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GEOTECHNICAL DESIGN REPORT
HAMLIN BRIDGE NO. 3286
ROUTE 133 OVER WILSON STREAM
FARMINGTON, MAINE

Prepared for:
McFarland Johnson, Inc.
Freeport, Maine

October 2020
09.0026052.00

Prepared by:
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VIA EMAIL

October 6, 2020
File No. 09.0026052.00

Ms. Theresa McAuliffe, P.E.
McFarland Johnson, Inc.
5 Depot Street, Suite 25
Freeport, Maine 04032

Re: Geotechnical Design Report
Hamlin Bridge No. 3286, Route 133 over Wilson Stream
Maine Department of Transportation WIN 22236.00
Farmington, Maine

Dear Theresa:

We are pleased to provide this Geotechnical Design Report (GDR) to McFarland Johnson, Inc. (MJ) for the subject project. Our work was completed in accordance with the Agreement between MJ and GZA GeoEnvironmental, Inc. (GZA) dated December 16, 2019, which incorporates proposal No. 09.P000077.20, dated October 24, 2019, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to MJ, who is under contract with the Maine Department of Transportation (MaineDOT) for final design of the proposed bridge.

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

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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Attachment: Geotechnical Design Report

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1.0 INTRODUCTION

This report presents the results of the geotechnical evaluation by GZA GeoEnvironmental, Inc. (GZA) for the subject project. Our work was completed in accordance with the Agreement between McFarland Johnson, Inc. (MJ) and GZA dated December 16, 2019, which incorporates proposal No. 09.P000077.20, dated October 24, 2019, and the Limitations included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to MJ, who is under contract with the Maine Department of Transportation (MaineDOT) for final design of the proposed bridge.

1.1 BACKGROUND

The project includes replacement of the Hamlin Bridge No. 3286 Carrying Route 133 over Wilson Stream in Farmington, Maine, the location of which is shown on **Figure 1**. Built in 1938, the existing bridge is comprised of a non-composite cast-in-place deck superstructure with painted steel beams. The bridge spans 60 feet along the skew with an overall length of about 66 feet and is supported on concrete stub abutments bearing on soil. Repairs were made to the railings, curb, and substructure in 1990 and the upstream wingwalls were rehabilitated in 2017.

Based on our review of the information provided by MJ, plans are to construct a replacement bridge on the existing alignment consisting of a 115-foot-long, single span, 25-degree skew, integral abutment bridge with five metalized steel plate girder beams and approximately 11.3-foot-high backwalls and in-line wingwalls. The bridge will be supported on driven piles. 1.75 horizontal to 1 vertical (1.75H:1V) riprapped slopes are proposed in front of the abutments and will include 4-foot-wide wildlife benches at El. 361 on both sides of the stream. Elevations used herein are in feet and are referenced to North American Vertical Datum of 1988 (NAVD88) datum.

Each abutment will be supported on five piles. Based on information provided by MJ, the bridge designer, we understand the maximum factored axial pile load is 345 kips for the strength loading condition and a 5-pile configuration. The estimated thermal deformation of the bridge superstructure is 0.663 inches for the Strength I condition.

An approximate 3.0- to 3.5-foot raise in grade is proposed at the existing abutments to facilitate estimated river flood levels and required freeboard. The horizontal alignment of the bridge is being shifted 2 feet upstream of the existing alignment. A new driveway will be constructed to the east of the south approach with new fills expected to be up to 6 feet locally.

Route 133 will be closed at the bridge crossing and detoured during construction of the new bridge.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data provided by MaineDOT;
- Conducted geotechnical engineering analyses for soil and bedrock properties; stability and settlement of approach embankments; frost susceptibility and drainage of approach embankments; AASHTO LRFD load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; pile downdrag loads; pile drivability; lateral pile evaluations; and seismic design considerations;
- Developed geotechnical engineering recommendations including foundation design recommendations for driven piles, lateral earth pressures and seismic design parameters; and
- Prepared this report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Prior to GZA's engagement in the project, an exploration program was completed by MaineDOT in 2018. Details of this program are described below. GZA did not perform additional explorations for this evaluation.

2.1 TEST BORINGS

MaineDOT drilled two test borings, BB-FWS-101 and BB-FWS-102, between July 10 and July 12, 2018. A. Van Buskirk of MaineDOT logged the borings. The as-drilled boring locations and elevations were surveyed and provided by MaineDOT (in station/offset format for the locations) and are included on the logs in **Appendix B**.

BB-FWS-101 and BB-FWS-102 were located behind the existing south and north abutments, respectively, as shown on the Boring Location Plan (prepared by MaineDOT), **Figure 2**. The test borings were drilled through the overburden soil and terminated approximately 10 to 12 feet into bedrock. Depths of borings ranged from approximately 46.0 to 54.8 feet below ground surface (bgs). The borings were drilled using 3-and 4-inch casing and drive-and-wash techniques. Standard penetration testing (SPT) and split-spoon sampling were performed at 5-foot intervals in the borings. Bedrock cores were obtained using NQ2 wire line coring equipment. The hammer efficiency at the time of the borings was 0.928 as provided on the boring logs.

Drafts of the logs for BB-FWS-101 and -102 were prepared by MaineDOT and provided to GZA. GZA subsequently reviewed the logs and made edits to reflect laboratory soil test results and our analysis of stratification. The final logs, including GZA's edits, are provided in **Appendix B**.

3.0 LABORATORY TESTING

Laboratory testing was conducted by MaineDOT on split-spoon soil samples retrieved during the 2018 investigation. The testing program consisted of gradation analysis / AASHTO Classification / Frost

Classification assessments of eight samples, hydrometer testing of five samples, water content of 13 soil samples and Atterberg Limits of five soil samples. Results of the testing are included in **Appendix C**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping[1], the surficial unit in the vicinity of the bridges consists of the Presumpscot Formation (Glaciomarine Bottom Deposit), described as silt and clay, with local sandy beds. Glacial till was also mapped in the area of the bridge consisting of a mixture of sand, silt, and clay.

Bedrock in the vicinity of the site is mapped[2] as the Sangerville Formation, and is described as thickly bedded, fine- to coarse-grained, light gray, calcareous, graywacke metasandstone, with thinly interlaminated metasiltstone or metapelite.

4.2 SUBSURFACE PROFILE

Two soil units were encountered in the test borings overlying bedrock: Fill and Glacial Till. Approximately 6 to 8 inches of asphalt pavement was encountered in the borings. The thicknesses and generalized descriptions of the soil units are presented in the following table, in descending order from existing ground surface. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs in **Appendix B**. An interpretive subsurface profile was developed representing generalized stratification along the proposed bridge alignment and is presented on **Figure 3**. The stratum thicknesses and elevations of the borings are summarized in **Table 1**.

GENERALIZED SUBSURFACE CONDITIONS		
Subsurface Unit	**Approximate Encountered Thickness (ft)**	**Generalized Description**
Fill	12 to 15	Varying From: Brown, very loose to dense, Gravelly SAND, little Silt, To: Sandy GRAVEL, little Silt. (USCS: SW-SM, SM, GM). MaineDOT Frost Classification: 0-III *Encountered in all borings.*
Glacial Till	19 to 31	Varying From: Gray, stiff to hard, SILT, little to some sand, trace to little clay, trace to little Gravel; To: Silty GRAVEL, some Sand. (USCS: ML, GM) MaineDOT Frost Classification: IV *Encountered in all borings.*
Top of Bedrock Elevation	Encountered Top of Rock:	Abutment 1: Approximate El. 335.4 Abutment 2: Approximate El. 324.8

[1] Weddle, Thomas K., 2003. Surficial geology of the Farmington quadrangle, Maine: Maine Geological Survey, Open-File Map 03-51, map, scale 1:24,000. *Maine Geological Survey Maps*. 1072. http://digitalmaine.com/mgs_maps/1072

[2] Pankiwsky, Kost A., 1978, Reconnaissance bedrock geology of the Farmington [15-minute] quadrangle, Maine: Maine Geological Survey, Open-File Map 78-16, map, scale 1:62,500. *Maine Geological Survey Maps*. 300. http://digitalmaine.com/mgs_maps/300

4.2.1 Bedrock

Bedrock was cored in both borings and was described as hard, fresh, fine grained, gray, Metasandstone. Two joint sets were identified on the boring logs as horizontal to moderately dipping and steep angle. In general, the joints are described as close to moderately spaced, and tight to open with minor silty sand infilling and intermittent zones of highly fractured rock. The Rock Quality Designation (RQD) in the core runs ranged from 0 to 81 percent.

4.2.2 Groundwater

Groundwater was measured by MaineDOT in the boreholes. The times, borehole depths, and casing conditions at the time of the measurements were not provided to GZA. The measured groundwater level in BB-FWS-101 was 7.8 feet bgs, corresponding to approximately El. 362.2. The water level was not observed in BB-FWS-102.

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.

GZA did not observe the test borings during drilling or the recovered soil or rock samples during our work. We relied on descriptions made by MaineDOT combined with laboratory test results for our understanding of the soil materials.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with *2017 AASHTO LRFD Bridge Design Specifications, 8th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide* (BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations are included in **Appendix D**.

5.2 APPROACH EMBANKMENTS

The roadway profile will be raised approximately 3.0 to 3.5 feet above existing grades in the vicinity of the bridge. The roadway will also be widened, but the top of the existing embankment is relatively level several feet past the outside edges of the existing pavement, so the maximum permanent new fill thickness is on the order of 4 feet over the embankment footprint.

Approach embankment side slopes will be constructed with an inclination of 2H:1V or less.

The ground surface in front of each abutment will slope down to the river level at an inclination of 1.75H:1V and will be protected by riprap, except for a 4-foot-wide wildlife path at El. 361.0. Based on the proposed bridge length, this will require demolition of the existing abutments and removal of soil and/or rockfill extending approximately 15 feet into each existing riverbank, resulting in 15-foot-tall, 1.75H:1V riprap slopes in front of each abutment with the wildlife shelf.

We anticipate that the embankments will be constructed over primarily stiff to hard silt or dense gravel Glacial Till supporting soil. Due to the typical strength and low compressibility, embankment settlement and global stability are not considered a concern for this project.

The added fill has the potential to increase stresses and induce pile downdrag forces. However, due to the density of the glacial till layer and the anticipated unload due to removal of the existing abutments and backfill, we anticipate less than 0.5 inches of embankment settlement adjacent to the piles. Therefore, downdrag forces can be neglected.

5.3 SEISMIC DESIGN CONSIDERATIONS

The subsurface profile for seismic design includes the approach fills (including backfill behind abutments), and Glacial Till overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1, considering the average SPT N-values in glacial till which encompass most of the soil encountered in the borings. The average SPT N-value is generally between 15 and 50 blows per foot in the abutment borings, which is the range defining Site Class D. Therefore, the bridge is assigned to Site Class D.

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

5.4 EVALUATION OF FOUNDATIONS

5.4.1 Foundation Type Assessment

During preliminary design MaineDOT conducted preliminary evaluations for pile foundations. Based on the results during preliminary design and the Interpretive Subsurface Profile we anticipate that ASTM A572 Grade 50 steel HP14x89 piles may be driven to refusal on or near bedrock to support integral abutments at this site. We anticipate that the expected pile embedment lengths of 25 to 35 feet will be sufficient for this pile to resist lateral loads due to thermal deformations. Design considerations and evaluations are presented below.

5.4.2 Pile Design Considerations

Considering the relatively thin overburden at the abutments, we anticipate that piles will be driven to or near the bedrock surface. The skin friction resistance was evaluated for use as an input in drivability analyses using the computer analytical software *APile* by Ensoft. The results of our evaluations indicate the piles will gain support primarily through end bearing in glacial till or on bedrock. The estimated side resistance was estimated to be approximately 15 percent of the nominal resistance and was used as an input in the pile drivability analyses.

Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. Axial tensile geotechnical (uplift) resistance was not evaluated because the structural loads provided by MJ do not include uplift loading on the piles.

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

5.4.3 Load and Resistance Factors

In GZA's experience, for piles gaining a significant portion of their geotechnical resistance in very dense soil or bedrock the drivability resistance will control the geotechnical static resistance of the pile. The piles will be driven to a nominal resistance calculated by dividing the maximum factored pile load (Strength I load case) divided by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1. Resistance factors for service and extreme limit state design should be taken as 1.0.

Structural resistance of the piles should be checked at the strength limit state considering a resistance factor ϕ_c=0.50, per AASHTO LRFD Article 10.7.3.2.3 for hard driving condition. Since the piles will be subject to lateral loading, the piles should also be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.70 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).

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, per MaineDOT BDG Section 5.4.2.11.

5.4.4 Pile Type

The abutments are planned to be supported on ASTM A572, Grade 50 (fy=50 kips per square inch [ksi]) steel HP14x89 piles. Each abutment will include five piles.

5.4.5 Pile Loads

MJ provided a maximum factored axial load of 345 kips per pile for the strength condition, therefore, piles should be installed to a nominal axial resistance of at least 531 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65 for piles installed under hard driving conditions. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted during construction in accordance with AASHTO requirements to assess nominal geotechnical pile resistance.

5.4.6 Design-Phase Pile Drivability Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 531 kips at the abutments. Analyses were completed using a Delmag D25-32 diesel hammer with a ram weight of 5,510 pounds and a maximum rated energy of 66,340 foot-pounds (ft-lbs). A 25-foot-long pile was assumed to encounter very hard driving conditions (toe quake of 0.04) on bedrock. The results are summarized below.

