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GEOTECHNICAL DESIGN REPORT WHITNEY BROOK BRIDGE NO. 2942 ROUTE 1 OVER WHITNEY BROOK BRIDGEWATER, MAINE

Prepared for:

Hoyle, Tanner & Associates, Inc.
Portsmouth, New Hampshire

September 2022
09.0026100.01

Prepared by:

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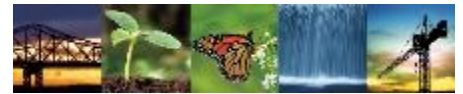
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September 27, 2022
File No. 09.0026100.01

Mr. Aaron Lachance, P.E.
Hoyle, Tanner & Associates, Inc.
100 International Drive, Suite 360
Portsmouth, New Hampshire 03801

Re: Geotechnical Design Report
Whitney Brook Bridge No. 2942, Route 1 over Whitney Brook
Maine Department of Transportation WIN 24783.00
Bridgewater, Maine

Dear Aaron:

We are pleased to provide this revised Geotechnical Design Report (GDR) to Hoyle, Tanner & Associates, Inc. (HTA) for the subject project. This report supersedes our September 16, 2022 report and has been revised based on comments received from HTA. Our work was completed in accordance with the Agreement for Professional Services between Hoyle, Tanner and GZA GeoEnvironmental, Inc. (GZA) dated February 28, 2022, and the *Limitations* included in **Appendix A** of this report. GZA is providing geotechnical engineering services as a Subconsultant to HTA, who is under contract with the Maine Department of Transportation for preliminary design of the proposed bridge.

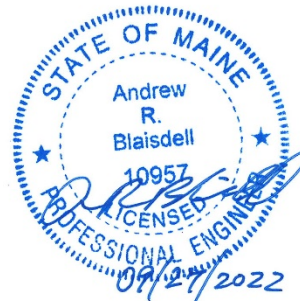
It has been a pleasure serving Hoyle, Tanner on this phase of the project, and we look forward to our continued work with you through project completion. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

Michael P. Smith, P.E. (MA)
Senior Project Manager

Christopher L. Snow, P.E.
Consultant Reviewer



Andrew R. Blaisdell, P.E.
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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) geotechnical evaluation for the subject project. Our services were completed in accordance with the Agreement for Professional Services between Hoyle, Tanner and Associates, Inc. (HTA) and GZA dated February 28, 2022. This report is subject to the Limitations included in **Appendix A**.

1.1 BACKGROUND

The project includes replacement of the Whitney Brook Bridge No. 2942 carrying Route 1 over Whitney Brook in Bridgewater, Maine, the location of which is shown on **Figure 1**. Built in 1923, the existing bridge is a 33-foot-long, single span structure consisting of a concrete T-beam superstructure supported by cast-in-place concrete abutments and wingwalls. It has been widened and/or rehabilitated several times during its service life. Available plans indicate the bridge is supported on spread footings bearing on soil at approximately El. 387, roughly 19 feet below existing pavement level and 7 feet below the riverbed level.

We understand that a bridge replacement is proposed consisting of a 66-foot-long, single-span, integral abutment bridge constructed on the existing alignment. Proposed roadway grades are within about 1 foot of existing pavement elevations. A temporary detour bridge is planned to the west of the existing bridge to carry two-way traffic throughout bridge construction.

1.2 OBJECTIVES AND SCOPE OF SERVICES

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

- Reviewed mapped surficial and bedrock geology of the site;
- Reviewed existing subsurface data;
- Reviewed rock core samples at MaineDOT's storage facility;
- Summarized subsurface explorations by others;
- Completed geotechnical evaluations for soil and bedrock properties; frost susceptibility and drainage of approach embankments; AASHTO load and resistance factors associated with geotechnical design elements; nominal resistance of pile foundations; 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 geotechnical design report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

2.1 PREVIOUS (1988) BORINGS

In 1988, the Maine Department of Transportation (MaineDOT) drilled two test borings, designated CM-2-88 and CM-3-88, to explore subsurface conditions for a bridge widening. The locations of borings CM-2-88 and CM-3-88 were estimated by GZA by measuring from features of the existing bridge shown on the 1988 boring location plan and are shown on **Figure 2**.



Therefore, the locations should be considered approximate. The borings were advanced into bedrock to depths ranging from approximately 21.5 to 30 feet below grade. The boring log sheet from the 1989 geotechnical report is included in **Appendix B**.

2.2 RECENT BORINGS

MaineDOT drilled two test borings, BB-BWB-101 and BB-BWB-102, on June 23 and 24, 2020. Bruce Wilder of MaineDOT logged the borings. The as-drilled boring locations and elevations were surveyed and provided to GZA by MaineDOT and are included on the logs in **Appendix C**.

BB-BWB-101 and BB-BWB-102 were located behind the existing south and north abutments, respectively, as shown on the Boring Location Plan, **Figure 2**. The test borings were drilled through the overburden soil and terminated approximately 9.7 to 10 feet into bedrock. Depths of borings ranged from approximately 38.7 to 45 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 overburden using a 24-inch-long, 1 3/8-inch inside-diameter sampler. The sampler was driven with a 140-lb calibrated automatic hammer with a 30-inch drop from a truck mounted drill rig. The hammer efficiency at the time of the borings was 0.886 as provided on the boring logs. Bedrock cores were obtained using NQ2 wire line coring equipment.

Drafts of the logs for BB-BWB-101 and -102 were prepared by MaineDOT and provided to GZA. GZA personnel visited MaineDOT's storage facility in Bangor, Maine to review, classify and photograph the rock core samples collected at the boring locations. The boring logs were revised based on GZA's review. The revised borings logs are provided in **Appendix C**. Photographic logs of the recovered rock core specimens are included in **Appendix D**.

3.0 LABORATORY TESTING

Laboratory testing was conducted by MaineDOT on split-spoon soil samples retrieved during the 2020 investigation. The testing program consisted of gradation analysis / AASHTO Classification / Frost Classification/water content assessments of six samples, and one loss on ignition test. Results of the testing are included in **Appendix E**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

Based on available geologic mapping¹, the surficial units in the vicinity of the bridge consist of thin drift (glacial till) and patchy gravel deposits. Generally, gravel deposits consist of thin, poorly-sorted deposits with little internal structure, including ice contact deposits such as eskers and kames found in the southern part of the Bridgewater quadrangle, and post-glacial alluvial deposits on the Meduxnekeag River.

Bedrock in the vicinity of the site is mapped² as the Carys Mills Formation, primarily consisting of Silurian-Ordovician metamorphosed pelitic rocks, silty limestones, and pelitic shales.

¹ Neil, Craig D. (compiler) , 2002, Surficial materials of the Bridgewater quadrangle, Maine: Maine Geological Survey, Open-File Map 02-70, map, scale 1:24,000. Maine Geological Survey Maps. 1105. http://digitalmaine.com/mgs_maps/1105

² Horton, J.D., San Juan, C.A., and Stoeser, D.B., 2017, The State Geologic Map Compilation (SGMC) geodatabase of the conterminous United States (ver. 1.1, August 2017): U.S. Geological Survey Data Series 1052, 46 p., <https://doi.org/10.3133/ds1052>.



4.2 SUBSURFACE PROFILE

4.2.1 Soil

Four soil units were encountered beneath 3 to 3.5 inches of asphalt pavement and above bedrock at the site: Fill, Relic Topsoil/Alluvium, Glacial Stream Deposits, and Glacial Till. The approximate thicknesses and generalized descriptions of the subsurface units are presented in the table below, 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 and C**.

GENERALIZED SUBSURFACE CONDITIONS		
Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	8.7 to 13.7	Brown, medium dense to dense, Gravelly fine to coarse SAND, trace to some silt, with occasional cobbles. (USCS: SP, SM). MaineDOT Frost Classification = II <i>Encountered in both borings.</i>
Relic Topsoil/Alluvium	5.5	Grey, medium dense, fine to coarse SAND, little gravel, little silt, with wood and organics. (USCS: SM) <i>Encountered in BB-BWB-101</i>
Glacial Stream Deposit	5 to 14.2	Brown, medium dense to dense, Gravelly, fine to coarse SAND, little to trace silt. Occasional cobbles were observed in <i>BB-BWB-101</i> . (USCS: SW-SM, SM). <i>Encountered in boring BB-BWB-101 beneath Relic Topsoil and in BB-BWB-102 directly beneath Fill.</i>
Glacial Till	16.3	Variable <u>from</u> Grey, hard, SILT, some sand, little gravel; <u>to</u> gray, very dense, Gravelly, fine to coarse SAND, little silt, occasional cobbles. (USCS: ML, SM). <i>Encountered in BB-BWB-102.</i>
Estimated Top of Bedrock	Abutment 1: Approx. El. 375.7 to El. 381.2 (24 to 29 feet bgs) Abutment 2: Approx. El. 369.1 to El. 372.8 (32 to 35 feet bgs)	

4.2.2 Bedrock

Bedrock was cored in both BB-BWB-100 series borings and was described as hard, fresh to slightly weathered, fine grained to aphanitic, grey, CALCAREOUS PELITE with calcite stringers/veins. Primary joint sets are extremely close to moderately spaced, high angle to vertical, rough to smooth, undulating to planar, fresh to discolored, very tight to open. Secondary joint sets are close to moderately spaced, horizontal to low angle, planar to stepped, rough to smooth, fresh, very tight to open. The Rock Quality Designation (RQD) in the core runs ranged from 0 to 49 percent.

4.2.3 Groundwater

Groundwater depth was measured in boring BB-BWB-101 and BB-BWB-102 at approximately 14 and 15 feet bgs, corresponding to El. 390.4 and 389.7, respectively. Groundwater levels in the borings were measured during or immediately after drilling and may have been affected by drilling procedures, which included introduction of water for drilling purposes. The groundwater observations were made at the times and under the conditions stated in the boring logs. Fluctuations in groundwater level occur due to variations in season, precipitation, river levels and construction activities in the area. Consequently, water levels during construction are likely to vary from those encountered at the time the observations were made.



5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in accordance with *2020 AASHTO LRFD Bridge Design Specifications, 9th Edition* (herein designated as AASHTO) and the *MaineDOT Bridge Design Guide, 2003 Edition*, with updates through 2018 (MaineDOT BDG). The sections that follow describe the evaluations and the geotechnical basis for each element. Supporting calculations developed by GZA for the project are attached in **Appendix F** of this report.

5.2 EVALUATION OF FOUNDATIONS

We understand the proposed bridge replacement will consist of an integral abutment bridge. In Maine, driven H-piles are the most conventional and cost-effective pile foundation type installed to support integral abutments. Given the depth to bedrock and the nature of the overburden soils, we anticipate that the piles will be supported predominantly in end bearing on bedrock. Design considerations and evaluations are presented below.

5.3 PILE DESIGN CONSIDERATIONS

5.3.1 Pile Type and Loading

Seven (7) steel HP14x73 piles are proposed to support each abutment. HTA provided a maximum factored structural load of 295 kips per pile and an imposed pile head deflection of 0.625 inches (fixed head, no rotation). The required nominal axial resistance is 454 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 to assess nominal geotechnical pile resistance in accordance with AASHTO requirements.

5.3.2 Load and Resistance Factors

Piles should be driven to achieve the required nominal geotechnical resistance of the piles. In GZA's experience for piles gaining a significant portion of their geotechnical resistance on 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) 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.

Since the piles will be subject to lateral loading, the 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 $\phi_{cc}=0.7$ and the flexural resistance factor $\phi_f=1.0$ should be applied to the combined axial and flexural resistance of the pile in the interaction equation (AASHTO LRFD Eq. 6.9.2.2-2) for the upper pile segment. Structural resistance of the lower segment should be checked at the strength limit state considering a resistance factor $\phi_c=0.50$, per AASHTO LRFD Article 10.7.3.2.3 for hard driving conditions versus $\phi_{cc}=0.7$ since the pile is pinned and lateral fixity is not obtained (see Section 5.2.5 for details).

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV), earth surcharge (ES), live load surcharge (LS) loads, and components and attachments (DC) loads using the load factors for permanent loads (γ_p) provided in LRFD Table 3.4.1-2 for strength limit state foundation design. A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall 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.



5.3.3 Pile Resistance Distribution and Settlement

Based on our experience with similar soils, we anticipate that the proposed H-piles will be driven to refusal on or near to the top of bedrock to achieve the required axial geotechnical resistance. The axial geotechnical resistance of piles was estimated using the computer analytical software *APile* by Ensoft based on the Nordlund method. Based on the proposed bottom of integral backwall elevation (approximately El. 395) and the range in encountered bedrock elevations, pile lengths of 14 to 20 feet are anticipated at Abutment 1 and lengths of 19 to 26 feet are anticipated at Abutment 2 for piles driven to bedrock. The results indicate that the piles will gain on the order of 24 to 50 kips side resistance when driven to the top of bedrock and that the remainder of the nominal geotechnical resistance will be developed in bearing on bedrock. Therefore, the mobilized side friction during installation is anticipated to be about 10 percent of the nominal geotechnical resistance. Since the piles will gain support largely in end bearing, there is no reduction for group interaction in axial compression. We understand HTA performed a cursory wind load evaluation on the piles and determined that uplift will not occur in the piles; therefore, axial tensile geotechnical (uplift) resistance was not evaluated.

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 1/2 inch; the majority of which will occur as the bridge is constructed.

5.3.4 Wave Equation Analysis

GZA completed preliminary wave equation analyses to assess the drivability of an ASTM A572 Grade 50 Steel HP14x73 pile with a nominal geotechnical resistance of 454 kips. Analyses were completed using a Delmag D22-23 diesel hammer operating at its lowest of four fuel settings, with a ram weight of 4,850 pounds and a maximum rated energy of 51,216 foot-pounds (ft-lbs).

The analyzed pile lengths anticipate that the piles would be driven to the top of rock elevation at each abutment. The results are summarized below.

Pile Location and Type	Pile Length (ft)	Pile Penetration (ft)	Nominal Resistance = 454 kips			
			Compressive Driving Stress ¹ (ksi)	Transferred Energy ² (kip-ft)	Stoke (feet)	Final Penetration Resistance (blows per inch)
Abutment 1	19	14	36	12.5	6.3	7
Abutment 2	24	19	32	12.9	6.3	7

Notes:

1. The maximum allowable driving stress is 45 ksi for Grade 50 steel.
2. Hammer was operated on Fuel Setting 4 (800 psi [72%]).

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. A number of commonly available diesel pile hammers are available that can provide similar rated energy to the hammer assumed for preliminary analyses. Construction-phase drivability analyses should be conducted for the hammer(s) proposed by the contractor.

Results of the preliminary wave equation analyses are provided in **Appendix F**.



5.3.5 Lateral Pile Analysis

GZA developed the soil profile tabulated below based on the soil conditions encountered in the test borings and laboratory testing results. The analysis was performed for Abutment 1 where the highest bedrock is anticipated to be encountered. The pile was an ASTM A572 Grade 50 steel HP14x73 pile oriented for weak-axis bending in the longitudinal direction.

L-PILE® INPUT PARAMETERS						
ABUTMENT 1, TYPICAL SHORTEST PILE LENGTH APPROX. 14 FT						
Stratum	Soil Model	Top of Layer Elevation (ft-NAVD 88)	Layer Thickness (ft)	k (pci) / E50	φ' (deg)/ Su (psf)	γ_e (pcf)
Fill/Relic Topsoil	Reese Sand	395	5.0	105	33	125
Glacial Stream Deposit**	Reese Sand	390	5.0	35	30	58
Glacial Till	Reese Sand	385	4.0	145	40	68
Top of Bedrock	--	381	--	--	--	--

Notes:

1. Soil strata were modelled after boring BB-BWB-101.
2. ** Indicates the top of layer is the approximate ground water elevation based on the boring logs.
3. pci = pounds per cubic inch, deg = degrees, psf = pounds per square foot, γ_T = total unit weight (used above anticipated groundwater level), γ_e = effective unit weight (used below anticipated groundwater), pcf = pounds per square foot.

GZA completed a preliminary lateral pile analysis using *LPile* to assess pile fixity. Initial analysis used pile head boundary conditions and loads provided by HTA, including a factored axial load of 290 kips, thermal deflection of 0.625 inches, and a fixed-head condition (zero slope condition). The pile tip did not achieve a fixed condition, but with 0.047 inches of tip movement, the pile was interpreted to have achieved a “pinned” condition. Therefore, we conclude that the effect of adding shear resistance at the pile tip to account resistance between the pile tip and the rock surface will result in sufficient lateral support for the driven piles to adequately support the integral abutments. This methodology is described in *MaineDOT Technical Report ME 01-7 (ME 01-7)*, which allows a maximum theoretical resistance of 70 percent of the applied load. Lateral pile input parameters are included in **Appendix F**. We note that our preliminary analysis considered the anticipated shortest pile in the group, while the average length is more likely on the order of 17 feet at Abutment 1. In addition, the use of rock injector tips in the low RQD, high angle to vertically jointed bedrock will likely achieve some penetration in bedrock which will further enhance the pinned condition.

