7 Lateral Earth Pressure

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move towards the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.4** of this report.

A report prepared for the Virginia Transportation Research Council (VTRC) entitled, "The Behavior of Integral Abutment Bridges," evaluated earth pressures on abutment backwalls and concluded that Rankine or Coloumb passive earth pressure could be conservatively used for design of most integral abutment bridges. The relevant discussion from this reference is attached in **Appendix D**.

Lateral earth pressure evaluations for abutments are based on the BDG and the VTRC reference indicated above and are presented in **Appendix D** and summarized below:

- Passive earth pressure coefficients were developed using Rankine theory for Type 4 soil.
- AASHTO Commentary C3.10.9.1 specifies that single-span bridges are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

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

5.4.8 Frost Penetration

Fill soils are anticipated to be present at the abutments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1625, and with low to moderate moisture content (±15%) soils, the estimated depth of frost penetration is 6.5 feet.

6.0 RECOMMENDATIONS

6.1 SEISMIC DESIGN

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

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.117 g
SDs (Period = 0.2 sec)	0.248 g
SD1 (Period = 1.0 sec)	0.109 g

Per AASHTO Article 3.10.6, the site is assigned to Seismic Zone 1 based on a calculated SD1 of 0.11 g. Per AASHTO Article 4.7.4, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but the minimum requirements specified in AASHTO Articles 4.7.4.4 and 3.10.9 apply.

6.2 EMBANKMENT CONSTRUCTION

Fill and marine deposits were encountered in borings taken through the approach embankments. Pavement subgrade soils are anticipated to consist of loose sand and gravel embankment fill on the west. On the east a combination of loose sand and gravel embankment fill and stiff clayey silt marine deposits are anticipated at proposed pavement subgrade level. Ground preparation for the pavement section should include proof-compaction of loose sand and gravel fill where exposed. The clayey silt marine deposits do not require compaction.

We recommend that sand and gravel subgrade materials shall be proof-compacted after excavation is made to the proposed subgrade level. Proof compaction should consist of making at least 4 passes with a heavy roller. Subgrade that exhibits weaving, rutting or otherwise unsuitable performance during rolling should be excavated and the unsuitable material replaced with compacted fill meeting the requirements of MaineDOT Section 203.

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill behind new abutments should consist of Maine DOT 703.19 Granular Borrow for Underwater Backfill, BDG Type 4 soil. Recommended soil properties for Type 4 soils to be used as backfill are as follows:

- Internal Friction Angle of Soil = 32°
- Soil Total Unit Weight = 125 pcf
- Coefficient of Passive Earth Pressure, K_p= 3.3 (use for design of backwalls and wingwalls)

- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (Heq) values developed in accordance with AASHTO Article 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 BDG.
  - We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. Foundation drains should be sloped to drain by gravity and should daylight on the side slope of the approach embankment and/or through weep holes in the abutments.

6.4 PILE DESIGN

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed primarily in end-bearing on or near the bedrock surface.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load of 350 kips by a resistance factor of 0.65, resulting in a required nominal pile resistance of 539 kips.
- Preliminary wave equation analyses indicate that the piles can be driven to a nominal resistance of 575 kips using a diesel hammer with a rated energy of about 40,000 foot-pounds for 15-foot-long ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel). The final penetration resistance was 9 to 10 bpi, which is within the Maine DOT range of 3 to 15 blows per inch. In GZA's experience, the preferred range of final penetration resistance is 6 to 10 blows per inch.
- For the purpose of estimating quantities, it should be assumed that the piles will penetrate to approximately El. 108 at Abutment 1 and El. 106 at Abutment 2. These assumes that the top of rock elevations encountered in borings BB-BB-102 and BB-BB-103 are representative for Abutments 1 and 2, respectively, and the piles may penetrate a short distance into the bedrock.
- We recommend that one pile at each abutment be dynamically tested to assess driving stress at the maximum factored load and to develop blow count criteria to achieve the required nominal resistance.
- Since the pile lengths are very short, no splices should be allowed.
- Due to the short pile lengths and integral abutment design configuration, it is desirable to achieve at least a pinned condition at the pile tips. We recommend using Associated Pile

and Fitting, Rock Injector, HP-80500 Pile Points to protect the piles during driving and enhance their ability to achieve the pinned condition.

- The structural engineer should evaluate the minimum required pile embedment for the integral abutment bridge. The recommended lower bound (LB) and upper bound (UB) geotechnical parameters for L-Pile® analyses are provided in the tables below. The top of the upper layer (Fill) corresponds to the bottom of the proposed pile cap, and the bottom lower layer (Glacial Till) corresponds to anticipated top of rock. Therefore, the combined layer thicknesses represent the anticipated embedded pile lengths.

SOIL PARAMETERS FOR L-PILE® INPUT ABUTMENT 1 (Boring BB-BB-102)				
Soil Model	**El. Range (ft-NAVD 88)**	**Total Unit Weight (pcf), LB / UB**	**φ' (deg) / Su (psf) LB / UB**	**k (pci), or E_{50} (unitless) LB / UB**
Reese Sand	Bottom of Cap to 120	120 / 135	31° / 36°	40 / 100
Stiff Clay	El. 120 to 113	115 / 120	2000 psf	E_{50} = 0.005 / E_{50} = 0.007
Reese Sand	El. 113 to 109	125 / 135	36° / 40°	100 / 200

SOIL PARAMETERS FOR L-PILE® INPUT ABUTMENT 2 (Boring BB-BB-103 / -104)					
Soil Model	**El. Range (ft-NAVD 88), L=15 ft**	**El. Range (ft-NAVD 88), L=18 ft**	**Total Unit Weight (pcf), LB / UB**	**φ' (deg) / Su (psf) LB / UB**	**k (pci), or E_{50} (unitless) LB / UB**
Reese Sand	Bottom of Cap to 122	Bottom of Cap to 118	120 / 130	32° / 36°	50 / 100
Reese Sand	El. 122 to 114	El. 118 to 110	120 / 125	31° / 35°	40 / 80
Reese Sand	El. 114 to 110	El. 110 to 107	125 / 135	36° / 40°	100 / 200

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection. These items are given in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

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

AASHTO Table 10.5.5.2.3-1 requires that at least one load test with signal matching be performed per substructure to use a resistance factor of 0.65. Therefore, it is recommended that

two PDA tests with Signal Matching be completed for Bridge No. 5069, including one pile at each abutment.

Consistent with MaineDOT practice, one restrike test is recommended at the first abutment constructed. Additional PDA testing may be recommended if unanticipated conditions are encountered during installation, including early pile take-up, pile driving out-of-plumb, or otherwise unexplained variations in hammer performance.

7.2 PILE OBSTRUCTIONS

Pre-drilling, pre-excavation or spudding may be necessary to bypass potential obstructions, such as the boulders and rock fill encountered in boring BB-BB-102 near Abutment 1.

7.3 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to range from 0 to 2 feet below existing grades, along the southern end of each foundation, and up to 10 to 12 feet below existing grades, in the vicinity of the current bridge alignment. It is our understanding that Route 139 will remain in service during construction of the new bridge. As a result, temporary lateral support systems such and cantilevered or tied back sheeting will be required to support the roadway during abutment construction. Sheet pile installation may require predrilling or spudding to penetrate the rock fill in the vicinity of Abutment 1. In areas where sufficient space is available, the excavation slopes may consist of sloped, open-cuts. In all cases, temporary excavations should comply with OSHA excavation safety requirements.

Groundwater measurements indicated that static groundwater levels corresponded to the approximate stream level at the time of drilling. These levels will vary depending on numerous factors, primarily river level, and may approach or exceed the bottom of excavation level during periods of high water. If the foundation work is planned to be completed during normal water level periods, we anticipate that the inflow of groundwater or surface water to excavations can be handled by open pumping from sumps installed at the bottom of excavations or by gravity flow through temporary drainage ditches.

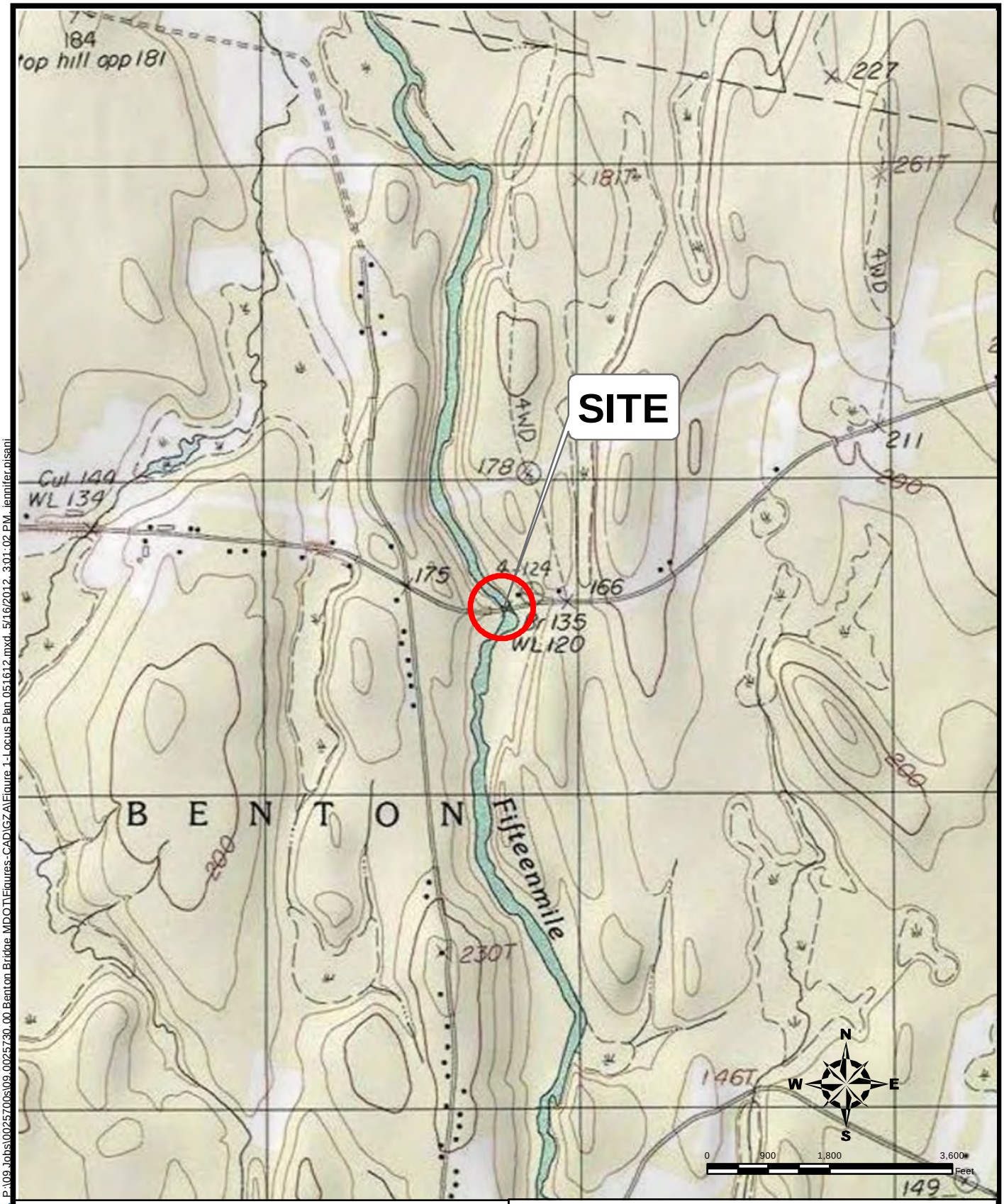
The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater and river water should comply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