SUMMARY OF WEAP ANALYSES					
Pile Analysis and Type	**Embedded Pile Length**	**Driving System[1]**	**Required Nominal Geotechnical Resistance (kips)**	**Max Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
Abutment 1 HP 14x89	25 feet	Delmag D 25-32 (Fuel setting 4, 72% of maximum pressure)	531	40	8

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

5.4.7 Lateral Pile Analysis

GZA developed a soil profile for lateral pile evaluations. The subsurface stratum encountered near the top of the piles includes primarily Glacial Till, but the uppermost portion of the piles may be within the bottom of the existing embankment fill. The following soil profiles were developed for lateral pile evaluations using LPile by Ensoft.

Abutment 1					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci)**	**ϕ' (deg)**	**γ_e (pcf)**
Existing Fill	Reese Sand	Top of Pile	65	33	63
Glacial Till	Reese Sand	354.8	145	40	68
Top of Rock	--	335.4	--	--	--

Abutment 2					
Stratum	**Soil Model**	**Top of Layer Elevation (NAVD88 ft)**	**k (pci)**	**ϕ' (deg)**	**γ_e (pcf)**
Existing Fill	Reese Sand	Top of Pile	65	33	63
Glacial Till	Reese Sand	355.7	145	40	68
Top of Rock	--	324.8	--	--	--

Notes:
1. Soil strata were modelled after BB-FWS-101 & 102.
2. Recommended modulus and unit weight values assume groundwater level at El. 360.
3. pci = pounds per cubic inch, deg = degrees, psf = pounds per square foot,
   a. γ_T = total unit weight (used above anticipated groundwater level),
   b. γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per square foot.
4. These parameters do not include reductions for group interaction. Reduction Factors should be applied in accordance with AASHTO 10.7.2.4 for spacing of 3 to 5 pile diameters.

We understand that MJ is completing lateral pile analyses to evaluate pile fixity and combined stresses in conjunction with the structural design. GZA reviewed the results of MJ's lateral pile analyses to check that the input parameters were applied in accordance with our recommendations.

5.4.8 Lateral Earth Pressures

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move toward 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.

Based on the estimated thermal bridge expansion of 0.663 inches and the abutment height of approximately 11.3 feet, the calculated abutment rotation is 0.0049 feet/foot. In accordance with the requirements of the BDG Section 5.4.2.11, integral abutment reinforcement is to be designed for full Coloumb passive pressure if the wall rotation is greater than 0.005 feet/foot. Therefore, we conclude that a lateral earth pressure less than full Coloumb passive can be considered for design. The *Massachusetts Department of Transportation LRFD Bridge Design Manual* provides the empirical equation, below, to calculate lateral earth pressure coefficient (K) based on the ratio of deflection (δ_t) and wall height (H).

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

Design lateral earth pressure recommendations were developed based on this equation, as presented in **Appendix D**, and are provided in **Section 6.3** of this report. AASHTO Commentary C3.10.9.1 specifies that single-span bridges are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

5.4.9 Frost Protection

Fill soils are anticipated to be present at the abutments and embankments, either as existing fill or imported backfill. Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 1,820, and with low-moisture content (<10 percent) soils, the estimated depth of frost penetration is approximately 7.6 feet. Additionally, Section 5.2.1 states that pile-supported integral abutments will be embedded no less than 4.0 feet for frost protection, therefore the abutment should be embedded a minimum of 4.0 feet (vertically) from the bottom of abutment, when installed per Figure 5-2 of the BDG, Fixed Pile Head, Full Integral Abutment Details -Steel Superstructures.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN AND REINFORCEMENT CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter, except in front of integral abutments where a 1.75H:1V slope angle will be used. Slopes should be provided with loam and seed for permanent erosion protection. Where steeper slopes are planned riprap should be employed to limit surface erosion. Riprap should also be provided where the embankment side slopes will be near or below typical water levels in the Wilson Stream to protect from scour.

6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, and short- and long-period spectral acceleration coefficients were interpolated from the AASHTO LRFD Acceleration Coefficient Maps (3.10.2.1-1 through -21 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectrum (Site Class D) is developed 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)	0.130 g
SDs (Period = 0.2 sec)	0.274 g
SD1 (Period = 1.0 sec)	0.113 g

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill between new abutments and wingwalls and a 1.5H:1V plane extending up from the bottom of the abutment to the pavement subgrade 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
  - Passive Earth Pressure, K_p = 3.89 (use for design of backwalls and wingwalls)
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height (H_{eq}) values developed in accordance with LRFD Section 3.11.6.4, based on the abutment/wingwall height and distance from the wall backface to the edge of traffic. A minimum H_{eq} of 2 feet is recommended.
- Foundation drainage should be provided in accordance with Section 5.4.1.9 of the MaineDOT BDG. We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center. Alternatively, prefabricated drainage geocomposite material can be placed against the uphill side of abutments, after holes have been created through the backing material at the weep hole locations.

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 through a combination of side friction and end-bearing on the bedrock surface.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 531 kips, calculated by dividing the maximum factored pile load of 345 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 66,340 ft-lbs for the anticipated 25- to 35 foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance of 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. 335.4 at Abutment 1, and approximately El. 324.9 at Abutment 2). A provision is recommended in the drawings for extra pile length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into the bedrock.
- We recommend that the first pile be dynamically tested at each abutment during initial driving to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a 24-hour restrike test on each test pile, to assess potential relaxation.
- Piles shall be spliced in accordance with MaineDOT Section 501.047.
- Piles should be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor ϕ_{cc}=0.7 and the flexural resistance factor ϕ_f=1.0 should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-1).
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankment to the bridge. The slabs should be positively connected to the backwalls.

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 discussed in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

We recommend that the H-pile installation be controlled using wave equation analysis of the contractor's proposed driving system, field logging of the pile installation, and determination of final penetration resistance based on dynamic pile testing with signal matching analysis.

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. We recommend that one Dynamic Load Test with Signal Matching be completed at each abutment, including at least one pile restrike at each abutment.

7.2 PILE OBSTRUCTIONS

Although no evidence of cobbles and boulders was seen in the shallower strata (Fill) in the test borings, it is possible that pre-drilling, pre-excavation or spudding may be necessary to bypass potential obstructions, such as boulders, rock fill or existing foundations and stone masonry.

7.3 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to be on the order of 11 feet below existing pavement grades. It is our understanding that Route 133 will be out of service during construction of the new bridge. In areas where sufficient space is available and water conditions permit, the excavation adjacent to the approaches may be constructed with sloped, open cuts. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

Considering the proximity of the required abutment excavations to the river water level, management of water will be related to stream water levels at the time of construction. Considering bottom of existing abutment elevations at approximately El. 352.5 to El. 351.0 and Q1.1 at El. 359.4 for the existing structure, and El. 362.0 to El. 361.0 for the proposed structure, water levels may be at or above the bottom of excavation level during construction. It may be desirable to over-excavate and place an 8- to 12-inch-thick crushed stone working mat to improve accessibility and allow dewatering.

We anticipate that the inflow of groundwater or surface water to excavations could be handled by open pumping from sumps installed at the bottoms of excavations if cofferdams are installed. Stacked sand bags or a porta-dam type system may be sufficient to limit inflow of surface water in lieu of a sheet pile cofferdam, given the relatively small anticipated head. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade and permit concrete placement in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring results, the four upper fill samples tested had approximately 15 percent passing the No. 200 sieve, indicating the fill will meet MaineDOT specifications for Granular Borrow and is suitable for embankment fill. The existing fill is not suitable for Granular Borrow for Underwater Backfill.

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.

TABLES

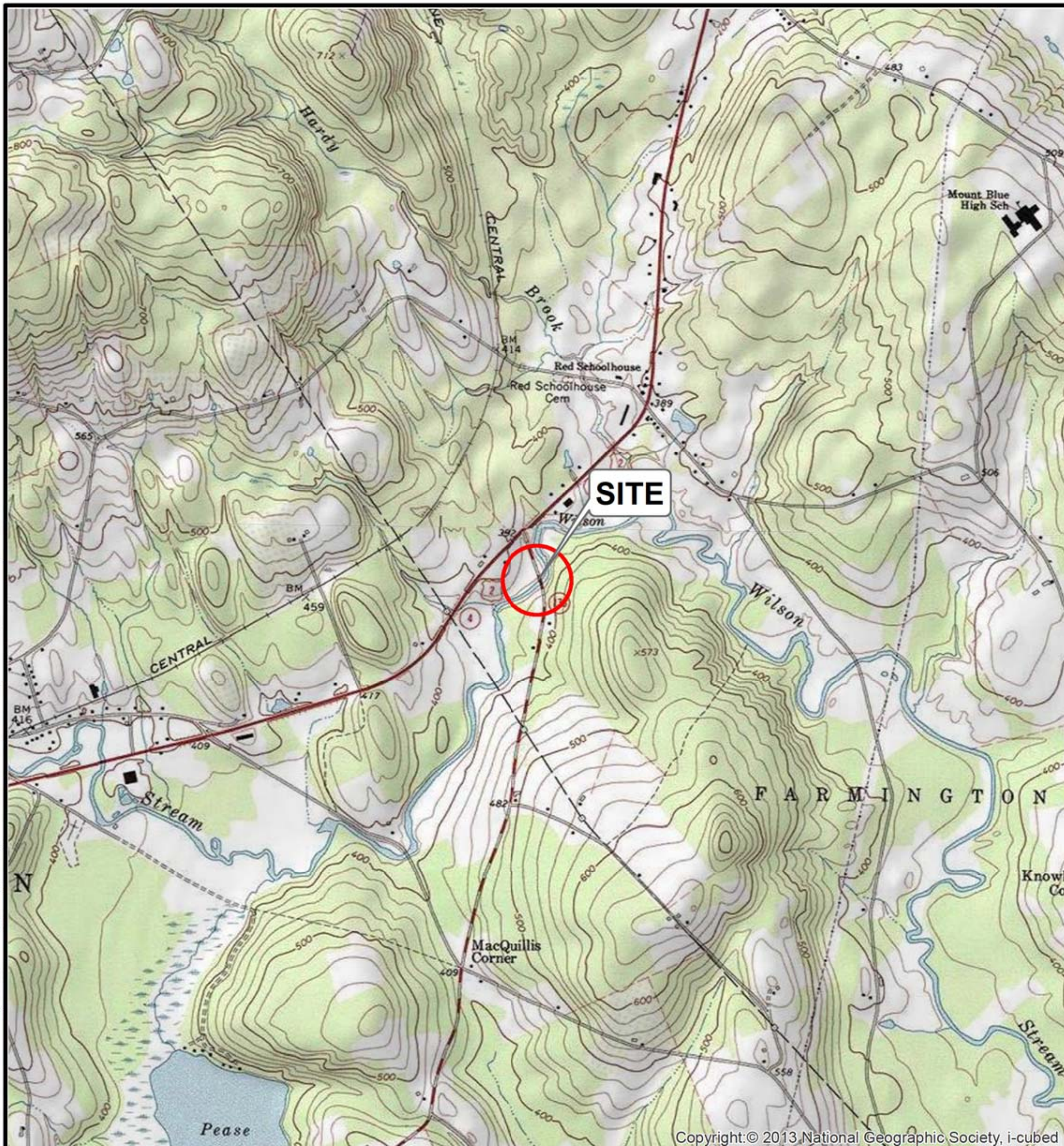
TABLE 1
Summary of Subsurface Explorations
Hamlin Bridge No. 3286, Route 113 over Wilson Stream
Farmington, ME
WIN 22236.00

Boring ID	Ground Surface El. (ft)	Top of Stratum Elevation (ft)				Stratum Thickness (ft)			Depth to Top of Probable Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
		Pavement	Fill	Glacial Till	Possible Bedrock	Pavement	Fill	Glacial Till				El. (ft)	Depth (ft)
BB-FWS-101	370	370	369.5	354.8	335.4	0.5	14.7	19.4	34.6	46.0	324.0	362.2	7.8
BB-FWS-102	369.2	369.2	368.5	355.8	324.9	0.8	12.7	30.9	44.4	54.8	314.4	NM	NM

El. = Elevation, NE = Not Encountered, NM = Not Measured, NP = Not Penetrated, > = Boring Terminated in Stratum

Notes:
1. Refer to the boring logs in Appendix B for additional information.
2. Project elevation datum is North American Vertical Datum (NAVD 88), unless noted otherwise.
3. As-drilled locations were surveyed by MaineDOT and provided to GZA by MFJ.
4. Stratum depths, thickness and elevations are rounded to the nearest 0.1 foot as interpreted on the boring logs, but this does not represent the precision of the data.

FIGURES



USGS QUADRANGLE LOCATION

N
W E
S

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. THIS MAP ALSO CONTAINS THE ESRI ARCGIS ONLINE USA COUNTIES WHICH PROVIDES DETAILED BOUNDARIES THAT ARE CONSISTENT WITH THE TRACT, BLOCK GROUP, AND STATE DATA SETS AND ARE EFFECTIVE AT REGIONAL AND STATE LEVELS.