We understand that HTA has completed final lateral pile evaluations using the *LPile* input parameters presented above with final loads and thermal deflections as well structural resistance calculations. The structural resistance should be checked based on the provisions of Article 6 of AASHTO and as per the recommendations of Section 5.4.2.4 of the MaineDOT BDG.

5.4 SEISMIC DESIGN CONSIDERATIONS

Seismic site class was determined in general accordance with LRFD Table C3.10.3.1 using the average SPT N-value of the soil materials encountered in the borings. The subsurface profile for seismic design includes the approach fills (including backfill behind abutments) and natural soils overlying bedrock. SPT N-values generally range between 20 and 60 blows per foot, with only one value less than 15. Based on Table C3.10.3.1 of LRFD, we conclude the bridge should be classified as Site Class D.

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



5.5 ADDITIONAL FOUNDATION CONSIDERATIONS

5.5.1 Frost Protection

Based on the MaineDOT BDG, Section 5.2.1, the Freezing Index for the site is 2400, and with the assumption of low-moisture content (+-10 percent) soils, the estimated depth of frost penetration is approximately 7.5 feet.

Granular fill soils encountered near the surface at the abutments typically were classified as AASHTO A-1-a and A-1-b with MaineDOT Frost Classification from 0 to II, indicating they are considered to exhibit low to moderate frost susceptibility. These materials are judged to be suitable for continued use beneath the approach roadway after reconstruction provided proper drainage of the pavement section is provided.

5.5.2 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 Granular Borrow for Underwater Backfill, BDG Type 4 soil.

Based on the estimated thermal bridge expansion of approximately 0.625 inches and the abutment height of 9.97 feet, the calculated abutment rotation is 0.0052 feet/foot. GZA evaluated passive earth pressure coefficient using Massachusetts DOT LRFD Bridge Design Manual, Section 3.10.8. We recommend using a Coefficient of Passive Earth Pressure, $K_p = 4.02$ for design of backwalls and wingwalls.

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

5.5.3 Gradation for Scour Analysis

Available laboratory data for the soil at the approximate elevation of the integral abutments was evaluated to estimate D_{50} values, as summarized in the table below. Full laboratory testing results are included in **Appendix E**.

Boring and Sample ID Number	Depth (feet)	W.C. (%)	D_{50} (mm)	Classification	
				USCS	AASHTO
BB-BWB-101, 3D	15.0-17.0	13.8	7.27	SW-SM	A-1-a
BB-BWB-101, 4D	20.0-22.0	9.1	11.00	SW-SM	A-1-a
BB-BWB-102, 3D	15.0-17.0	12.2	2.86	SM	A-1a
BB-BWB-102, 4D	20.0-22.0	18.7	+/-0.03*	CL	A-4

*Hydrometer not performed, D_{50} approximated by straight line extrapolation.

6.0 RECOMMENDATIONS

6.1 EMBANKMENT DESIGN CONSIDERATIONS

Embankment side slopes should be designed with MaineDOT-typical slope angles of 2H:1V or flatter. Slopes should be provided with loam and seed for permanent erosion protection. If steeper slopes are planned at the abutment approaches, 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 Whitney Brook to protect from scour.



6.2 SEISMIC DESIGN

The peak ground acceleration coefficient, short- and long-period spectral acceleration coefficients were interpolated from the AASHTO design guide maps (3.10.2.1-1 through -3 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years are summarized for the site are as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s (Period = 0.0 sec)	0.115 g
SDs (Period = 0.2 sec)	0.262 g
SD1 (Period = 1.0 sec)	0.122 g

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

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill behind new abutments and wingwalls should consist of MaineDOT 703.19 Granular Borrow for Underwater Backfill, MaineDOT BDG Type 4 soil. Recommended soil properties for Type 4 soils are as follows:
 - Internal Friction Angle of Soil = 32°
 - Soil Total Unit Weight = 125 pcf
 - Coefficient of Passive Earth Pressure, $K_p = 4.02$ (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.

6.4 RECOMMENDATIONS FOR FOUNDATIONS

6.4.1 Pile Design

- The proposed abutments may be supported on HP14x73 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed predominantly through end-bearing on or near the bedrock surface.
- To promote penetration into bedrock and limit driving damage, the steel H-piles should be fitted with HP Rock Injector pile tips (Associated Pile and Fitting HP-80500 Pile Point or similar), which have a chisel-like point intended to limit the potential for the pile to slide on a sloping rock surface.



- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 454 kips, calculated by dividing the maximum factored pile load of 295 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the ASTM A572 Grade 50 HP14x73 piles can be driven to a nominal resistance of 454 kips using a diesel hammer with a rated energy of approximately 50,000 ft-lbs without exceeding the allowable driving stress of 45 ksi (0.9F_y), with a final penetration resistance within the MaineDOT preferred range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the top of bedrock elevations encountered in the borings (approximately El. 376 at Abutment 1 and approximately El. 369 at Abutment 2), which are deeper than the 1988 borings and likely represent the longest pile lengths. 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 one pile in each substructure be dynamically tested at the end of initial drive to assess driving stress and establish the penetration resistance criteria to achieve the required nominal resistance for the production piles. The plans should also require a restrrike test on each test pile approximately 24 hours after initial drive.
- Piles shall be spliced in accordance with MaineDOT Standard Specification Section 501.047.

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 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 restrrike test on each test pile, to assess potential relaxation.

7.2 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavation will be required to about 10 feet below roadway level to construct the integral backwalls. This is about 2 feet above the bottom of brook level, and 2 feet below Q1.1 water level in Whitney Brook. A bridge closure is planned during construction. Based on these conditions, we anticipate that temporary excavation support may not be required, and sloped open cuts may be feasible to the required maximum depths.

Temporary cofferdams constructed with stacked supersacks/sandbags are anticipated to be necessary to control groundwater inflow into the excavations from the brook. It may also be feasible to leave portions of the existing abutments in-place during backwall construction to cut off brook flow from the work area. If water levels in the brook extend above El. 395, additional protective measures may be needed, such as sheet piles, to allow construction of the backwall in the dry.



The plans call for removal of existing abutments below new riprap, which extends to approximately El. 390; corresponding to about 5 feet below Q1.1 water level in Whitney Brook. Temporary cofferdams constructed with stacked supersacks/sandbags, porta-dam or similar systems are anticipated to be feasible to control groundwater inflow in order to demolish the abutments and place new riprap. The contractor should be responsible for selection of the cofferdam type, controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade.

It is anticipated that inflow into the excavation can be handled by open pumping from sumps installed at the bottoms of excavations. In order to expedite dewatering and provide a stable working surface it may be desirable to over-excavate at least 12 inches below the bottom of abutment level and replace with a crushed stone working mat placed on a layer of nonwoven geotextile. Sumps should be fitted with geotextile or sand filters to prevent loss of subgrade fines during pumping. The method of dewatering will depend on the type of cofferdam, size of the open excavation, the length of time the excavation is left open, and the time of year, particularly with regard to stream levels. Dewatering discharge should be managed in accordance with MaineDOT Best Management Practices.

7.3 REUSE OF ON-SITE MATERIALS

Based on the test boring and laboratory testing results, the soil typically had more than 10 percent passing the No. 200 sieve. Consequently, the fill typically does not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and would not be suitable for use as structural backfill. The material is considered suitable for use as Common Borrow.

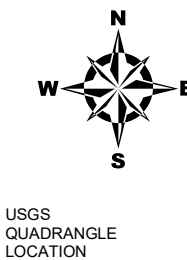
If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution, and moisture content. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.



FIGURES



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SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCGIS 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.

Data Supplied by :



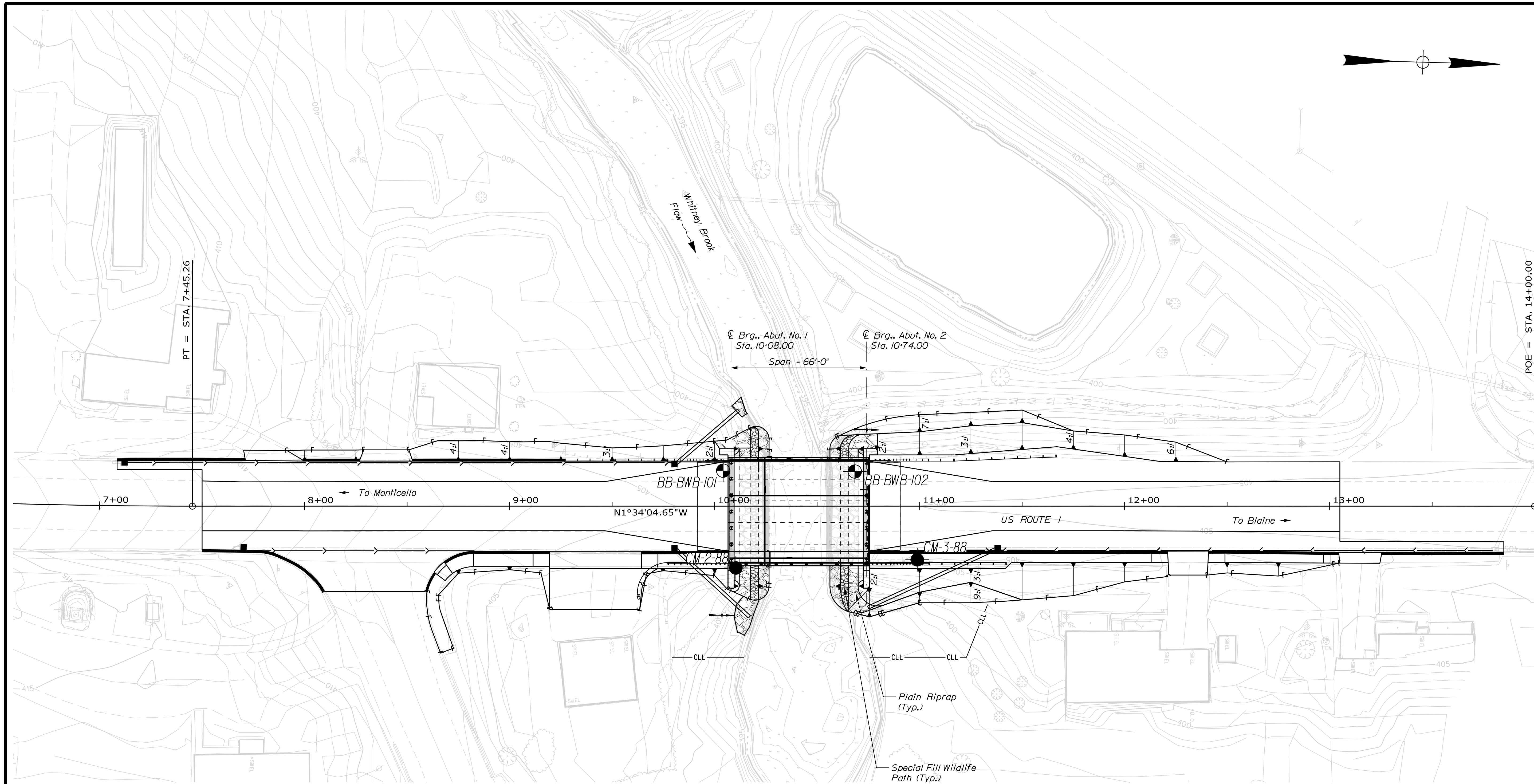
PROJ. MGR.: ARB
 DESIGNED BY: LCN
 REVIEWED BY: ARB
 OPERATOR: ADM
 DATE: 07-26-2021

LOCUS PLAN

**WHITNEY BROOK BRIDGE #2942
 BRIDGEWATER, MAINE**

JOB NO.
 09.0026100.01

FIGURE NO.
1

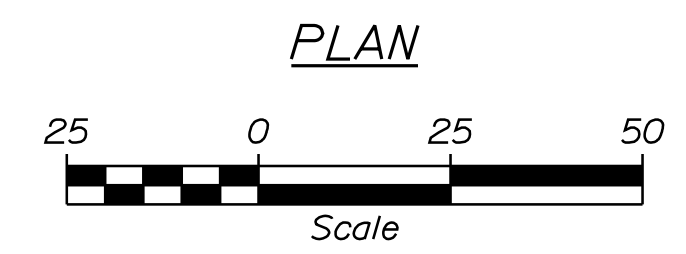


NOTES

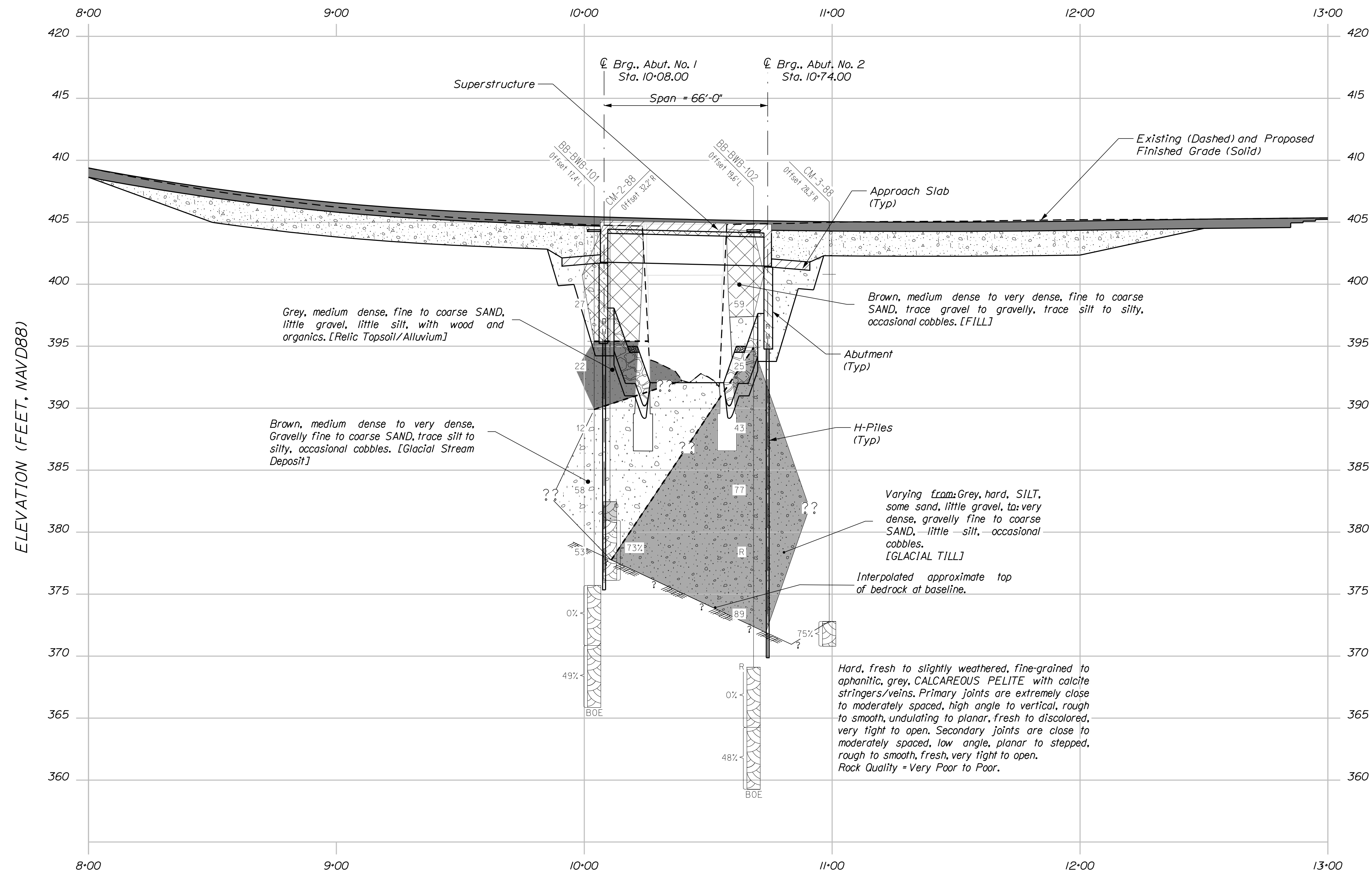
- 1) Base map developed from electronic files (Bridge.dwg, Topo.dwg, Text.dwg, Contours.dwg, Alignments.dwg, z_abutment_2.dwg, z_superstructure.dwg, and z_Existing.dwg) provided by Hoyle, Tanner & Associates on June 30, 2022.
- 2) The as-drilled locations of the 100-Series test borings were surveyed by MaineDOT and developed from test borings logs provided by MaineDOT on September 23, 2020.
- 3) The locations of the 1988 borings were estimated by measuring test borings from the 1988 boring location plan to existing bridge structural elements and should be considered approximate.

