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MEMORANDUM

TO: Ms. Laura Krusinski
Maine Department of Transportation

FROM: Blaine Cardali, P.E.
Christopher Snow, P.E.
Andrew R. Blaisdell, P.E.

DATE: April 30, 2021

FILE NO.: 09.0026073.00

SUBJECT: Geotechnical Design Basis Memorandum
Spring Brook Bridge No. 2794
Maine Department of Transportation WIN 022608.00
Camden, Maine



GZA GeoEnvironmental, Inc. (GZA) has prepared this memorandum for the proposed Spring Brook Bridge which carries Route 1 over Spring Brook in Camden, Maine. Our work was completed under GZA's June 3, 2020 General Consulting Agreement (GCA CTM202060300000000709) with the Maine Department of Transportation (MaineDOT) Bridge Program, and incorporates GZA's Proposal No. 09.P000069.21, dated August 4, 2020, and the Limitations Included in **Appendix A** of this memorandum. Calderwood Engineering is serving as the bridge designer for MaineDOT.

BACKGROUND

Spring Brook Bridge carries U.S. Route 1 over Spring Brook in Camden, Maine. The existing structure was constructed in 1918 and is a reinforced concrete box culvert with a 10-foot 6-inch span and a 10-foot rise. The culvert was extended upstream in 1933 when the roadway was widened. The culvert spans Spring Brook and sits at the base of a gully. Both approaches consist of tall embankment fills and there is approximately 22 feet of fill over the top of the concrete box. The culvert is generally in poor condition with severe undermining at the downstream end, significant amounts of exposed rebar in the culvert top slab, and large spalls at the downstream end.

This project will coincide with a highway reconstruction project on U.S. Route 1 in Camden. The road reconstruction includes an improvement to the existing vertical alignment at the Spring Brook Crossing which will result in an additional grade raise of approximately 6 feet at the culvert. It has been determined that with the addition of 6 feet of fill on the existing culvert, the top slab will be overstressed, and the culvert needs replacement.

In January 2020 the project was advertised as a "detail-build" with a culvert replacement to consist of either a precast concrete arch or a composite arch bridge on cast-in-



place concrete spread-footing foundations on bedrock. The project bids were rejected, and the project progressed into redesign as the current CM/GC project.

PROPOSED CONSTRUCTION

The CM-GC bridge designer, Calderwood Engineering, developed the current design as a 140-foot, single-span, pile-supported integral abutment bridge.

SUBSURFACE EXPLORATIONS

MaineDOT provided a Geotechnical Design Report on January 17, 2020, which included subsurface investigation programs completed in 2003 and 2015. GZA's evaluations are based on two of those test borings (BB-CSB-101 and BB-CSB-102A) conducted in 2015, as well as four test borings (BB-SB-101 through BB-SB-104) and eight rod soundings (RS-101 through RS-105A) conducted in 2003.

The borings were drilled using, 3- and 4-inch casing, and drive- or spin-and-wash drilling techniques. Standard penetration testing (SPT) and split spoon sampling were performed at standard 5-foot intervals using a 24-inch-long, 1-3/8-inch inside diameter sampler. SPTs were conducted using a rope and cathead hammer system except for borings BB-CSB-101 and -102A which were conducted using an automatic hammer with an energy transfer ratio of 0.908.

The as-drilled boring locations were surveyed by MaineDOT and are shown on **Figure 1**. Elevations referenced in this report are in feet and refer to the National American Vertical Datum of 1988 (NAVD 88). The test boring logs and additional description of these exploration programs may be found in MaineDOT's January 17, 2020 Geotechnical Design Report which is included as **Appendix B** of this report.

LABORATORY TESTING

Laboratory testing was conducted by MaineDOT on split-spoon soil samples retrieved during the 2015 investigation. The testing program included:

- Four (4) standard gradation analysis / MaineDOT Frost Classification / Unified Soil Classification System (USCS) assessments;
- Six (6) gradation analysis with hydrometer / MaineDOT Frost Classification / USCS assessments; and
- Ten (10) water content evaluations;

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



SUBSURFACE PROFILE

Two soil units were interpreted by MaineDOT beneath 2 to 9 inches of asphalt pavement and above bedrock at the site: Fill and Glacial Till, as summarized in the table that follows.

Soil Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	28 to 33	Varying <u>from</u> brown to olive brown, medium dense to dense, Gravelly SAND, little to some silt, <u>to</u> stiff, SILT, some sand, little to some gravel. Typical MaineDOT Frost Classification Range= II to IV <i>Encountered in all borings.</i>
Glacial Till	3.2 to 17	Varying <u>from</u> brown, very dense, GRAVEL, some sand, trace to little silt <u>to</u> very stiff, SILT, some sand, little to some gravel. (USCS: GW-GM, GC-GM). MaineDOT Frost Classification = 0 to II <i>Encountered in boring BB-CSB-101, BB-SB-101, BB-SB-103, and BB-SB-104.</i>
Estimated Top of Bedrock		Abutment 1: Approx. El. 34 to 46 Abutment 2: Approx. El. 49 to 51

Refer to the **Figure 2, Interpretive Subsurface Profile** for additional details regarding variations in stratification and unit thickness. Please note that GZA did not observe any of the soil samples for this project. The generalized descriptions above are interpreted from the boring logs provided by MaineDOT.

BEDROCK

Bedrock was cored in all four borings and was described by MaineDOT as hard, fresh, fine grained, grey, GNEISS and highly fractured fine-grained METASANDSTONE. In general, the joints are described as close to moderately spaced, low to high angle. The Rock Quality Designation (RQD) in the core runs ranged from 11 to 90 percent, which corresponds to Very Poor to Good rock quality.

GROUNDWATER

The groundwater levels were not observed during the borings. Fluctuations in groundwater levels will occur due to variations in season, precipitation, and construction activity in the area. Consequently, water levels during and after construction are likely to vary at the time the observations were made.

ENGINEERING EVALUATIONS

FOUNDATION TYPE

The January 2020 plans called for either a precast concrete arch or a composite arch bridge to be supported on cast-in-place concrete spread footing foundations bearing on bedrock. Both culvert design alternatives required a very deep support of excavation system. Review of the bid results indicated that support of excavation was a major factor in the high original bid prices. In order to enhance constructability, a single span, pile supported, integral abutment bridge is now the preferred CM/GC alternative foundation system.



PILE DESIGN CONSIDERATIONS

Considering the overburden thickness of approximately 20 to 30 feet below bottom of backwall at the abutments, we anticipate that piles can be driven to gain sufficient resistance primarily through end bearing in glacial till or on bedrock.

Axial tensile geotechnical (uplift) resistance was not evaluated because the integral abutment configuration will not impose uplift loading on the piles. Since the piles will gain support in primarily dense granular soil and/or bedrock, there is no reduction for group interaction in axial compression.

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

LOAD AND RESISTANCE FACTORS

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

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

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV) and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in AASHTO Table 3.4.1-2 for strength and extreme limit state design. A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck, per MaineDOT BDG Section 5.4.2.11.

PILE TYPE

The abutments are planned to be supported on ASTM A572, Grade 50 ($f_y=50$ kips per square inch [ksi]) steel HP14x89 piles oriented for strong axis bending. Each abutment will include six piles.

PILE LOADS

Calderwood Engineering provided a maximum factored axial load of 340 kips per pile for the strength condition. Therefore, piles should be installed to a nominal axial resistance of at least 523 kips, calculated by dividing the maximum factored axial load by a geotechnical resistance factor of 0.65. The resistance factor assumes dynamic pile testing with signal matching analysis will be conducted during construction in accordance with AASHTO requirements to assess nominal geotechnical pile resistance.



DESIGN-PHASE PILE DRIVABILITY ANALYSIS

Evaluations were conducted for axial compressive geotechnical resistance of the piles. Side friction was calculated using the Meyerhof (SPT) for granular layers (Existing Fill and Glacial Till) in accordance with AASHTO Article 10.7. Based on our experience with similar soils, we anticipate that the piles will be driven on or near bedrock to achieve the required resistance. The results of our evaluations indicate the piles will gain support primarily through end bearing on glacial till or bedrock, with limited contribution from friction in overburden soils. The geotechnical side resistance was used as an input in wave equation analyses conducted to assess the pile drivability. The contribution of side friction was estimated to be approximately 10 percent of the required nominal resistance.

GZA completed design phase wave equation analyses to assess the drivability of an HP 14x89 pile with a nominal geotechnical resistance of 523 kips at the abutments. Analyses were completed using a Delmag D16-32 diesel hammer with a ram weight of 3,520 pounds and a maximum rated energy of 40,200 foot-pounds (ft-lbs). A 20-foot-long pile was found to be the shortest anticipated pile length and was assumed to encounter very hard driving conditions (toe quake of 0.04) on bedrock. The results are summarized below.

SUMMARY OF WEAP ANALYSES					
Pile Analysis and Type	Embedded Pile Length	Driving System ¹	Required Nominal Geotechnical Resistance (kips)	Max Driving Stress (ksi)	Final Penetration Resistance (blows per inch)
Abutment 1 HP 14x89	20-30 feet	Delmag D 16-32 (Fuel setting 1, 100% of maximum pressure)	523	42	9

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

LATERAL PILE ANALYSIS

GZA developed a soil profile for lateral pile evaluations based on the materials encountered in BB-SB-101 through -104 located near the proposed abutments. The controlling profile for lateral analyses is expected, where the shortest piles are anticipated, at Abutment 2. The prevalent subsurface stratum in the abutment area is existing fill with a possible thin mantle of glacial till near the bedrock. The design soil profile developed for lateral pile evaluations using LPILE by Ensoft is summarized in the table below.