0 1,000 2,000 4,000 6,000

SCALE IN FEET

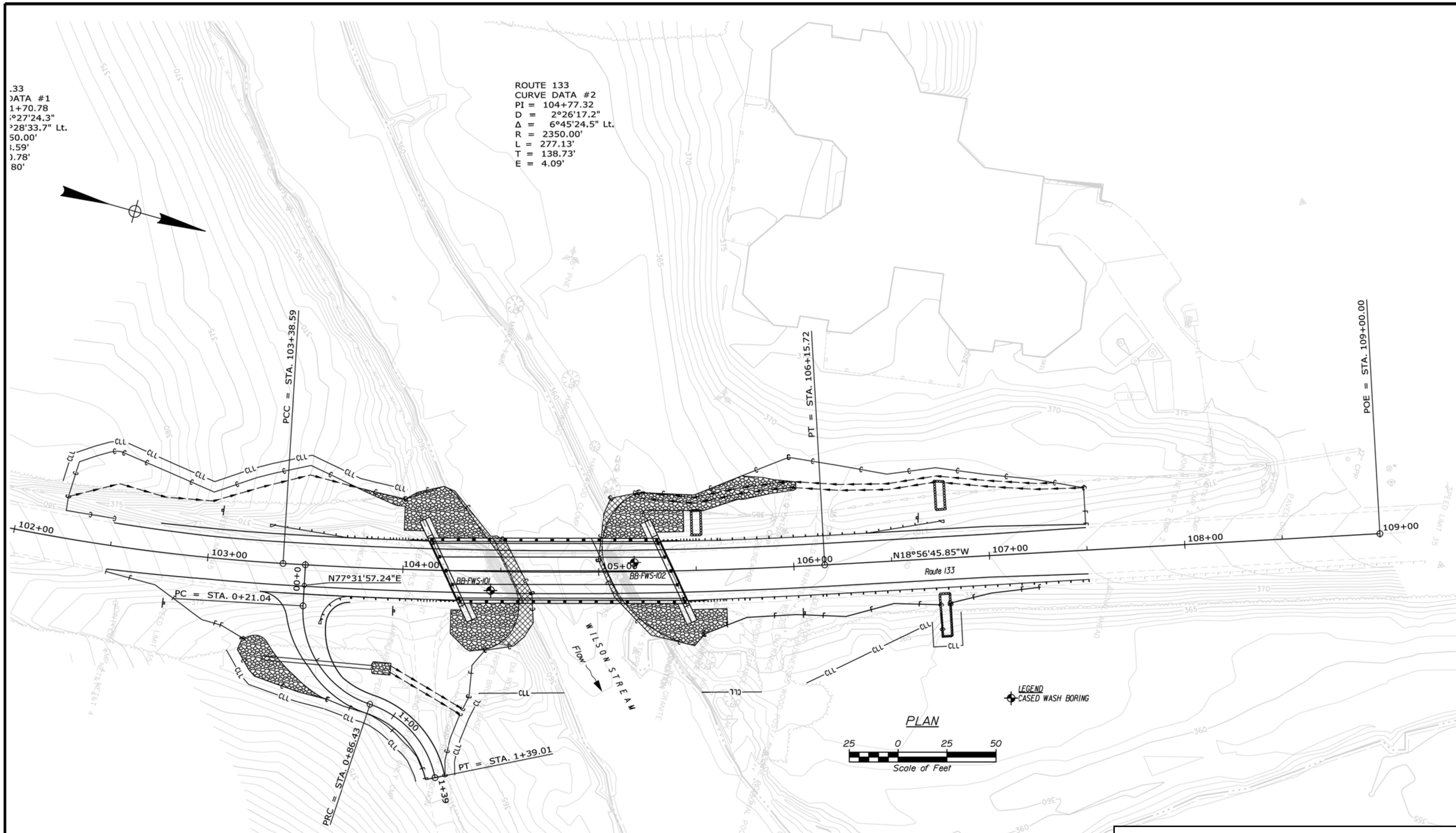
Data Supplied by :

esri

USGS science for a changing world

GZA	PROJ. MGR.: BMC DESIGNED BY: BMC REVIEWED BY: CLS OPERATOR: ENT DATE: 07-20-2020	**LOCUS PLAN** HAMLIN BRIDGE NO. 3286 ROUTE 113, FARMINGTON, MAINE	JOB NO. 09.0026052.00 FIGURE NO. **1**

Filename: ...\00\GEOTECH\MSTA\007_BLP1.dgn Division: GEOTECH Username: Terry.White Date:9/30/2020



HAMLIN BRIDGE NO. 3286
MAINEDOT WIN 22236.00
FARMINGTON, ME

BORING LOCATION PLAN

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

PREPARED FOR:
McFarland Johnson, Inc.

PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	2
DATE: 10/02/2020	PROJECT NO. 09.0026052.00	REVISION NO. 0	SHEET NO. 2 OF 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PROJECT NO. 022236.00

WIN 022236.00

BRIDGE PLANS

BRIDGE NO. 3286

	D.EATON	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	T.WHITE	B.CARDALI	JUN 2020
CHECKED-REVIEWED	C.SNOW	A.BLAISDELL	JUN 2020
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

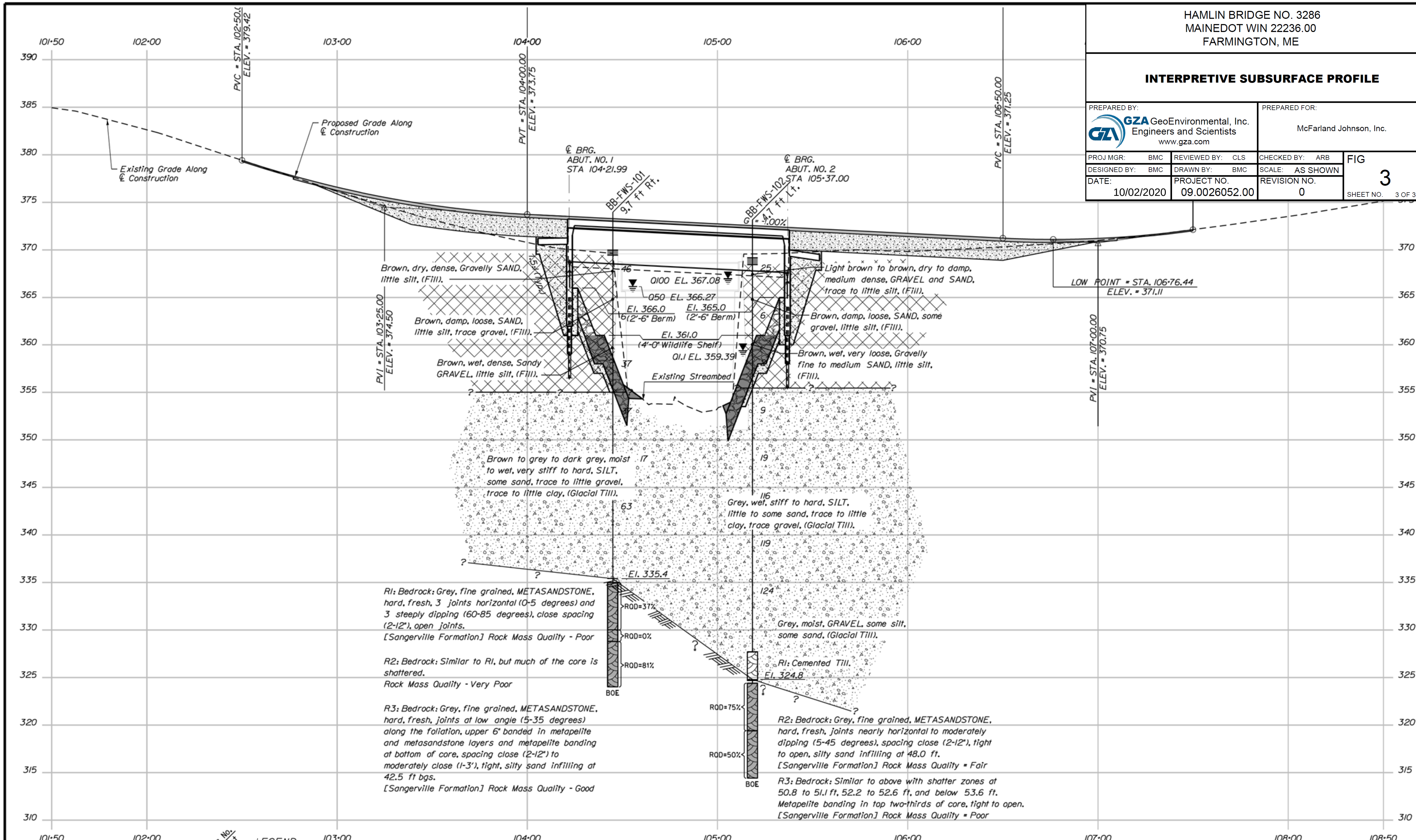
SIGNATURE
P.E. NUMBER
DATE

HAMLIN BRIDGE
WILSON STREAM
FARMINGTON FRANKLIN COUNTY

BORING LOCATION PLAN

SHEET NUMBER
7
OF 35

INTERPRETIVE SUBSURFACE PROFILE



PROFILE

HORIZ 25 0 25 50
VERT 5 0 5 10
SCALE

LEGEND

Boring No. / Offset
- Pavement Thickness, if applicable
- 10 — Energy Corrected Standard Penetration Test (SPT) N60-Value (Blows Per Foot)
- V1=855 psf — Denotes in-situ Vane Shear Test Performed at depth shown with Peak Undrained Shear Strength Provided
- Cored through Glacial Till
- RQD = Rock Quality Designation of Bedrock Core Sample
- Weathered Bedrock, if applicable
- Approximate Top of Bedrock
- NR — No Refusal
- R — Refusal
- WOH = Weight of Hammer
- WOR = Weight of Rods

Note: 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 and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

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

PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: ARB	FIG 3
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	
DATE: 10/02/2020	PROJECT NO. 09.0026052.00	REVISION NO. 0	SHEET NO. 3 OF 3

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
PROJECT NO. 022236.00
BRIDGE PLANS
WIN 022236.00
BRIDGE NO. 3286

	BY	DATE
PROJ. MANAGER	D.EATON	
DESIGN-DETAILED	T.WHITE	B.CARDALI JUN 2020
CHECKED-REVIEWED	C.SNOW	A.BLAISDELL JUN 2020
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE / P.E. NUMBER / DATE

HAMLIN BRIDGE
WILSON STREAM
FARMINGTON FRANKLIN COUNTY
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER 8 OF 35

Date: 9/30/2020 Username: Terry.White Division: GEOTECH Filename: ...\00\GEOTECH\MSTA\008_ISP1.dgn

APPENDIX A – LIMITATIONS

GEOTECHNICAL LIMITATIONS

Use of Report

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

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

4. In conducting our work, GZA relied upon certain information made available by public agencies, Client and/or others. GZA did not attempt to independently verify the accuracy or completeness of that information. Inconsistencies in this information which we have noted, if any, are discussed in the Report.

Subsurface Conditions

5. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs. The nature and extent of variations between these explorations may not become evident until further exploration or construction. If variations or other latent conditions then become evident, it will be necessary to reevaluate the conclusions and recommendations of this report.

6. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

7. Water level readings have been made in test holes (as described in this Report) and monitoring wells at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

8. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

9. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

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

Cost Estimates

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

Additional Services

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

APPENDIX B – 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, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

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

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

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

Rock Quality Designation (RQD):

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

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

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location: Farmington, Maine

Boring No.: BB-FWS-101
WIN: 22236.00

Driller:	MaineDOT	**Elevation (ft.)**	370.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/10/2018-7/11/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+44.9, 9.7 ft Rt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	7.8 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	369.5		6½" HMA. 0.5	
	1D	24/8	1.00 - 3.00	14/15/15/13	30	46				Brown, dry, dense, Gravelly SAND, little silt, (Fill).	G#296520 A-1-a, SW-SM WC=2.5%
5	2D	24/17	5.00 - 7.00	2/2/2/2	4	6				Brown, damp, loose, SAND, little silt, trace gravel, (Fill).	G#296521 A-2-4, SM WC=9.4%
10	3D	24/9	10.00 - 12.00	11/18/6/8	24	37	34			Brown, wet, dense, Sandy GRAVEL, little silt, (Fill).	G#296525 A-1-b, GM WC=15.5%
							38				
							31				
							52				
							98				
15	4D	24/9	15.00 - 17.00	16/13/11/11	24	37	85	354.8		Brown to grey, moist, hard, SILT, some sand, little gravel, trace clay. See Remarks 1. 15.2 Color change observed in spoon at 15.2 ft bgs from brown to grey.	G#296522 A-4, ML WC=14.5% Non-Plastic
							58				
							50				
							70				
	5D	24/20	19.50 - 21.50	9/5/6/4	11	17	9				
20							24			Grey, wet, very stiff, SILT, some sand, trace gravel, trace clay, (Glacial Till).	G#296523 A-4, ML WC=16.4% Non-Plastic
							36				
							51				
							74				
	6D	24/20	24.00 - 26.00	12/16/25/45	41	63	12			Similar to 5D, except hard.	
25											

Remarks:

1. NW Casing encountered a cobble at 9.5 ft, went crooked, and was subsequently pulled at 20.0 ft bgs (below ground surface). HW Casing was then driven to 15.0 ft, followed by Roller Coning ahead to 20.0 ft, and then telescoping NW.