7.4 REUSE OF ON-SITE MATERIALS

Based on the test boring results, three of the four fill samples tested had greater than 20 percent passing the No. 200 sieve, indicating the fill does not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and is unsuitable for use as structural backfill. The material is considered suitable for use as Common Borrow.

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

FIGURES



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 THE 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.

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS

BRIDGE #5069 OVER FIFTEEN MILE STREAM
MAINE DOT PIN 18231.00
BENTON, MAINE

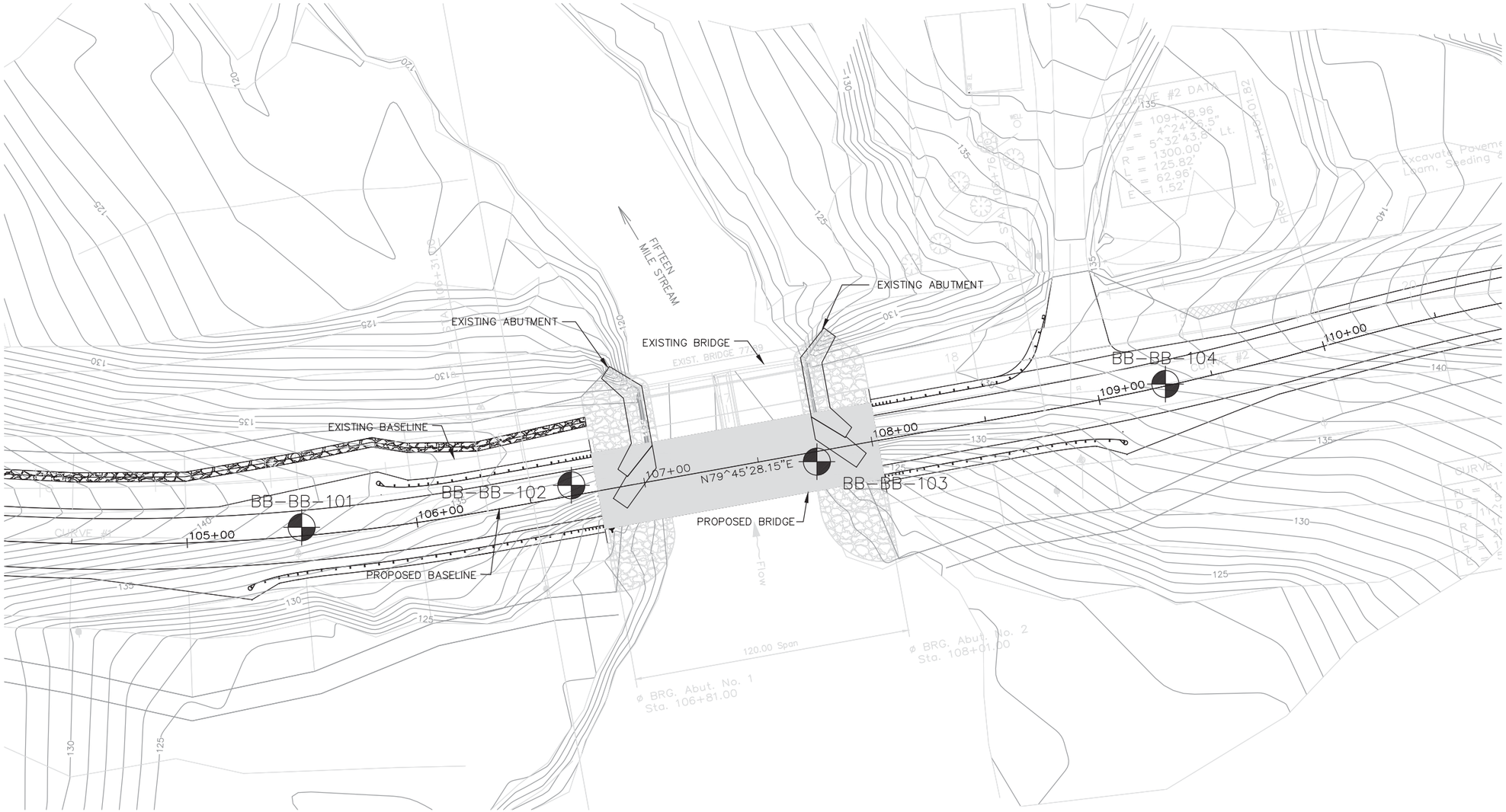
PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
ERDMAN ANTHONY

LOCUS PLAN

PROJ MGR: ARB	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE
DESIGNED BY: EJB	DRAWN BY: JAP	SCALE: 1 in = 2,000 ft	1
DATE: 05/16/2012	PROJECT NO. 09.0025730.00	REVISION NO.	

© 2012 - GZA GeoEnvironmental, Inc. P:\09 Jobs\00257200s\09.0025730.00 Benton Bridge MDOT\Figures-CAD\GZA\Figure 1-Locus Plan 051612.mxd, 5/16/2012, 3:01:02 PM, jennifer.nisani



NOTES:

1) BASE MAP DEVELOPED FROM ELECTRONIC FILES DATED NOVEMBER 14, 2013 PROVIDED BY ERDMAN ANTHONY (FILES INCLUDED Alignments.dgn, Bridge.dgn, Contours.dgn, Topo.dgn, z_Layout.dgn, and z_Alignments_Exist.dgn).

2) THE APPROXIMATE LOCATION OF THE GZA BORINGS WERE ESTIMATED USING TAPED MEASUREMENTS FROM SITE FEATURES AND SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHODS USED.

3) GZA BORINGS WERE PERFORMED BY MAINE TEST BORINGS OF HERMON, MAINE BETWEEN MAY 17, 2012 AND MAY 18, 2012 AND OBSERVED BY GZA PERSONNEL.

4) 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.

LEGEND

BB-BB-102

APPROXIMATE LOCATION AND DESIGNATION OF TEST BORING (DRILLED BY MAINE TEST BORING FOR GZA)