Abutments 1 & 2					
Stratum	Soil Model	Top of Layer Depth Below Pile Cap (ft)	k (pci)	ϕ' (deg)	γ_e (pcf)
Existing Fill	Reese Sand	0.0	143	35	125
Top of Rock	--	20.0 – 30.0	--	--	159

Notes:

1. These parameters do not include reductions for group interaction. Reduction Factors should be applied in accordance with AASHTO 10.7.2.4 if spacing is less than 5 pile diameters.

We understand that Calderwood Engineering is completing lateral pile analyses to evaluate pile fixity and combined stresses in conjunction with the structural design. GZA completed a review of Calderwood Engineering’s lateral pile analyses and found that the input parameters were applied in general accordance with our recommendations.

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 MaineDOT BDG Type 4 soil.

Based on the estimated thermal bridge expansion of 0.328 inches and the abutment height of approximately 12 feet, the calculated abutment rotation is 0.0023 feet/foot. In accordance with the requirements of the BDG Section 5.4.2.11, integral abutment reinforcement is to be designed for full Rankine passive pressure if the wall rotation is less than 0.005 feet/foot. Therefore, we conclude that Rankine passive earth pressure is appropriate for design. We recommend using a Rankine Coefficient of Passive Earth Pressure, $K_p = 3.25$ for design of backwalls and wingwalls. AASHTO Commentary C3.10.9.1 specifies that single-span bridges in Seismic Zone 2 are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

SEISMIC DESIGN

The subsurface profile for seismic design includes the embankment fill and Glacial Till overlying bedrock. Seismic site class was determined in general accordance with LRFD Table C3.10.3.1. Since the average SPT N-value within the soil profile is between 15 and 50 blows per foot in the borings, the bridge is assigned to Seismic Site Class D.

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

The peak ground acceleration coefficient, and short- and long-period spectral acceleration coefficients were interpolated from the AASHTO LRFD Acceleration Coefficient Maps (3.10.2.1-1 through -21 as appropriate). Based on the site coordinates, the recommended AASHTO Response Spectrum (Site Class D) is developed for a 7 percent probability of exceedance in 75 years. These results are summarized for the site as follows:



SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.10 g
SDs (Period = 0.2 sec)	0.22 g
SD1 (Period = 1.0 sec)	0.10 g

RECOMMENDATIONS

PILE DESIGN

- The proposed abutments may be supported on HP14x89 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed through a combination of side friction and end-bearing on or near the bedrock surface.
- Cobbles and high penetration resistances were encountered in several of the test borings that may create very hard driving conditions. Although not anticipated, the contractor should be prepared to spud, predrill or pre-excavate to penetrate these materials.
- To limit driving damage, the steel H-piles should be fitted with cast steel driving tips in accordance with MaineDOT Standard Specification Section 501.10 – Pile Tips.
- Pile installation should be controlled using wave equation analysis and field logging of the pile installation with final penetration resistance based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance of 523 kips, calculated by dividing the maximum factored pile load of 340 kips by a resistance factor of 0.65.
- Preliminary wave equation analyses indicate that the piles can be driven to the required nominal resistance using a diesel hammer with a rated energy of about 40,200 ft-lbs for the approximately 20- to 30-foot-long, ASTM A572 Grade 50 HP14x89 piles without exceeding the allowable driving stress of 45 ksi (0.9F_y for 50 ksi steel), and with a final penetration resistance of 9 blows per inch, which is within the MaineDOT range of 6 to 15 blows per inch.
- The pile tip elevations used in the drawings should correspond to the bedrock elevations encountered in the borings (approximately El. 34 to 46 at Abutment 1, and approximately El. 49 to 51 at Abutment 2). A provision is recommended in the drawings for extra pile length to account for variability in the top of rock surface and the potential for piles to penetrate a short distance into the bedrock.
- We recommend that the first pile be dynamically tested at each abutment during initial driving to assess driving stress and establish the penetration resistance criteria to achieve the required penetration and



nominal resistance for the production piles. The plans should also require a 24-hour restrike test on each test pile, to assess potential relaxation.

- Piles shall be spliced in accordance with MaineDOT Section 501.047.
- Piles should be checked for resistance to combined axial compression and flexure per AASHTO LRFD Articles 6.9.2.2 and 6.15.2. Per LRFD Article 6.5.4.2, the axial resistance factor $\phi_{cc}=0.7$ 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. 6.9.2.2-1), or other methods acceptable to MaineDOT.
- Approach slabs should be constructed at each abutment to smooth the transition from the approach embankments to the bridge. The slabs should be positively connected to the backwalls.

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
 - Rankine Coefficient of Passive Earth Pressure, $K_p = 3.25$ (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 or wing wall height and distance from the wall's back face 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 embankment side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. The drains should be sloped to drain by gravity and should outlet through a series of 4-inch-diameter weep holes, spaced approximately 10 feet center-to-center. Alternatively, prefabricated drainage geocomposite material can be placed against the uphill side of abutments, after holes have been created through the backing material at the weep hole locations.

CONSTRUCTION CONSIDERATIONS

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 a Dynamic Load Test with Signal Matching be completed at each abutment at the end of drive and beginning of restrike approximately 24 hours later.



PILE OBSTRUCTIONS

Cobbles were noted within the Fill and Glacial Till in the test borings. This is likely to result in hard driving conditions that may require pre-drilling, pre-excavation or spudding to bypass potential obstructions.

EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavations for abutment foundations are anticipated to be on the order of 12 feet below existing pavement grades. It is our understanding that Route 1 will be out of service during construction of the new bridge. In areas where sufficient space is available and water conditions permit, the excavation adjacent to the approaches may be constructed with sloped, open cuts. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

Since the bottom of proposed abutment elevations are proposed at approximately El. 69 to El. 75 and the Q1.1 water level is approximately El. 52 stream levels are not anticipated to impact abutment construction.

We anticipate that any inflow of groundwater or surface water to excavations could be handled by open pumping from sumps installed at the bottoms of excavations. The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the subgrade and permit concrete placement in-the-dry. Discharge of pumped water should comply with all local, State, and federal regulations.

CLOSURE

We trust that this information meets current project needs. Please feel free to call Chris Snow at (207) 358-5118 for additional information.

BMC/CLS/ARB:dim

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Attachments: Figure 1 – Boring Location Plan
Figure 2 - Interpretive Subsurface Profile

Table 1 – Subsurface Summary Table

Appendix A – Limitations
Appendix B.1 – 2003 Test Boring Logs
Appendix B.2 – 2015 Test Boring Logs
Appendix C – Laboratory Testing
Appendix D – Engineering Calculations



FIGURES

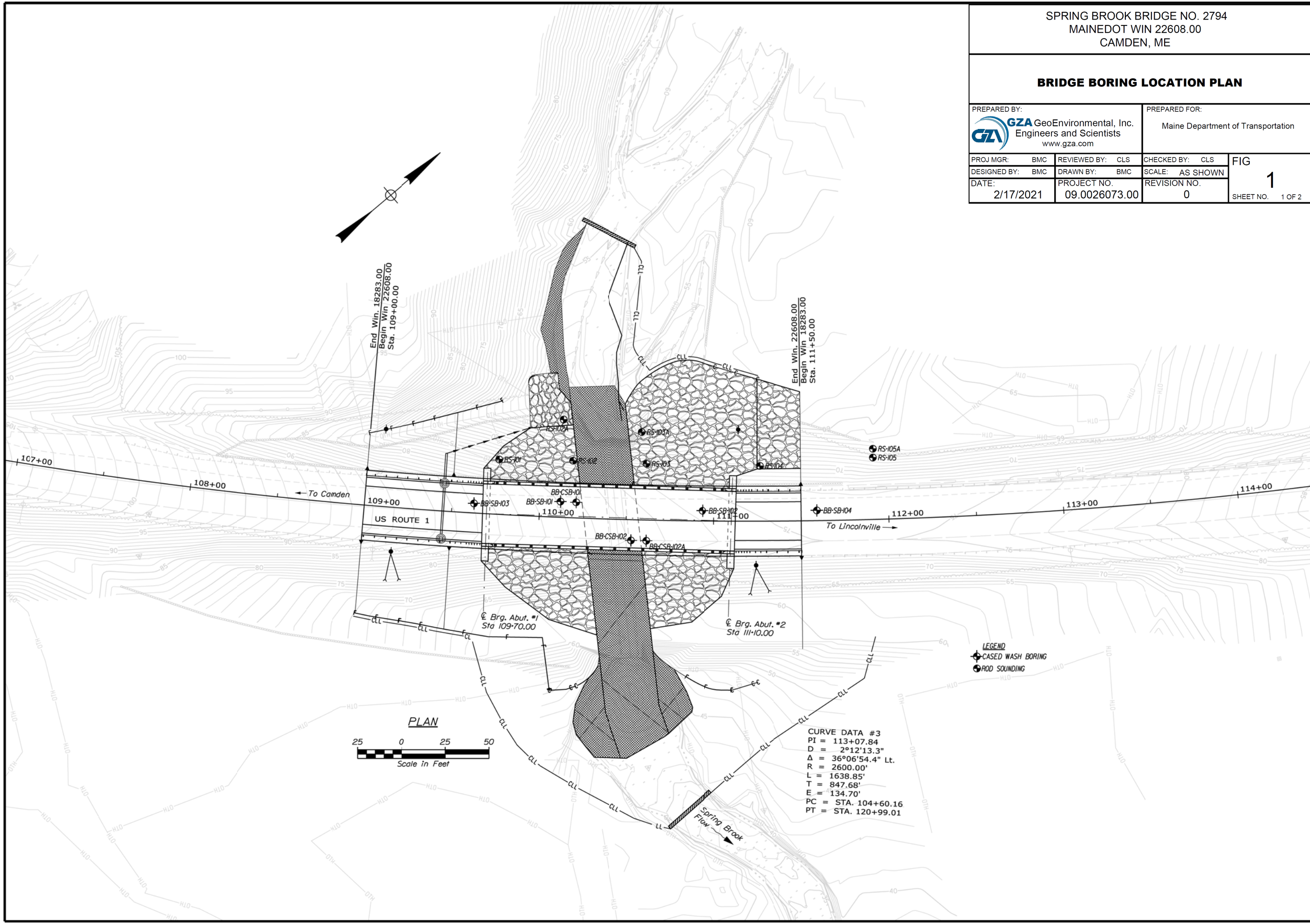
SPRING BROOK BRIDGE NO. 2794
 MAINEDOT WIN 22608.00
 CAMDEN, ME