BORING LOCATION PLAN LEGEND

- BB-BWB-102 Location and designation of 100-Series cased wash boring
- CM-3-88 Approximate location and designation of historic explorations provided in 1989 Geotechnical Report



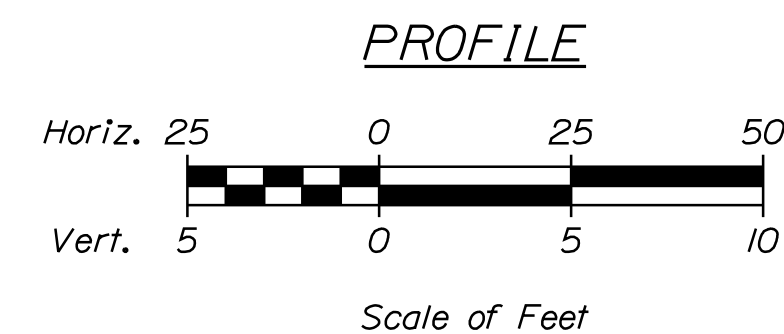
STATE OF MAINE DEPARTMENT OF TRANSPORTATION 24783.01	BRIDGE NO. 2942 WIN 24783.01 BRIDGE PLANS
WHITNEY BROOK BRIDGE WHITNEY BROOK AROOSTOOK COUNTY BRIDGEWATER	BORING LOCATION PLAN SHEET NUMBER <div style="font-size: 2em; font-weight: bold; text-align: center;">2</div>
PREPARED BY: <div style="text-align: right;"> </div>	DATE: JULY 2022 BY: ENT ABB J. STEINSON DESIGN-DETAILED: MPS CHECKED-REVIEWED: CLS DESIGN-DETAILED: CLS REVISIONS: 1 REVISIONS: 2 REVISIONS: 3 REVISIONS: 4 FIELD CHANGES
SIGNATURE: _____ P.E. NUMBER: _____ DATE: _____	



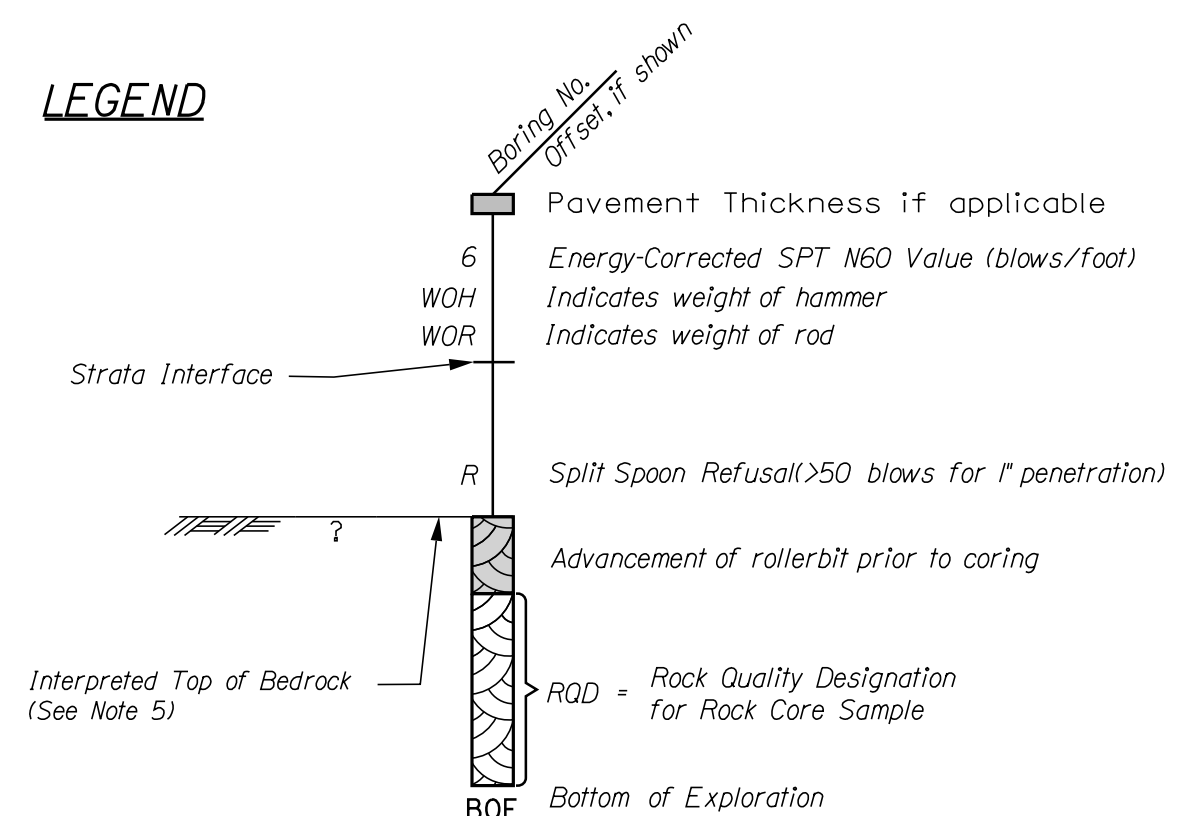
NOTES

- 1) Base map developed from electronic files provided by Hoyle, Tanner & Associates, Inc. on May 18, 2022. Files include 003_Prelim Profile.dgn.
- 2) The as-drilled boring locations were surveyed by a MaineDOT survey crew and provided to GZA in an electronic file (Borings.dgn).
- 3) BB-BWB-100 series borings were performed and logged by MaineDOT personnel between June 23 and 24, 2020. Soil and bedrock samples were reviewed by GZA at the MaineDOT storage facility in May 2022.
- 4) The CM-series borings were conducted by MaineDOT in 1988.

5) 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 rock transitions may vary and are probably more erratic. Boring data are shown for borings drilled off alignment, but interpreted strata are based on the borings drilled closest to the project baseline. For more specific information refer to the exploration logs.



LEGEND



PREPARED BY:

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	24783.01 WIN 24783.01 BRIDGE NO. 2942 BRIDGE PLANS																																													
WHITNEY BROOK BRIDGE WHITNEY BROOK ARROOSTOOK COUNTY BRIDGEWATER	INTERPRETIVE SUBSURFACE PROFILE																																													
<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <th>PROJ. MANAGER</th> <th>J. STEINSON</th> <th>DATE</th> <th>DATE</th> </tr> <tr> <td>DESIGN-DETAILED</td> <td>MPS</td> <td>ENT</td> <td>JULY 2022</td> </tr> <tr> <td>CHECKED-REVIEWED</td> <td>CLS</td> <td>ARB</td> <td></td> </tr> <tr> <td>DESIGN-DETAILED</td> <td></td> <td></td> <td></td> </tr> <tr> <td>REVISIONS 1</td> <td></td> <td></td> <td></td> </tr> <tr> <td>REVISIONS 2</td> <td></td> <td></td> <td></td> </tr> <tr> <td>REVISIONS 3</td> <td></td> <td></td> <td></td> </tr> <tr> <td>REVISIONS 4</td> <td></td> <td></td> <td></td> </tr> <tr> <td colspan="4" style="text-align: center;">FIELD CHANGES</td> </tr> </table>	PROJ. MANAGER	J. STEINSON	DATE	DATE	DESIGN-DETAILED	MPS	ENT	JULY 2022	CHECKED-REVIEWED	CLS	ARB		DESIGN-DETAILED				REVISIONS 1				REVISIONS 2				REVISIONS 3				REVISIONS 4				FIELD CHANGES				<table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td>SIGNATURE</td> <td>P.E. NUMBER</td> <td>DATE</td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> <tr> <td> </td> <td> </td> <td> </td> </tr> </table>	SIGNATURE	P.E. NUMBER	DATE						
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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

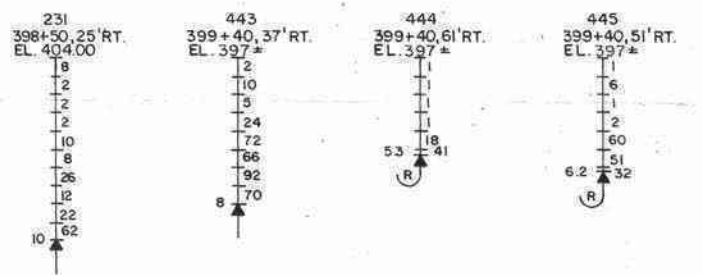
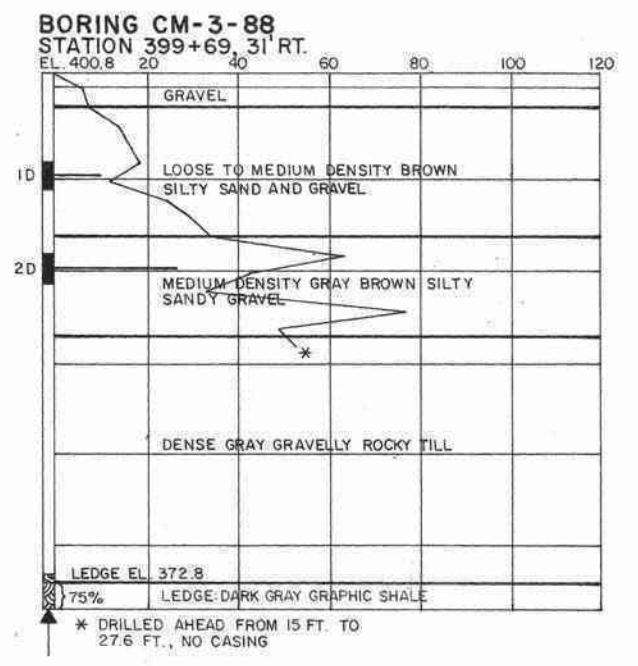
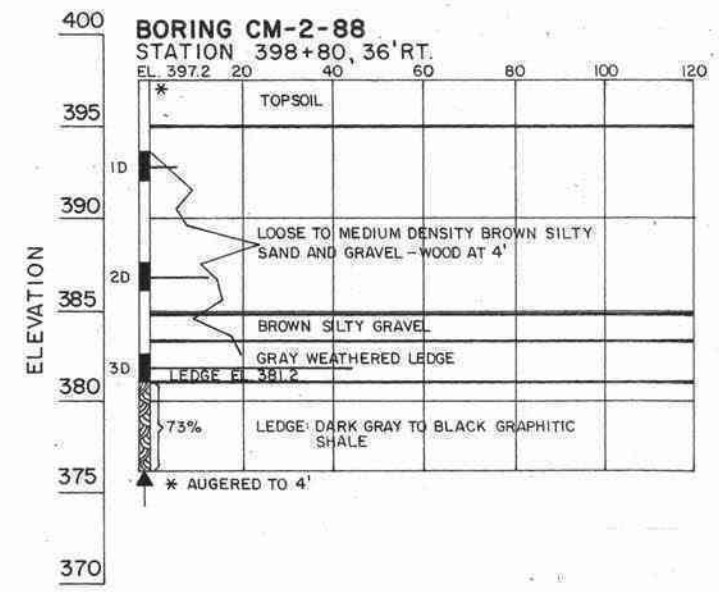
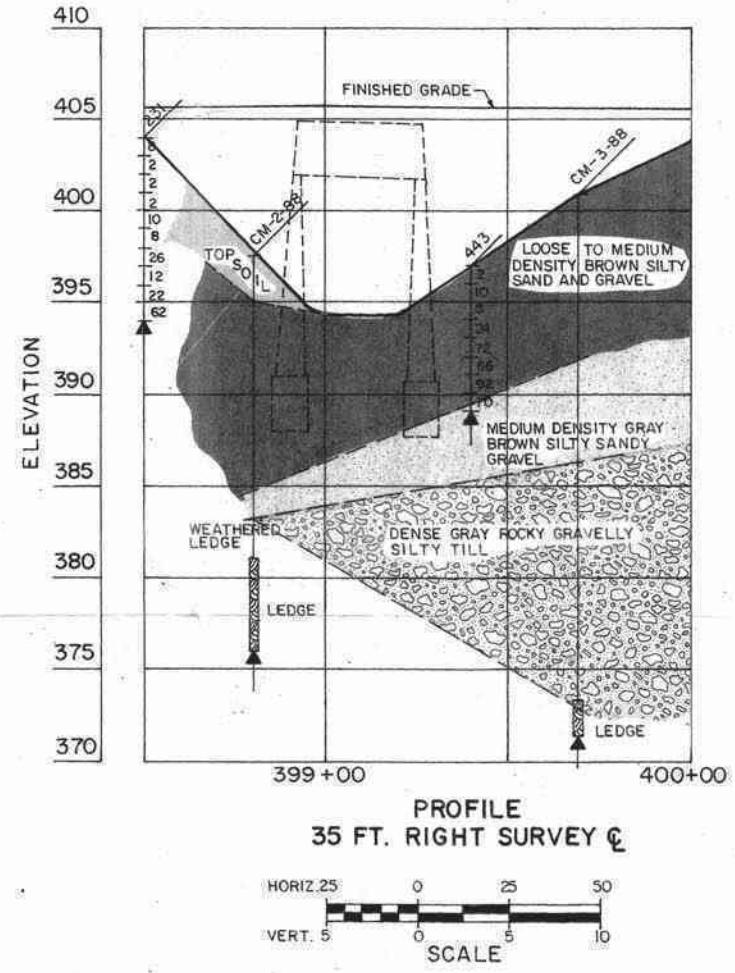
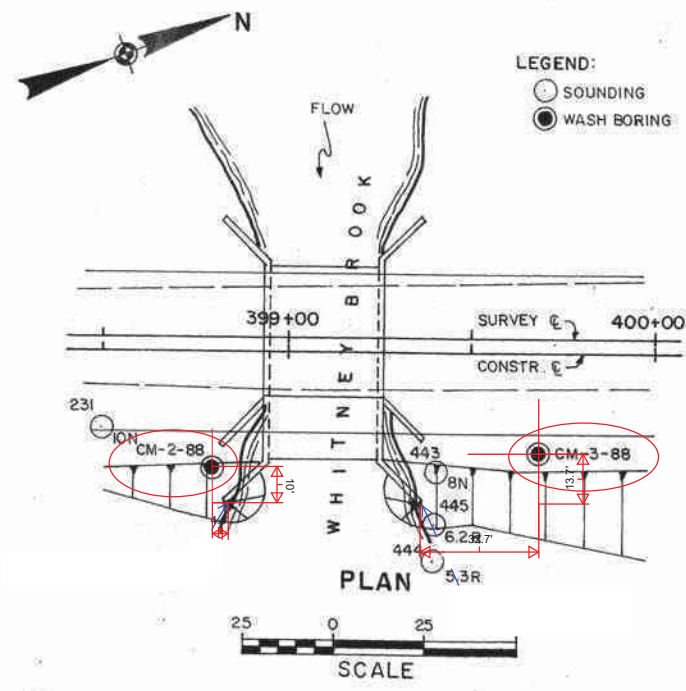
10. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

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 – PREVIOUS (1988) TEST BORING LOG SHEET



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WHITNEY BROOK BRIDGE
over
WHITNEY BROOK
in the town of
BRIDGEWATER
AROOSTOOK COUNTY
FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE



APPENDIX C – RECENT TEST BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																																																									
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES																																																									
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.			<u>Descriptive Term</u> trace 0 - 10 little 11 - 20 some 21 - 35 adjective (e.g. Sandy, Clayey) 36 - 50																																																						
		(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.			TERMS DESCRIBING DENSITY/CONSISTENCY <u>Coarse-grained soils</u> (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). <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr><td>Very loose</td><td>0 - 4</td></tr> <tr><td>Loose</td><td>5 - 10</td></tr> <tr><td>Medium Dense</td><td>11 - 30</td></tr> <tr><td>Dense</td><td>31 - 50</td></tr> <tr><td>Very Dense</td><td>> 50</td></tr> </tbody> </table> <u>Fine-grained soils</u> (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. <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value (blows per foot)</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr> <tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr> <tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr> <tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr> <tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr> <tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr> </tbody> </table> <u>Rock Quality Designation (RQD):</u> RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core}^*}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) <table border="1"> <thead> <tr> <th>Rock Quality</th> <th>RQD (%)</th> </tr> </thead> <tbody> <tr><td>Very Poor</td><td>≤25</td></tr> <tr><td>Poor</td><td>26 - 50</td></tr> <tr><td>Fair</td><td>51 - 75</td></tr> <tr><td>Good</td><td>76 - 90</td></tr> <tr><td>Excellent</td><td>91 - 100</td></tr> </tbody> </table> <u>Desired Rock Observations (in this order, if applicable):</u> 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))				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	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	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100
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SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.																																																											
	CH	Inorganic clays of high plasticity, fat clays.																																																											
	OH	Organic clays of medium to high plasticity, organic silts.																																																											
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.																																																											
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level																																																													
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information				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: Whitney Brook Bridge #2942 carries U.S. Route 1 over Whitney Brook
Location: Bridgewater, Maine

Boring No.: BB-BWB-101

WIN: 24783.00

Driller: MaineDOT	Elevation (ft.): 404.4	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Westrack	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B.Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2020; 06:30-11:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: 10+69, 16.8 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 14.0 ft bgs.