N

0 25 50 75
SCALE IN FEET

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(100)X

PIN
18231.00

BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

BRIDGE #5069 OVER
FIFTEEN MILE STREAM

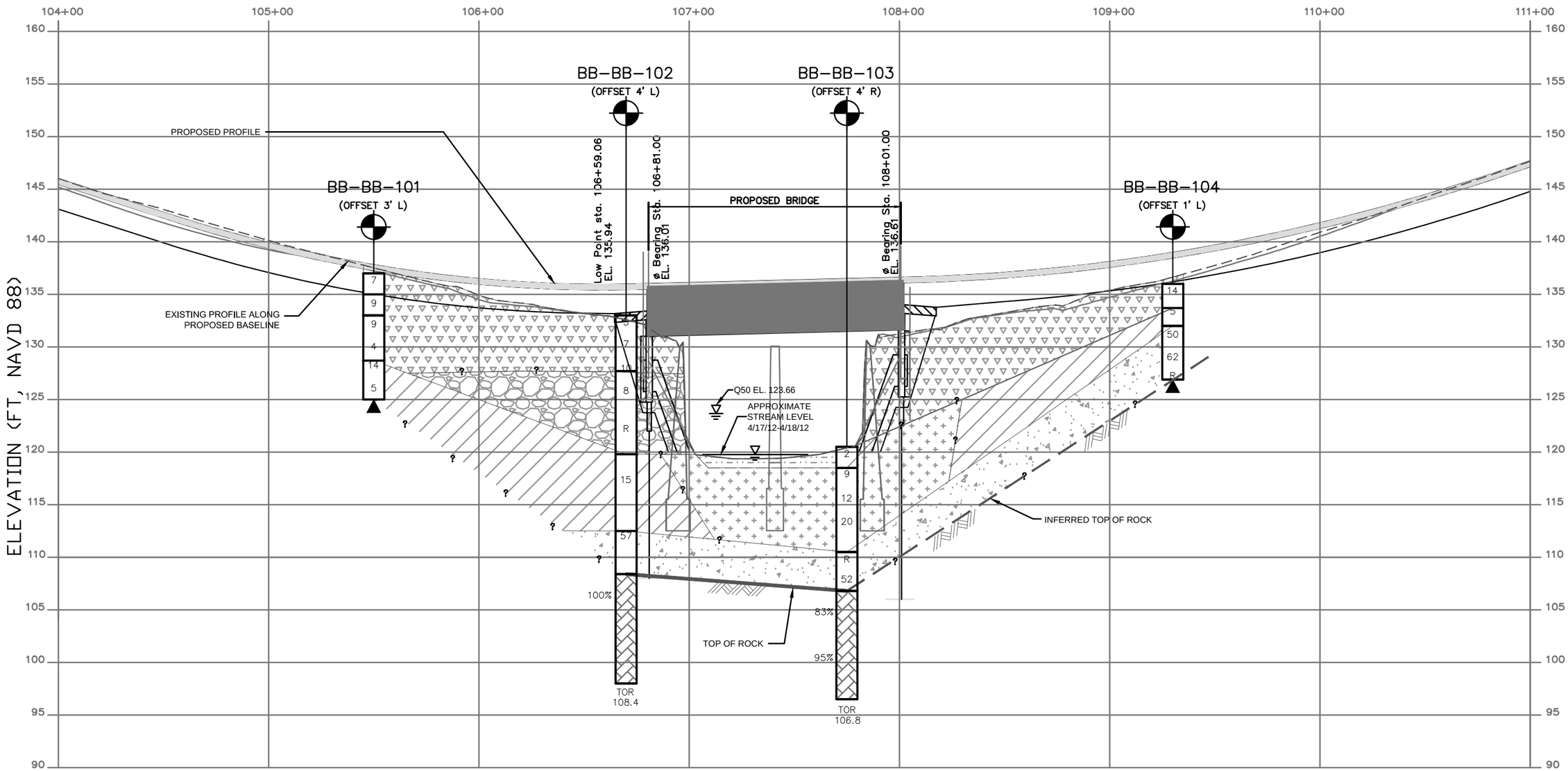
BENTON KENNEBEC COUNTY

BORING LOCATION PLAN

SHEET NUMBER

2

GZA-P:\09 Jobs\0025700s\09.0025730.00 Benton Bridge MDOT\Figures-CAD\GZA\25730 BLP - Profile 122313 ETL.dwg [LOC PLAN] January 02, 2014 - 2:17pm evan.lonstein



LEGEND:

BB-BB-101 — INDICATES BOREHOLE DESIGNATION

(OFFSET 0') — INDICATES LOCATION AND DIRECTION OFFSET FROM BASELINE OF PROPOSED BRIDGE

30 — INDICATES ENERGY-CORRECTED STANDARD PENETRATION TEST (SPT) VALUE, N60

R — INDICATES REFUSAL

30% — INDICATES RQD OF CORE RUN

ROCK CORE

DRILLING TERMINATED WITHOUT ROCK CORE

TOR 106.8 — TOP OF CORED BEDROCK (FT-NAVD 1988)

FILL

POSSIBLE ROCK FILL

RECENT ALLUVIUM

SAND AND GRAVEL

MARINE DEPOSIT

GLACIAL TILL

BEDROCK

NOTES:

1) BASE PROFILE DEVELOPED FROM ELECTRONIC FILE DATED NOVEMBER 14, 2013 PROVIDED BY ERDMAN ANTHONY ENTITLED z_Profile.DGN.

2) THIS GENERALIZED INTERPRETIVE SUBSURFACE 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 AND BEDROCK TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO THE EXPLORATION LOGS.

3) GROUND SURFACE ELEVATIONS OF AS-DRILLED BORINGS WERE INTERPOLATED BETWEEN CONTOURS ON SITE PLANS PROVIDED BY ERDMAN ANTHONY ON APRIL 9, 2012 ENTITLED "CONTOURS.DGN." THESE ELEVATIONS ARE APPROXIMATE AND SHOULD BE CONSIDERED ACCURATE TO THE DEGREE IMPLIED BY THE METHOD USED TO ESTIMATE THEM.

4) 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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(100)X

PIN 18231.00

BRIDGE PLANS

SIGNATURE

P.E. NUMBER

DATE

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

BENTON

BRIDGE #5069 OVER
FIFTEEN MILE STREAM
KENNEBEC COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

GZA-P:\09 Jobs\0025700s\09.00025730.00 Benton Bridge MDOT\Figures-CAD\GZA\25730 BLP - Profile 010214 ETL.dwg [PROFILE] January 15, 2014 - 4:11pm erlc.bacon

APPENDIX A

LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Further, reliance by any party not expressly identified in the agreement, 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 GZA's 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.

Subsurface Conditions

4. 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.

5. 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.

6. Water level readings have been made in test holes (as described in the Report) 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.

7. 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.

8. 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

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

Cost Estimates

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

Additional Services

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.

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APPENDIX B

TEST BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: 15 Mile Stream Bridge
Location: Benton, Maine

Boring No.: BB-BB-101
PIN: 18231.00

Driller:	Tom Schafer	**Elevation (ft.)**	137±	**Auger ID/OD:**	2 1/4
Operator:	Tom Schafer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	Mobile B48	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	04-18-12	**Drilling Method:**	SSA/HSA	**Core Barrel:**	-
Boring Location:	See Plan	**Casing ID/OD:**	-	**Water Level*:**	Dry
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	24/16	0.0 - 2.0	4-4-3-4	7	7	SSA		Loose, brown, fine to coarse SAND, little Gravel, trace Silt, frequent inclusions of gray SILT, dry. -FILL-	
								135.0	2.0	
	2D	24/12	2.0 - 4.0	4-4-5-8	9	9			Loose, brown, fine to coarse SAND, some Gravel, trace Silt, dry. -FILL-	G#1 A-1-b, SW-SM WC = 5.2%
								133.0	4.0	
	3D	24/14	4.0 - 6.0	6-5-4-2	9	9			Loose, brown-gray, fine to coarse SAND, some to little Gravel, some Silt, moist. (Possible reworked till.) -FILL-	G#2 A-1-b, SM WC = 13.7%
5										
	4D	24/7	6.0 - 8.0	2-2-2-3	4	4			Very loose, brown, fine to coarse SAND, little to some Silt, little Gravel, moist to wet. (Possible reworked till.)	
	5D	24/10	8.0 - 10.0	3-5-9-9	14	14	HSA	128.7	Top 4": Brown, fine to medium SAND, some Silt, trace Gravel, wet. Organics throughout.	
									8.3	
10	6D	24/16	10.0 - 12.0	4-2-3-2	5	5			Bottom: Very stiff, olive-gray, Clayey SILT, some fine to medium Sand, little Gravel, moist. -MARINE DEPOSIT-	WC = 29:0%
									Stiff, olive-gray, Clayey SILT, little fine to medium Sand, wet.	
								125.0	12.0	
									Bottom of Exploration at 12.00 feet below ground surface.	
15										
20										
25										

Remarks:

1. Boring terminated at 12.0 feet BGS, No Refusal.

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: 15 Mile Stream Bridge
Location: Benton, Maine

Boring No.: BB-BB-102
PIN: 18231.00

Driller:	Tom Schafer	**Elevation (ft.)**	133±	**Auger ID/OD:**	2 1/4
Operator:	Tom Schafer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	Mobile B-48	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	04-17-12	**Drilling Method:**	Driven & Spun Casing	**Core Barrel:**	-
Boring Location:	See Plan	**Casing ID/OD:**	4" / 3"	**Water Level*:**	14.2
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	24/6	0.0 - 2.0	1-3-2-1	5	5		132.8	Topsoil (0.2)	
									Loose, brown, fine to coarse SAND, some Silt, rootlets throughout, moist. -FILL-	
	2D	24/8	2.0 - 4.0	3-4-3-3	7	7	SSA		Loose, brown, fine to coarse SAND, some Gravel, some Silt, moist. -FILL-	G#4 A-1-b, SM WC = 14.7%
	3D	24/10	4.0 - 6.0	2-1-9-7	10	10			Loose, brown-olive, fine to coarse Silty SAND, some Gravel, moist. Possible Reworked Till. -FILL-	G#5 A-2, SM WC = 14.8%
5								127.7	(5.3) See Remark 1	
	4D	24/3	6.0 - 8.0	1-4-4-3	8	8			Loose, brown, fine to medium Sandy GRAVEL, little Silt, moist. -FILL-	
									Drilling efforts indicate persistent Cobbles/Boulders from 6.0' to 13.2'. (Possible Rock Fill.)	
10	5D	7/2	10.0 - 10.6	46-50/1"	-		RC		Gray GRAVEL, little Silt, trace fine to medium Sand, moist. -FILL- (Possible Rock Till)	
								119.8	(13.2) Drill action indicates change from cobbles/boulders to soil at 13.2 feet.	
15	6D	24/14	15.0 - 17.0	4-6-9-9	15	15	21		Very stiff, olive-gray to gray, Clayey SILT, some fine to medium Sand, trace Gravel, wet. -MARINE DEPOSIT-	WC = 18.8%
							26			
							34			
							37			
							42			
20	7D	19/5	20.5 - 22.1	27-25-32-50/1"	57	57		112.5	(20.5) Very dense, gray, GRAVEL, some fine to coarse Sand, little Silt, wet. -GLACIAL TILL-	
	R1	34/8	22.2 - 25.0	RQD = 0%			NQ2	111.0	See Remark 2. (22.0) R1 Top: Cobbles and Boulders, Gravel Pieces between Cobbles. See Remark 2.	
25	R2	60/60	25.0 - 30.0	RQD = 100%				108.4	(24.6) R1 Bottom 5 inches: Hard, fresh, fine grained, gray METASANDSTONE. Hard, fresh, fine-grained, gray, METASANDSTONE with	

Remarks:

1. Cobble at 5.4', likely giving higher blows from 5' - 6'.
2. Practical casing refusal at 22.1'. Removed casing, added spin shoe, and spun 3" casing to 25'.
3. R3 was not recovered on first attempt; barrel catch did not work, had to recore to retrieve Rock. Core was highly fractured upon retrieval. Original drill action was similar to R1, fracturing and lost recovery believed to have resulted from recoring; not representative of rock mass. Hole open to 34.7' after core retrieved.
4. Boring was terminated at 35' BGS in bedrock.