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: Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location: Farmington, Maine

Boring No.: BB-FWS-101
WIN: 22236.00

Driller:	MaineDOT	**Elevation (ft.)**	370.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/10/2018-7/11/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+44.9, 9.7 ft Rt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	7.8 ft bgs.
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							20				
							27				
							79				
							112				
	7D	12/10	29.00 - 30.00	32/57(6")	---		64			Dark grey, wet, SILT, some sand, little clay, trace gravel, (Glacial Till).	G#296524 A-4, ML WC=8.9% Non-Plastic
30							109				
							157				
							119				
							180			b220 blows for 0.6 ft.	
	8D	6/3	34.00 - 34.50	50 (6")	---		b220	335.4		Grey, wet, Silty GRAVEL, some sand, (Glacial Till). 34.6	
35	R1	60/46	35.00 - 40.00	RQD = 37%			NQ-2	335.0		Top of Bedrock at Elev. 335.4 ft. Roller Coned ahead to 35.0 ft bgs. 35.0	
										R1: Bedrock: Grey, fine grained, METASANDSTONE, hard, fresh, 3 joints horizontal (0-5 degrees) and 3 steeply dipping (60-85 degrees), close spacing (2-12"), open joints. Rock Mass Quality - Poor Sangerville Formation R1: Core Times (min:sec) 35.0-36.0 ft (1:24) 36.0-37.0 ft (2:28) 37.0-38.0 ft (1:37) 38.0-39.0 ft (4:19) 39.0-40.0 ft (2:18) 67% Recovery	
40	R2	14.4/10	40.00 - 41.20	RQD = 0%						R2: Bedrock: Similar to R1, but much of the core is shattered. Rock Mass Quality - Very Poor R2: Core Times (min:sec) 40.0-41.0 ft (2:00) 41.0-41.2 ft (2:46) Core Blocked 69% Recovery	
	R3	57.6/55	41.20 - 46.00	RQD = 81%				324.0		R3: Bedrock: Grey, fine grained, METASANDSTONE, hard, fresh, joints at low angle (5-35 degrees) along the foliation, upper 6" banded in metapelite and metasandstone layers and metapelite banding at bottom of core, spacing close (2-12") to moderately close (1-3'), tight, silty sand infilling at 42.5 ft bgs. Rock Mass Quality - Good Sangerville Formation R3: Core times (min:sec) 41.2-42.2 ft (1:37) 42.2-43.2 ft (1:52) 43.2-44.2 ft (1:34)	
45											
50											

Remarks:

1. NW Casing encountered a cobble at 9.5 ft, went crooked, and was subsequently pulled at 20.0 ft bgs (below ground surface). HW Casing was then driven to 15.0 ft, followed by Roller Coning ahead to 20.0 ft, and then telescoping NW.

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: Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location: Farmington, Maine

Boring No.: BB-FWS-101
WIN: 22236.00

Driller:	MaineDOT	**Elevation (ft.)**	370.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/10/2018-7/11/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	104+44.9, 9.7 ft Rt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	7.8 ft bgs.
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										44.2-45.2 ft (1:37) 45.2-46.0 ft (2:43) 96% Recovery	
										46.0	
										Bottom of Exploration at 46.0 feet below ground surface.	
55											
60											
65											
70											
75											

Remarks:

1. NW Casing encountered a cobble at 9.5 ft, went crooked, and was subsequently pulled at 20.0 ft bgs (below ground surface). HW Casing was then driven to 15.0 ft, followed by Roller Coning ahead to 20.0 ft, and then telescoping NW.

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:	Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location:	Farmington, Maine
Boring No.:	BB-FWS-102
WIN:	22236.00

Driller:	MaineDOT	**Elevation (ft.)**	369.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/11/2018-7/12/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	105+18.3, 4.7 ft Lt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.75 - 2.75	9/9/7/4	16	25	SSA	368.5		9" HMA. 0.8 Top 6": Light brown, dry, medium dense, GRAVEL, little sand, trace silt, (Fill). Bottom 14": Brown, damp, medium dense, fine to medium SAND, little silt, trace gravel, (Fill).	
5	2D	24/8	5.00 - 7.00	3/2/2/2	4	6	22			Brown, damp, loose, SAND, some gravel, little silt, (Fill).	G#296526 A-1-b, SM WC=4.4%
							23				
							22				
							24				
							16				
10	3D	24/2	10.00 - 12.00	1/WOH/WOH/1	1	2	2			Brown, wet, very loose, Gravelly fine to medium SAND, little silt, (Fill).	
							14				
							26				
							28	355.7		13.5	
							45				
15	4D	24/9	15.00 - 17.00	2/3/3/5	6	9	25			Grey, wet, stiff, SILT, little sand, trace clay, trace gravel, (Glacial Till).	G#296527 A-4, ML WC=18.4% Non-Plastic
							46				
							68				
							65				
							65				
20	5D	24/22	20.00 - 22.00	6/6/6/9	12	19	52			Grey, wet, very stiff, SILT, little sand, little clay, trace gravel, (Glacial Till).	G#296528 A-4, ML WC=15.3% Non-Plastic
							50				
							63				
							126				
25	6D	24/20	24.00 - 26.00	39/51/24/31	75	116	OPEN			Grey, wet, hard, SILT, some sand, trace gravel, (Glacial Till).	G#296529 A-4, ML

Remarks:

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* 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: Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location: Farmington, Maine

Boring No.: BB-FWS-102
WIN: 22236.00

Driller:	MaineDOT	**Elevation (ft.)**	369.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/11/2018-7/12/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	105+18.3, 4.7 ft Lt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.928	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							HOLE				WC=12.6%
	7D	24/22	29.00 - 31.00	24/32/45/47	77	119				Grey, wet, hard, SILT, little sand, trace gravel, (Glacial Till).	G#296530 A-4, ML WC=12.5%
30											
	8D	24/24	34.00 - 36.00	33/40/40/38	80	124				Similar to 7D.	G#296531 A-4, ML WC=15.2%
35											
	9D	13.2/10	39.50 - 40.60	41/55/39(1.2")	---					Grey, wet, hard SILT, some sand, little gravel, (Glacial Till).	G#296532 A-4, ML WC=8.8%
40											
	R1		41.10 - 44.10								
	10D	3/1	44.10 - 44.35	50(3")	---		NQ-2	324.8		Grey, moist, GRAVEL, some silt, some sand, (Glacial Till).	
45	R2	60/56	44.80 - 49.80	RQD = 75%			NQ-2	324.4		R1: Cemented Till R1: Core Times (min:sec) 41.1-42.1 ft (1:08) 42.1-43.1 ft (0:40) 43.1-44.1 ft (6:40) Grey, moist, GRAVEL, some silt, some sand, (Glacial Till). 44.4 Top of Bedrock at Elev. 324.8 ft. Roller Coned ahead to 44.8 ft bgs. 44.8	
50	R3	60/56	49.80 - 54.80	RQD = 43%							

Remarks:

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: Hamlin Bridge #3286 carries Route 133 over Wilson Stream
Location: Farmington, Maine

Boring No.: BB-FWS-102
WIN: 22236.00

Driller:	MaineDOT	**Elevation (ft.)**	369.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Wilder/Daggett/Niles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	A. Van Buskirk	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/11/2018-7/12/2018	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	105+18.3, 4.7 ft Lt.	**Casing ID/OD:**	NW-3"/HW-4"	**Water Level*:**	None Observed
Hammer Efficiency Factor: 0.928		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R2: Bedrock: Grey, fine grained, METASANDSTONE, hard, fresh, joints nearly horizontal to moderately dipping (5-45 degrees), spacing close (2-12"), tight to open, silty sand infilling at 48.0 ft. Rock Mass Quality = Fair Sangerville Formation R2: Core Times (min:sec) 44.8-45.8 ft (1:28) 45.8-46.8 ft (1:22) 46.8-47.8 ft (1:29) 47.8-48.8 ft (1:54) 48.8-49.8 ft (2:08) 93% Recovery	
55								314.4		R3: Bedrock: Grey, fine grained, METASANDSTONE, hard, fresh, joints nearly horizontal to moderately dipping (5-45 degrees), spacing close (2-12") with shatter zones at 50.8-51.1 ft, 52.2 to 52.6 ft, and below 53.6 ft. Metapelite banding in top 2/3 of core, tight to open. Rock Mass Quality = Poor Sangerville Formation R3: Core Times (min:sec) 49.8-50.8 ft (1:54) 50.8-51.8 ft (2:04) 51.8-52.8 ft (1:55) 52.8-53.8 ft (1:58) 53.8-54.8 ft (2:48) 93% Recovery 54.8 **Bottom of Exploration at 54.8 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

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 TEST RESULTS

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Farmington   Work Number: 22236.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-FWS-101, 1D	104+44.9	9.7 Rt.	1.0-3.0	296520	1	2.5			SW-SM	A-1-a	0
BB-FWS-101, 2D	104+44.9	9.7 Rt.	5.0-7.0	296521	1	9.4			SM	A-2-4	II
BB-FWS-101, 3D	104+44.9	9.7 Rt.	10.0-12.0	296525	1	15.5			GM	A-1-b	I
BB-FWS-101, 4D	104+44.9	9.7 Rt.	15.0-17.0	296522	1	14.5	-N	P-	ML	A-4	IV
BB-FWS-101, 5D	104+44.9	9.7 Rt.	19.5-21.5	296523	1	16.4	-N	P-	ML	A-4	IV
BB-FWS-101, 7D	104+44.9	9.7 Rt.	29.0-30.0	296524	1	8.9	-N	P-	ML	A-4	IV
BB-FWS-102, 2D	105+18.3	4.7 Lt.	5.0-7.0	296526	2	4.4			SM	A-1-b	II
BB-FWS-102, 4D	105+18.3	4.7 Lt.	15.0-17.0	296527	2	18.4	-N	P-	ML	A-4	IV
BB-FWS-102, 5D	105+18.3	4.7 Lt.	20.0-22.0	296528	2	15.3	-N	P-	ML	A-4	IV
BB-FWS-102, 6D	105+18.3	4.7 Lt.	24.0-26.0	296529	2	12.6			ML	A-4	IV
BB-FWS-102, 7D	105+18.3	4.7 Lt.	29.0-31.0	296530	3	12.5			ML	A-4	IV
BB-FWS-102, 8D	105+18.3	4.7 Lt.	34.0-36.0	296531	3	15.2			ML	A-4	IV
BB-FWS-102, 9D	105+18.3	4.7 Lt.	39.5-40.6	296532	3	8.8			ML	A-4	IV

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 MaineDOT and Corps of Engineers Classification Systems.

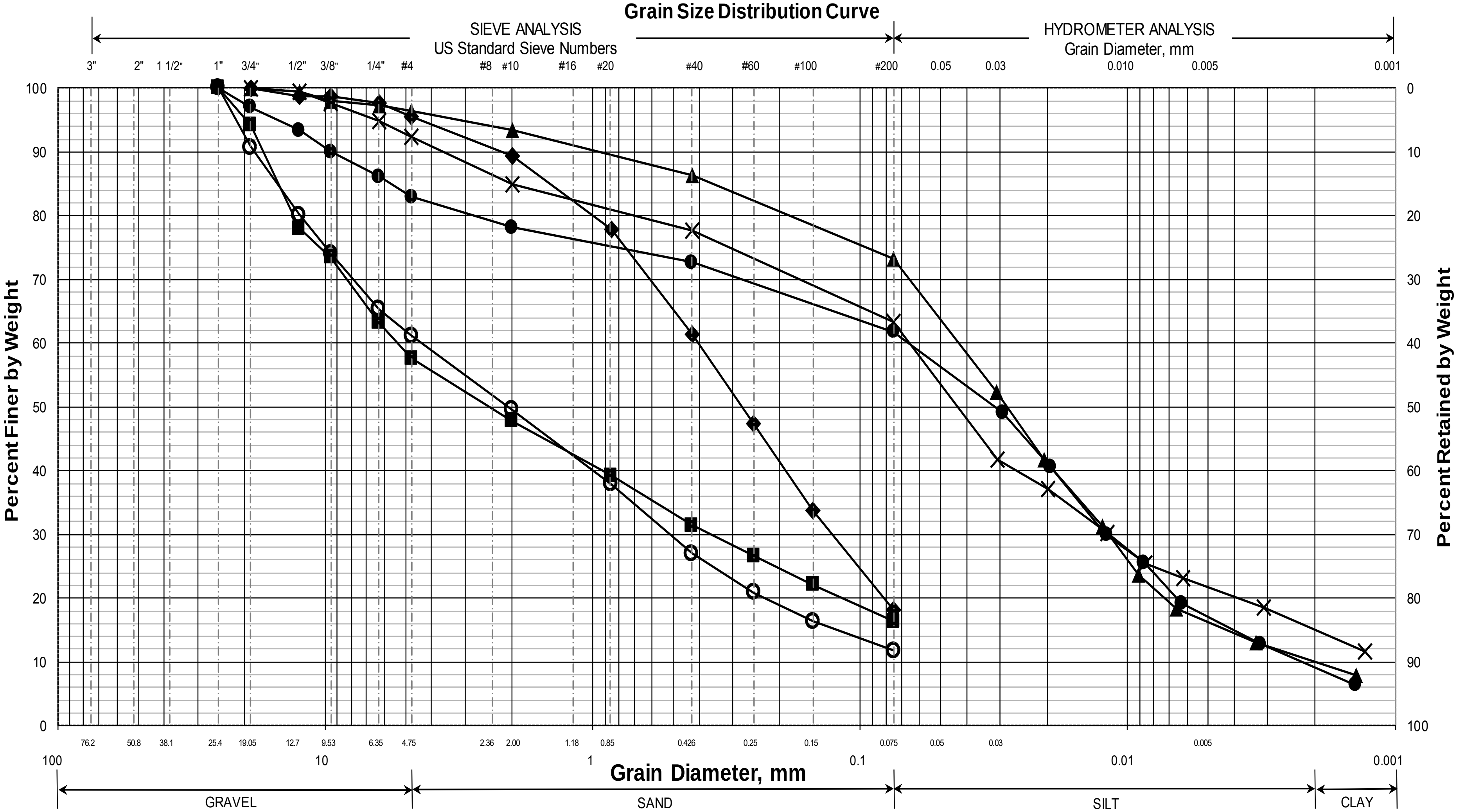
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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   NP = Non Plastic