Hammer Efficiency Factor: 0.886

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
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							404.1	SSA	3 1/2" HMA.		
5	1D	24/12	5.0 - 7.0	9/7/11/15	18	27			Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt, occasional small cobbles, (Fill).		
10	2D	24/16	10.0 - 12.0	12/9/6/7	15	22	395.4	18	Grey, wet, medium dense, fine to coarse SAND, little gravel, little silt, wood, little organics, (Relic Topsoil).	#336979 Loss of Ignition 23.7%	
15	3D	24/10	15.0 - 17.0	7/4/4/6	8	12	389.9	17	Brown, wet, medium dense, Gravelly, fine to coarse SAND, trace silt, (Glacial Stream Deposit).	G#336980 A-1-a, SW-SM WC=13.8%	
20	4D	24/13	20.0 - 22.0	15/23/16/20	39	58	385.4	17	Brown, wet, very dense, Gravelly, fine to coarse SAND, trace silt, occasional cobble, (Glacial Stream Deposit).	G#336981 A-1-a, SW-SM WC=9.1%	
25								63	Roller Coned ahead to 25.0 ft bgs.		

Remarks:

N 943338.6120 E 2304521.6451

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: Whitney Brook Bridge #2942 carries U.S. Route 1 over Whitney Brook
Location: Bridgewater, Maine

Boring No.: BB-BWB-101

WIN: 24783.00

Driller: MaineDOT	Elevation (ft.): 404.4	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Westrack	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B.Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2020; 06:30-11:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: 10+69, 16.8 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 14.0 ft bgs.
Hammer Efficiency Factor: 0.886	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

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
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	5D	24/17	25.0 - 27.0	17/15/21/26	36	53	26			Brown, wet, very dense, Silty fine to coarse SAND, some gravel, (Glacial Stream Deposit).		
							56			Roller Coned ahead to 28.7 ft bgs.		
							102					
	R1	60/43	28.7 - 33.7	RQD = 0%			^a 106	375.7		≈106 blows for 0.7 ft.		
							NQ-2			Top of Bedrock at Elev. 375.7 ft.		
30										R1: Hard, fresh to slightly weathered, aphanitic to fine grained, grey, CALCAREOUS PELITE, with calcite veins. Primary joints are extremely close to close, high angle to vertical, planar to undulating, smooth to rough, fresh to discolored, tight to partially open. Secondary joints are close, low angle, planar, rough to smooth, fresh, tight to partially open.		
										[Carys Mill Formation]		
										Rock Quality = Very Poor		
										Recovery = 72%		
										R1: Core Times (min:sec)		
										28.7-29.7 ft (2:24)		
										29.7-30.7 ft (3:50)		
										30.7-31.7 ft (2:16)		
										31.7-32.7 ft (2:58)		
										32.7-3.7 ft (3:00)		
											33.7	
										R2: Hard, fresh to slightly weathered, aphanitic to fine grained, grey, CALCAREOUS PELITE, with calcite veins. Primary joints are extremely close to moderately spaced, high angle to vertical, planar, rough to smooth, fresh, very tight to open.		
										Rock Quality = Poor		
										Recovery = 97%		
										R2: Core Times (min:sec)		
										33.7-34.7 ft (2:36)		
										34.7-35.7 ft (2:04)		
										35.7-26.7 ft (2:25)		
										36.7-37.7 ft (2:28)		
										37.7-38.7 ft (2:46)		
											38.7	
										Bottom of Exploration at 38.7 feet below ground surface.		

Remarks:
N 943338.6120 E 2304521.6451

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Whitney Brook Bridge #2942 carries U.S. Route 1 over Whitney Brook
Location: Bridgewater, Maine

Boring No.: BB-BWB-102

WIN: 24783.00

Driller: MaineDOT	Elevation (ft.): 404.4	Auger ID/OD: 5" Solid Stem
Operator: Daggett/Westrack	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B.Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/23/2020; 07:00-13:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2
Boring Location: 10+03.2, 17.3 ft Lt.	Casing ID/OD: NW-3"	Water Level*: 15.0 ft bgs.
Hammer Efficiency Factor: 0.886	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

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
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	5D	12/6	25.0 - 26.0	21/52	---	29					Grey, wet, very dense, Gravelly, fine to coarse SAND, little silt. occasional cobbles, (Glacial Till). Cobble from 26.0-26.3 ft bgs. Roller Coned ahead to 30.0 ft bgs.	
						22						
						37						
						46						
						84						
30	6D	24/18	30.0 - 32.0	18/29/31/48	60	89	30				Grey, wet, hard, SILT, some sand, little gravel, (Glacial Till). G#336985 A-4, CL WC=17.4%	
							22					
							20					
							33					
							124					
35	7D R1	3.6/3 56.4/54	35.0 - 35.3 35.3 - 40.0	50(3.6") RQD=0%	---	a30	369.1				a30 blows for 0.3 ft. Similar to above, weathered rock in tip of spoon. Top of Bedrock at Elev. 369.1 ft. R1: Hard, fresh, aphanitic to fine grained, grey, CALCAREOUS PELITE, with calcite stringers. Primary joints are extremely close to close, vertical to high angle, planar, smooth, fresh, very tight to partially open. Secondary joints are close to moderately spaced, horizontal to low angle, planar to stepped, rough to smooth, fresh, tight to open. [Carys Mill Formation] Rock Quality = Very Poor Recovery = 96% R1: Core times (min:sec) 35.3-36.3 ft (1:58) 36.3-37.3 ft (3:07) 37.3-38.3 ft (2:45) 38.3-39.3 ft (3:11) 39.3-40.0 ft (3:01) Core Blocked R2: Hard, fresh, aphanitic to fine grained, grey, CALCAREOUS PELITE, with calcite stringers. Joints are close to moderately spaced, high angle to vertical, undulating to planar, rough to smooth, fresh, very tight to partially open. [Carys Mill Formation] Rock Quality = Poor Recovery = 100% R2: Core Times (min:sec) 40.0-41.0 ft (2:14) 41.0-42.0 ft (2:26) 42.0-43.0 ft (2:21) 43.0-44.0 ft (2:28)	
							35.3					
40	R2	60/60	40.0 - 45.0	RQD = 48%			364.4					
45							359.4					
50												

Remarks:
 N 943272.8322 E 2304523.0270

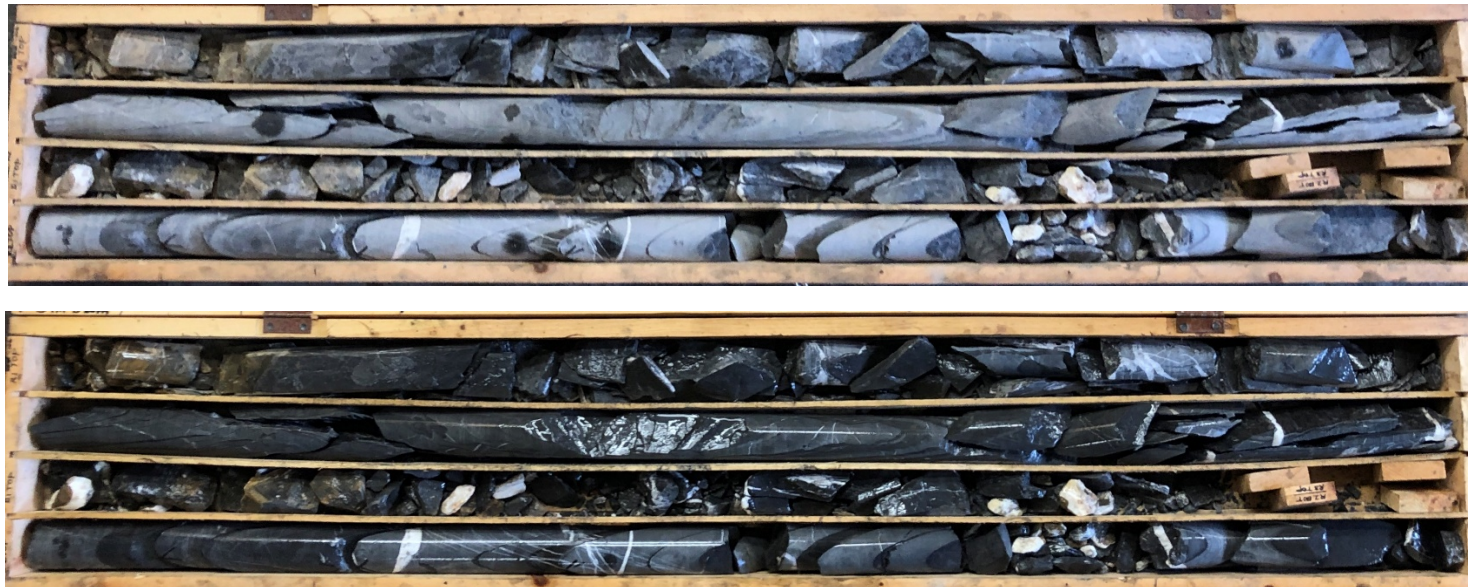


APPENDIX D – ROCK CORE PHOTOGRAPHS



Whitney Brook Bridge No. 2942
Bridgewater, ME
Rock Core Photographs

Boring No.	Run	Depth (ft)	Recovery (in)	Recovery (%)	RQD (in)	RQD (%)	Rock Type	Box Row
BB-BWB-101	R1	28.7 - 33.7	43	72	0	0	CALCAREOUS PELITE	3
BB-BWB-101	R2	33.7 - 38.7	58	97	29.5	49	CALCAREOUS PELITE	4
BB-BWB-102	R1	35.3 - 40.0	54	96	0	0	CALCAREOUS PELITE	1
BB-BWB-102	R2	40.0 - 45.0	60	100	29	48	CALCAREOUS PELITE	2

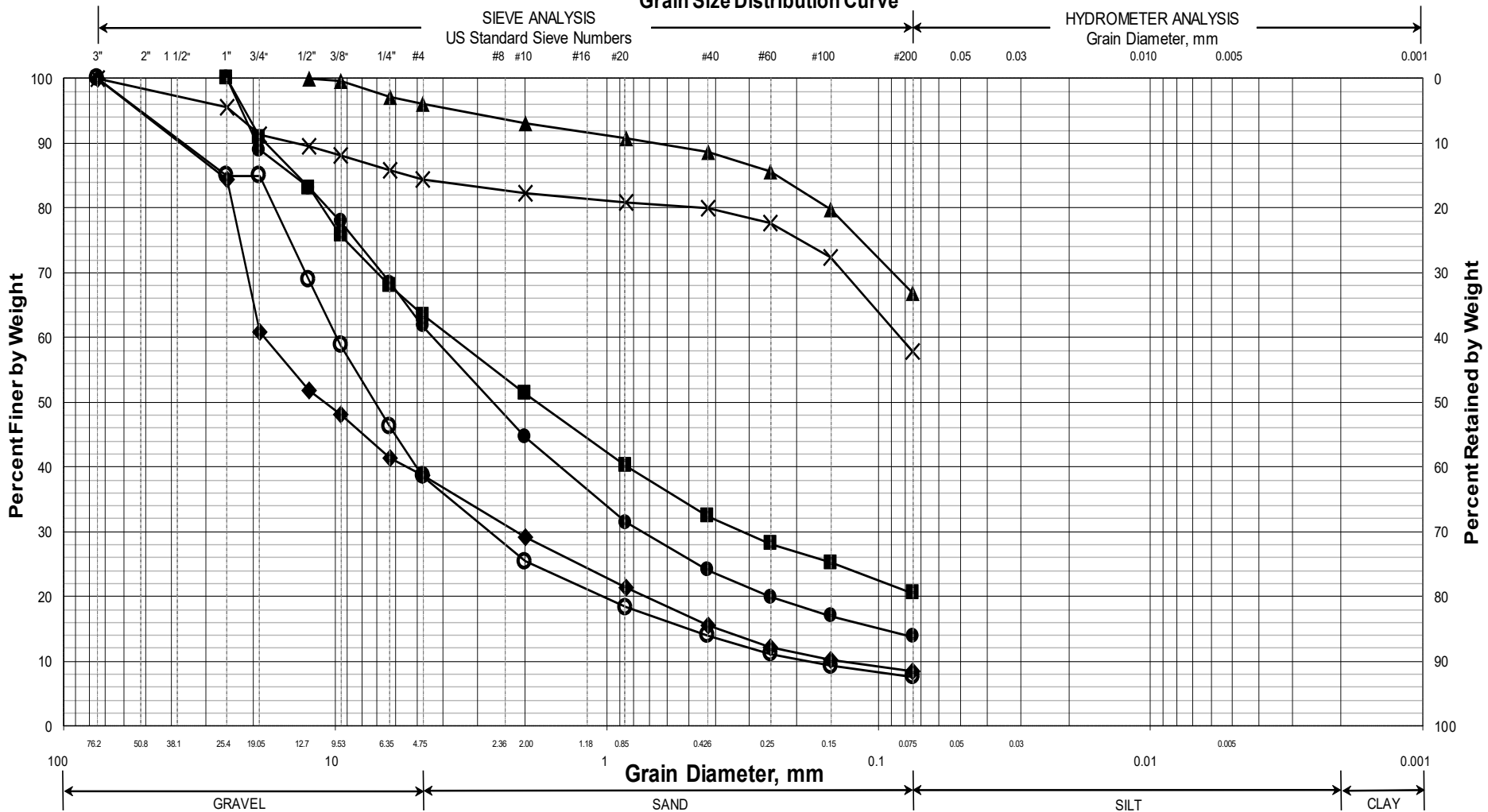


- Notes:**
1. Box row corresponds to the core box section in which the rock core sample is contained; Row 1=Top, Row 4=Bottom.
 2. Top photo is dry, bottom photo is wet.