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: 15 Mile Stream Bridge
Location: Benton, Maine

Boring No.: BB-BB-102
PIN: 18231.00

Driller:	Tom Schafer	**Elevation (ft.)**	133±	**Auger ID/OD:**	2 1/4
Operator:	Tom Schafer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	Mobile B-48	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	04-17-12	**Drilling Method:**	Driven & Spun Casing	**Core Barrel:**	-
Boring Location:	See Plan	**Casing ID/OD:**	4" / 3"	**Water Level*:**	14.2
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.
										Calcite Stringers throughout. Joint is high angle, planar, rough, fresh, tight. Core Rates (minutes/foot): 2.0, 2.25, 2, 2, 2 Rock Mass Quality = Excellent	
30	R3	60/39	30.0 - 35.0							Hard, fresh, fine-grained gray METASANDSTONE with Calcite Stringers throughout. See Remark 3. Core Rates (minutes/foot): 2, 2.25, 1.75, 2, 2	
35								98.0		35.0 **Bottom of Exploration at 35.00 feet below ground surface.**	
40											
45											
50											

Remarks:

1. Cobble at 5.4', likely giving higher blows from 5' - 6'.
2. Practical casing refusal at 22.1'. Removed casing, added spin shoe, and spun 3" casing to 25'.
3. R3 was not recovered on first attempt; barrel catch did not work, had to recore to retrieve Rock. Core was highly fractured upon retrieval. Original drill action was similar to R1, fracturing and lost recovery believed to have resulted from recoring; not representative of rock mass. Hole open to 34.7' after core retrieved.
4. Boring was terminated at 35' BGS in bedrock.
5. Water level measured on 4/18/12, about 16 hours after completion of 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: 15 Mile Stream Bridge
Location: Benton, Maine

Boring No.: BB-BB-103
PIN: 18231.00

Driller:	Tom Schafer	**Elevation (ft.)**	120.5±	**Auger ID/OD:**	2 1/4
Operator:	Tom Schafer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	Mobile B-48	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	04-18-12	**Drilling Method:**	Driven Casing	**Core Barrel:**	NQ2
Boring Location:	See Plan	**Casing ID/OD:**	3"	**Water Level*:**	1.0
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	24/13	0.0 - 2.0	1-1-1-4	2	2	SSA		Brown/gray SILT, some fine to coarse Sand, with organics, moist. -RECENT ALLUVIUM-	
								118.5	2.0	
	2D	24/10	2.0 - 4.0	4-5-4-5	9	9			Loose, gray, fine to medium Sandy SILT, some Gravel, wet. -SAND and GRAVEL-	
	3D	24/4	4.0 - 6.0	9-7-5-6	12	12			Medium dense, gray, fine to coarse Gravelly SAND, little Silt, wet. -SAND and GRAVEL-	
5	4D	24/14	6.0 - 8.0	15-8-12-12	20	20			6-6.8': Medium dense, olive, fine to medium SAND, little Gravel, trace Silt, wet. 6.8-7.2': Medium dense, olive, fine SAND, trace Silt, wet. -SAND and GRAVEL-	
							36			
							34			
10	5D	11/9	10.0 - 10.9	22-75/5"			12	110.5	10.0 Brown-gray, fine to coarse SAND, some Silt, little Gravel, oxidation staining on matrix, wet. -GLACIAL TILL- Cobbles at 10.6 - 11.2'.	G#7 A-2, SM WC = 12.2%
	6D	24/16	11.5 - 13.5	18-22-30-27	52	52	28		Very dense, brown, fine to coarse SAND, some Gravel, trace to little Silt, wet. -GLACIAL TILL-	
							47			
								106.8	13.7	
	R1	60/57	14.0 - 19.0	RQD = 83%			NQ2		Hard, fresh, medium-grained, gray METASANDSTONE with Calcite Stringers throughout, Primary Joints are close to moderately spaced, low angle, undulating, rough, fresh, tight. Secondary Joints close to moderately spaced, high angle, planar, rough, fresh to discolored, tight. Core Rates (minutes/foot): 2.5, 2.5, 2.25, 2.5, 2.5 Rock Mass Quality = Good	
15										
	R2	60/59	19.0 - 24.0	RQD = 95%					Hard, fresh, fine-grained gray METASANDSTONE with Calcite Stringers throughout, Primary Joints are moderately spaced, low angle, planar, rough, iron stained, tight. Secondary Joint is near vertical, planar, rough, fresh, tight. Core Rates (minutes/foot): 1.5, 1.5, 2, 1.5, 2 Rock Mass Quality = Excellent	
20										
								96.5	24.0	
									Bottom of Exploration at 24.00 feet below ground surface.	
25										

Remarks:

1. Boring terminated at 24' BGS in Bedrock.
2. Water level measured about 1 hour after completion of drilling; corresponded roughly to stream water level at time of measurement.

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: 15 Mile Stream Bridge
Location: Benton, Maine

Boring No.: BB-BB-104
PIN: 18231.00

Driller:	Tom Schafer	**Elevation (ft.)**	136±	**Auger ID/OD:**	2 1/4
Operator:	Tom Schafer	**Datum:**	NAVD 88	**Sampler:**	Split Spoon
Logged By:	Eric Baron	**Rig Type:**	Mobile B-48	**Hammer Wt./Fall:**	140/30
Date Start/Finish:	04-18-12	**Drilling Method:**	HSA	**Core Barrel:**	-
Boring Location:	See Plan	**Casing ID/OD:**	-	**Water Level*:**	Dry
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	24/16	0.0 - 2.0	3-6-8-6	14	14	HSA		Top 6": Dark brown, fine to medium SAND, trace Silt. Bottom: Medium dense, brown, fine to coarse SAND, some Gravel, little Silt, dry. -FILL-	
	2D	24/15	2.0 - 4.0	6-3-2-5	5	5		133.7	Top 4" same as 1D — 2.3	G#8 A-6, CL LL=32, PI=13 WC=21.8%
	3D	24/19	4.0 - 6.0	9-23-27-24	50	50		132.0	Medium stiff to stiff, olive-gray, CLAY & SILT, some fine to medium Sand, trace Gravel, moist, oxidation staining throughout. -MARINE DEPOSIT- — 4.0	G#9 A-1-a, SM WC=9.1%
5	4D	24/20	6.0 - 8.0	22-32-30-28	62	62			Very dense, olive-brown, Gravelly, fine to coarse SAND, little Silt, moist. -GLACIAL TILL-	
	5D	13/8	8.0 - 9.1	18-23-50/1"	-				Very dense, olive-brown, Gravelly, fine to coarse SAND, little Silt, moist. -GLACIAL TILL-	
								126.9	Dense to very dense, brown, fine to medium SAND, little Gravel, trace Silt, moist. -GLACIAL TILL- — 9.1	
10									**Bottom of Exploration at 9.10 feet below ground surface.**	
15										
20										
25										

Remarks:

1. Boring terminated at 9.1' BGS on split spoon refusal.

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

LABORATORY TESTING RESULTS

THIELSCH ENGINEERING

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Benton Bridge

MDOT Project Number:

Town(s): Benton

GZA Project Number: 09.0025730.00

Boring & Sample Identification Number	Station (Feet)	Sample No.	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-BB-101		2D	2-4			5.2			SW-SM	A-1-b	0
BB-BB-101		3D	4-6			13.7			SM	A-1-b	II
BB-BB-101		6D	10-12			29.0					
BB-BB-102		2D	2-4			14.7			SM	A-1-b	II
BB-BB-102		3D	4-6			14.8			SM	A-2	II
BB-BB-102		6D	15-17			18.8					
BB-BB-103		5D	10-10.9			12.2			SM	A-2	II
BB-BB-104		2D	2.3-4			21.8	32	13	CL	A-6	IV
BB-BB-104		3D	4-6			9.1			SM	A-1-a	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.

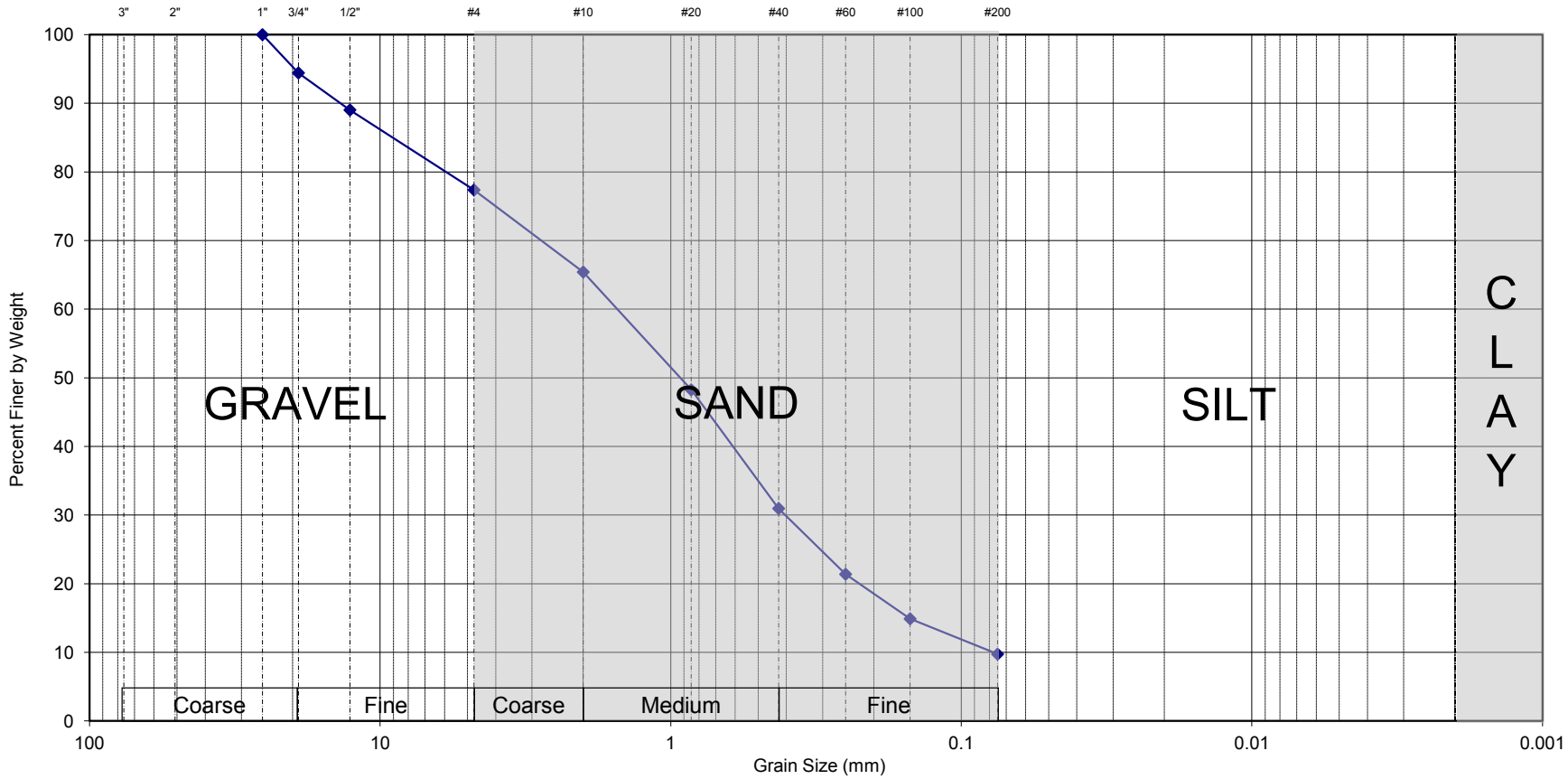
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
22.6%	67.6%	9.7%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
1	BB-BB-101	S-2D	2-4'	Gray-brown f-c SAND, some f-c Gravel, trace Silt	5.2			