BRIDGE BORING LOCATION PLAN

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: Maine Department of Transportation	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: CLS	FIG
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	1
DATE: 2/17/2021	PROJECT NO. 09.0026073.00	REVISION NO. 0	

STATE OF MAINE
 DEPARTMENT OF TRANSPORTATION
 NHP-2260(800)
 WIN 22608.00
 BRIDGE NO. 2794
 BRIDGE PLANS

Date: 10/2/2020
 Username: common
 Division: BRIDGE
 Filename: ... \Final_Details\005_BLP1.dgn



PROJ. MANAGER	ALATHE	BY	DATE
DESIGN-DETAILED	KCN	JAB	03.07.2018
CHECKED-REVIEWED	LARUSINSKI	T WHITE	DEC. 2019
DESIGN-DETAILED	REVISIONS 1		
	REVISIONS 2		
	REVISIONS 3		
	REVISIONS 4		
	FIELD CHANGES		

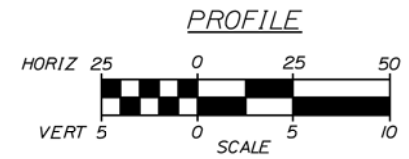
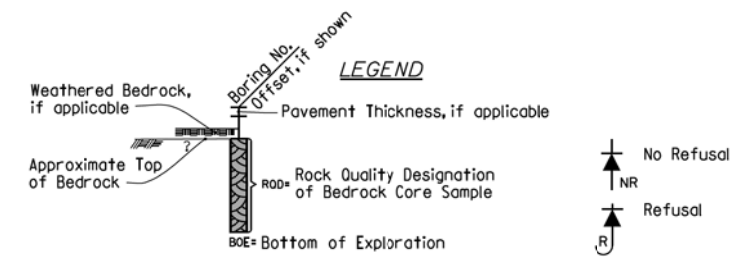
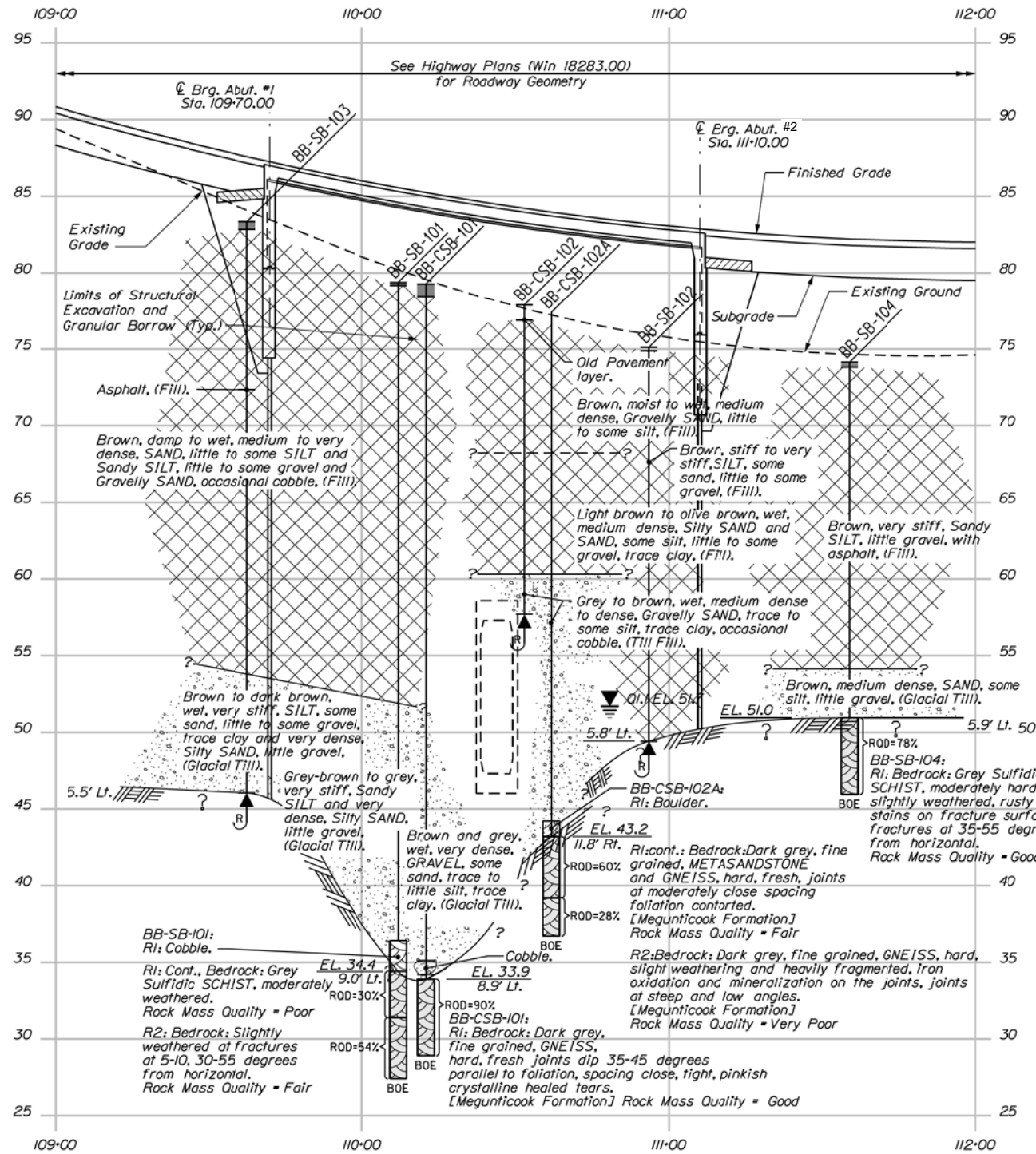
SPRING BROOK BRIDGE
 SPRING BROOK
 KNOX COUNTY
 CAMDEN
 BORING LOCATION PLAN

SHEET NUMBER
A5
 OF A30

INTERPRETIVE SUBSURFACE PROFILE

PREPARED BY: GZA GeoEnvironmental, Inc. Engineers and Scientists www.gza.com		PREPARED FOR: Maine Department of Transportation	
PROJ MGR: BMC	REVIEWED BY: CLS	CHECKED BY: CLS	FIG 2 SHEET NO. 2 OF 2
DESIGNED BY: BMC	DRAWN BY: BMC	SCALE: AS SHOWN	
DATE: 2/17/2021	PROJECT NO. 09.0026073.00	REVISION NO. 0	

STATE OF MAINE
 DEPARTMENT OF TRANSPORTATION
 NHPP-2260(800)
 WIN 22608.00
 BRIDGE NO. 2794
 BRIDGE PLANS



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil and bedrock transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

PROJ. MANAGER	DATE	BY	DATE
ALATHE <td>03.07.2018 <td>JAB <td></td> </td></td>	03.07.2018 <td>JAB <td></td> </td>	JAB <td></td>	
CHECKED-REVIEWED <td></td> <td>KCN <td></td> </td>		KCN <td></td>	
DESIGN-REVIEWED <td></td> <td>JUN/SUNSA <td></td> </td>		JUN/SUNSA <td></td>	
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REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SPRING BROOK BRIDGE
 SPRING BROOK
 CAMDEN
 KNOX COUNTY
 INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
A6
 OF A30

Date: 10/2/2020

Username: common

Division: BRIDGE

Filename: ... \Final_Details\006_ISP1.dgn



TABLE



TABLE 1
Summary of Subsurface Explorations
 Spring Brook Bridge No. 2794, Route 1 over Spring Brook
 Camden, ME
 WIN 22608.00

Boring ID	Ground Surface El. (ft)	Top of Stratum Elevation (ft)				Stratum Thickness (ft)			Depth to Top of Probable Bedrock (ft)	Bottom of Boring Depth (ft)	Bottom of Boring El. (ft)	Groundwater	
		Pavement	Fill	Glacial Till	Possible Bedrock	Pavement	Fill	Glacial Till				El. (ft)	Depth (ft)
BB-CSB-101	79.3	79.3	78.5	50.8	33.9	0.8	27.7	16.9	45.4	50.4	28.9	NE	NE
BB-CSB-102	77.9	NE	77.9	NE	NE	NE	20.2	NE	NE	20.2	57.7	NE	NE
BB-CSB-102A	77.4	NE	77.4	NE	43.2	NE	34.2	NE	34.2	40.7	36.7	NE	NE
BB-SB-101	79.4	79.4	79.2	51.9	34.4	0.2	27.3	17.5	45.0	52.0	27.4	NE	>28
BB-SB-102	75.1	75.1	74.9	NE	49.5	0.3	25.4	NE	25.7	25.7	49.4	NE	>8.5
BB-SB-103	83.3	83.3	82.8	53.8	46.0	0.5	29.0	7.8	37.3	37.3	46.0	61.0	22.3
BB-SB-104	74.2	74.2	73.9	54.2	51.0	0.3	19.7	3.2	23.2	28.2	46.0	NE	27.0

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

Notes:

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



APPENDIX A – LIMITATIONS



LIMITATIONS

Use of Report

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

Standard of Care

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

Subsurface Conditions

4. The generalized soil profile(s) provided in our Report are based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs.
5. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.
6. Water level readings have been made in test holes (as described in the Report) 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.
7. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

Compliance with Codes and Regulations

8. 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.