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

Maine Department of Transportation
Grain Size Distribution Curve



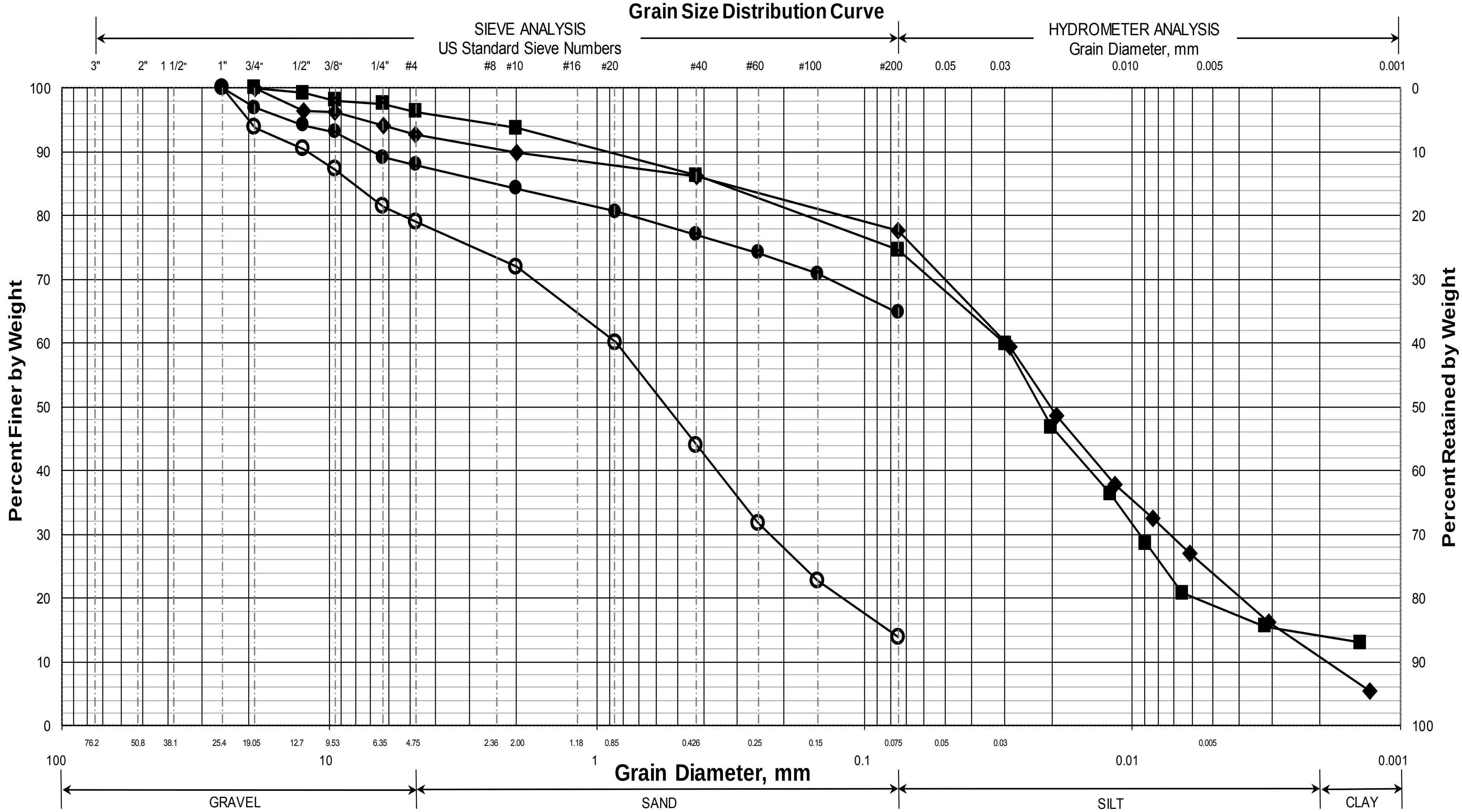
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FWS-101/1D	104+44.9	9.7 RT	1.0-3.0	Gravelly SAND, little silt.	2.5			
◆	BB-FWS-101/2D	104+44.9	9.7 RT	5.0-7.0	SAND, little silt, trace gravel.	9.4			
■	BB-FWS-101/3D	104+44.9	9.7 RT	10.0-12.0	Sandy GRAVEL, little silt.	15.5			
●	BB-FWS-101/4D	104+44.9	9.7 RT	15.0-17.0	SILT, some sand, little gravel, trace clay.	14.5			NP
▲	BB-FWS-101/5D	104+44.9	9.7 RT	19.5-21.5	SILT, some sand, trace clay, trace gravel.	16.4			NP
×	BB-FWS-101/7D	104+44.9	9.7 RT	29.0-30.0	SILT, some sand, little clay, trace gravel.	8.9			NP

WIN	
022236.00	
Town	
Farmington	
Reported by/Date	
WHITE, TERRY A	10/22/2018

SHEET 1

Maine Department of Transportation
Grain Size Distribution Curve



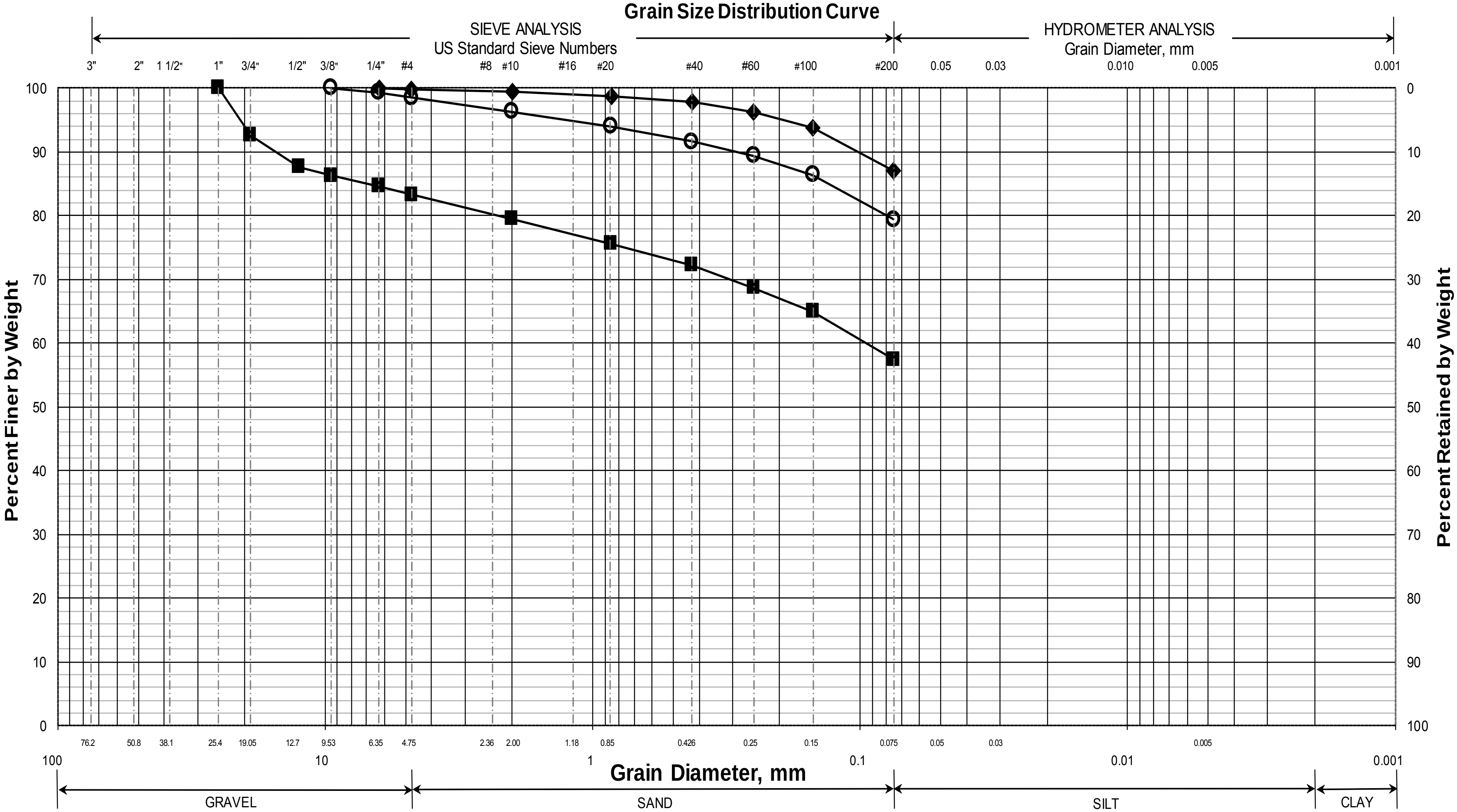
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FWS-102/2D	105+18.3	4.7 LT	5.0-7.0	SAND, some gravel, little silt.	4.4			
◆	BB-FWS-102/4D	105+18.3	4.7 LT	15.0-17.0	SILT, little sand, trace clay, trace gravel.	18.4			NP
■	BB-FWS-102/5D	105+18.3	4.7 LT	20.0-22.0	SILT, little sand, little clay, trace gravel.	15.3			NP
●	BB-FWS-102/6D	105+18.3	4.7 LT	24.0-26.0	SILT, some sand, little gravel.	12.6			
▲									
×									

WIN	
022236.00	
Town	
Farmington	
Reported by/Date	
WHITE, TERRY A	10/22/2018

SHEET 2

Maine Department of Transportation
Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-FWS-102/7D	105+18.3	4.7 LT	29.0-31.0	SILT, little sand, trace gravel.	12.5			
◆	BB-FWS-102/8D	105+18.3	4.7 LT	34.0-36.0	SILT, little sand, trace gravel.	15.2			
■	BB-FWS-102/9D	105+18.3	4.7 LT	39.5-40.6	SILT, some sand, little gravel.	8.8			
●									
▲									
×									

WIN	
022236.00	
Town	
Farmington	
Reported by/Date	
WHITE, TERRY A	10/22/2018

SHEET 3

APPENDIX D – ENGINEERING CALCULATIONS

Seismic Calculation

Seismic Site Class Calculation Summary

Project:	Hamlin Bridge over Wilson Stream	**Project No.:**	09.0026052.00
Location:	Farmington, ME		
Evaluated By/Date:	BMC	**Date**	6/3/2020
Checked By/Date:	CLS	**Date**	6/XX/2020

Objective:
Determine seismic site class by performing calculations in accordance with the MaineDOT Bridge Manual 2003 Edition with updates in 2014, which references the AASHTO LRFD Seismic Bridge Design Specifications, 8th Edition.

Subsurface Data: Borings BB-FWS-101 and -102 were drilled by MaineDOT between July 10 and 12, 2018.

Assumptions: Soil borings extended to depths between 46 and 55 feet below the roadway level and bedrock was encountered in the soil borings.

Approach: 1) Evaluate if the procedure in AASHTO LRFD Seismic Section 3.10.2.1 for classifying a site is appropriate for the site. Sites with highly variable subsurface conditions or very large sites may require multiple site class determinations or a site-specific seismic response analysis. Furthermore, classifying a site based on the 100 feet of soil and rock beneath the ground surface may be inappropriate if deep deposits of weak soils are present below 100 feet, or if foundation structures are supported on firm soil or rock below soft soils which can be justified as having little effect on the structure's seismic response.

2) Evaluate if soil properties are known in sufficient detail to determine site class. If data is not known in sufficient detail, AASHTO permits the use of Site Class D, unless conditions for Site Class E or Site Class F are likely to be present.

3) Check for the four categories of Site Class F requiring site-specific evaluation:
- Soils vulnerable to potential failure (liquefiable soils, sensitive clays, weakly cemented soils)
- Peats or highly organic clays greater than 10 feet in thickness
- Thick layers (greater than 25 feet) of highly plastic clay (PI > 75)
- Very thick soft/medium stiff clays (greater than 125 feet)

4) Check for existence of greater than 10 feet of soft clay (where s_u < 500 psf, w > 40%, and PI > 20). If these conditions are met, classify as Site Class E.

5) Categorize the site using one of the following three methods in AASHTO C3.10.3.1-1:
- $\overline{v}_s$ (Method A) - $\overline{N}$ (Method B) - $\overline{N}_{ch}$ and $\overline{s}_u$ (Method C)

If shear wave velocity data are available, they should be used to classify the site. The $\overline{N}$ and $\overline{s}_u$ methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with $\overline{v}_s$.

Results: Calculations of the Seismic Site Class based on Method B as described in section 3.10.3.1 of the LRFD Seismic Bridge Design Specifications are attached. Calculations results are summarized in the table below.