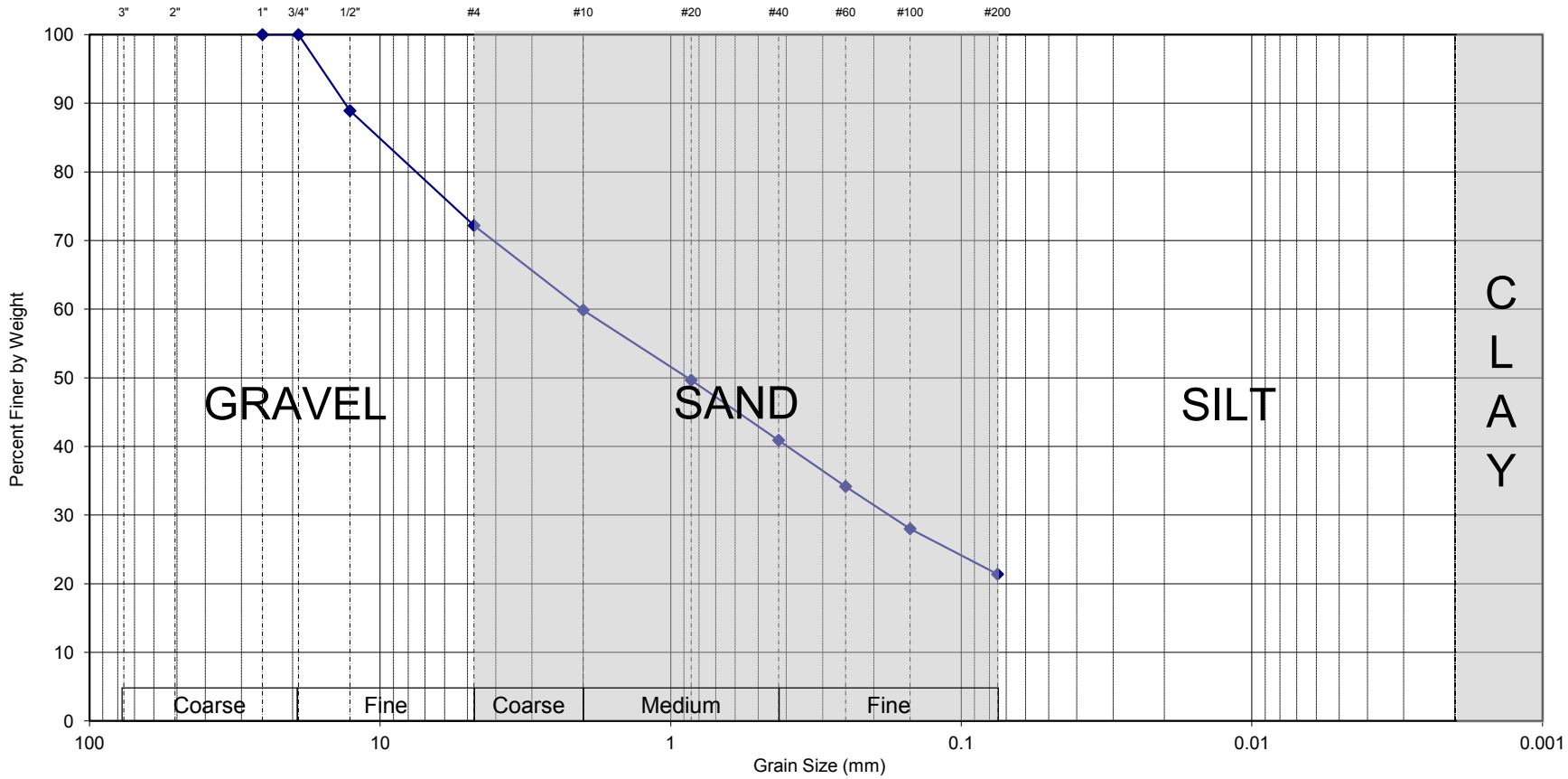
Sieve Size	% Passing
¾"	94.4
½"	89.0
#4	77.4
#10	65.4
#20	48.2
#40	31.0
#60	21.4
#100	14.9
#200	9.7

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
27.8%	50.8%	21.4%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
2	BB-BB-101	S-3D	4-6'	Brown f-c SAND, some fine Gravel, some Silt	13.7			

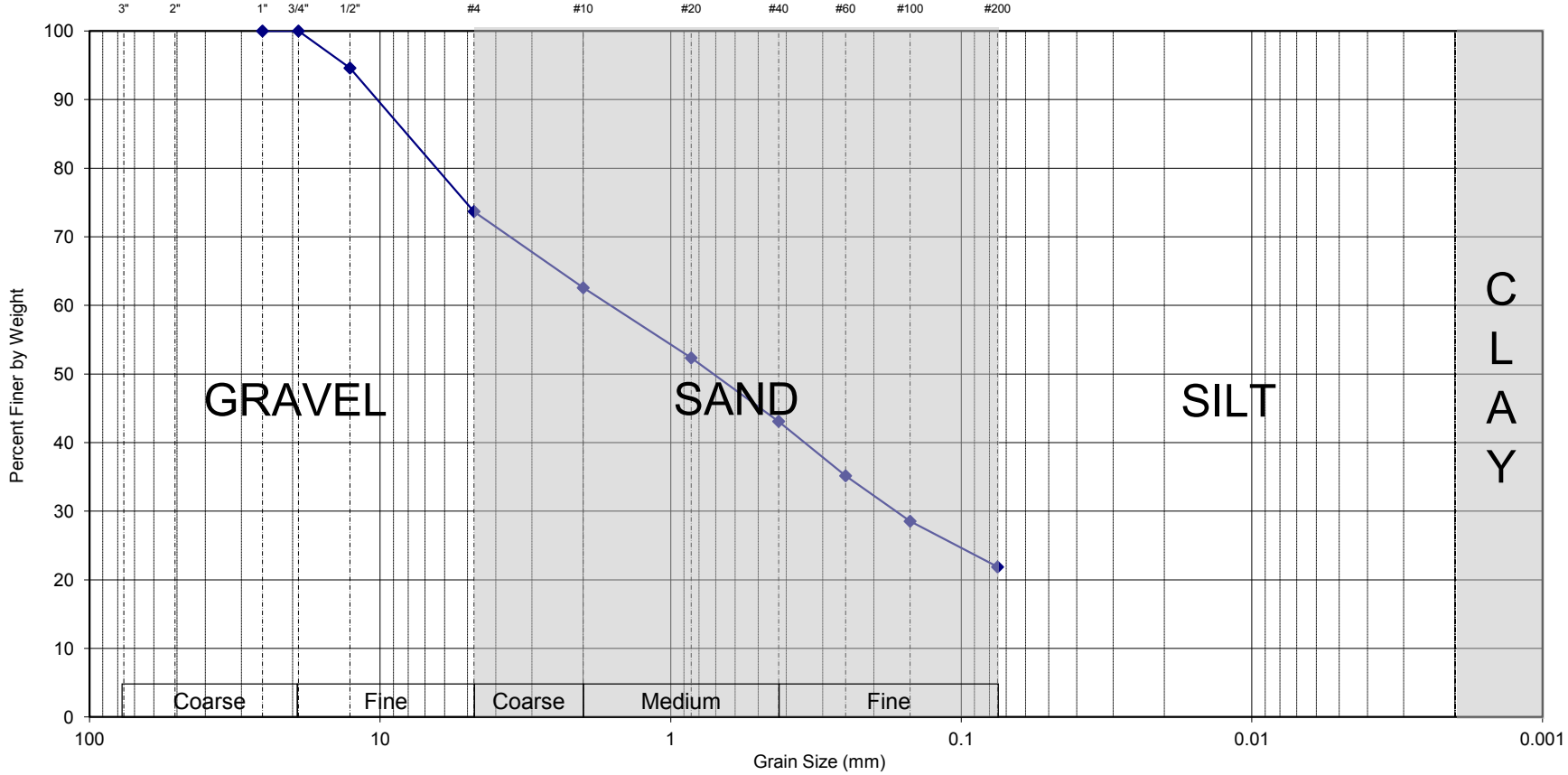
Sieve Size	% Passing
¾"	100.0
½"	88.9
#4	72.2
#10	59.9
#20	49.7
#40	40.9
#60	34.2
#100	28.0
#200	21.4

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
26.3%	51.8%	21.9%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
4	BB-BB-102	S-2D	2-4'	Brown f-c SAND, some fine Gravel, some Silt	14.7			