APPENDIX B – BORING LOGS

APPENDIX B.1 - BORINGS BY MAINEDOT

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																																																							
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)																																																						
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>trace</u> 0 - 10 <u>little</u> 11 - 20 <u>some</u> 21 - 35 <u>adjective (e.g. sandy, clayey)</u> 36 - 50	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="0"> <tr> <td><u>Density of Cohesionless Soils</u></td> <td><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <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> </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="0"> <tr> <td><u>Consistency of Cohesive soils</u></td> <td><u>SPT N-Value (blows per foot)</u></td> <td><u>Approximate Undrained Shear Strength (psf)</u></td> <td><u>Field Guidelines</u></td> </tr> <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> </table> <u>Rock Quality Designation (RQD):</u> RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) <table border="0"> <tr> <td colspan="2">Correlation of RQD to Rock Mass Quality</td> </tr> <tr> <td><u>Rock Mass Quality</u></td> <td><u>RQD (%)</u></td> </tr> <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> </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 mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))	<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50	<u>Consistency of Cohesive soils</u>	<u>SPT N-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>	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	Correlation of RQD to Rock Mass Quality		<u>Rock Mass Quality</u>	<u>RQD (%)</u>	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100
		<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>																																																								
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Excellent	91 - 100																																																										
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.																																																								
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																																																								
		OL	Organic silts and organic silty clays of low plasticity.																																																								
	SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.																																																								
CH		Inorganic clays of high plasticity, fat clays.																																																									
OH	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				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 Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information																																																											

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook Location: Camden, Maine	Boring No.: BB-CSB-101 WIN: 22608.00
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Driller: MaineDOT	Elevation (ft.): 79.3	Auger ID/OD: 5" Solid Stem
Operator: Giles/Daggett/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2015; 06:30-12:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 110+20.9, 8.9 ft Lt.	Casing ID/OD: NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.908	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 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 _u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
	T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample 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								SSA	78.5	10" Pavement		
	1D	24/12	1.00 - 3.00	9/13/16/10	29	44				Brown, damp, dense, SAND, some gravel, little silt, (Fill).	0.8	
5	2D	24/15	5.00 - 7.00	4/6/7/9	13	20				Brown, moist, very stiff, Sandy SILT, little gravel, (Fill).	G#264783 A-4, SM WC=11.6%	
10	3D	24/13	10.00 - 12.00	3/3/4/6	7	11	11			Brown, moist, stiff, SILT, some sand, some gravel, trace clay, (Fill).	G#264784 A-4, SC-SM WC=12.4%	
15	4D	24/14	15.00 - 17.00	6/5/8/9	13	20	13			Brown, wet, medium dense, SAND, some gravel, little silt (Fill).	G#264785 A-1-b, SM WC=15.2%	
20	5D	24/13	20.00 - 22.00	14/7/6/14	13	20	28			Brown, wet, very stiff, SILT, some sand, some gravel, trace clay.	G#264786 A-4, SC-SM WC=14.5%	
25							174			Roller Coned ahead to 25.5 ft bgs.		

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook Location: Camden, Maine	Boring No.: BB-CSB-101 WIN: 22608.00
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Driller: MaineDOT Operator: Giles/Daggett/Giles Logged By: B. Wilder Date Start/Finish: 6/24/2015; 06:30-12:30 Boring Location: 110+20.9, 8.9 ft Lt.	Elevation (ft.): 79.3 Datum: NAVD88 Rig Type: CME 45C Drilling Method: Cased Wash Boring Casing ID/OD: NW	Auger ID/OD: 5" Solid Stem Sampler: Standard Split Spoon Hammer Wt./Fall: 140#/30" Core Barrel: NQ-2" Water Level*: None Observed
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Hammer Efficiency Factor: 0.908 <small>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</small>	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/> <small>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</small>	<small>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</small>
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	6D	24/10	25.50 - 27.50	9/12/27/18	39	59	79		50.8	Similar to above, except very dense and occasional cobble.		
							62	Roller Coned ahead to 30.0 ft bgs.				
							57					
							40					
30	7D	24/14	30.00 - 32.00	11/19/19/19	38	58	26		28.5	Brown, wet, very dense, GRAVEL, some sand, trace silt, (Glacial Till).	G#264787 A-1-a, GW-GM WC=7.3%	
							43					
							82					
							80					
35	8D	24/16	35.00 - 37.00	17/15/13/9	28	42	36		28.5	Similar to above, except dense.		
							40	Roller Coned ahead to 40.0 ft bgs.				
							86					
							66					
40	9D	8.4/6	40.00 - 40.70	57/40(2.4")	---		48		33.9	Grey, wet, very dense, GRAVEL, some sand, little silt, trace clay, (Glacial Till).	G#264788 A-1-b, GC-GM WC=6.0%	
							64	Roller Coned ahead to 45.4 ft bgs.				
							73					
							75					
45	R1	60/60	45.40 - 50.40	RQD = 90%			NQ-2		33.9	a40 blows for 0.2 ft. Cobble from 44.2-45.1 ft bgs.		
								45.4-46.4 ft (4:34)				
								46.4-47.4 ft (3:11)				
50										Top of Bedrock at Elev. 33.9 ft. R1: Bedrock: Dark grey, fine grained, GNEISS, hard, fresh joints dip 35-45 degrees parallel to foliation, spacing close, tight, pinkish crystalline healed tears, iron staining on some joints. [Megunticook Formation] Rock Mass Quality = Good R1:Core Times (min:sec) 45.4-46.4 ft (4:34) 46.4-47.4 ft (3:11)		

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook	Boring No.: BB-CSB-101
	Location: Camden, Maine	WIN: 22608.00

Driller: MaineDOT	Elevation (ft.): 79.3	Auger ID/OD: 5" Solid Stem
Operator: Giles/Daggett/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2015; 06:30-12:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 110+20.9, 8.9 ft Lt.	Casing ID/OD: NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.908	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 N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
	T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50								28.9		47.4-48.4 ft (2:52) 48.4-49.4 ft (3:49) 49.4-50.4 ft (3:50) 100% Recovery Bent Casing 50.4- Bottom of Exploration at 50.4 feet below ground surface.		
55												
60												
65												
70												
75												

Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook Location: Camden, Maine	Boring No.: BB-CSB-102 WIN: 22608.00
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Driller: MaineDOT	Elevation (ft.): 77.9	Auger ID/OD: 5" Solid Stem
Operator: Giles/Daggett/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2015; 12:30-16:30	Drilling Method: Cased Wash Boring	Core Barrel: N/A
Boring Location: 110+53, 11.8 ft Rt.	Casing ID/OD: NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.908	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 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_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test		

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0	1D	24/16	0.00 - 2.00	8/4/5/4	9	14	SSA		[Hatched Pattern]	Brown, moist, medium dense, Gravelly, SAND, little silt, (Fill). Old pavement layer at 1.0 ft bgs.		
5	2D	24/8	5.00 - 7.00	3/4/5/6	9	14					Brown, wet, medium dense, Gravelly, SAND, some silt, (Fill).	G#264789 A-1-b, SM WC=8.7%
10	3D	24/14	10.00 - 12.00	4/5/7/4	12	18	28			Light brown, wet, medium dense, Silty SAND, little gravel, trace clay, (Fill).	G#264790 A-4, SC-SM WC=15.6%	
							26					
							23					
							37					
15	4D	24/15	14.00 - 16.00	12/6/5/13	11	17	22			Olive-brown, wet, medium dense, GRAVEL, some silt, some sand, trace clay, (Fill).	G#264791 A-2-4, GC-GM WC=16.0%	
							65					
							160					
							83			Roller Coned ahead to 17.0-19.0 ft bgs.		
							22					
20	5D	14.4/10	19.00 - 20.20	8/4/30(2.4")	---					Grey, wet, very dense, GRAVEL, some sand, some silt, trace clay, occasional cobble, (Fill).	G#264792 A-2-4, GC-GM WC=10.5%	
										Bottom of Exploration at 20.2 feet below ground surface. Top of Concrete Box, moved to BB-CSB-102A		
25												


Remarks:

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook Location: Camden, Maine	Boring No.: BB-CSB-102A WIN: 22608.00
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Driller: MaineDOT	Elevation (ft.): 77.4	Auger ID/OD: 5" Solid Stem
Operator: Giles/Daggett/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2015; 12:30-16:30	Drilling Method: Cased Wash Boring	Core Barrel: N/A
Boring Location: 110+61.7, 11.8 ft Rt.	Casing ID/OD: NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.908 **Hammer Type:** Automatic Hydraulic Rope & Cathead

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)				
0							SSA			Auger to 20.0 ft bgs and set in NW Casing. No soil samples recovered. Soils similar to BB-CSB-102.		
5												
10												
15												
20	1D	24/14	20.00 - 22.00	22/14/9/6	23	35	10	57.4		Brown, wet, dense, Gravelly, SAND, some silt, occasional cobble, (Till Fill).		
							13					
							9					
							14					
25							36					

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Spring Brook Bridge #2794 carries US Route 1 over Spring Brook Location: Camden, Maine	Boring No.: BB-CSB-102A WIN: 22608.00
--	---	--

Driller: MaineDOT	Elevation (ft.): 77.4	Auger ID/OD: 5" Solid Stem
Operator: Giles/Daggett/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/24/2015; 12:30-16:30	Drilling Method: Cased Wash Boring	Core Barrel: N/A
Boring Location: 110+61.7, 11.8 ft Rt.	Casing ID/OD: NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.908	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 N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected
	T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample 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	2D	24/4	25.00 - 27.00	12/8/9/11	17	26	20			Similar to above, except medium dense.		
							36					
							61					
							112					
							89					
30	3D	24/10	30.00 - 32.00	3/7/12/13	19	29	27			Brown, wet, medium dense, Gravelly, SAND, trace silt, (Till Fill).	G#264793 A-1-a, SW-SM WC=13.4%	
							65					
							141					
	R1	60/60	33.20 - 38.20	RQD = 60%			NQ-2	44.2		R1:Boulder. R1:Core Times (min:sec) 33.2-34.2 ft (13:52)		
								43.2				
35										Top of Bedrock at Elev. 43.2 ft. R1:Bedrock: Dark grey, fine grained, METASANDSTONE and GNEISS, hard, fresh, joints at moderately close spacing, foliation contorted. [Megunticook Formation]. Rock Mass Quality = Fair R1:Core Times (min:sec) 34.2-35.2 ft (3:22) 35.2-36.2 ft (2:45) 36.2-37.2 ft (2:17) 37.2-38.2 ft (2:28) 100% Recovery		
	R2	30/30	38.20 - 40.70	RQD = 11%						R2: Bedrock: Dark grey, fine grained, GNEISS, hard, slight weathering and heavily fragmented, iron oxidation and mineralization on the joints, joints at steep and low angles. [Megunticook Formation] Rock Mass Quality = Very Poor R2:Core Times (min:sec) 38.2-39.2 ft (3:10) 39.2-40.2 ft (3:05) 40.2-40.7 ft (5:00) 100% Recovery Core Blocked		
40								36.7				
45												
50												

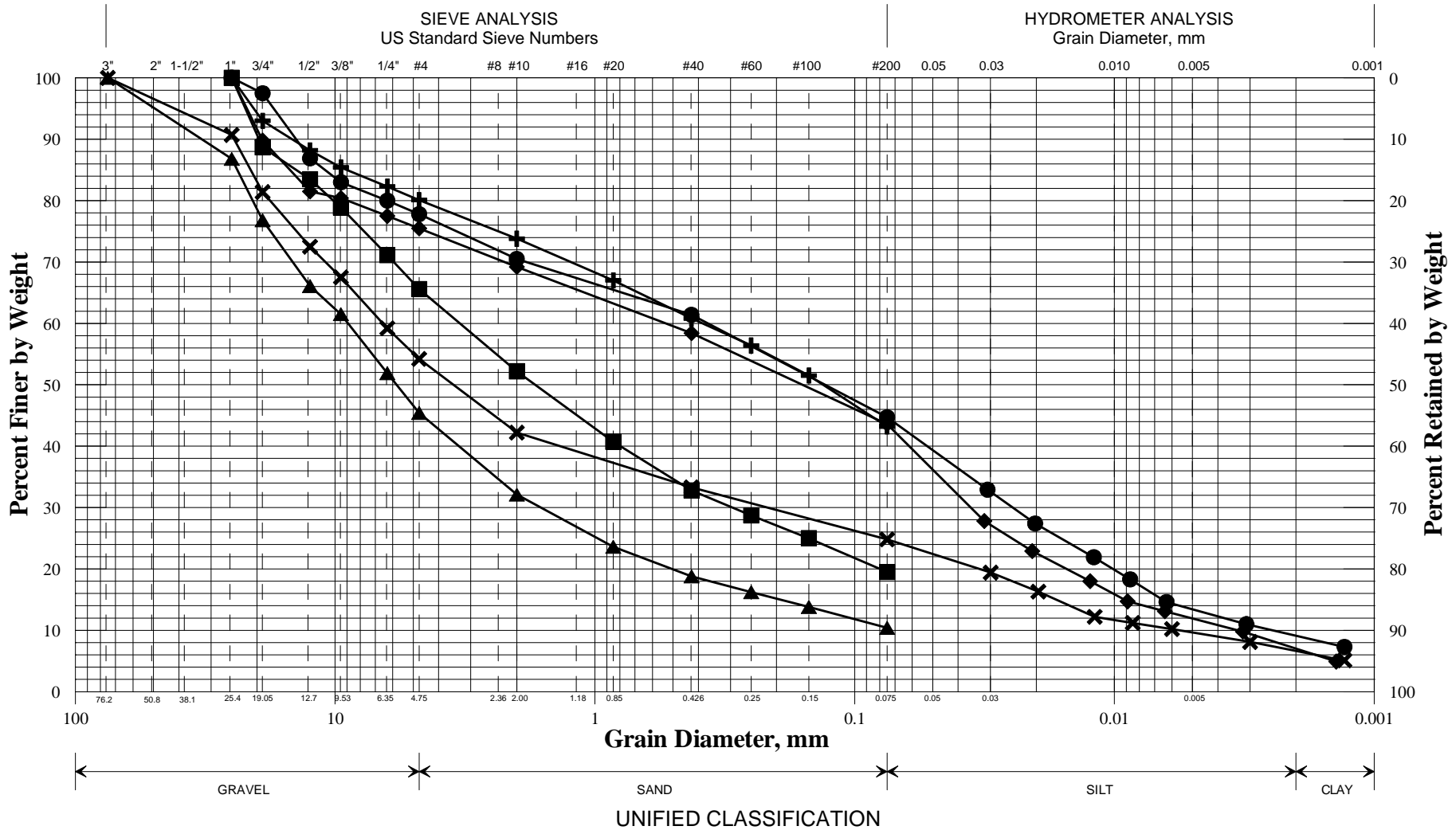
Remarks:

APPENDIX B.1 - BORINGS BY OTHERS



APPENDIX C – LABORATORY TESTING

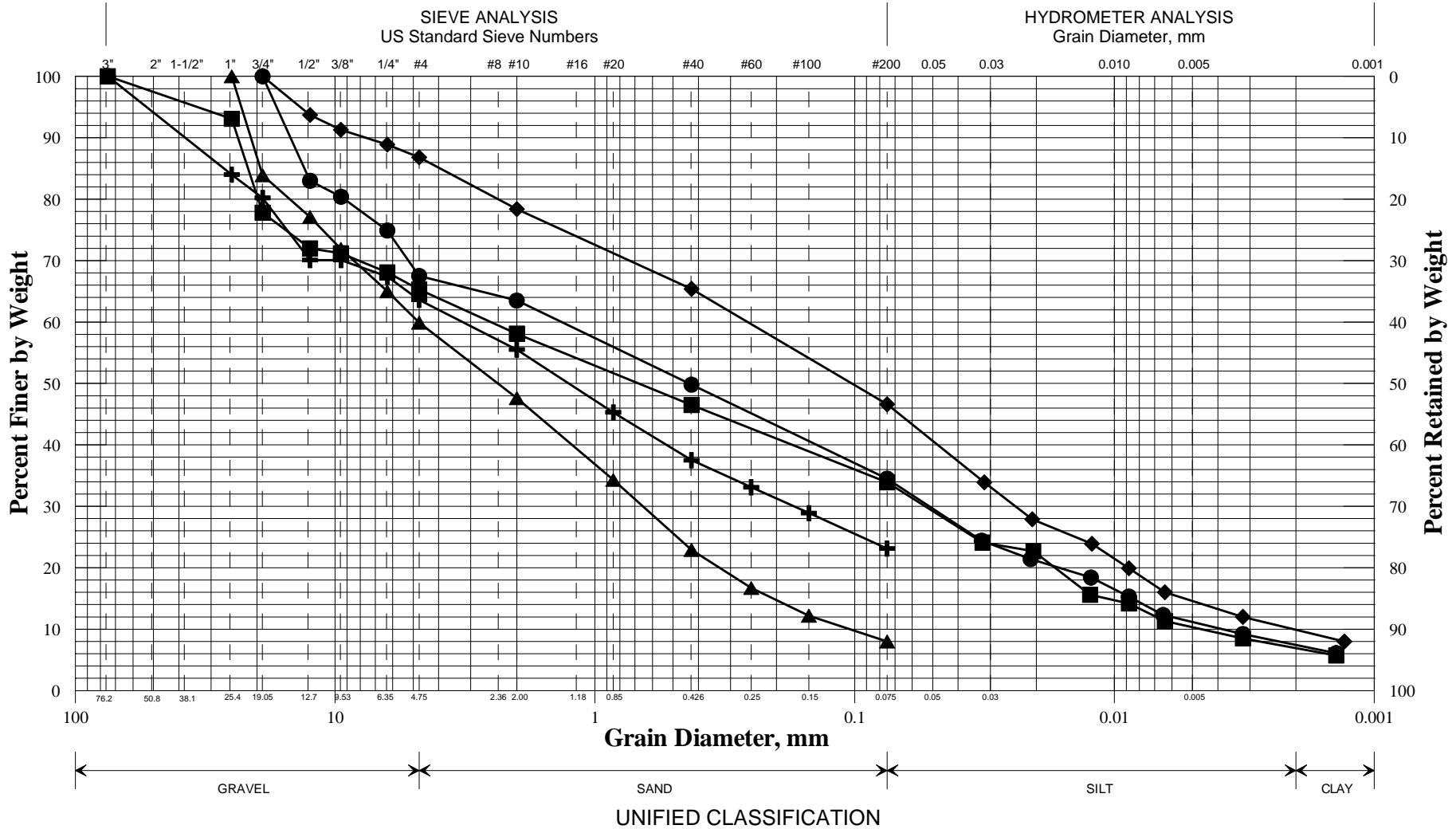
**State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE**