APPENDIX E – LABORATORY TEST RESULTS

Maine Department of Transportation Grain Size Distribution Curve



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-BWB-101/3D	10+69	16.8 LT	15.0-17.0	Gravelly SAND, trace silt.	13.8			
◆	BB-BWB-101/4D	10+69	16.8 LT	20.0-22.0	Gravelly SAND, trace silt.	9.1			
■	BB-BWB-102/1D	10+69	16.8 LT	5.0-7.0	Gravelly SAND, some silt.	9.2			
●	BB-BWB-102/3D	10+03.2	17.3 LT	15.0-17.0	Gravelly SAND, little silt.	12.2			
▲	BB-BWB-102/4D	10+03.2	17.3 LT	20.0-22.0	SILT, some sand, trace gravel.	18.7			
X	BB-BWB-102/6D	10+03.2	17.3 LT	30.0-32.0	SILT, some sand, little gravel.	17.4			

WIN
024783.00
Town
Bridgewater
Reported by/Date
WHITE, TERRY A 7/29/2021



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336979	BB-BWB-101/2D	GEOTECHNICAL (DISTURBED)	6/24/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+69	Offset, ft: 16.8 LT Dbfg, ft: 10.0-12.0	
WIN/Town 024783.00 - BRIDGEWATER		Sampler: BRUCE WILDER		

TEST RESULTS

Sieve Analysis (T 27)	
Wash Method	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	

Miscellaneous Tests	
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	23.7
Water Content (T 265), %	

Consolidation (T 216)					
Trimming, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)						
Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft²	tons/ft²	tons/ft²	tons/ft²		

Comments:

Cup is labeled 5'-7'

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336980	BB-BWB-101/3D	GEOTECHNICAL (DISTURBED)	6/24/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+69	Offset, ft: 16.8 LT Dbfg, ft: 15.0-17.0	
WIN/Town 024783.00 - BRIDGEWATER		Sampler: BRUCE WILDER		

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	85.0
¾ in. [19.0 mm]	85.0
½ in. [12.5 mm]	68.9
⅜ in. [9.5 mm]	58.8
¼ in. [6.3 mm]	46.2
No. 4 [4.75 mm]	38.6
No. 10 [2.00 mm]	25.4
No. 20 [0.850 mm]	18.3
No. 40 [0.425 mm]	14.0
No. 60 [0.250 mm]	11.1
No. 100 [0.150 mm]	9.3
No. 200 [0.075 mm]	7.6

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	13.8

Consolidation (T 216)

Trimblings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336981	BB-BWB-101/4D	GEOTECHNICAL (DISTURBED)	6/24/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+69	Offset, ft: 16.8 LT Dbfg, ft: 20.0-22.0	
WIN/Town 024783.00 - BRIDGEWATER		Sampler: BRUCE WILDER		

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	84.4
¾ in. [19.0 mm]	60.8
½ in. [12.5 mm]	51.9
⅜ in. [9.5 mm]	48.1
¼ in. [6.3 mm]	41.3
No. 4 [4.75 mm]	38.7
No. 10 [2.00 mm]	29.1
No. 20 [0.850 mm]	21.3
No. 40 [0.425 mm]	15.5
No. 60 [0.250 mm]	12.1
No. 100 [0.150 mm]	10.3
No. 200 [0.075 mm]	8.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.1

Consolidation (T 216)

Trimming, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336982	BB-BWB-102/1D	GEOTECHNICAL (DISTURBED)	6/23/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+69	Offset, ft: 16.8 LT Dbfg, ft: 5.0-7.0	
WIN/Town 024783.00 - BRIDGEWATER		Sampler: BRUCE WILDER		

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	90.9
½ in. [12.5 mm]	83.1
⅜ in. [9.5 mm]	75.8
¼ in. [6.3 mm]	68.0
No. 4 [4.75 mm]	63.5
No. 10 [2.00 mm]	51.3
No. 20 [0.850 mm]	40.2
No. 40 [0.425 mm]	32.4
No. 60 [0.250 mm]	28.1
No. 100 [0.150 mm]	25.2
No. 200 [0.075 mm]	20.5

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	9.2

Consolidation (T 216)

Trimblings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**

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GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336983	BB-BWB-102/3D	GEOTECHNICAL (DISTURBED)	6/23/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+03.2 Offset, ft: 17.3 LT Dbfg, ft: 15.0-17.0	Sampler: BRUCE WILDER	
WIN/Town 024783.00 - BRIDGEWATER				

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	89.0
½ in. [12.5 mm]	83.1
⅜ in. [9.5 mm]	77.9
¼ in. [6.3 mm]	68.4
No. 4 [4.75 mm]	61.8
No. 10 [2.00 mm]	44.6
No. 20 [0.850 mm]	31.4
No. 40 [0.425 mm]	24.1
No. 60 [0.250 mm]	19.9
No. 100 [0.150 mm]	17.0
No. 200 [0.075 mm]	13.8

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	12.2

Consolidation (T 216)

Trimblings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336984	BB-BWB-102/4D	GEOTECHNICAL (DISTURBED)	6/23/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+03.2 Offset, ft: 17.3 LT Dbfg, ft: 20.0-22.0	Sampler: BRUCE WILDER	
WIN/Town 024783.00 - BRIDGEWATER				

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	100.0
⅜ in. [9.5 mm]	99.6
¼ in. [6.3 mm]	97.1
No. 4 [4.75 mm]	96.0
No. 10 [2.00 mm]	93.1
No. 20 [0.850 mm]	90.7
No. 40 [0.425 mm]	88.6
No. 60 [0.250 mm]	85.6
No. 100 [0.150 mm]	79.8
No. 200 [0.075 mm]	66.8

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	18.7

Consolidation (T 216)

Trimblings, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**

Paper Copy: Lab File; Project File; Geotech File



GEOTECHNICAL TEST REPORT

Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
336985	BB-BWB-102/6D	GEOTECHNICAL (DISTURBED)	6/23/2020	8/13/2020
Sample Type: GEOTECHNICAL Location:		Station: 10+03.2 Offset, ft: 17.3 LT Dbfg, ft: 30.0-32.0		
WIN/Town 024783.00 - BRIDGEWATER		Sampler: BRUCE WILDER		

TEST RESULTS

Sieve Analysis (T 27, T 11)	
Wash Method	
Procedure A	
SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	100.0
1 in. [25.0 mm]	95.6
¾ in. [19.0 mm]	91.3
½ in. [12.5 mm]	89.5
⅜ in. [9.5 mm]	88.1
¼ in. [6.3 mm]	85.8
No. 4 [4.75 mm]	84.4
No. 10 [2.00 mm]	82.2
No. 20 [0.850 mm]	80.8
No. 40 [0.425 mm]	79.9
No. 60 [0.250 mm]	77.7
No. 100 [0.150 mm]	72.3
No. 200 [0.075 mm]	57.9

Miscellaneous Tests	
Liquid Limit @ 25 blows (T 89), %	
Plastic Limit (T 90), %	
Plasticity Index (T 90), %	
Specific Gravity, Corrected to 20°C (T 100)	
Loss on Ignition, % (T 267)	
Water Content (T 265), %	17.4

Consolidation (T 216)					
Trimming, Water Content, %					
	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft ³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Vane Shear Test on Shelby Tubes (Maine DOT)						
Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear	Remold	U. Shear	Remold		
	tons/ft ²	tons/ft ²	tons/ft ²	tons/ft ²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **GREGORY LIDSTONE**

Date Reported: **8/17/2020**

Paper Copy: Lab File; Project File; Geotech File



APPENDIX F – GEOTECHNICAL CALCULATIONS

F.1 Soil Parameters

F.2 Apile Results

F.3 Lateral Pile Analysis

F.4 Preliminary WEAP

F.5 Earth Pressure Coefficients

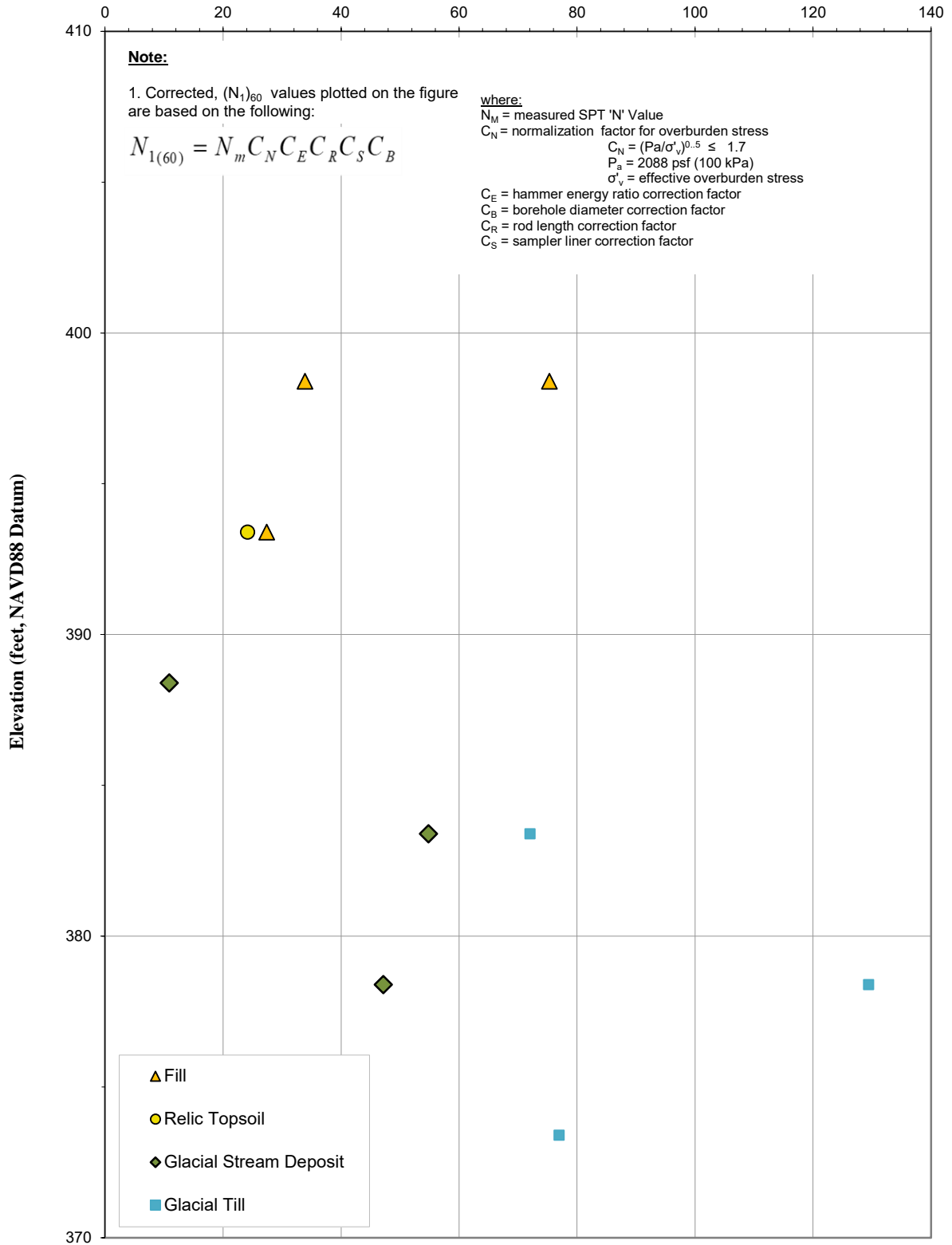
F.6 Seismic Coefficients

F.7 Frost Depth



F.1 Soil Parameters

SPT ($N_{1(60)}$) Values



PROJ MGR: MRJ
 DESIGNED BY: MRJ
 REVIEWED BY: ARB

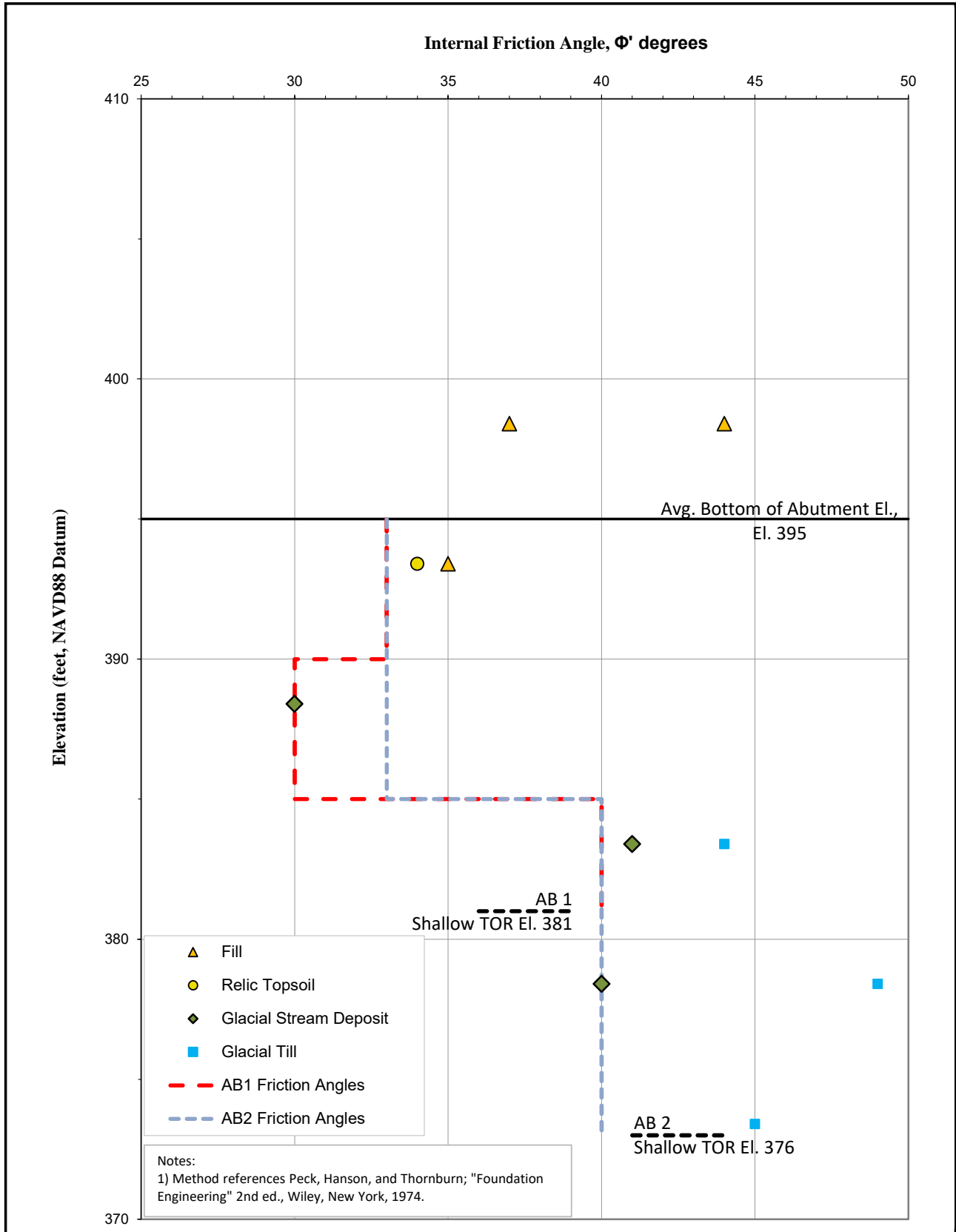
DRAWN BY: MRJ
 DATE: 3/23/21

WHITNEY BROOK BRIDGE #2942 BRIDGEWATER, MAINE



CORRECTED STANDARD PENETRATION TEST (SPT) $N_{1(60)}$ -VALUES FOR ALL STRATA

FIGURE A



PROJ MGR: MRJ DESIGNED BY: MRJ REVIEWED BY: ARB	DRAWN BY: MRJ DATE: 3/23/21	WHITNEY BROOK BRIDGE #2942 BRIDGEWATER, MAINE	FIGURE B
		INTERNAL FRICTION ANGLES BASED ON STANDARD PENETRATION TEST (SPT) N-VALUES FOR ALL STRATA	



F.2 Apile Results



ABUTMENT 1

APILE for Windows, Version 2019.9.11

Serial Number : 315773170

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.
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=====

This program is licensed to :

GZA GeoEnvironmental, Inc.
Norwood, MA

Path to file locations : \\gzaport1\Jobs\09 Jobs\0026100s\09.0026100.00 -
MEDOT Whitney Brook Bridge #2942\01 - Final Design\Work\Apile\
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Name of output file : Abutment1-HP14x73 - L=14ft.ap9o
Name of plot output file : Abutment1-HP14x73 - L=14ft.ap9p

Time and Date of Analysis

Date: April 26, 2022 Time: 08:40:28

1

* INPUT INFORMATION *

New Pile

DESIGNER :

JOB NUMBER :

METHOD FOR UNIT LOAD TRANSFERS :



- FHWA (Federal Highway Administration)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)

TYPE OF LOADING :

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 21.40 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 14.00 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 0.00 FT.
- PERIMETER OF PILE = 56.40 IN.
- TIP AREA OF PILE = 21.40 IN2
- INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/FT^3	FRICTION ANGLE DEGREES	Nq FACTOR FHWA
0.00	SAND	0.80*	125.00	33.00	47.20**
5.00	SAND	0.80*	125.00	33.00	47.20**
5.00	SAND	0.80*	58.00	30.00	30.00**
10.00	SAND	0.80*	58.00	30.00	30.00**



10.00	SAND	0.80*	68.00	40.00	160.00**
14.00	SAND	0.80*	68.00	40.00	160.00**
14.00	SAND	0.80*	106.00	45.00	475.00**
100.00	SAND	0.80*	106.00	45.00	475.00**

* VALUE ASSUMED BY THE PROGRAM

** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE

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
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
10.00	1.000	1.000
10.00	1.000	1.000
14.00	1.000	1.000
14.00	1.000	1.000
100.00	1.000	1.000

 * COMPUTATION RESULT *



 * FED. HWY. METHOD *

PILE PENETRATION FT.	SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.3	0.3
1.00	0.1	0.6	0.7
2.00	0.5	1.1	1.6
3.00	1.1	1.6	2.7
4.00	2.0	1.9	3.8
5.00	3.1	2.0	5.1
6.00	4.2	2.0	6.3
7.00	5.3	1.9	7.2
8.00	6.5	2.7	9.2
9.00	7.7	6.2	13.9
10.00	9.1	9.8	18.9
11.00	11.6	13.8	25.4
12.00	15.5	20.5	36.0
13.00	19.6	33.5	53.1
14.00	23.9	46.8	70.7

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.

 * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
 * CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.4167E-01	0.0000E+00	0.0000E+00
			0.4585E-02	0.2872E-01
			0.7641E-02	0.5565E-01
			0.1146E-01	0.1023E+00
			0.1375E-01	0.1436E+00
			0.1528E-01	0.1795E+00
			0.1528E-01	0.3591E+00
			0.1528E-01	0.5386E+00



			0.1528E-01	0.8976E+00
			0.1528E-01	0.3591E+01
2	10	0.2500E+01	0.0000E+00	0.0000E+00
			0.2751E+00	0.2872E-01
			0.4585E+00	0.5565E-01
			0.6877E+00	0.1023E+00
			0.8252E+00	0.1436E+00
			0.9169E+00	0.1795E+00
			0.9169E+00	0.3591E+00
			0.9169E+00	0.5386E+00
			0.9169E+00	0.8976E+00
			0.9169E+00	0.3591E+01
3	10	0.4958E+01	0.0000E+00	0.0000E+00
			0.5456E+00	0.2872E-01
			0.9093E+00	0.5565E-01
			0.1364E+01	0.1023E+00
			0.1637E+01	0.1436E+00
			0.1819E+01	0.1795E+00
			0.1819E+01	0.3591E+00
			0.1819E+01	0.5386E+00
			0.1819E+01	0.8976E+00
			0.1819E+01	0.3591E+01
4	10	0.5042E+01	0.0000E+00	0.0000E+00
			0.5464E+00	0.2872E-01
			0.9106E+00	0.5565E-01
			0.1366E+01	0.1023E+00
			0.1639E+01	0.1436E+00
			0.1821E+01	0.1795E+00
			0.1821E+01	0.3591E+00
			0.1821E+01	0.5386E+00
			0.1821E+01	0.8976E+00
			0.1821E+01	0.3591E+01
5	10	0.7500E+01	0.0000E+00	0.0000E+00
			0.5177E+00	0.2872E-01
			0.8629E+00	0.5565E-01
			0.1294E+01	0.1023E+00
			0.1553E+01	0.1436E+00
			0.1726E+01	0.1795E+00
			0.1726E+01	0.3591E+00
			0.1726E+01	0.5386E+00
			0.1726E+01	0.8976E+00
			0.1726E+01	0.3591E+01
6	10	0.9958E+01	0.0000E+00	0.0000E+00
			0.6136E+00	0.2872E-01
			0.1023E+01	0.5565E-01



			0.1534E+01	0.1023E+00
			0.1841E+01	0.1436E+00
			0.2045E+01	0.1795E+00
			0.2045E+01	0.3591E+00
			0.2045E+01	0.5386E+00
			0.2045E+01	0.8976E+00
			0.2045E+01	0.3591E+01
7	10	0.1004E+02	0.0000E+00	0.0000E+00
			0.6582E+00	0.2872E-01
			0.1097E+01	0.5565E-01
			0.1646E+01	0.1023E+00
			0.1975E+01	0.1436E+00
			0.2194E+01	0.1795E+00
			0.2194E+01	0.3591E+00
			0.2194E+01	0.5386E+00
			0.2194E+01	0.8976E+00
			0.2194E+01	0.3591E+01
8	10	0.1200E+02	0.0000E+00	0.0000E+00
			0.1761E+01	0.2872E-01
			0.2935E+01	0.5565E-01
			0.4403E+01	0.1023E+00
			0.5284E+01	0.1436E+00
			0.5871E+01	0.1795E+00
			0.5871E+01	0.3591E+00
			0.5871E+01	0.5386E+00
			0.5871E+01	0.8976E+00
			0.5871E+01	0.3591E+01
9	10	0.1396E+02	0.0000E+00	0.0000E+00
			0.1984E+01	0.2872E-01
			0.3307E+01	0.5565E-01
			0.4961E+01	0.1023E+00
			0.5953E+01	0.1436E+00
			0.6615E+01	0.1795E+00
			0.6615E+01	0.3591E+00
			0.6615E+01	0.5386E+00
			0.6615E+01	0.8976E+00
			0.6615E+01	0.3591E+01
10	10	0.1404E+02	0.0000E+00	0.0000E+00
			0.1989E+01	0.2872E-01
			0.3315E+01	0.5565E-01
			0.4973E+01	0.1023E+00
			0.5967E+01	0.1436E+00
			0.6630E+01	0.1795E+00
			0.6630E+01	0.3591E+00
			0.6630E+01	0.5386E+00
			0.6630E+01	0.8976E+00



11	10	0.5700E+02	0.6630E+01	0.3591E+01
			0.0000E+00	0.0000E+00
			0.1989E+01	0.2872E-01
			0.3315E+01	0.5565E-01
			0.4973E+01	0.1023E+00
			0.5967E+01	0.1436E+00
			0.6630E+01	0.1795E+00
			0.6630E+01	0.3591E+00
			0.6630E+01	0.5386E+00
			0.6630E+01	0.8976E+00
			0.6630E+01	0.3591E+01
12	10	0.9996E+02	0.0000E+00	0.0000E+00
			0.1989E+01	0.2872E-01
			0.3315E+01	0.5565E-01
			0.4973E+01	0.1023E+00
			0.5967E+01	0.1436E+00
			0.6630E+01	0.1795E+00
			0.6630E+01	0.3591E+00
			0.6630E+01	0.5386E+00
			0.6630E+01	0.8976E+00
			0.6630E+01	0.3591E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.2924E+01	0.8976E-02
0.5847E+01	0.1795E-01
0.1169E+02	0.3591E-01
0.2339E+02	0.2334E+00
0.3508E+02	0.7540E+00
0.4210E+02	0.1311E+01
0.4678E+02	0.1795E+01
0.4678E+02	0.2693E+01
0.4678E+02	0.3591E+01

LOAD VERSUS SETTLEMENT CURVE

TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.5818E-01	0.1137E-03	0.3257E-01	0.1000E-03



0.5818E+00	0.1137E-02	0.3257E+00	0.1000E-02
0.2909E+01	0.5683E-02	0.1629E+01	0.5000E-02
0.5818E+01	0.1137E-01	0.3257E+01	0.1000E-01
0.1164E+02	0.2273E-01	0.6514E+01	0.2000E-01
0.2367E+02	0.5550E-01	0.1253E+02	0.5000E-01
0.2951E+02	0.8676E-01	0.1431E+02	0.8000E-01
0.3324E+02	0.1076E+00	0.1549E+02	0.1000E+00
0.4519E+02	0.2103E+00	0.2141E+02	0.2000E+00
0.5316E+02	0.5125E+00	0.2938E+02	0.5000E+00
0.5945E+02	0.8142E+00	0.3566E+02	0.8000E+00
0.6197E+02	0.1015E+01	0.3819E+02	0.1000E+01
0.7056E+02	0.2017E+01	0.4678E+02	0.2000E+01



ABUTMENT 2

APILE for Windows, Version 2019.9.11

Serial Number : 315773170

A Program for Analyzing the Axial Capacity
and Short-term Settlement of Driven Piles
under Axial Loading.

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This program is licensed to :

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Norwood, MA

Path to file locations : \\gzaport1\Jobs\09 Jobs\0026100s\09.0026100.00 -
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Name of plot output file : Abutment2-HP14x73 - L=19ft.ap9p

Time and Date of Analysis

Date: April 20, 2022 Time: 14:12:04

1

* INPUT INFORMATION *

New Pile

DESIGNER :

JOB NUMBER :

METHOD FOR UNIT LOAD TRANSFERS :



- FHWA (Federal Highway Administration)
Unfactored Unit Side Friction and Unit Side Resistance are used.

COMPUTATION METHOD(S) FOR PILE CAPACITY :

- FHWA (Federal Highway Administration)

TYPE OF LOADING :

- COMPRESSION

PILE TYPE :

H-Pile/Steel Pile

DATA FOR AXIAL STIFFNESS :

- MODULUS OF ELASTICITY = 0.290E+08 PSI
- CROSS SECTION AREA = 21.40 IN2

NONCIRCULAR PILE PROPERTIES :

- TOTAL PILE LENGTH, TL = 19.00 FT.
- BATTER ANGLE = 0.00 DEG
- PILE STICKUP LENGTH, PSL = 0.00 FT.
- ZERO FRICTION LENGTH, ZFL = 0.00 FT.
- PERIMETER OF PILE = 56.40 IN.
- TIP AREA OF PILE = 21.40 IN2
- INCREMENT OF PILE LENGTH
USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/FT^3	FRICTION ANGLE DEGREES	Nq FACTOR FHWA
0.00	SAND	0.80*	125.00	33.00	47.20**
5.00	SAND	0.80*	125.00	33.00	47.20**
5.00	SAND	0.80*	58.00	30.00	30.00**
10.00	SAND	0.80*	58.00	30.00	30.00**



10.00	SAND	0.80*	68.00	40.00	160.00**
19.00	SAND	0.80*	68.00	40.00	160.00**
19.00	SAND	0.80*	106.00	50.00	475.00**
100.00	SAND	0.80*	106.00	50.00	475.00**

* VALUE ASSUMED BY THE PROGRAM

** VALUE ESTIMATED BY THE PROGRAM BASED ON FRICTION ANGLE

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
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
10.00	1.000	1.000
10.00	1.000	1.000
19.00	1.000	1.000
19.00	1.000	1.000
100.00	1.000	1.000

 * COMPUTATION RESULT *



 * FED. HWY. METHOD *

PILE PENETRATION FT.	SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.3	0.3
1.00	0.1	0.6	0.7
2.00	0.5	1.1	1.6
3.00	1.1	1.6	2.7
4.00	2.0	1.9	3.8
5.00	3.1	2.0	5.1
6.00	4.2	2.0	6.3
7.00	5.3	1.9	7.2
8.00	6.5	2.7	9.2
9.00	7.7	6.2	13.9
10.00	9.1	9.8	18.9
11.00	11.6	13.8	25.4
12.00	15.5	18.0	33.5
13.00	19.6	20.0	39.5
14.00	23.9	21.2	45.1
15.00	28.6	22.4	50.9
16.00	33.4	23.6	57.0
17.00	38.6	28.0	66.6
18.00	44.0	43.2	87.1
19.00	49.6	59.4	109.0

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.

 * COMPUTE LOAD-DISTRIBUTION AND LOAD-SETTLEMENT *
 * CURVES FOR AXIAL LOADING *

T-Z CURVE NO.	NO. OF POINTS	DEPTH TO CURVE FT.	LOAD TRANSFER PSI	PILE MOVEMENT IN.
1	10	0.4167E-01	0.0000E+00	0.0000E+00
			0.4585E-02	0.2872E-01
			0.7641E-02	0.5565E-01



			0.1146E-01	0.1023E+00
			0.1375E-01	0.1436E+00
			0.1528E-01	0.1795E+00
			0.1528E-01	0.3591E+00
			0.1528E-01	0.5386E+00
			0.1528E-01	0.8976E+00
			0.1528E-01	0.3591E+01
2	10	0.2500E+01		
			0.0000E+00	0.0000E+00
			0.2751E+00	0.2872E-01
			0.4585E+00	0.5565E-01
			0.6877E+00	0.1023E+00
			0.8252E+00	0.1436E+00
			0.9169E+00	0.1795E+00
			0.9169E+00	0.3591E+00
			0.9169E+00	0.5386E+00
			0.9169E+00	0.8976E+00
			0.9169E+00	0.3591E+01
3	10	0.4958E+01		
			0.0000E+00	0.0000E+00
			0.5456E+00	0.2872E-01
			0.9093E+00	0.5565E-01
			0.1364E+01	0.1023E+00
			0.1637E+01	0.1436E+00
			0.1819E+01	0.1795E+00
			0.1819E+01	0.3591E+00
			0.1819E+01	0.5386E+00
			0.1819E+01	0.8976E+00
			0.1819E+01	0.3591E+01
4	10	0.5042E+01		
			0.0000E+00	0.0000E+00
			0.5464E+00	0.2872E-01
			0.9106E+00	0.5565E-01
			0.1366E+01	0.1023E+00
			0.1639E+01	0.1436E+00
			0.1821E+01	0.1795E+00
			0.1821E+01	0.3591E+00
			0.1821E+01	0.5386E+00
			0.1821E+01	0.8976E+00
			0.1821E+01	0.3591E+01
5	10	0.7500E+01		
			0.0000E+00	0.0000E+00
			0.5177E+00	0.2872E-01
			0.8629E+00	0.5565E-01
			0.1294E+01	0.1023E+00
			0.1553E+01	0.1436E+00
			0.1726E+01	0.1795E+00
			0.1726E+01	0.3591E+00
			0.1726E+01	0.5386E+00
			0.1726E+01	0.8976E+00



6	10	0.9958E+01	0.1726E+01	0.3591E+01
			0.0000E+00	0.0000E+00
			0.6136E+00	0.2872E-01
			0.1023E+01	0.5565E-01
			0.1534E+01	0.1023E+00
			0.1841E+01	0.1436E+00
			0.2045E+01	0.1795E+00
			0.2045E+01	0.3591E+00
			0.2045E+01	0.5386E+00
			0.2045E+01	0.8976E+00
			0.2045E+01	0.3591E+01
7	10	0.1004E+02	0.0000E+00	0.0000E+00
			0.6582E+00	0.2872E-01
			0.1097E+01	0.5565E-01
			0.1646E+01	0.1023E+00
			0.1975E+01	0.1436E+00
			0.2194E+01	0.1795E+00
			0.2194E+01	0.3591E+00
			0.2194E+01	0.5386E+00
			0.2194E+01	0.8976E+00
			0.2194E+01	0.3591E+01
8	10	0.1450E+02	0.0000E+00	0.0000E+00
			0.2046E+01	0.2872E-01
			0.3410E+01	0.5565E-01
			0.5115E+01	0.1023E+00
			0.6138E+01	0.1436E+00
			0.6820E+01	0.1795E+00
			0.6820E+01	0.3591E+00
			0.6820E+01	0.5386E+00
			0.6820E+01	0.8976E+00
			0.6820E+01	0.3591E+01
9	10	0.1896E+02	0.0000E+00	0.0000E+00
			0.2554E+01	0.2872E-01
			0.4257E+01	0.5565E-01
			0.6385E+01	0.1023E+00
			0.7662E+01	0.1436E+00
			0.8514E+01	0.1795E+00
			0.8514E+01	0.3591E+00
			0.8514E+01	0.5386E+00
			0.8514E+01	0.8976E+00
			0.8514E+01	0.3591E+01
10	10	0.1904E+02	0.0000E+00	0.0000E+00
			0.2559E+01	0.2872E-01
			0.4265E+01	0.5565E-01
			0.6397E+01	0.1023E+00



			0.7677E+01	0.1436E+00
			0.8530E+01	0.1795E+00
			0.8530E+01	0.3591E+00
			0.8530E+01	0.5386E+00
			0.8530E+01	0.8976E+00
			0.8530E+01	0.3591E+01
11	10	0.5950E+02	0.0000E+00	0.0000E+00
			0.2559E+01	0.2872E-01
			0.4265E+01	0.5565E-01
			0.6397E+01	0.1023E+00
			0.7677E+01	0.1436E+00
			0.8530E+01	0.1795E+00
			0.8530E+01	0.3591E+00
			0.8530E+01	0.5386E+00
			0.8530E+01	0.8976E+00
			0.8530E+01	0.3591E+01
12	10	0.9996E+02	0.0000E+00	0.0000E+00
			0.2559E+01	0.2872E-01
			0.4265E+01	0.5565E-01
			0.6397E+01	0.1023E+00
			0.7677E+01	0.1436E+00
			0.8530E+01	0.1795E+00
			0.8530E+01	0.3591E+00
			0.8530E+01	0.5386E+00
			0.8530E+01	0.8976E+00
			0.8530E+01	0.3591E+01

TIP LOAD KIP	TIP MOVEMENT IN.
0.0000E+00	0.0000E+00
0.3713E+01	0.8976E-02
0.7426E+01	0.1795E-01
0.1485E+02	0.3591E-01
0.2971E+02	0.2334E+00
0.4456E+02	0.7540E+00
0.5347E+02	0.1311E+01
0.5941E+02	0.1795E+01
0.5941E+02	0.2693E+01
0.5941E+02	0.3591E+01

LOAD VERSUS SETTLEMENT CURVE



TOP LOAD KIP	TOP MOVEMENT IN.	TIP LOAD KIP	TIP MOVEMENT IN.
0.9621E-01	0.1291E-03	0.4137E-01	0.1000E-03
0.9621E+00	0.1291E-02	0.4137E+00	0.1000E-02
0.4811E+01	0.6455E-02	0.2068E+01	0.5000E-02
0.9621E+01	0.1291E-01	0.4137E+01	0.1000E-01
0.1929E+02	0.2583E-01	0.8273E+01	0.2000E-01
0.3953E+02	0.6187E-01	0.1591E+02	0.5000E-01
0.5022E+02	0.9488E-01	0.1817E+02	0.8000E-01
0.5695E+02	0.1168E+00	0.1967E+02	0.1000E+00
0.7660E+02	0.2228E+00	0.2719E+02	0.2000E+00
0.8672E+02	0.5265E+00	0.3731E+02	0.5000E+00
0.9470E+02	0.8294E+00	0.4529E+02	0.8000E+00
0.9791E+02	0.1031E+01	0.4850E+02	0.1000E+01
0.1088E+03	0.2035E+01	0.5941E+02	0.2000E+01



F.3 Lateral Pile Analysis



Table 1 - LPile Input Parameters
MEDOT Whitney Brook Bridge #2942
 GZA GeoEnvironmental, Inc.