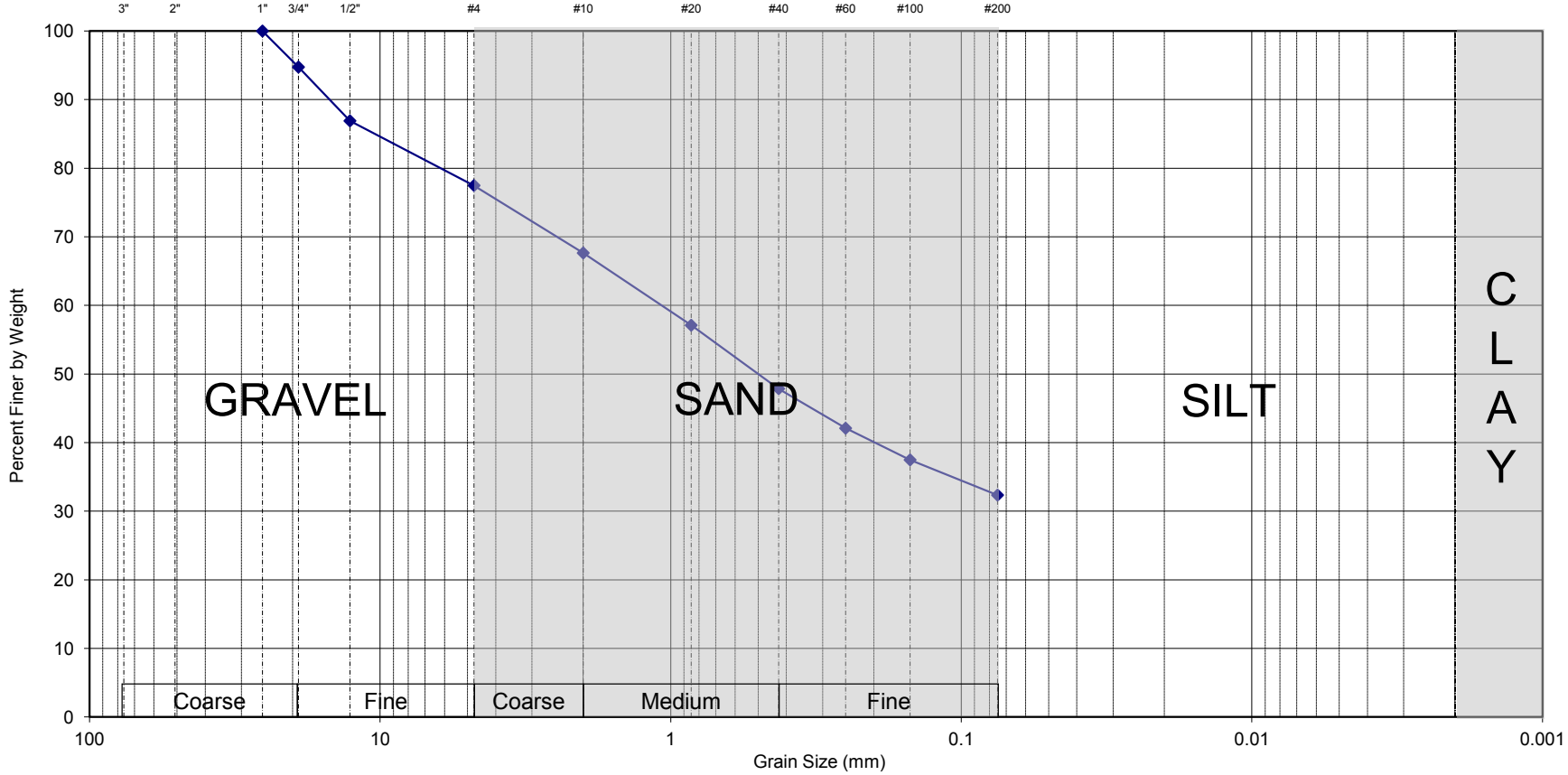
Sieve Size	% Passing
¾"	100.0
½"	94.6
#4	73.7
#10	62.6
#20	52.3
#40	43.1
#60	35.2
#100	28.5
#200	21.9

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
22.5%	45.1%	32.4%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
5	BB-BB-102	S-3D	4-6'	Brown f-c SAND, some Silt & Clay, some f-c Gravel	14.8			

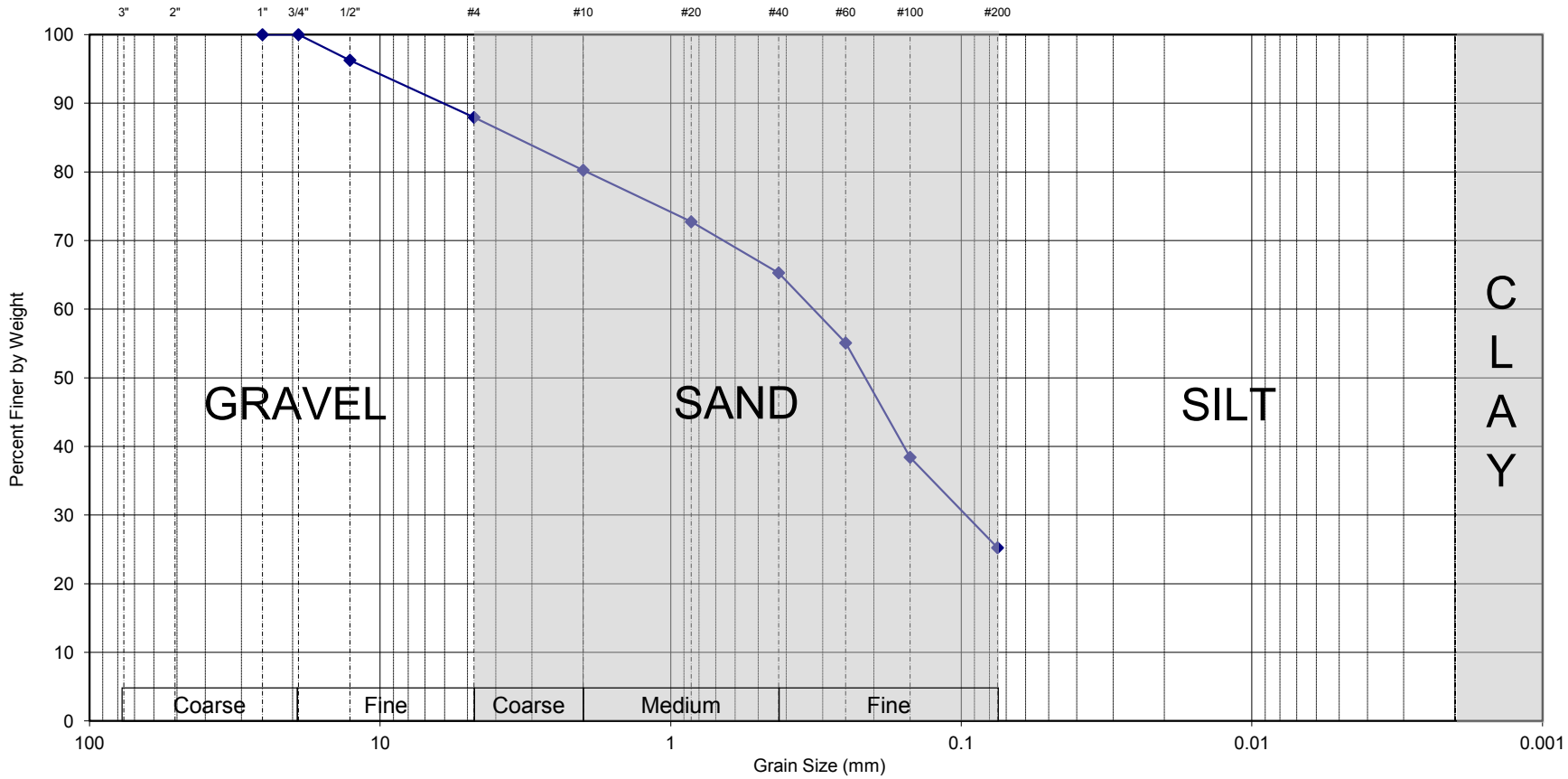
Sieve Size	% Passing
¾"	94.7
½"	86.9
#4	77.5
#10	67.6
#20	57.1
#40	47.9
#60	42.1
#100	37.5
#200	32.4

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
12.1%	62.7%	25.2%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
7	BB-BB-103	S-5D	10-10.9'	Gray-brown f-c SAND, some Silt, little fine Gravel	12.2			

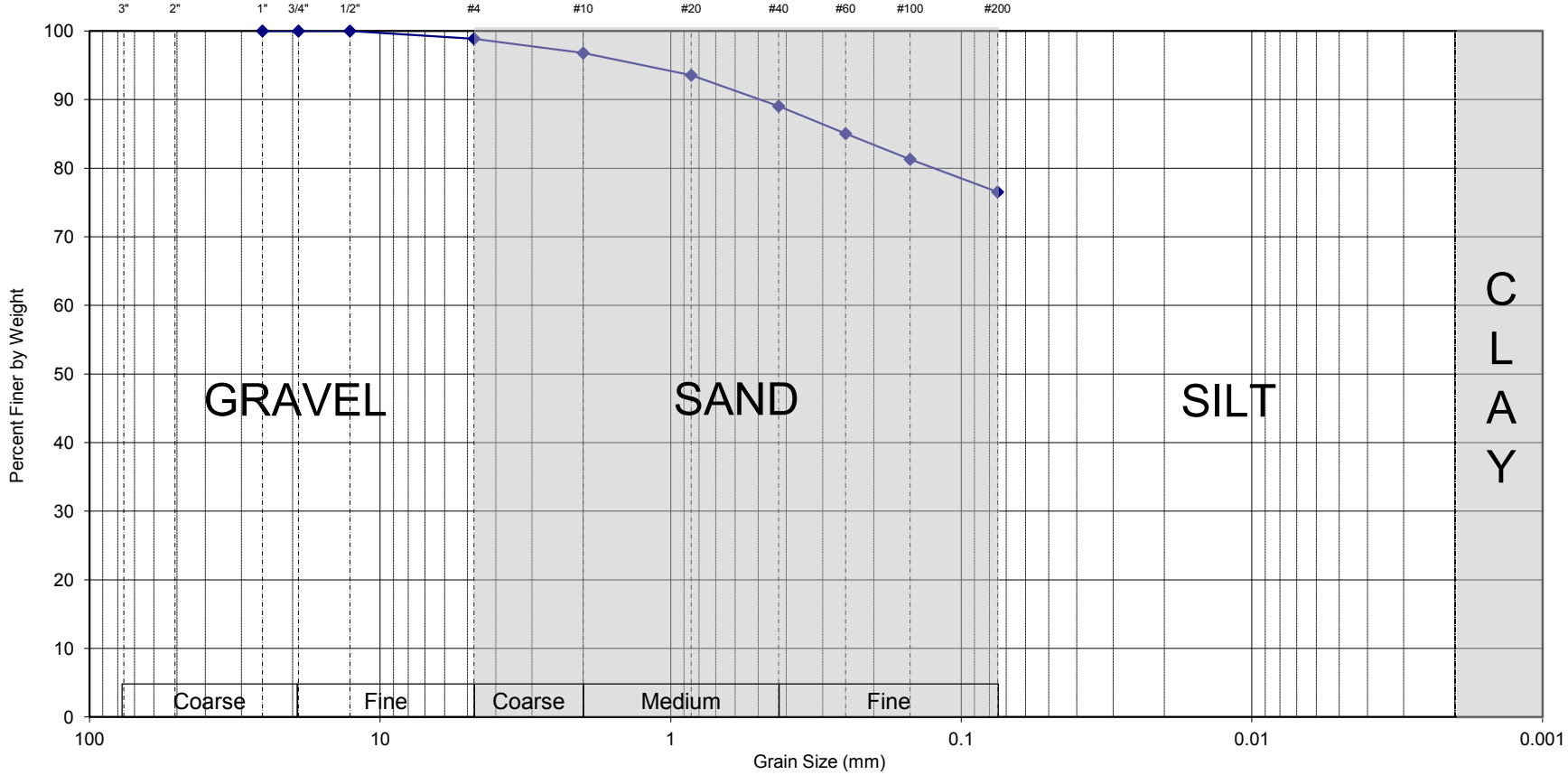
Sieve Size	% Passing
¾"	100.0
½"	96.3
#4	87.9
#10	80.2
#20	72.7
#40	65.3
#60	55.1
#100	38.4
#200	25.2