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-CSB-101/2D	110+20.9	8.9 LT	5.0-7.0	Sandy SILT, little gravel.	11.6			
◆	BB-CSB-101/3D	110+20.9	8.9 LT	10.0-12.0	SILT, some sand, some gravel, trace clay.	12.4			
■	BB-CSB-101/4D	110+20.9	8.9 LT	15.0-17.0	SAND, some gravel, little silt.	15.2			
●	BB-CSB-101/5D	110+20.9	8.9 LT	20.0-22.0	SILT, some sand, some gravel, trace clay.	14.5			
▲	BB-CSB-101/7D	110+20.9	8.9 LT	30.0-32.0	GRAVEL, some sand, trace silt.	7.3			
×	BB-CSB-101/9D	110+20.9	8.9 LT	40.0-40.7	GRAVEL, some sand, little silt, trace clay.	6.0			

WIN	
022608.00	
Town	
Camden	
Reported by/Date	
WHITE, TERRY A	7/15/2015

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-CSB-102/2D	110+53	11.8 RT	5.0-7.0	Gravelly SAND, some silt.	8.7			
◆	BB-CSB-102/3D	110+53	11.8 RT	10.0-12.0	Silty SAND, little gravel, trace clay.	15.6			
■	BB-CSB-102/4D	110+53	11.8 RT	14.0-16.0	GRAVEL, some sand, some silt, trace clay.	16.0			
●	BB-CSB-102/5D	110+53	11.8 RT	19.0-20.2	GRAVEL, some sand, some silt, trace clay.	10.5			
▲	BB-CSB-102A/3D	110+61.7	11.8 RT	30.0-32.0	Gravelly SAND, trace silt.	13.4			
×									

WIN	
022608.00	
Town	
Camden	
Reported by/Date	
WHITE, TERRY A	7/15/2015



APPENDIX D – ENGINEERING CALCULATIONS

SEISMIC

Seismic Site Class Calculation Summary

Project: Spring Brook Bridge **Project No.:** 09.0026073.00
Location: Camden, ME
Evaluated By/Date: ENT **Date** 11/30/2020
Checked By/Date: CLS **Date** 11/30/2020

Objective:

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

Subsurface Data:

Borings BB-SB-101 through -104 were drilled by MaineDOT between March 26 and April 1, 2003.
 Borings BB-CSB-101 through -102A were drilled by MaineDOT on June 24, 2015.

Assumptions:

Soil borings extended to depths between 25 and 52 feet below the roadway level and bedrock was encountered in the soil borings.

Approach:

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

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

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

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

- 5) Categorize the site using one of the following three methods in AASHTO C3.10.3.1-1:
 - v_s (Method A)
 - N (Method B)
 - N_{ch} and s_u (Method C)
 If shear wave velocity data are available, they should be used to classify the site. The N and s_u methods should only be used if shear wave velocity data is not available, as the correlation between site amplification and these geotechnical parameters is more uncertain (and therefore more conservative) than the correlation with v_s .

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

Boring ID	BB-SB-101	BB-SB-102	BB-SB-103	BB-SB-104	BB-CSB-101	BB-CSB-102/102A	Average
N-Value	42	38	43	18	36	25	33

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

INPUT

Exploration ID: BB-SB-101

Ground Surface Elevation: 79.4 ft

Depth of Boring: 52.0 ft

Depth to Bedrock: 45.0

EQUATIONS

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

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

CALCULATION

N = 41.7

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	4.0	6.0	74.4	40	8.0	0.20	
	10.0	12.0	68.4	14	5.5	0.39	
	15.0	17.0	63.4	20	5.0	0.25	
	20.0	22.0	58.4	44	5.0	0.11	
	25.0	27.0	53.4	79	5.0	0.06	
	30.0	32.0	48.4	102	5.0	0.05	
Glacial Till	35.0	35.0	44.4	50	4.0	0.08	
Top of Rock	40.0	40.1	39.4	50	62.5	1.25	
	45.0						

100.00

INPUT

Exploration ID: BB-SB-102 Ground Surface Elevation: 75.1 ft Depth of Boring: 25.7 ft
Depth to Bedrock: 25.7

EQUATIONS

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

CALCULATION

N = 37.7

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	5.0	7.0	69.1	18	8.5	0.47	
	10.0	12.0	64.1	46	5.0	0.11	
	15.0	15.9	59.7	50	4.5	0.09	
	20.0	21.7	54.3	12	5.4	0.45	
	25.0	25.7	49.8	50	76.7	1.53	
Top of Rock	25.7						

100.00

INPUT

Exploration ID: BB-SB-103

Ground Surface Elevation: 83.3 ft

Depth of Boring: 37.3 ft

Depth to Bedrock: 37.3

EQUATIONS

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

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

CALCULATION

N = 42.6

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	4.2	6.2	78.1	43	8.1	0.19	
	10.0	12.0	72.3	38	5.4	0.14	
	15.0	17.0	67.3	19	5.0	0.26	
	20.0	22.0	62.3	46	5.0	0.11	
	25.0	27.0	57.3	35	4.5	0.13	
	29.0	31.0	53.3	28	5.0	0.18	
Glacial Till	35.0	36.3	47.7	50	67.0	1.34	
Top of Rock	37.3						

100.00

INPUT

Exploration ID: BB-SB-104

Ground Surface Elevation: 74.2 ft

Depth of Boring: 28.2 ft

Depth to Bedrock: 23.2

EQUATIONS

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

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

CALCULATION

N = 17.9

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	10.0	12.0	63.2	24	13.5	0.56	
	15.0	17.0	58.2	23	5.0	0.22	
Glacial Till	20.0	22.0	53.2	17	81.5	4.79	
Top of Rock	23.2						

100.00

INPUT

Exploration ID: BB-CSB-101

Ground Surface Elevation: 79.3 ft

Depth of Boring: 50.4 ft

Depth to Bedrock: 45.4

EQUATIONS

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

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

CALCULATION

$N = 35.7$

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	1.0	3.0	77.3	44	4.0	0.09	
	5.0	7.0	73.3	20	4.5	0.23	
	10.0	12.0	68.3	11	5.0	0.45	
	15.0	17.0	63.3	20	5.0	0.25	
	20.0	22.0	58.3	20	5.3	0.26	
	25.5	27.5	52.8	59	5.0	0.08	
Glacial Till	30.0	32.0	48.3	58	4.8	0.08	
	35.0	37.0	43.3	42	5.0	0.12	
	40.0	40.7	39.0	50	61.5	1.23	
Top of Rock	45.4						

100.00

INPUT

Exploration ID: BB-CSB-102/102A

Ground Surface Elevation: 77.9 ft

Depth of Boring: 40.7 ft

Depth to Bedrock: 33.2

EQUATIONS

where: m = number of layers

d_i = the thickness of all layers between 0 and 100 feet.

d_c = the thickness of any clay layers between 0 and 100 feet.

N_i = the Standard Penetration Resistance (ASTM D 1586) of cohesionless soil layers not to exceed 100 blows/ft, corrected for hammer energy for calibrated auto hammers (i.e., N_{60}).