Boring ID	BB-FWS-101	BB-FWS-102	Average
N-Value	35	18	26

Conclusions: Based on the procedure outlined in section 3.10.3.1 and table 3.10.3.1-1 of the LRFD Seismic Bridge Design Specifications, we recommend that Site Class "D" be used for design.

INPUT

Exploration ID: **BB-FWS-101** Ground Surface Elevation: 370.0 ft Depth of Boring: 46.0 ft

Depth to Bedrock: 34.6

EQUATIONS

$$\overline{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

CALCULATION

$\overline{N}$ = 34.6

Soil Strata	SPT Interval Depth Top, ft	Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	1.0	3.0	368.0	46	4.0	0.09	
	5.0	7.0	364.0	6	4.5	0.75	
	10.0	12.0	359.0	37	5.0	0.14	
Glacial Till	15.0	17.0	354.0	37	4.8	0.13	
	19.5	21.5	349.5	17	4.5	0.26	
	24.0	26.0	345.0	63	4.8	0.08	
	29.0	30.0	340.5	50	4.5	0.09	
	34.0	34.5	335.8	50	68.0	1.36	
Top of Rock	34.6						

100.00

Hamlin Bridge over Wilson Stream
Farmington, ME

Calculated By: BMC Date: 6/3/2020
Checked By: CLS Date: 6/XX/2020

INPUT

Exploration ID: **BB-FWS-102** Ground Surface Elevation: 369.2 ft Depth of Boring: 54.8 ft

Depth to Bedrock: 44.4

EQUATIONS

$$\bar{N}_{ch} = \frac{\sum_{i=1}^{m} d_i}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

where: m = number of layers
d_i = the thickness of all layers between 0 and 100 feet.
d_c = the thickness of any clay layers between 0 and 100 feet.
N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).
Note: d_i calculated assuming breaks between sub-layers occur at the midpoint between SPT sample intervals (unless noted otherwise)

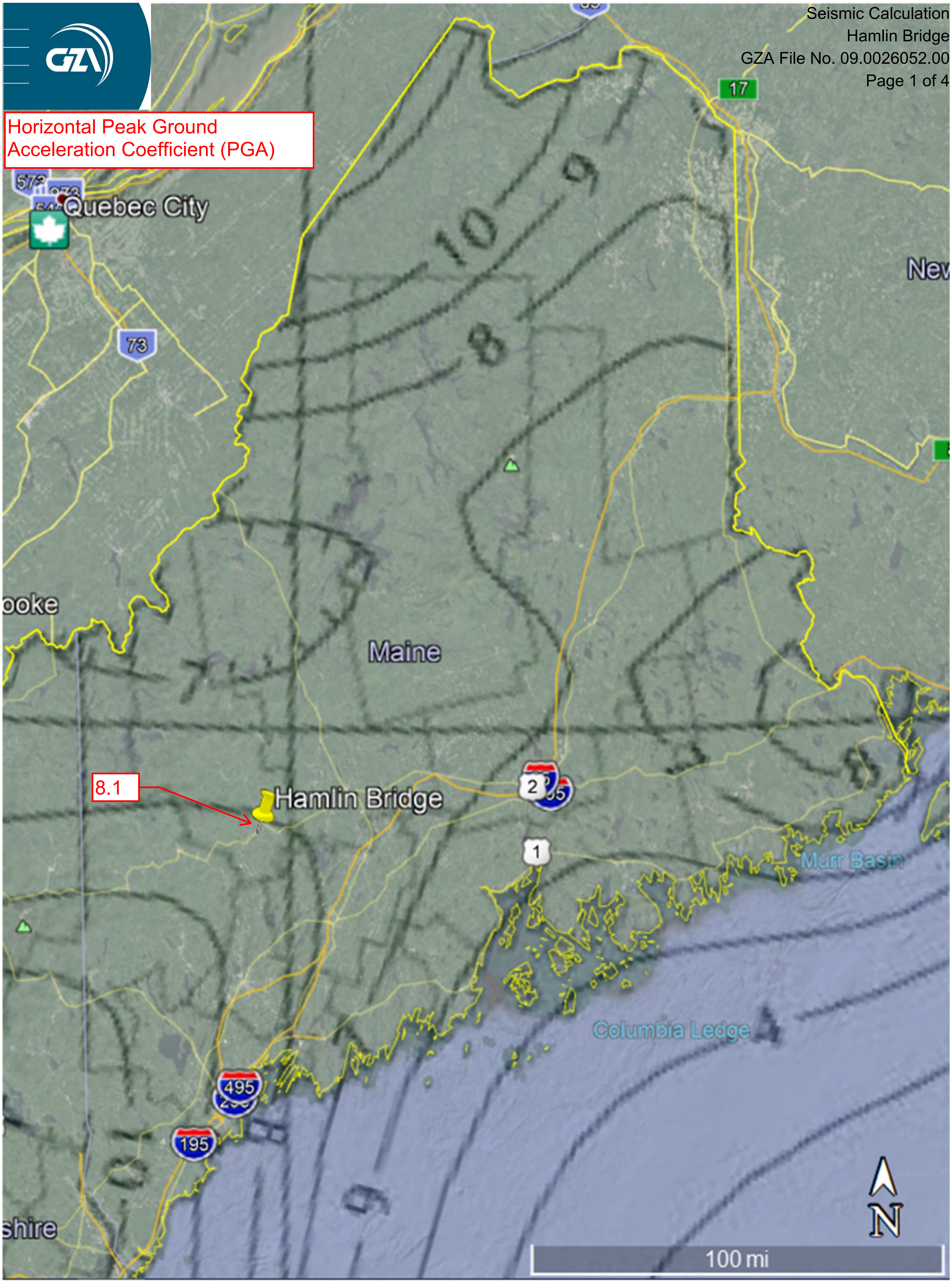
CALCULATION

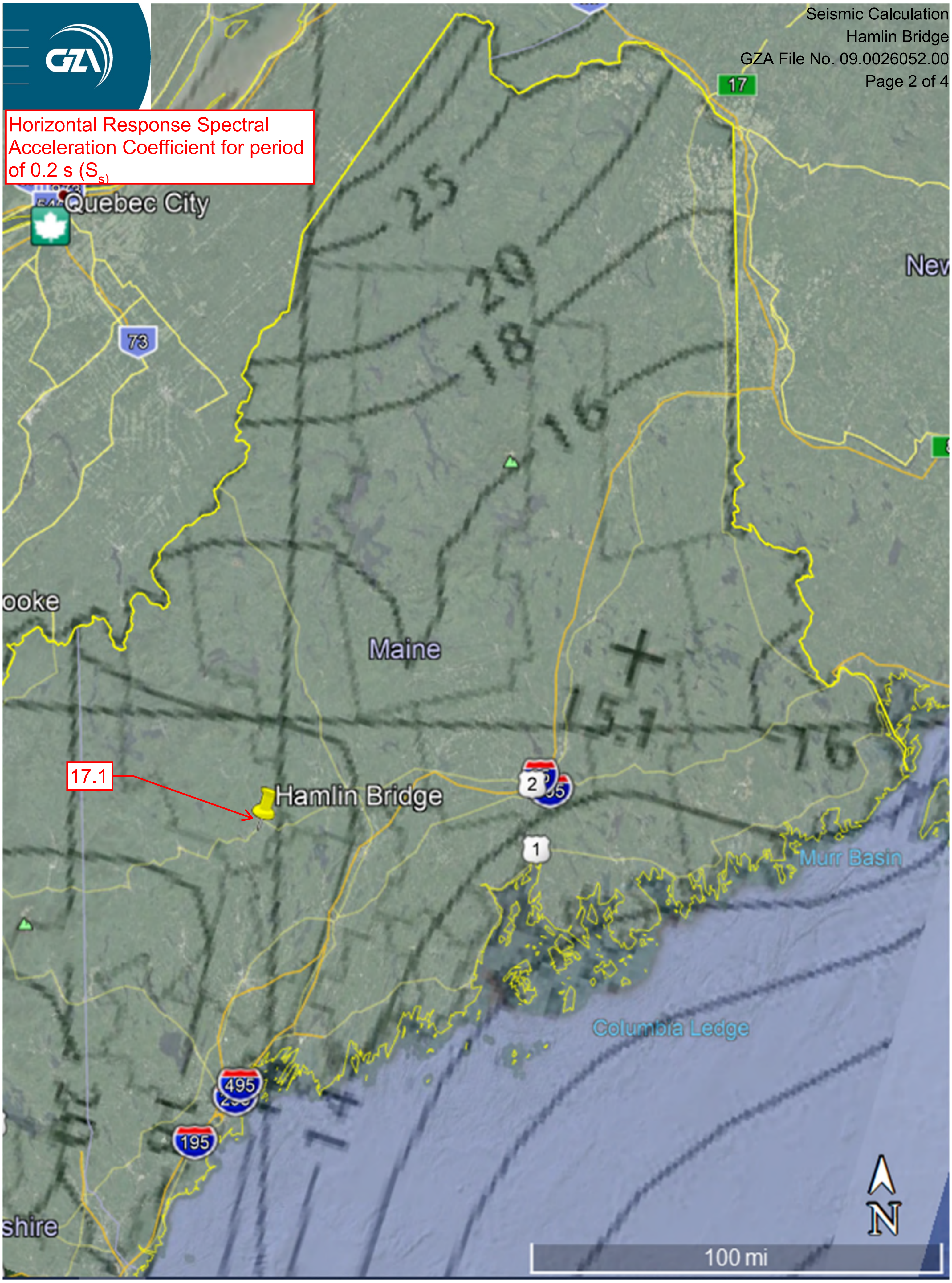
$\bar{N}$ = 17.9

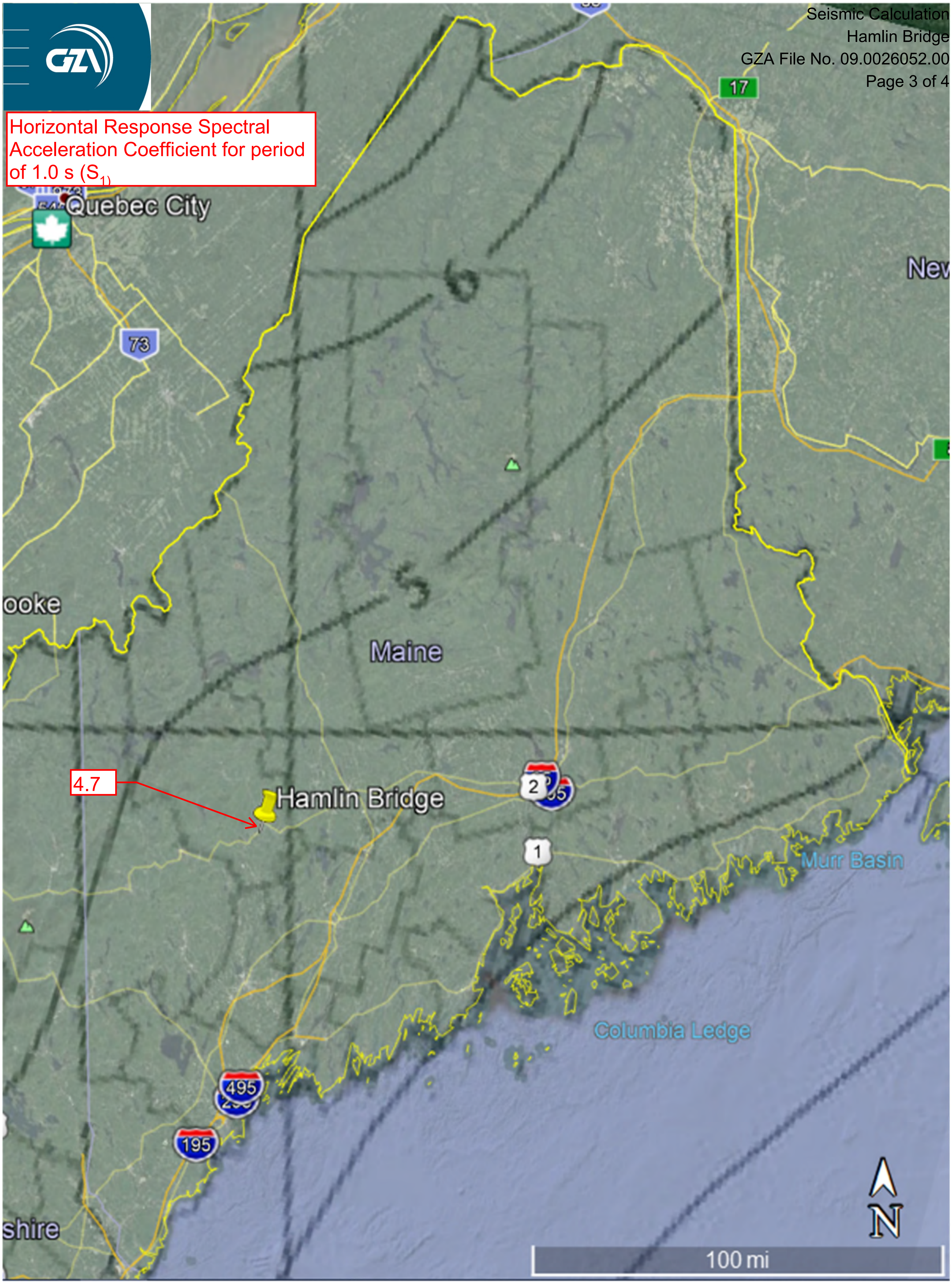
Soil Strata	SPT Interval Depth Top, ft	SPT Interval Depth Bottom, ft	SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
Fill	0.8	2.8	367.5	25	3.9	0.16	
	5.0	7.0	363.2	6	4.6	0.77	
	10.0	12.0	358.2	2	5.0	2.50	
Glacial Till	15.0	17.0	353.2	9	5.0	0.56	
	20.0	22.0	348.2	19	4.5	0.24	
	24.0	26.0	344.2	116	4.5	0.04	
	29.0	31.0	339.2	119	5.0	0.04	
	34.0	36.0	334.2	124	5.3	0.04	
	39.5	40.6	329.2	50	3.1	0.06	
	41.1	41.4	328.0	50	59.2	1.18	
Top of Rock	44.4						