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
1.1%	22.3%	76.5%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
8	BB-BB-104	S-2D	2.3-4'	Gray & Brown CLAY & SILT, some f-m Sand, trace Gravel	21.8	32	19	13

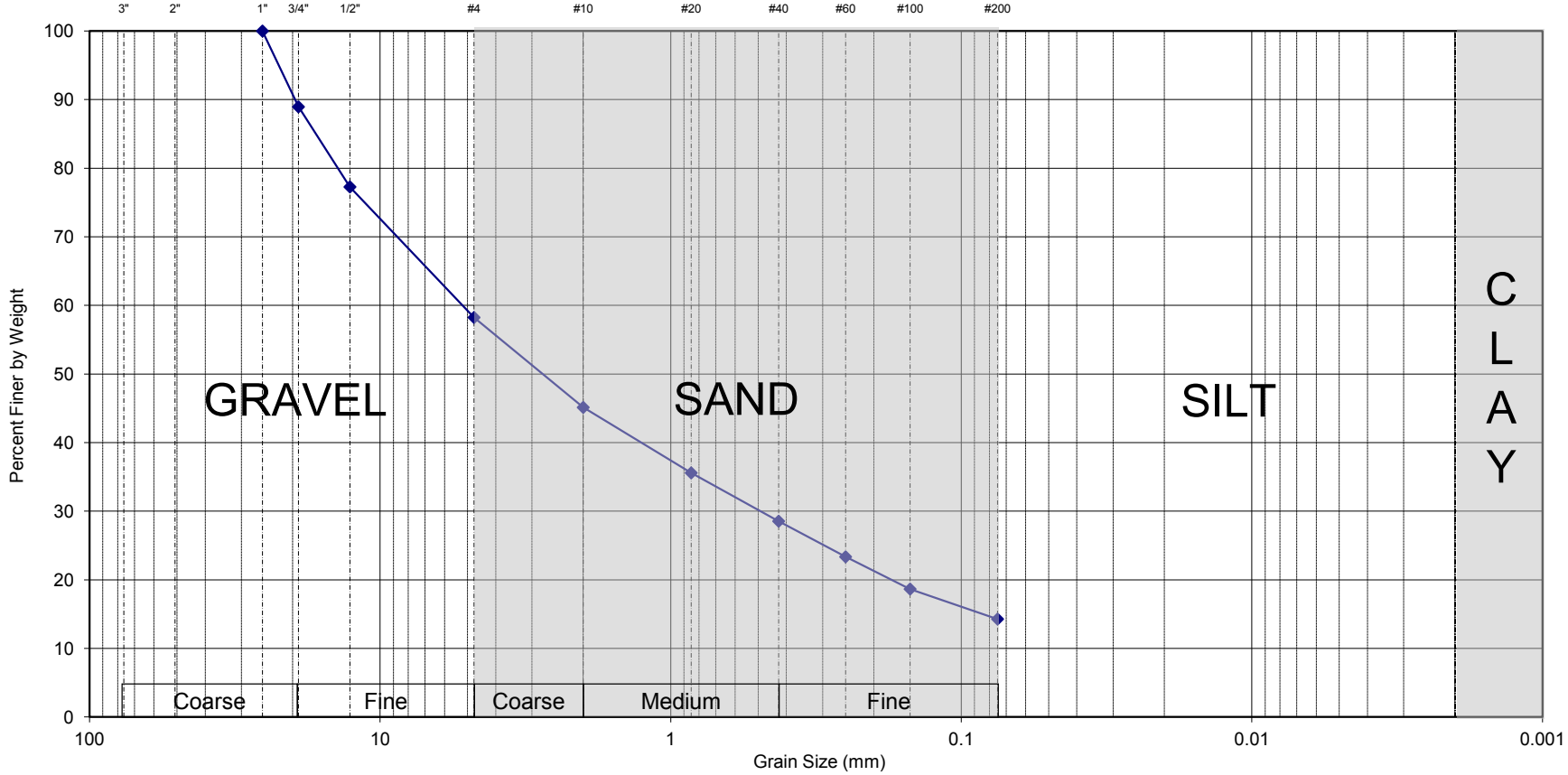
Sieve Size	% Passing
¾"	100.0
½"	100.0
#4	98.9
#10	96.8
#20	93.6
#40	89.0
#60	85.0
#100	81.3
#200	76.5

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

U.S. STANDARD SIEVE AND HYDROMETER



Gravel	Sand	Fines
41.8%	44.0%	14.3%

Lab #	Exploration	Sample	Depth	Description	WC	LL	PL	PI
9	BB-BB-104	S-3D	4-6'	Gray-brown f-c SAND and f-c GRAVEL, little Silt	9.1			

Sieve Size	% Passing
¾"	88.9
½"	77.3
#4	58.2
#10	45.1
#20	35.6
#40	28.5
#60	23.4
#100	18.7
#200	14.3

THIELSCH ENGINEERING

CTS-74-12-0003.07
Benton Bridge
Benton, ME
GZA File # 09.0025730.00

Tested by: AS/BB Date: 5/14/12
Reviewed by: MBP Date: 5/17/12

APPENDIX D

GEOTECHNICAL ENGINEERING CALCULATIONS

GZA
GeoEnvironmental, Inc.
Four Free Street
Portland, Maine 04101
207-879-9190
Fax: 207-879-0099
www.gza.com

Engineers and Scientists

JOB: 09.0025730.00
SUBJECT: Seismic Site Class Evaluations
SHEET: 1 OF 1
CALCULATED BY: ETL Date: 11/19/2013
CHECKED BY: ARB Date: 1/3/2014

BB-BB-102			
Elevation	Ni	di	di/Ni
136.0	20	5.0	0.3
130.8	7	2.0	0.3
128.8	10	2.0	0.2
126.8	8	4.0	0.5
122.8	50	4.5	0.1
117.8	15	5.0	0.3
112.3	57	4.5	0.1

*See Note 1

BB-BB-103			
Elevation	Ni	di	di/Ni
136.0	20	7.5	0.4
118.5	9	2.0	0.2
116.5	12	2.0	0.2
114.5	20	4.0	0.2
110.5	50	3.0	0.1
108.5	52	2.5	0.0

Sum=	27.0	1.7	*See Note 2	Sum=	21.0	1.1
	Σ(di)/Σ(di/Ni)	15.5			Σ(di)/Σ(di/Ni)	19.6

Navg	17.6

Navg>15 bpf, use Site Class D for design

Notes:

1. The upper layer consists of planned fill and is assumed to have an average N value of at least 20 blows per foot.
2. The total thickness included in the average includes existing overburden and planned granular fill. This calculation is conservative relative to Chapter 3 of AASHTO LRFD 6th Ed.

Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.60168°N, 69.46853°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 [1]	PGA = 0.073 g
From Figure 3.4.1-3 [2]	S_S = 0.155 g
From Figure 3.4.1-4 [3]	S_1 = 0.045 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1{,}200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.073 g, F_{PGA} = 1.600

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_s = 0.155 g, F_a = 1.600

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight–line interpolation for intermediate values of S_1

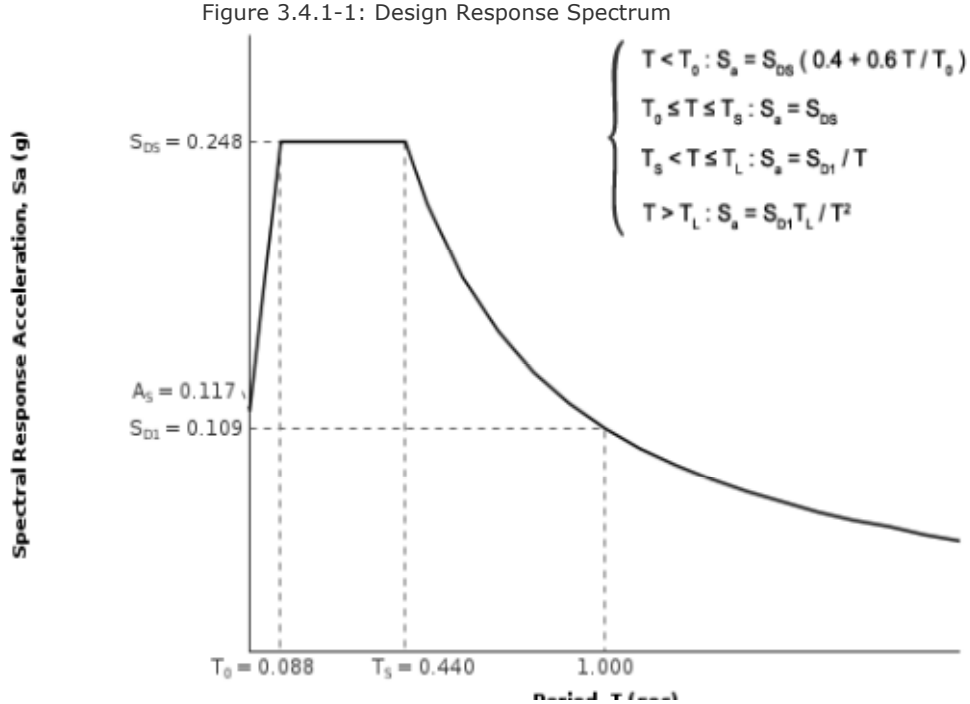
For Site Class = D and S_1 = 0.045 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}\ PGA = 1.600 \times 0.073 = 0.117$ g