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

CALCULATION

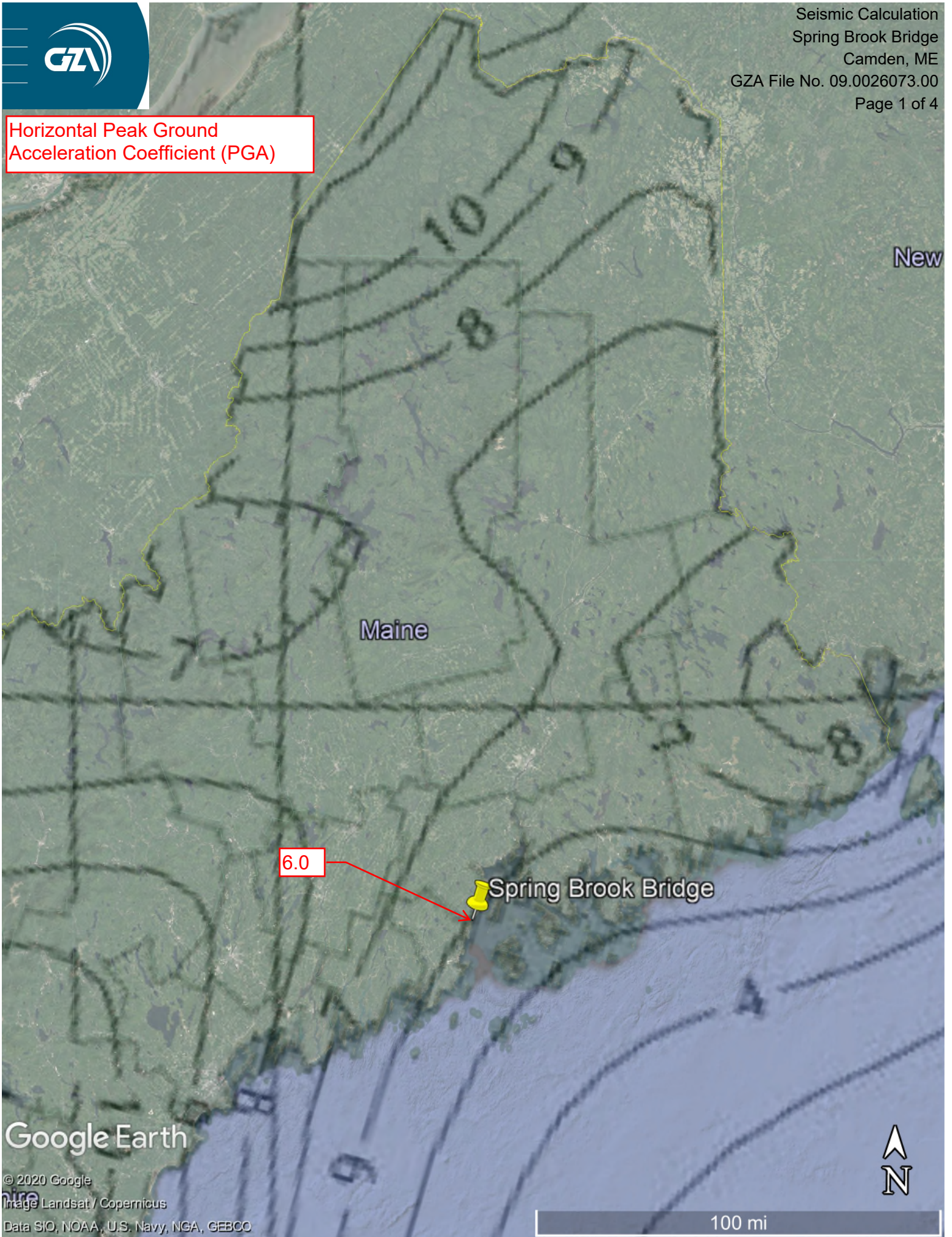
$N = 25.2$

Soil Strata	SPT Interval Depth		SPT Elevation (mid-interval)	SPT N-value	d_i	d_i / N_i	Comment
	Top, ft	Bottom, ft					
Fill	0.0	2.0	76.9	14	3.5	0.25	
	5.0	7.0	71.9	14	5.0	0.36	
	10.0	12.0	66.9	18	4.5	0.25	
	14.0	16.0	62.9	17	5.0	0.29	
Glacial Till	20.0	22.0	56.9	35	5.5	0.16	
	25.0	27.0	51.9	26	5.0	0.19	
	30.0	32.0	46.9	29	71.5	2.47	
Top of Rock	33.2						

100.00



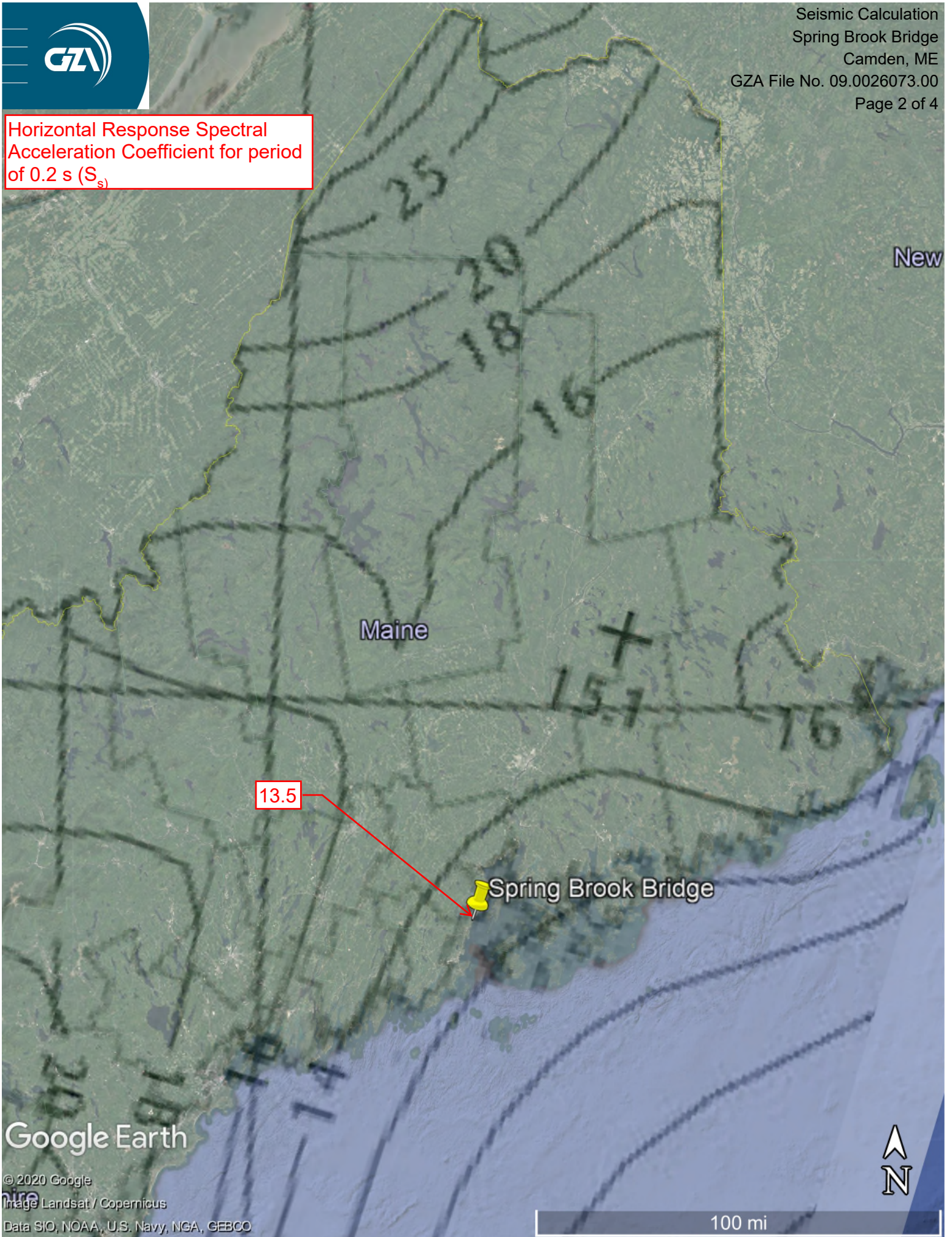
Horizontal Peak Ground
Acceleration Coefficient (PGA)



Google Earth



Horizontal Response Spectral
Acceleration Coefficient for period
of 0.2 s (S_s)