100.00

Seismic Design Parameters


GZA
Horizontal Peak Ground Acceleration Coefficient (PGA)
Quebec City
73
17
10
9
8
7
New
ooke
Maine
8.1
Hamlin Bridge
2
95
1
Murr Basin
Columbia Ledge
495
195
shire
N
100 mi


Horizontal Response Spectral Acceleration Coefficient for period of 0.2 s (S_s)
Quebec City
73
17
25
20
18
16
Maine
15.1
16
17.1
Hamlin Bridge
2
1
Murr Basin
Columbia Ledge
495
195
N
100 mi


Horizontal Response Spectral Acceleration Coefficient for period of 1.0 s (S_1)
Quebec City
73
17
6
5
Maine
4.7
Hamlin Bridge
2
1
Murr Basin
Columbia Ledge
495
195
ooke
shire
N
100 mi

Hamlin Seismic Interpolation for Coefficients		
Seismic Parameter	**Interpolated Value from Maps[1]**	**Design Parameter**
Horizontal Peak ground Acceleration Coefficient	8.1	$PGA = .081$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	17.1	$S_s = 0.171$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.7	$S_1 = .047$

Notes: 1. AASHTO Figures 3.10.2.1-1,-2, and -3 were overlaid within the Google Earth software. Coefficients were interpolated between lines on these figures as presented in pages 1 through 3 of this calculation.

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

Therefore:

$$A_s = F_{PGA} \times PGA = 1.6 \times 0.081 = 0.130\ g$$

$$S_{DS} = F_a \times S_s = 1.6 \times 0.171 = 0.274\ g$$

$$S_{D1} = F_v \times S_1 = 2.4 \times 0.047 = 0.113\ g$$

Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.130 g
SDs (Period = 0.2 sec)	0.274 g
SD1 (Period = 1.0 sec)	0.113 g

Frost Calculation

March 2014

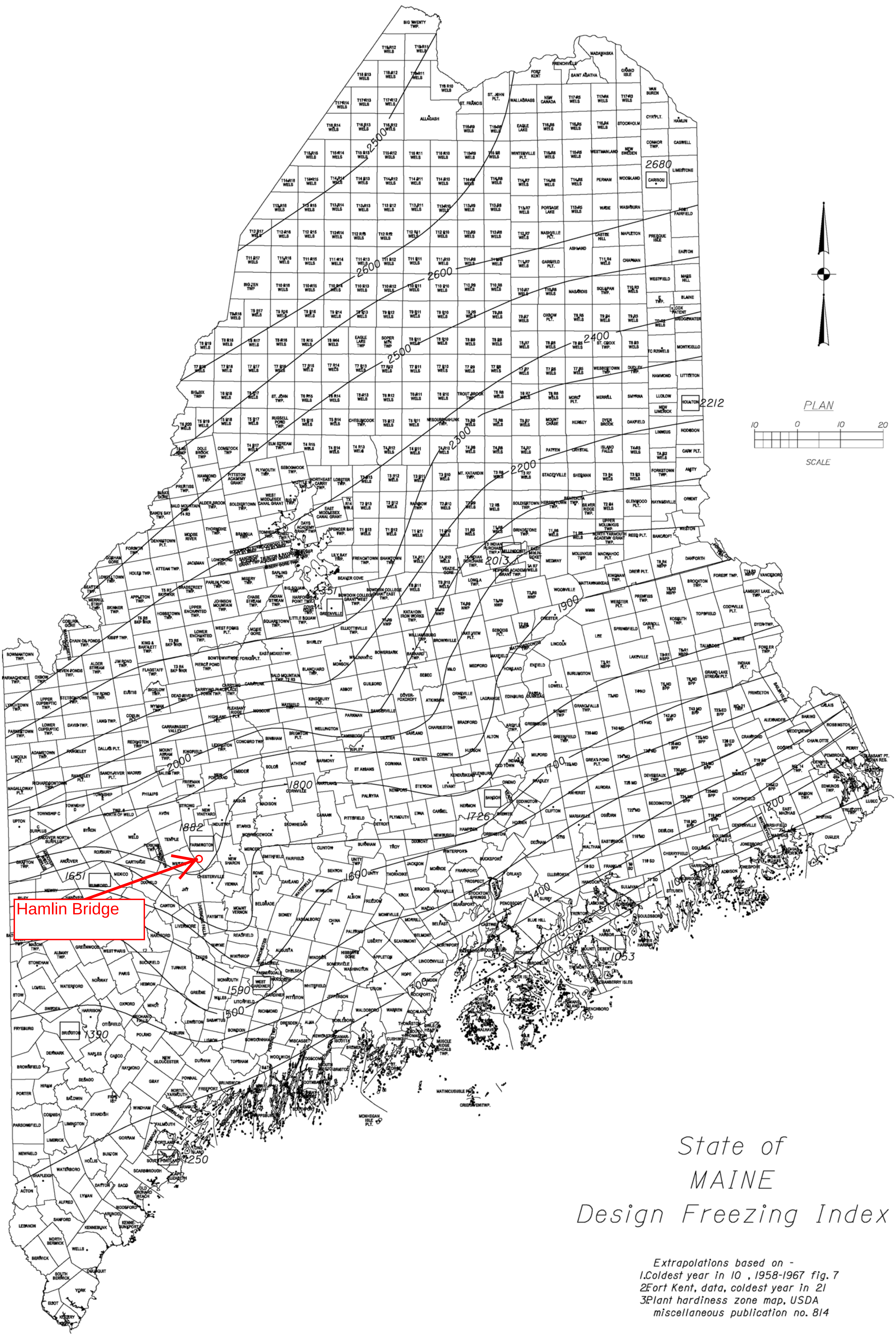


Figure 5-1 Maine Design Freezing Index Map

Table 5-1 Depth of Frost Penetration

1820

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
91 = 7.6'	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

Existing Fill soils Glacial Till depsosits are anticipated to be present at the abutments near the elevation of the footings. The material is coarse-grained with water contents less than 10% to 15%. Based on the MaineDOT BDG, Section 5.2.1 and a Freezing Index of 1817 the estimated depth of frost penetration is 91 inches.

Lateral Earth Pressures

GZA
GeoEnvironmental, Inc
707 Sable Oaks Drive
Suite 150
South Portland, Maine 04106
207-879-9190
Fax 207-879-0099

Engineers and Scientists

Subject:

Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)
3. Massachusetts Department of Transportation Highway Division LRFD Bridge Manual Part I, section 3.10.8

Input Parameters:

$\beta := 0\text{deg}$ — Angle of backfill to the horizontal

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

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

$\delta_f := 19.5\text{deg}$ — Average value of friction angle between, precast concrete and clean sand/silty sand-gravel mixture *(AASHTO LRFD Table 3.11.5.3-1)*

Passive Earth Pressure on Integral Backwall:

Per BDG Section 5.4.2.11, developing full passive pressure requires that ratio of lateral abutment movement (y) to abutment height (Hb) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered.

$y := 0.663\text{in}$ — From structural engineer

$H_b := 11.26\text{ft}$

$\frac{y}{H_b} = 0.0049$ — Ratio of lateral movement to abutment height is less than 0.005, use Rankine passive earth Pressure

Earth Pressure Coefficients:

Since the ratio of lateral movement is less than .005 but not considered significantly less GZA evaluated the typical Rankine coefficient and compared values to the Massachusetts DOT methodology presented in section 3.1.8 for design of the Hamlin bridge.

Rankine Passive Earth Pressure Coefficient

$$K_{pr} := \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \qquad \boxed{K_{pr} = 3.25}$$

MassDOT Section 3.10.8 presents the plot and calculation shown below for a gravel borrow material.

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

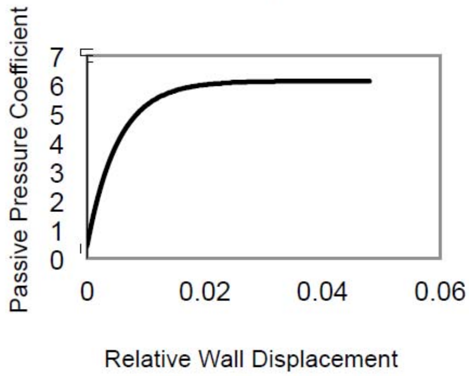


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H.

$$\omega := \frac{y}{H_b} = 0.0049$$

$$K_{p.mass} := 0.43 + 5.7\left(1 - \exp\left(-190 \cdot \frac{y}{H_b}\right)\right) = 3.89$$

$$\boxed{K_{p.mass} = 3.89}$$

Considering the ratio of lateral movement is approaching but less than the threshold of 0.005 for Coulomb passive coeffients, GZA recommends that a passive earth pressure coeffcient of 3.89, as calculated by the MassDOT methodology, is more conservative than the Rankine coeffcient and is recommended for the abutment design for the Hamlin bridge project.

Pile Drivability Analyses

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Hamlin Bridge Replacement in Farmington, ME. Evaluations were conducted to assess a suitable driving system to install piles to the required geotechnical nominal resistance of 531 kips for abutment piles.

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, 8th edition. (AASHTO LRFD)

Soil Properties

Consider Hamlin Bridge Interpretive Subsurface Profile (see Figure 3), subsurface layering and properties relative to pile design are presented in the Apile outputs on pages 7-10.

Structural Properties

HP14x89, ASTM A572, Gr. 50

Yield Strength of Steel $F_y := 50\text{ksi}$

Area of section $A_s := 26.1\text{in}^2$

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

Radius of gyration (weak axis) $r_x := 5.88\text{in}$

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.5.1-1

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 pile is fully embedded, AASHTO 10.7.3.13.1.

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 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State:

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

$P_r = 653 \cdot kip$

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 or by the drivability of the pile.

Required nominal resistance of 531 kips based on hard driving conditions, a maximum factored pile load of 345 kips, and a 0.65 resistance factor for Abutments 1 and 2.

5. Geotechnical Axial Resistance - Drivability Analysis

Side Friction:

GZA used static analysis to estimate the side friction on the pile to represent 15 to 20 % of the required nominal resistance of 531 kips.

Allowable Driving Stress:

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

$f_y := 50ksi$ 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

GZA evaulated the shorter pile length of 25 feet, where stresses are likely to be highest, using a 0.04 in toe quake.

Drive piles with a Delmag D25-32 open-ended diesel hammer with a rated energy of 66,340 ft-lb (fuel setting 4, 3 below maximum).

GRLWEAP Output is attached on Sheets 5 through 6.

$R_{ndr1} := 531kip$ Required nominal geotechnical resistance, pile driving stress =40 ksi, final penetration resistance=8 bpi.

Results:
The selected driving system can install the propos ed pile to a nominal resistance of 531 kips, with a final penetration resistance of 8 blows per foot, a stroke of 7.3 feet, and driving stress of approximately 40 ksi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

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

$$R_{ndr1\_factored} := R_{ndr1} \cdot \phi_{dyn}$$

$$R_{ndr1\_factored} = 345 \cdot kip$$

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

Service and Extreme Limit State Factored Drivability Resistance:

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

From Article 10.5.5.1 and 10.5.5.3

$$R_{ndr1\_serv\_ext} := R_{ndr1} \cdot \phi_{serv\_ext}$$

$$R_{ndr1\_serv\_ext} = 531 \cdot kip$$

Summary of Results - Axial Loading:

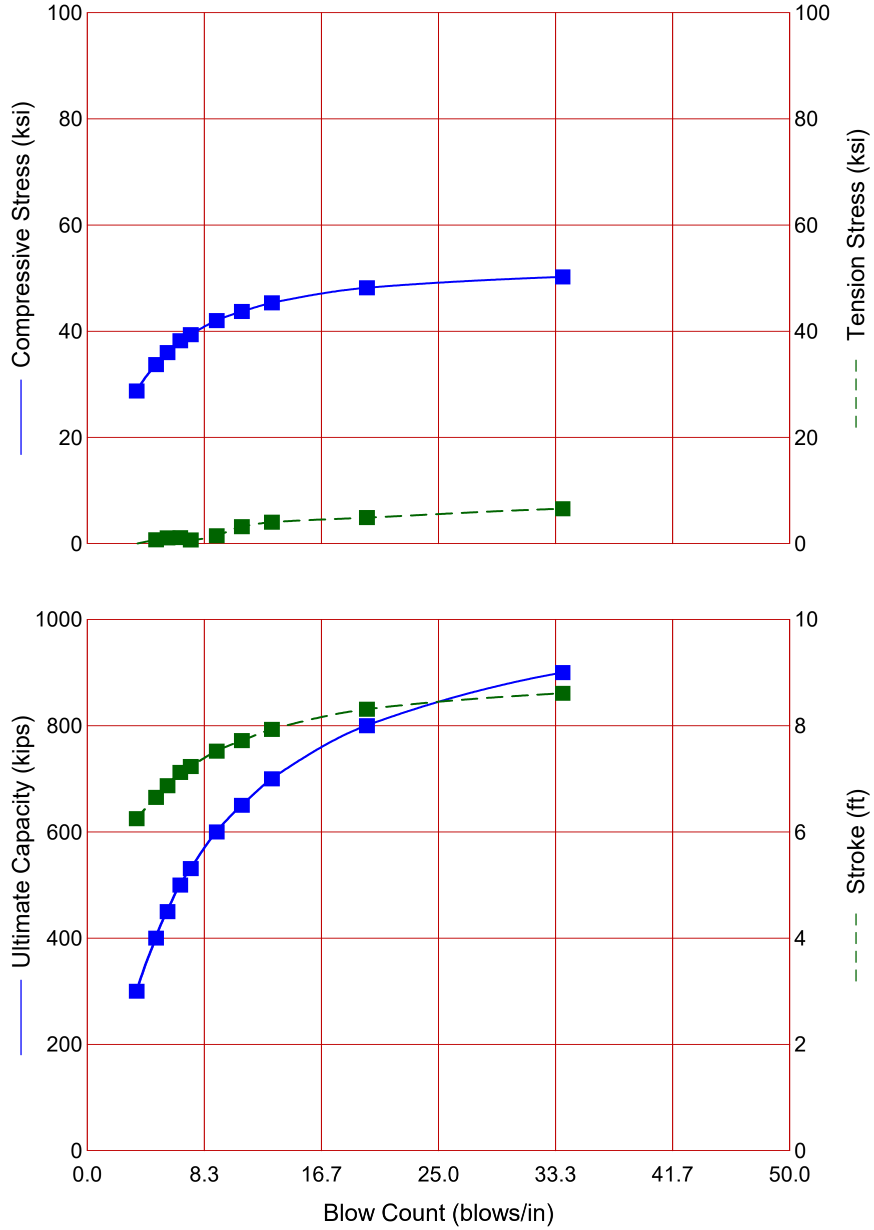
ASTM A572, HP 14x89	Factored Structural Resistance (kips)	Factored Geotechnical Static Resistance (kips)	Factored Geotechnical Resistance (kips)	Governing Resistance (kips)
Abutment 1 & 2: Strength Limit State, Design	653	n/a	345	345
butment 1 & 2: Service/Extreme Limit State, Desig	1305	n/a	531	531

Results indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 66,340 ft-lbs for the anticipated 25- to 35-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel), and with a final penetration resistance of 8 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
300.0	28.74	0.00	3.5	6.25	15.13
400.0	33.71	0.73	4.9	6.65	15.43
450.0	35.97	1.10	5.7	6.87	15.69
500.0	38.19	1.14	6.6	7.12	16.05
531.0	39.36	0.70	7.4	7.23	16.13
600.0	42.00	1.48	9.2	7.52	16.59
650.0	43.71	3.19	11.0	7.72	17.16
700.0	45.34	4.05	13.2	7.93	17.82
800.0	48.17	4.91	19.9	8.31	18.93
900.0	50.23	6.59	33.8	8.61	19.79

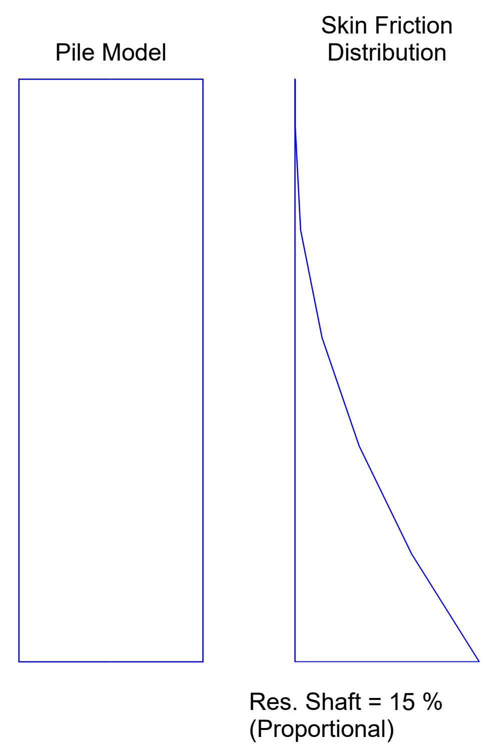
GZA Geo Environmental, Inc.
Hamlin Bridge 3286

21-Sep-2020
GRLWEAP Version 2010



DELMAG D 25-32

Ram Weight	5.51	kips
Efficiency	0.800	
Pressure	1057 (72%)	psi
Helmet Weight	1.90	kips
Hammer Cushion	60155	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	27.00	ft
Pile Penetration	25.00	ft
Pile Top Area	26.10	in2



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========================================================================
                 APILE for Windows, Version 2018.8.5

                  Serial Number :  653550831

                  A Program for Analyzing the Axial Capacity
                  and Short-term Settlement of Driven Piles
                            under Axial Loading.
                (c) Copyright ENSOFT, Inc., 1987-2015
                        All Rights Reserved

========================================================================


This program is licensed to :

GZA GeoEnvironmental, Inc.
Portland, OR

Path to file locations      : P:\09 Jobs\0026000s\09.0026052.00 - Hamlin Bridge 3286, Farmington\Work\Drivability\Apile\
Name of input data file     : Hamlin Apile 14x89.ap8d
Name of output file         : Hamlin Apile 14x89.ap8o
Name of plot output file    : Hamlin Apile 14x89.ap8p

------------------------------------------------------------------------
                     Time and Date of Analysis
------------------------------------------------------------------------

          Date:  June 03, 2020     Time:  13:58:42


1
     ********************
     * INPUT INFORMATION *
     ********************



     Hamlin Bridge

     DESIGNER   : B.Cardali

     JOB NUMBER : 09.0026052.00
```



```
     METHOD FOR UNIT LOAD TRANSFERS :

     - USACE (U.S. Army Corps of Engineers)
       Unfactored Unit Side Friction and Unit Side Resistance are used.



     COMPUTATION METHOD(S) FOR PILE CAPACITY :

     - USACE (U.S. Army Corps of Engineers)
       # Critical Depth Method for Sand:
         10 to 20 Pile Diameter based on the Density
       Use Long Pile Option


     TYPE OF LOADING :
     - COMPRESSION


     PILE TYPE :

     H-Pile/Steel Pile


     DATA FOR AXIAL STIFFNESS :

     - MODULUS OF ELASTICITY     = 0.290E+08 PSI
     - CROSS SECTION AREA        =    202.00 IN2


     NONCIRCULAR PILE PROPERTIES :

     - TOTAL PILE LENGTH, TL     =     37.00 FT.
     - BATTER ANGLE              =      0.00 DEG
     - PILE STICKUP LENGTH, PSL  =      2.00 FT.
     - ZERO FRICTION LENGTH, ZFL =      0.00 FT.
     - PERIMETER OF PILE         =     57.00 IN.
     - TIP AREA OF PILE          =    202.00 IN2
     - INCREMENT OF PILE LENGTH
       USED IN COMPUTATION       =      1.00 FT.



     SOIL INFORMATIONS :
```

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	1.25	63.00	33.00	0.00
5.00	SAND	1.25	63.00	33.00	0.00
5.00	SAND	1.25	68.00	40.00	0.00
40.00	SAND	1.25	68.00	40.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00
0.10E+08*	0.10E+08*	0.00	0.00	0.00	0.00	0.00

* MAXIMUM UNIT FRICTION AND/OR MAXIMUM UNIT BEARING WERE SET TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO LIMIT THE COMPUTED DATA.

DEPTH FT.	LRFD FACTOR ON UNIT FRICTION	LRFD FACTOR ON UNIT BEARING
0.00	1.000	1.000
5.00	1.000	1.000
5.00	1.000	1.000
40.00	1.000	1.000

1

****
* COMPUTATION RESULT *
****

****
* ARMY CORPS METHOD *
****

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.8	1.8
1.00	0.1	3.4	3.5
2.00	0.4	6.1	6.5
3.00	0.9	11.1	12.0
4.00	1.7	20.2	21.8
5.00	2.6	30.4	33.0
6.00	3.9	41.8	45.7
7.00	5.6	54.4	60.0
8.00	7.6	64.1	71.7
9.00	9.9	72.5	82.4
10.00	12.5	80.9	93.4
11.00	15.4	89.3	104.6
12.00	18.5	97.6	116.2
13.00	22.0	106.0	128.0
14.00	25.7	114.4	140.1
15.00	29.7	122.8	152.5
16.00	33.9	131.2	165.2
17.00	38.5	139.6	178.1
18.00	43.4	148.0	191.4
19.00	48.5	156.4	204.9
20.00	53.9	164.8	218.7
21.00	59.6	173.2	232.8
22.00	65.6	181.6	247.1
23.00	71.8	190.0	261.8
24.00	78.4	198.4	276.7
25.00	85.2	206.8	291.9
26.00	92.3	215.2	307.4
27.00	99.7	223.6	323.2
28.00	107.3	232.0	339.3
29.00	115.3	240.3	355.6
30.00	123.5	247.8	371.3
31.00	131.9	251.5	383.5
32.00	140.4	253.4	393.8
33.00	148.8	253.4	402.2
34.00	157.3	251.6	408.8
35.00	165.7	250.7	416.5

NOTES:

- AN ASTERISK IS PLACED IN THE END-BEARING COLUMN
  IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
  OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

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* COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
* CURVES FOR AXIAL LOADING                      *
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T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.0000E+00		
			0.0000E+00	0.0000E+00
			0.0000E+00	0.1000E-01
			0.0000E+00	0.2000E-01
			0.0000E+00	0.4000E-01
			0.0000E+00	0.6000E-01
			0.0000E+00	0.8000E-01
			0.0000E+00	0.1200E+00
			0.0000E+00	0.1600E+00
			0.0000E+00	0.5000E+00
			0.0000E+00	0.1000E+02
2	10	0.2525E+01		
			0.0000E+00	0.0000E+00
			0.3870E+00	0.1000E-01
			0.5132E+00	0.2000E-01
			0.6132E+00	0.4000E-01
			0.6557E+00	0.6000E-01
			0.6793E+00	0.8000E-01
			0.7047E+00	0.1200E+00
			0.7181E+00	0.1600E+00
			0.7470E+00	0.5000E+00
			0.7608E+00	0.1000E+02
3	10	0.4958E+01		
			0.0000E+00	0.0000E+00
			0.5156E+00	0.1000E-01
			0.7668E+00	0.2000E-01
			0.1014E+01	0.4000E-01
			0.1136E+01	0.6000E-01
			0.1208E+01	0.8000E-01
			0.1291E+01	0.1200E+00
			0.1337E+01	0.1600E+00

			0.1441E+01	0.5000E+00
			0.1492E+01	0.1000E+02
4	10	0.5000E+01		
			0.0000E+00	0.0000E+00
			0.6834E+00	0.1000E-01
			0.9406E+00	0.2000E-01
			0.1159E+01	0.4000E-01
			0.1255E+01	0.6000E-01
			0.1310E+01	0.8000E-01
			0.1370E+01	0.1200E+00
			0.1402E+01	0.1600E+00
			0.1472E+01	0.5000E+00
			0.1506E+01	0.1000E+02
5	10	0.2253E+02		
			0.0000E+00	0.0000E+00
			0.1100E+01	0.1000E-01
			0.1964E+01	0.2000E-01
			0.3234E+01	0.4000E-01
			0.4123E+01	0.6000E-01
			0.4780E+01	0.8000E-01
			0.5686E+01	0.1200E+00
			0.6282E+01	0.1600E+00
			0.7988E+01	0.5000E+00
			0.9091E+01	0.1000E+02
6	10	0.3996E+02		
			0.0000E+00	0.0000E+00
			0.1135E+01	0.1000E-01
			0.2079E+01	0.2000E-01
			0.3559E+01	0.4000E-01
			0.4666E+01	0.6000E-01
			0.5525E+01	0.8000E-01
			0.6772E+01	0.1200E+00
			0.7634E+01	0.1600E+00
			0.1031E+02	0.5000E+00
			0.1223E+02	0.1000E+02

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.2612E+01	0.1000E-03
0.1847E+02	0.5000E-02
0.2612E+02	0.1000E-01
0.5840E+02	0.5000E-01

0.8258E+02	0.1000E+00
0.1168E+03	0.2000E+00
0.1847E+03	0.5000E+00
0.2612E+03	0.1000E+01
0.3693E+03	0.2000E+01

LOAD VERSUS SETTLEMENT CURVE
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TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.2802E+01	0.3045E-03	0.2612E+01	0.1000E-03
0.6716E+01	0.1464E-02	0.5524E+01	0.1000E-02
0.2411E+02	0.6616E-02	0.1847E+02	0.5000E-02
0.3672E+02	0.1239E-01	0.2612E+02	0.1000E-01
0.9496E+02	0.5603E-01	0.5840E+02	0.5000E-01
0.1396E+03	0.1089E+00	0.8258E+02	0.1000E+00
0.3012E+03	0.5198E+00	0.1847E+03	0.5000E+00
0.3801E+03	0.1026E+01	0.2612E+03	0.1000E+01
0.4930E+03	0.2034E+01	0.3693E+03	0.2000E+01