Equation (3.4.1-2): $S_{DS} = F_a\ S_S = 1.600 \times 0.155 = 0.248$ g

Equation (3.4.1-3): $S_{D1} = F_v\ S_1 = 2.400 \times 0.045 = 0.109$ g

Figure 3.4.1-1: Design Response Spectrum



Period, T (sec)

Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For S_{D1} = 0.109 g, Seismic Design Category = A

Seismic Design Category ≡ "the design category in accordance with Table 3.5-1" = A

References

1. *Figure 3.4.1-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf

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Engineers and Scientists

JOB: 09.0025730.00
SUBJECT: Benton Bridge Axial Capacity
SHEET: 1 OF 6
CALCULATED BY E.Lonstein 12/20/2013
CHECKED BY A. Blaisdell 12/20/2013
REVIEWED BY C. Snow 12/20/2013

Objective

Evaluate the axial geotechnical resistance of the pile for the Bridge over Fifteen Mile Stream in Benton, ME

Methodology

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

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

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

Soil Properties

Consider Benton Bridge Interpretive Subsurface Profile (attached):
Abutment 1 pile soil profile (boring BB-102; top of pile 8' below existing ground): 5 feet of loose to dense sand (fill/rock fill) overlying 7.5 feet of stiff Clayey Silt (Marine Deposit) overlying 4 feet of very dense sand (Glacial Till), with bedrock at 16.5 feet below bottom of pile cap.
Abutment 2 pile soil profile (boring BB-103; top of pile 5' above existing ground): 3.5-7 feet of medium dense to dense sand (New FIll) overlying 8 feet of loose to medium dense sand (Sand & Gravel) overlying 3.5 feet of very dense sand (Glacial Till), with bedrock at 15-18.5 feet below bottom of pile cap.

Structural Properties

Try a HP14x89

Yield Strength of Steel $F_y := 50ksi$

Area of section $A_s := 26.1in^2$

Young's Modulus of Steel $E_s := 30000 \cdot ksi$

Section Properties $I_x := 904 \cdot in^4$

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.4.1-1 pg. 6-73

Determine normalized column slenderness factor λ

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi}\right)^2 \cdot \frac{F_y}{E}$$ AASHTO Eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the unbraced length l is 0 (fully supported)

Giving: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = 1305 \cdot kip$

2. Factored Structural Compressive Resistance - Strength Limit State:

Factor for piles in compression under hard driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance for Strength Limit State:

$P_r := \phi_c \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 783 \cdot kip$

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$P_r := \phi \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 1305 \cdot kip$

4. Geotechnical Axial Resistance - Static Analysis

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

Used GRLWEAP analytical axial capacity tool to estimate side friction resistance during driving, estimates 26 kips friction resistance. Also checked using Meyerhof SPT-method for Abutment 2 profile, estimated skin friction is 54 kips.

Required nominal resistance of 575 kips for design based on a maximum factored load of 374 kips and a 0.65 resistance factor.

Estimated friction distribution and % shaft resistance using GRLWEAP based on design loads.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8.1 Pg. 10-121

$f_y := 50ksi$ yield Strength of steel

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

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

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Engineers and Scientists

JOB: 09.0025730.00
SUBJECT: Benton Bridge Axial Capacity
SHEET: 4 OF 6
CALCULATED BY E.Lonstein 12/20/2013
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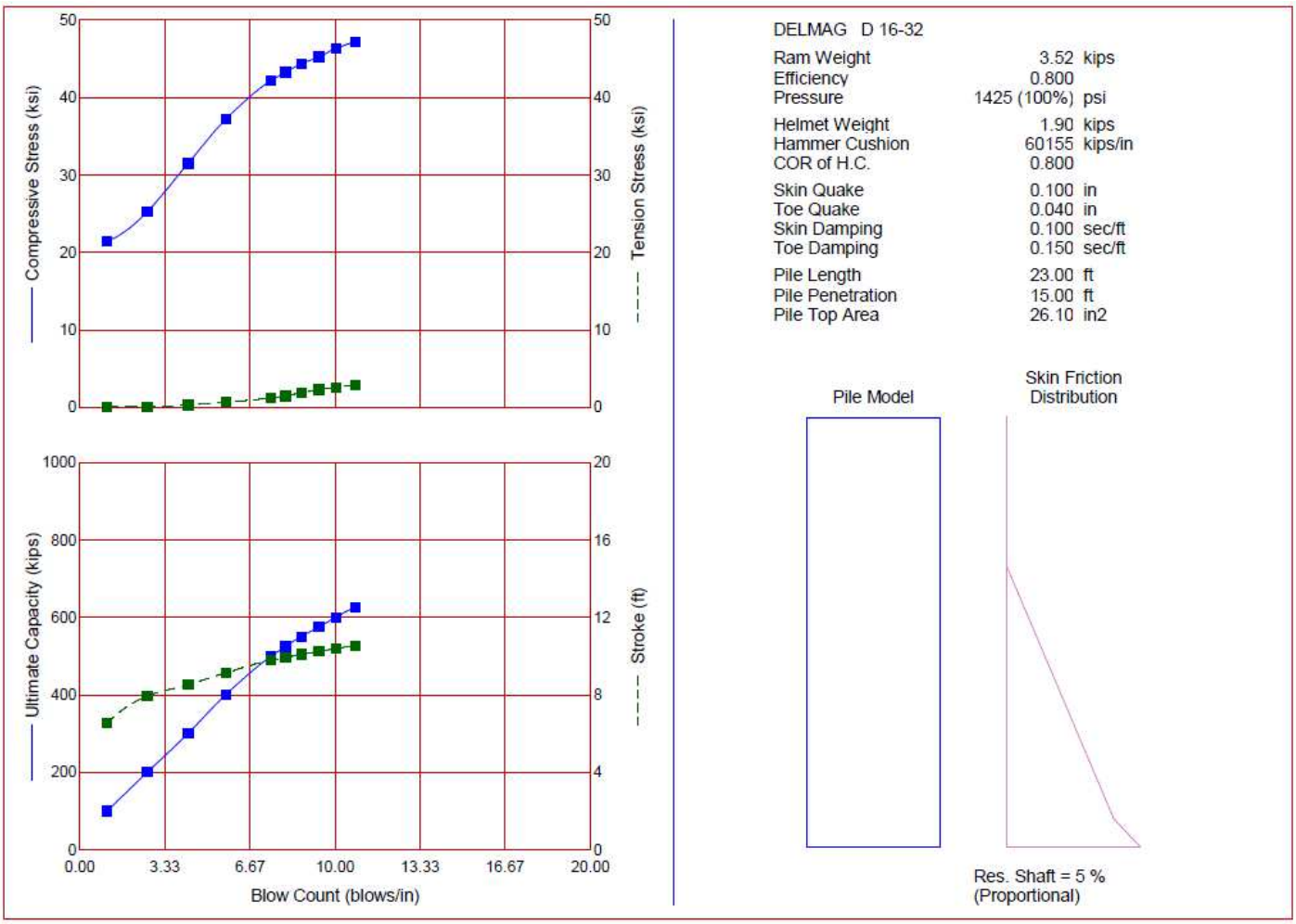
REVIEWED BY C. Snow 12/20/2013

Use GRLWEAP - Consider driving pile through 15 feet of medium dense sand and gravel and very dense glacial till to rock with toe quake representative of very hard driving conditions (0.04in). Pile length is modeled as 23 feet with 8 feet of stickup.

Estimated skin friction is 5% to 10% of required nominal resistance. For short pile; driving stress is anticipated to control resistance. Therefore, use lower friction contribution in GRLWEAP.



GZA Geo Environmental, Inc.
BENTON (Abutment 2)

20-Dec-2013
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
100.0	21.44	0.07	1.1	6.54	16.09
200.0	25.23	0.07	2.6	7.94	14.47
300.0	31.45	0.29	4.2	8.52	14.11
400.0	37.16	0.67	5.7	9.14	14.60
500.0	42.19	1.21	7.5	9.80	15.29
525.0	43.23	1.48	8.1	9.94	15.43
550.0	44.32	1.84	8.7	10.09	15.68
575.0	45.23	2.26	9.4	10.24	15.95
600.0	46.29	2.57	10.0	10.39	16.23
625.0	47.16	2.81	10.8	10.53	16.50

Allowable stress in piles during driving is 0.9 fy=45 ksi, Rndr = 575 kip.

$R_{ndr} := 575kip$

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

$R_{ndr_factored} := R_{ndr} \cdot \phi_{dyn}$

$\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1 PDA, WEAP and CAPWAP used to establishing driving criteria

$R_{ndr_factored} := R_{ndr} \cdot \phi_{dyn}$

$R_{ndr_factored} = 374 \cdot kip$

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

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi_{serv_ext} := 1$

$R_{ndr_serv_ext} := R_{ndr} \cdot \phi_{serv_ext}$

$R_{ndr_serv_ext} = 575 \cdot kip$

Summary of Results - Axial Loading:

			Abutments 1 and 2	
ASTM A570, HP 14x73	Structural Resistance (kips)	Geotechnical Static Resistance (kips)	Drivability Resistance (kips)	Governing Resistance (kips)
Strength Limit State	783	n/a	374	374
Service/Extreme Limit State	1305	n/a	575	575

Results indicate drivability controls, Delmag D16-32 is suitable to penetrate overburden on the maximum fuel setting and achieve strength limit state resistance of 374 kips with a final blow count of 9 to 10 bpi.