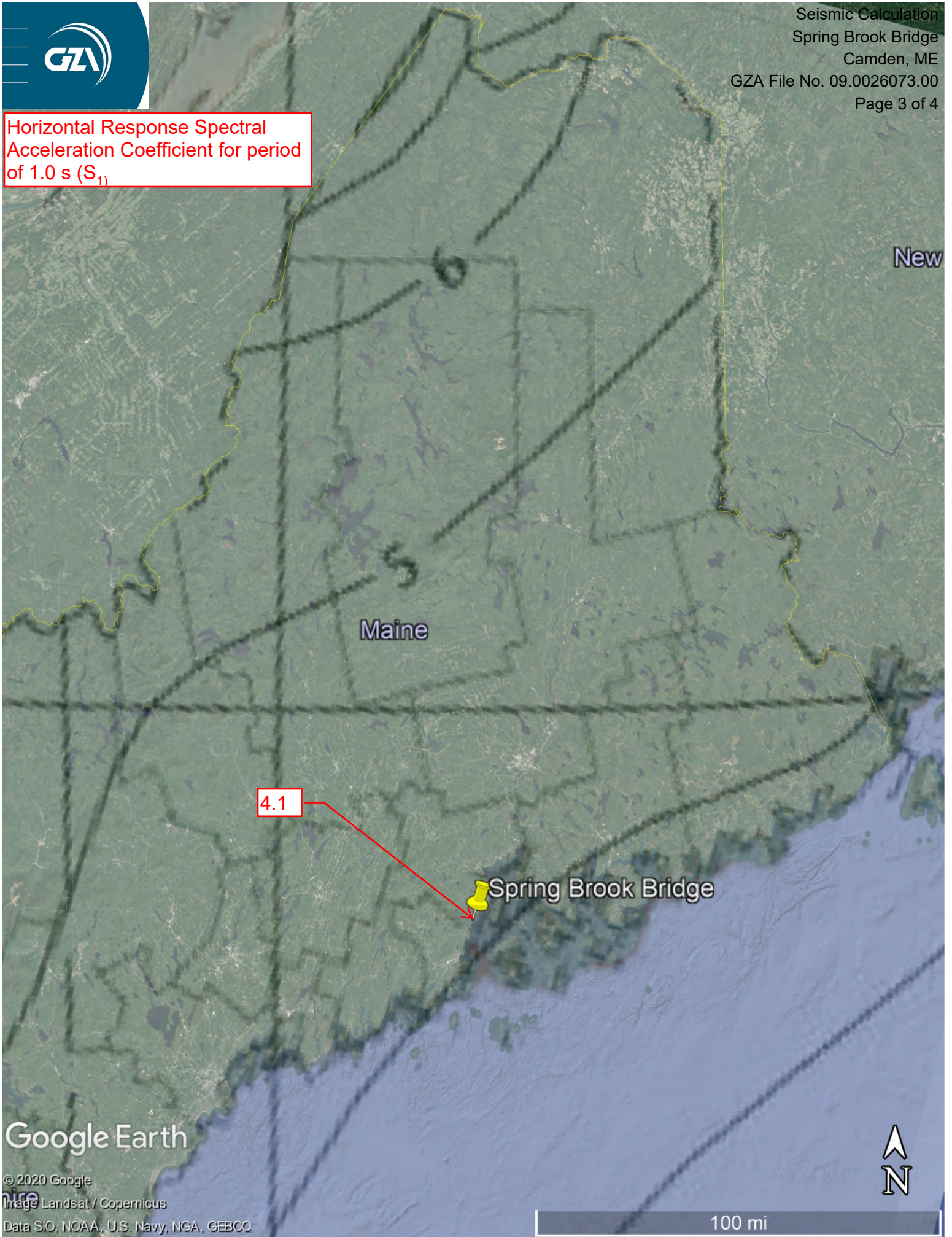


Google Earth

100 mi



Horizontal Response Spectral
Acceleration Coefficient for period
of 1.0 s (S_1)



Google Earth

Camden Seismic Interpolation for Coefficients		
Seismic Parameter	Interpolated Value from Maps¹	Design Parameter
Horizontal Peak ground Acceleration Coefficient	6.0	$PGA = .060$
Horizontal Response Spectral Acceleration Coefficient for Period of 0.2s	13.5	$S_s = 0.135$
Horizontal Response Spectral Acceleration Coefficient for Period of 1.0s	4.1	$S_1 = .041$

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

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

Therefore:

$$A_s = F_{PGA} \times PGA = 1.6 \times 0.060 = 0.096 \text{ g}$$

$$S_{DS} = F_a \times S_s = 1.6 \times 0.135 = 0.216 \text{ g}$$

$$S_{D1} = F_v \times S_1 = 2.4 \times 0.041 = 0.098 \text{ g}$$

Summary:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	Design Value
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.10 g
SDs (Period = 0.2 sec)	0.22 g
SD1 (Period = 1.0 sec)	0.10 g

LATERAL EARTH PRESSURES



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

*Engineers and
 Scientists*

JOB: 09.0026073.00 Camden Bridge
 SUBJECT: Lateral Earth Pressures
 SHEET: 1 OF 2
 CALCULATED BY E. Tome 1/22/2021
 CHECKED BY C. Snow 1/22/2020

Subject: Evaluate lateral earth pressure coefficients

References:

1. MaineDOT Bridge Design Guide, Chapter 3
2. AASHTO LRFD Bridge Design Specifications, 8th Edition (2017)

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*)
- $\delta_f := 21.0\text{deg}$ Average value of friction angle between, precast concrete and clean sand/silty sand-gravel mixture (*AASHTO LRFD Table 3.11.5.3-1*)

Passive Earth Pressure on Integral Backwall:

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

- $y := 0.328\text{in}$ From structural engineer
- $H_b := 12.0\text{ft}$
- $\frac{y}{H_b} = 0.0023$ Ratio of lateral movement to abutment height is less than 0.005, use Rankine passive earth Pressure

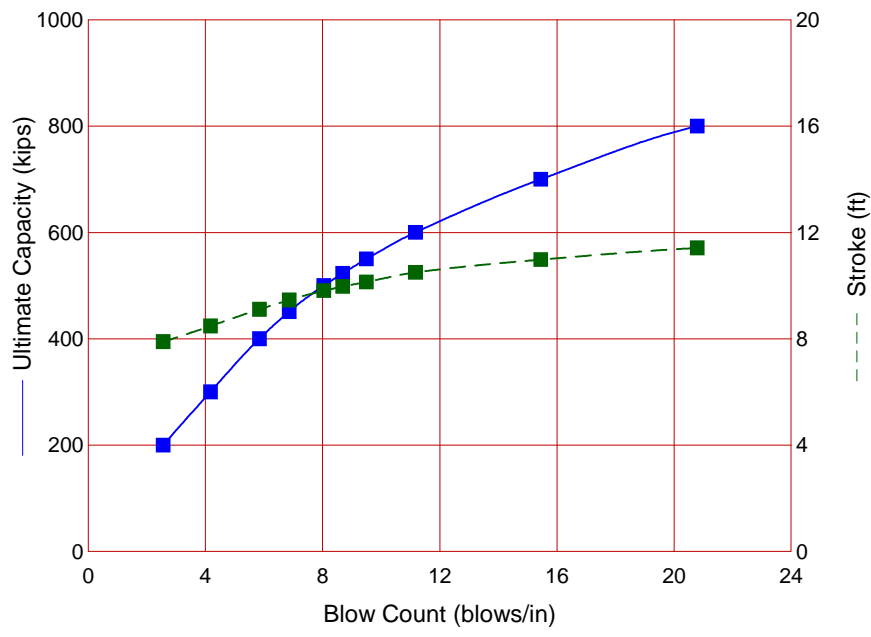
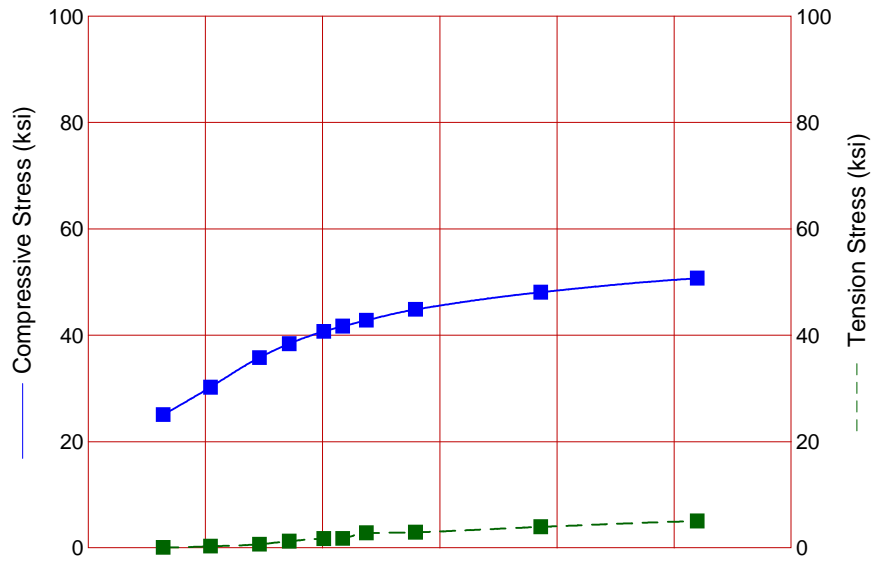
Earth Pressure Coefficients:

Since the ratio of lateral movement is less than .005, GZA evaluated the typical Rankine coefficient.

Rankine Passive Earth Pressure Coefficient

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

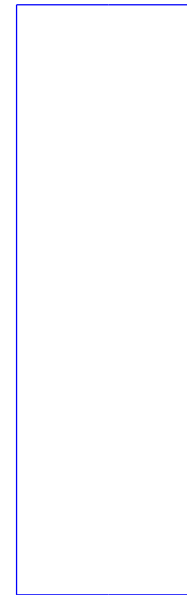
DESIGN PHASE DRIVABILITY ANALYSIS



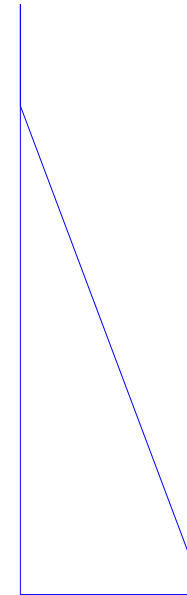
DELMAG D 16-32

Ram Weight	3.52 kips
Efficiency	0.800
Pressure	1425 (100%) psi
Helmet Weight	1.90 kips
Hammer Cushion	60155 kips/in
COR of H.C.	0.800
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	29.00 ft
Pile Penetration	24.00 ft
Pile Top Area	26.10 in ²

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
200.0	25.06	0.08	2.6	7.89	14.63
300.0	30.22	0.32	4.2	8.48	14.41
400.0	35.78	0.70	5.8	9.11	14.90
451.0	38.39	1.26	6.9	9.46	15.57
500.0	40.71	1.75	8.0	9.81	16.25
523.0	41.68	1.79	8.7	9.97	16.56
550.0	42.80	2.82	9.5	10.14	16.92
600.0	44.84	2.94	11.2	10.50	17.63
700.0	48.09	3.97	15.4	10.98	18.61
800.0	50.70	5.06	20.8	11.42	19.51

6-Pile Configuration