GZA FILE NO. 09.0026100.00
CALCULATED BY M. Johnescu, 4/1/2021
CHECKED BY A. Blaisdell, 4/1/2021
REVISED BY M. Smith, 5/19/2022
CHECKED BY A. Blaisdell, 5/19/2022

Abutment 1, Pile length = 14.00'						
Stratum	Soil Model	Top of Layer Elevation (ft)	Layer Thickness (ft)	k (pci) / E50	ϕ' (deg) / Su (psf)	γ_e (pcf)
Fill/Relic Topsoil	Reese Sand	395.0	5	105	33	125
Glacial Stream Deposit	Reese Sand	390.0	5	35	30	58
Glacial Till	Reese Sand	385.0	4	145	40	68
		Total	14			



F.4 Preliminary WEAP



GZA GeoEnvironmental, Inc.

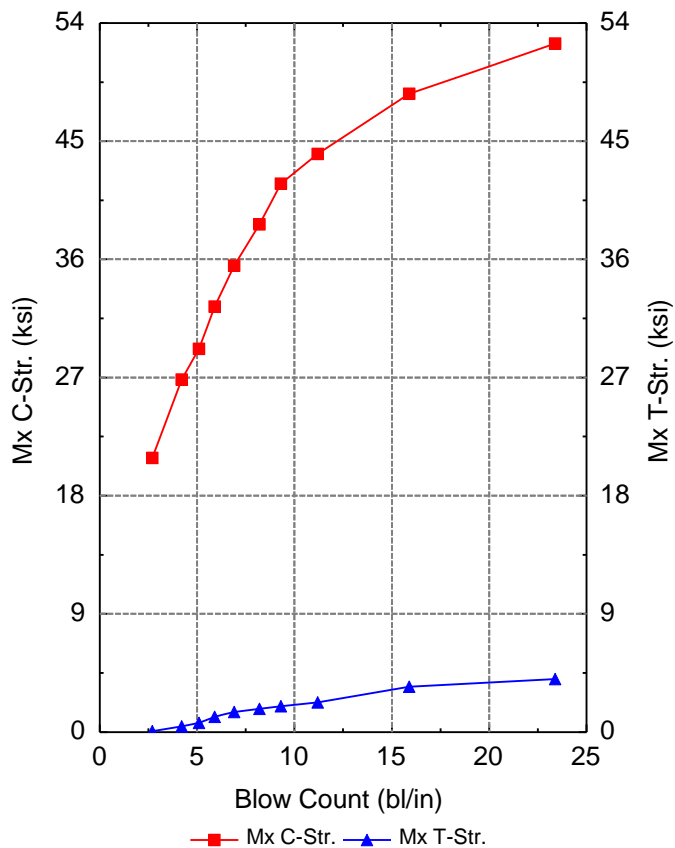
GZA FILE NO. 09.0026100.10
 CALCULATED BY M. Johnescu, 5/24/2022
 CHECKED BY M. Smith, 5/24/2022
 REVISED BY M. Smith, 9/26/2022

**TABLE 1 - SUMMARY OF PRELIMINARY WAVE EQUATION RESULTS
 WHITNEY BROOK BRIDGE #2942
 BRIDGEWATER, MAINE
 HP 14x73 PILES**

Abutment	Length = Penetration + 5 (ft)	Nominal Resistance (kips)	% Side Friction	Skin Quake (in)	Toe Quake (in)	Skin Damping (s/ft)	Toe Damping (s/ft)	Hammer	Ram Weight (kips)	Weap Analysis Results				Notes
										Max Compression Stress (ksi)	Final Penetration Resistance (bl/in)	Stroke (ft)	Transferred Energy (kip-ft)	
1	19	454	10	0.1	0.04	0.05	0.15	Delmag D22-23	4.85	35.9	7.1	6.36	12.5	Fuel Setting 4 (72%)
2	24	454	20	0.1	0.04	0.05	0.15	Delmag D22-23	4.85	32.7	7.1	6.35	12.9	Fuel Setting 4 (72%)

Notes:

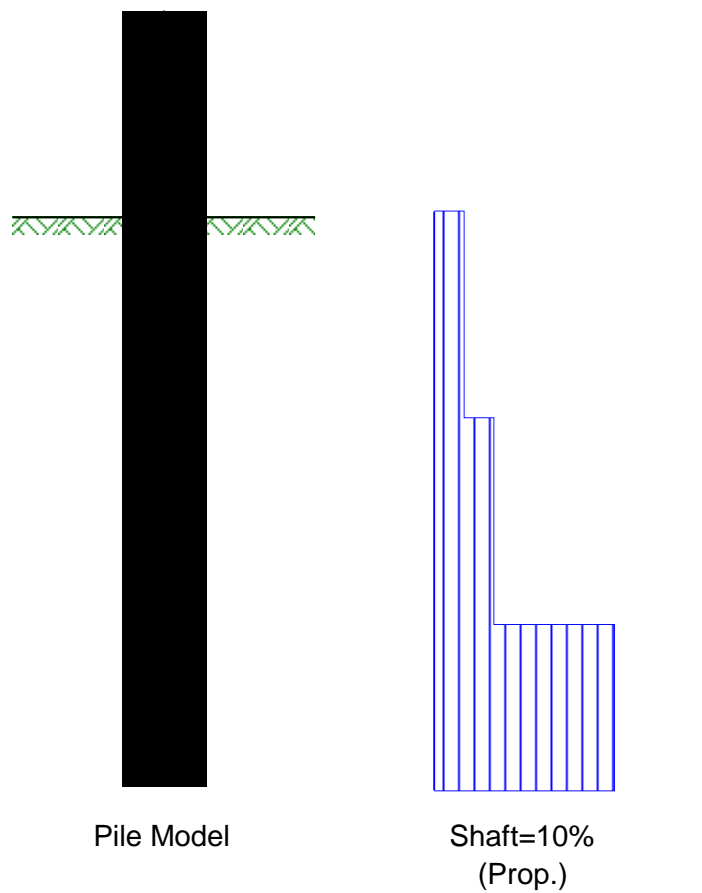
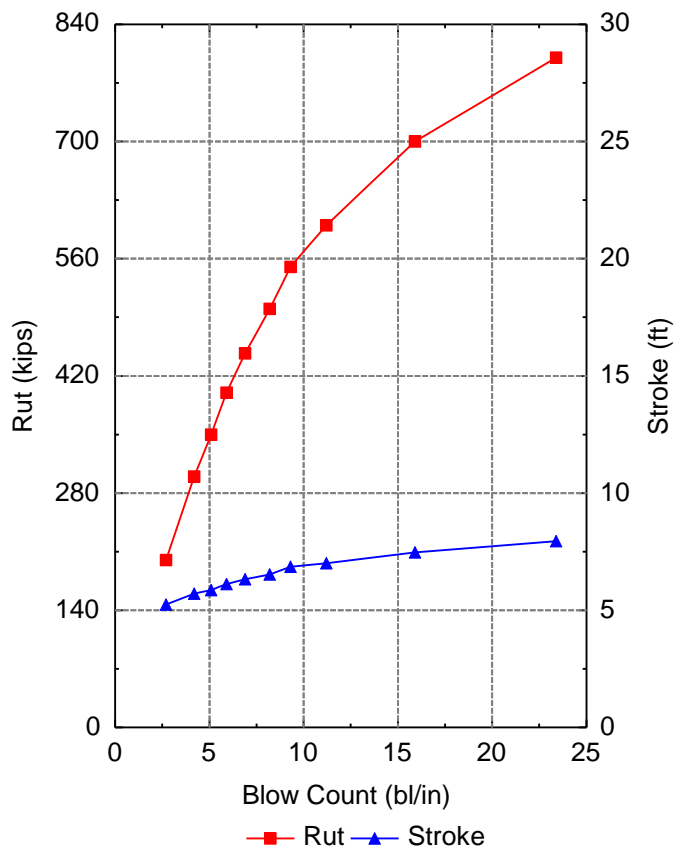
1. The maximum allowable driving stress is 45 ksi for Grade 50 steel.
2. Final penetration resistance represents the average over the final few inches of penetration. MaineDOT preferred range of 6 to 15 blows per inch.



DELMAG D 22-23

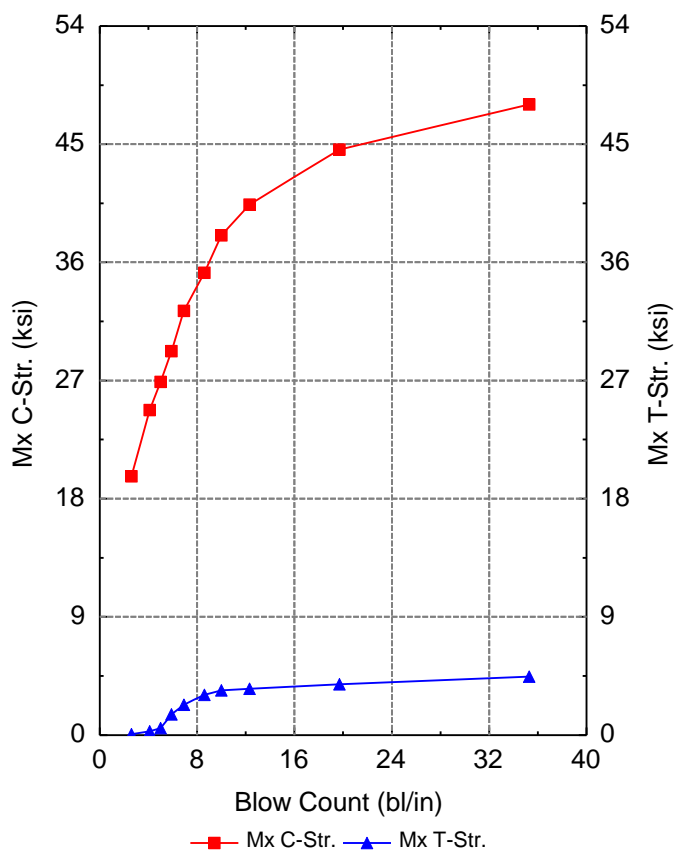
Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	800.0 (72%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	19.000	ft
Pile Penetration	14.000	ft
Pile Top Area	21.400	in ²

RSA No



Bearing Graph Summary — DELMAG D 22-23

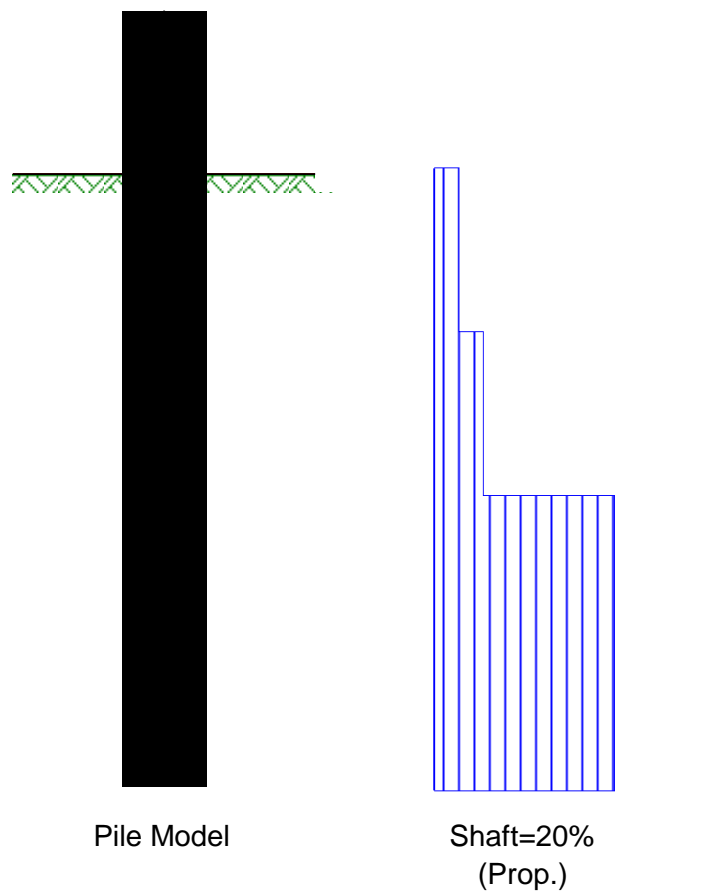
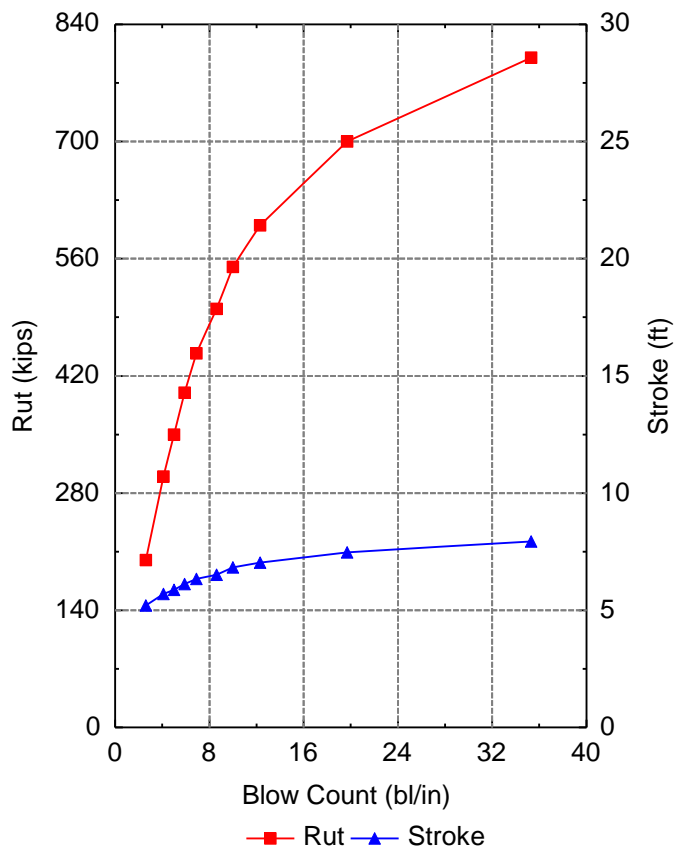
Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/in	Stroke ft	ENTHRU kip-ft	Hammer -
200.0	20.87	0.06	2.7	5.24	11.61	D 22-23
300.0	26.84	0.42	4.2	5.72	11.64	D 22-23
350.0	29.20	0.69	5.1	5.87	11.67	D 22-23
400.0	32.40	1.17	5.9	6.12	12.13	D 22-23
447.0	35.51	1.53	6.9	6.33	12.49	D 22-23
500.0	38.66	1.77	8.2	6.54	12.82	D 22-23
550.0	41.76	1.97	9.3	6.86	13.58	D 22-23
600.0	44.04	2.25	11.2	7.01	13.82	D 22-23
700.0	48.61	3.45	15.9	7.48	14.76	D 22-23
800.0	52.43	4.03	23.4	7.96	15.79	D 22-23



DELMAG D 22-23

Ram Weight	4.85	kips
Efficiency	0.800	
Pressure	800.0 (72%)	psi
Helmet Weight	3.100	kips
Hammer Cushion	109976.0	kips/in
COR of H.C.	0.800	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	s/ft
Toe Damping	0.150	s/ft
Pile Length	24.000	ft
Pile Penetration	19.000	ft
Pile Top Area	21.400	in ²

RSA No



Bearing Graph Summary — DELMAG D 22-23

Rut kips	Mx C-Str. ksi	Mx T-Str. ksi	Blow Ct bl/in	Stroke ft	ENTHRU kip-ft	Hammer -
200.0	19.70	0.07	2.6	5.21	11.77	D 22-23
300.0	24.75	0.28	4.1	5.70	11.91	D 22-23
350.0	26.90	0.53	5.0	5.88	12.11	D 22-23
400.0	29.23	1.56	5.9	6.12	12.50	D 22-23
447.0	32.31	2.32	6.9	6.33	12.92	D 22-23
500.0	35.22	3.05	8.5	6.51	13.14	D 22-23
550.0	38.05	3.41	10.0	6.83	13.84	D 22-23
600.0	40.41	3.53	12.3	7.03	14.21	D 22-23
700.0	44.59	3.87	19.7	7.48	15.03	D 22-23
800.0	48.04	4.45	35.3	7.94	16.02	D 22-23



F.5 Earth Pressure Coefficients



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Engineers and
 Scientists

JOB: 09.0026100.01 Whitney Brook Br.
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 2
 CALCULATED BY M. Smith 5/6/2022
 CHECKED BY A. Blaisdell 5/19/2022

Subject: Evaluate lateral earth pressure coefficients

- References:**
1. MaineDOT Bridge Design Guide, Chapter 3
 2. AASHTO LRFD Bridge Design Specifications, 9th Edition (2020)
 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/silt 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 (H_b) exceeds 0.005. If the calculated rotation is significantly less, Rankine earth pressure may be considered.

- $y := 0.625\text{in}$ Expansion deflection From structural engineer
- $H_b := 9.97\text{ft}$ Abutment Height
- $\frac{y}{H_b} = 0.0052$ Ratio of lateral movement to abutment height.

Earth Pressure Coefficients:

Since the ratio is greater than 0.005, GZA evaluated the passive earth pressure coefficient by using the Massachusetts DOT methodology.

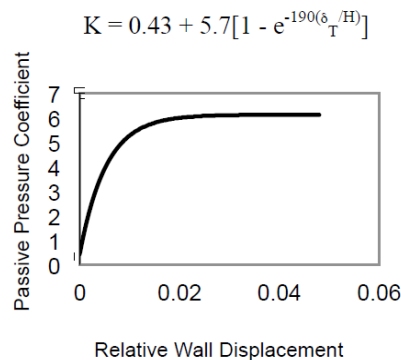


Figure 3.10.8-1: Plot of Passive Pressure Coefficient, K, vs. Relative Wall Displacement, δ_T/H .

$$K_{p.mass} := \left[0.43 + 5.7 \left(1 - \exp \left(-190 \cdot \frac{y}{H_b} \right) \right) \right] = 4.02$$

$K_{p.mass} = 4.02$

The passive earth pressure coefficient (K_p) for granular borrow is 4.02 using the MassDOT methodology.



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*Engineers and
Scientists*

JOB: 09.0026100.01 Whitney Brook Br.
SUBJECT: Lateral Earth Pressures
SHEET: 2 OF 2
CALCULATED BY M. Smith 5/6/2022
CHECKED BY A. Blaisdell 5/19/2022

Rankine Active Earth Pressure Coefficient

$$K_{ar} := \cos(\beta) \cdot \frac{\left[\cos(\beta) - \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2} \right]}{\left[\cos(\beta) + \sqrt{(\cos(\beta))^2 - (\cos(\phi))^2} \right]}$$

$$K_{ar} = 0.31$$

The active earth pressure coefficient (Ka) for granular borrow is 0.31.



F.6 Seismic Coefficients



Site Class Seismic Design Parameters

Whitney Brook Bridge No. 2942
 Route 1 over Whitney Brook, Bridgewater, Maine
 GZA Job#: 09.0026100.10

Seismic Interpolation Coefficients		
Seismic Parameter	Interpolated Value	Design Parameter
Horizontal Peak Ground Acceleration Coefficient (PGA)	7.2	0.072
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s, S _s	16.4	0.164
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s, S ₁	5.1	0.051

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 2 through 4 of this calculation.

For Site Class D	
F _{PGA} =	1.6
F _a =	1.6
F _v =	2.4

Therefore:

A _s = F _{PGA} × PGA =	0.115
S _{DS} = F _a × S _s =	0.262
S _{D1} = F _v × S ₁ =	0.122

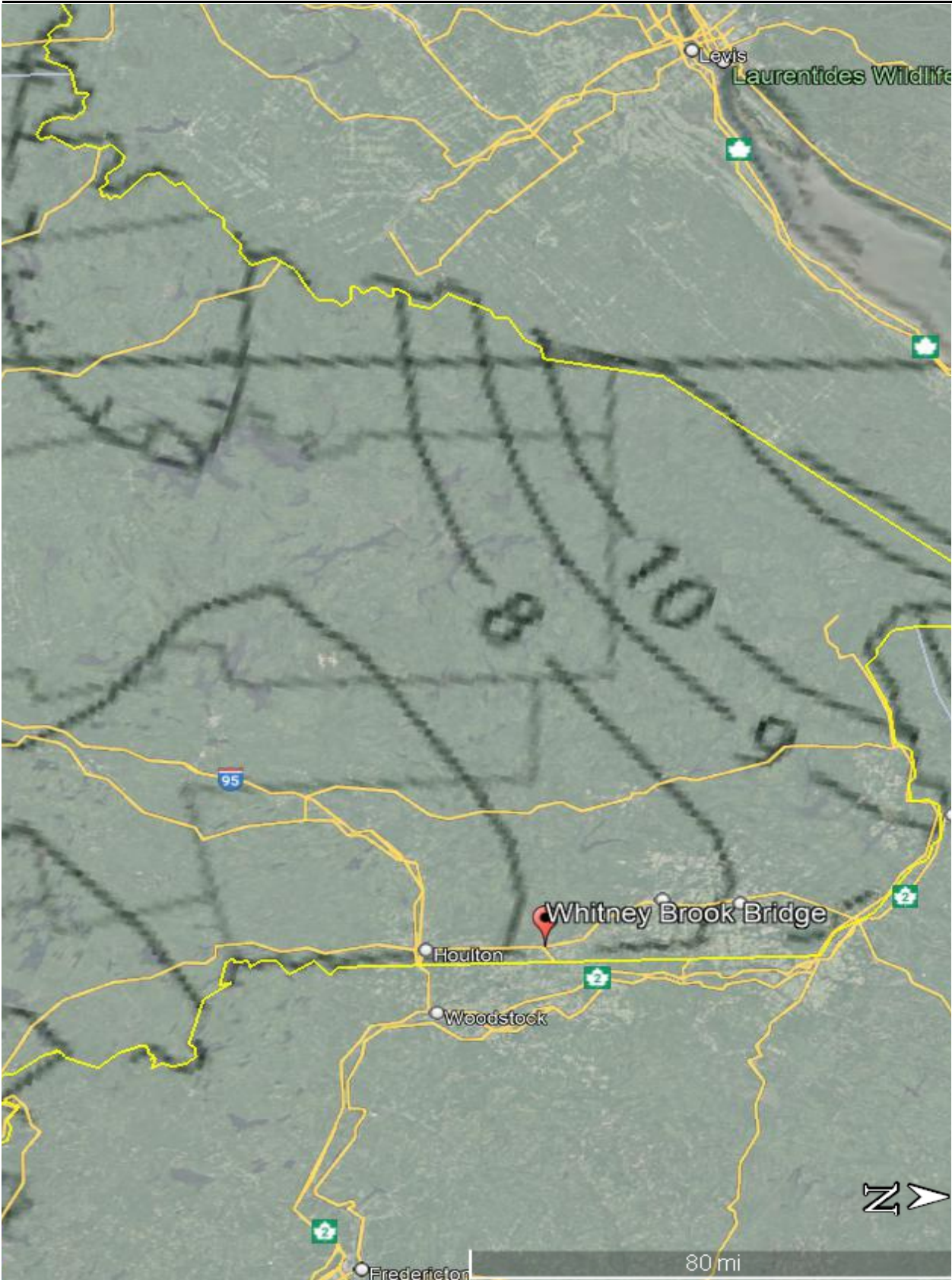
Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s (Period = 0.0 sec)	0.115
S _{Ds} (Period = 0.2 sec)	0.262
S _{D1} (Period = 1.0 sec)	0.122



Site Class Seismic Design Parameters

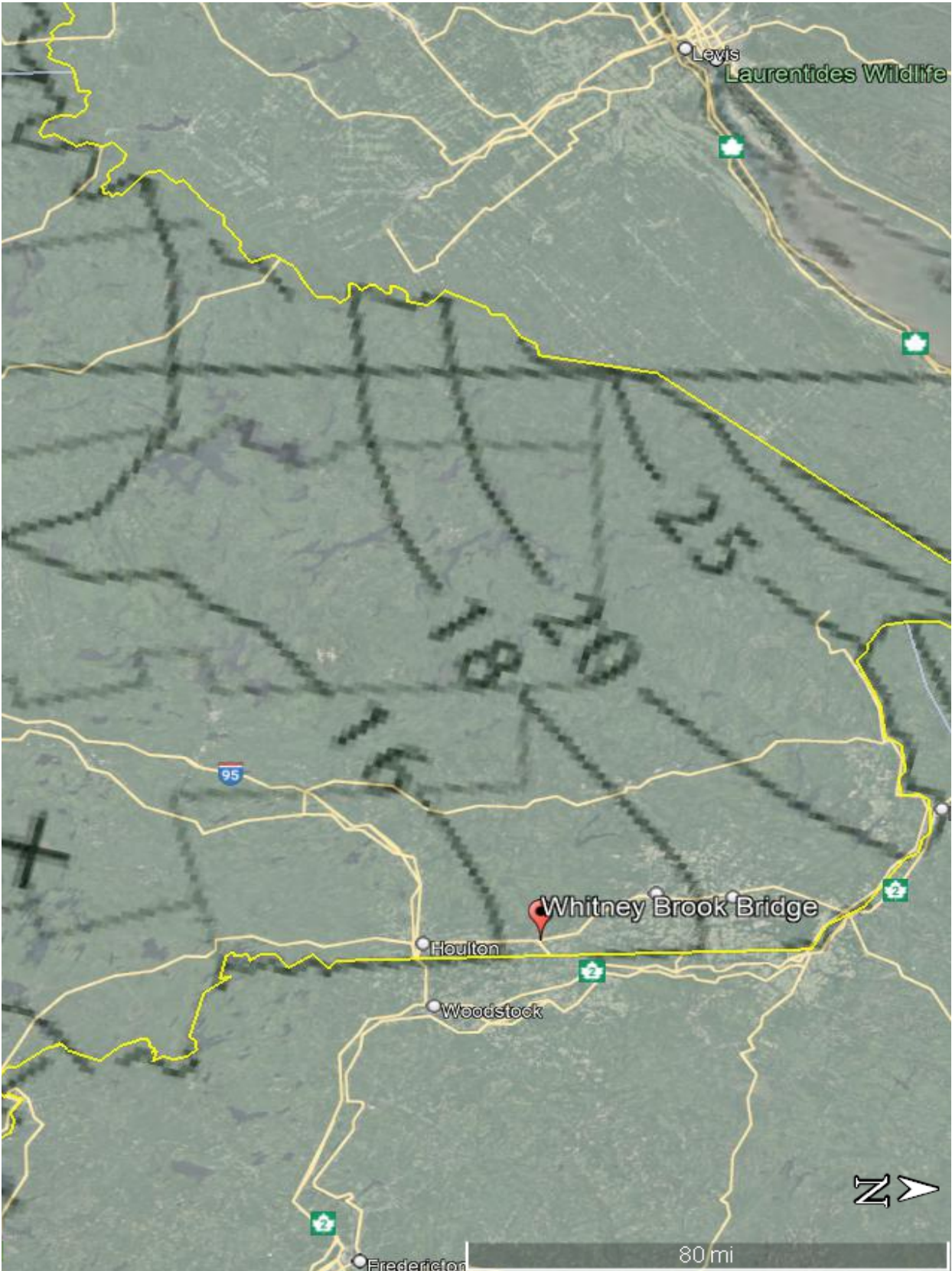
Whitney Brook Bridge No. 2942
Route 1 over Whitney Brook, Bridgewater, Maine
GZA Job#: 09.0026100.10





Site Class Seismic Design Parameters

Whitney Brook Bridge No. 2942
Route 1 over Whitney Brook, Bridgewater, Maine
GZA Job#: 09.0026100.10





Site Class Seismic Design Parameters

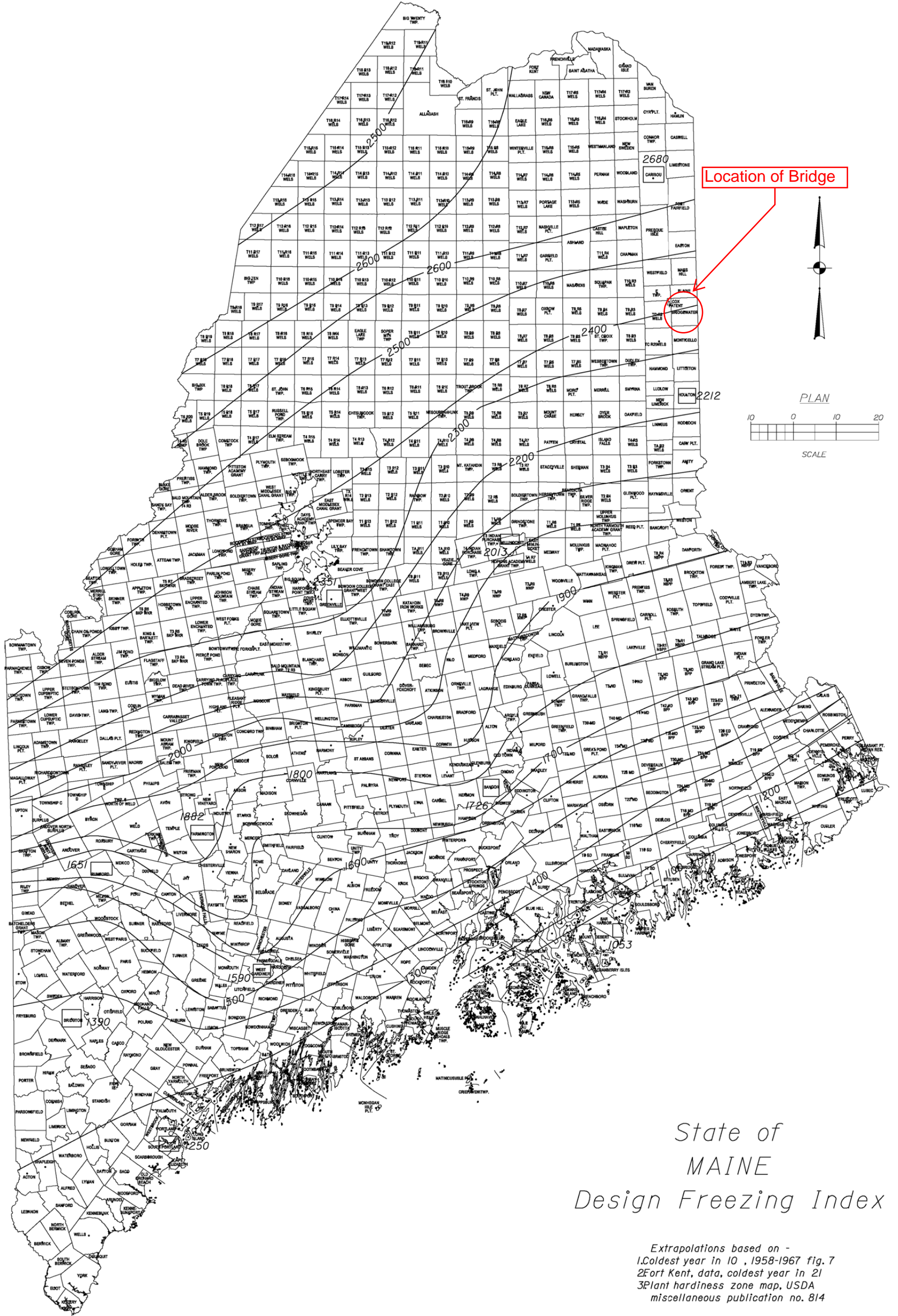
Whitney Brook Bridge No. 2942
Route 1 over Whitney Brook, Bridgewater, Maine
GZA Job#: 09.0026100.10





F.7 Frost Depth

Figure 5-1 Maine Design Freezing Index Map



5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Note: Testing and descriptions of the top 15 feet of soil typically showed Fine to Coarse Sand with varying amount of Silt and Gravel, so the average of fine-grained and coarse grained for 2400 Freezing Index was used for Frost depth = 7.45'. We recommend 7.5' for frost.

CHAPTER 5 - SUBSTRUCTURES

- 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.



GZA GeoEnvironmental, Inc.