State of Maine

Department of Transportation
Bridge Program - Geotechnical
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Addendum #1 to Soils Report 2017-38

To:	File
From:	Laura Krusinski, MaineDOT
Subject:	Addendum #1 to "Geotechnical Design Report for the Replacement of Gilman Bridge, Athens, Maine" Soils Report No. 2017-38
Date:	September 18, 2019
WIN:	18952.00
Town:	Athens

The following changes are made to the Geotechnical Design Report for the Replacement of Gilman Bridge, Athens, Maine" Soils Report No. 2017-38:

1. Replace the 1st three sentences in paragraph 3 on page 1, with: *"*The replacement superstructure will consist of five weathering steel girders creating a 117-foot single span with a bridge deck width of approximately 32 feet. The superstructure will be constructed with hinged, integral abutments founded on driven H-piles at Abutment No. 1 and H-piles installed in bedrock sockets at Abutment No. 2.*"*

2. Replace the 2rd paragraph on page 7 with: "H-piles are recommended for foundation support. Due to the relatively shallow bedrock surface, H-pile lengths at Abutment No. 2 are insufficient to control moments and develop enough lateral soil support to create a fixed or pinned condition at the pile tip. To provide sufficient pile length to control moments in the piles, and to prevent translation of the pile tip, H-piles at Abutment No. 2 shall be installed in bedrock sockets."

3. Replace the 1st paragraph of Section 7.1 on page 7 with the following: "Abutments No. 1 and No. 2 will be hinged, integral abutments founded on a single row of H-piles. H-piles shall be 50 ksi, Grade A572 steel. Piles at Abutment No. 2 shall be installed in bedrock sockets with a steel plate welded to the pile web and flanges at the pile tip. Abutment No. 1 piles may be driven to the required nominal resistance on or within bedrock."

4. Replace Table 3 on page 8 with:

Location	Bottom of Pile Cap Elevation (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Tip Elevation of Pile (feet)	Estimated Pile Lengths[1] (feet)
Abutment No. 1	501	477 - 482	477 - 482	21-26
Abutment No. 2	500	483 - 488	478	24

Table 1 – Estimated Pile Lengths for Abutments No. 1 and No. 2

5. Add the following Section 7.1.6 to the Report on page 13:

7.1.6 Abutment No. 1 Driven Piles.

Drivability analyses were performed to determine the pile resistance that might be achieved considering available diesel hammers. To reduce the risk of piles "walking" prior to developing the required nominal pile resistance, the nominal drivability resistance was estimated using (1) a Delmag D36-32 hammer and a maximum driving of 40 ksi and (2) a Delmag D19-42 hammer with a maximum driving resistance of 12 blows per inch (bpi). Factored drivability resistances were calculated using a resistance factor, $\varphi_{dyn,}$ of 0.65 for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

The recommended nominal and factored resistances of HP 14x117 pile at the strength limit states for Abutment No. 1 are provided the table below

Pile Section	Abutment No. 1 Axial Pile Resistance Strength Limit State	
	Nominal Drivability Resistance (kips)	Factored Drivability Resistance $\varphi_{dyn} = 0.65$ (kips)
HP 14 x 117	750 (800)[2]	487 (520)[2]

LRFD Article 10.7.3.2.3 states that the nominal axial compressive resistance of piles driven to hard rock is typically controlled by the structural resistance with a resistance factor for severe driving conditions applied. However, the estimated factored axial pile resistances from the drivability analyses for the Abutment No. 1 H-pile sections are less than the

[1] includes 2-foot pile embedment in cap

[2] Drivability resistance based on a Delmag D19-42 and limiting penetration resistance of 12 bpi. Drivability resistance with a Delmag D36-32 and a limiting driving stress of 40 ksi shown in parentheses.

controlling factored axial geotechnical and structural resistance per LRFD Article 10.7.3.2.3. Therefore, drivability resistance governs for Abutment No. 1 pile design.

6. Add the following Section 7.1.7 to the Report on page 13:

   7.1.7 Driven Pile Quality Control

   The contract plans shall require the contractor to perform a wave equation analysis of the proposed pile-hammer system and conduct one dynamic pile load test with signal matching. The first pile driven at Abutment No. 1 should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. 24-hour (minimum) restrike tests are required because piles may "walk" out of position due to sloping rock at Abutment No. 1.

   With this level of quality control, the ultimate resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn}, of 0.65. The maximum factored axial pile load should be shown on the plans.

   Piles should be driven to an acceptable penetration resistance as determined by the contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required pile resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving may be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

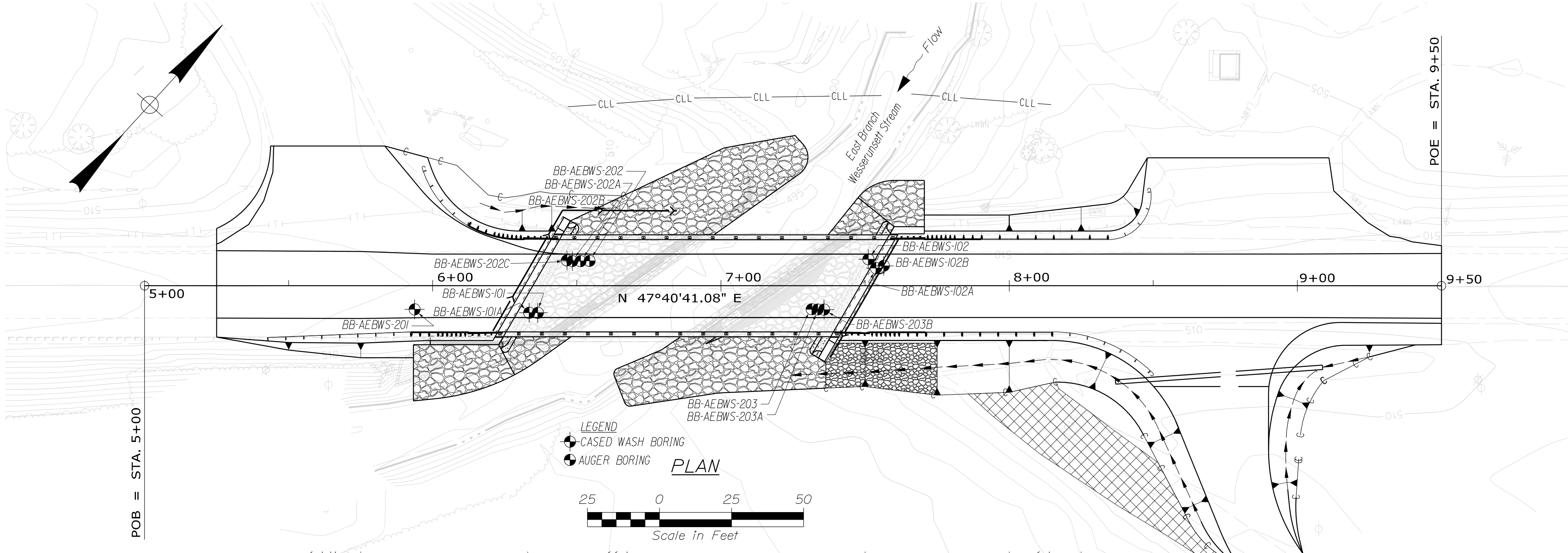
7. Replace Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile with the attached Sheet 2 - Boring Location Plan & Interpretive Subsurface Profile which has been updated with the current span and abutment configurations.

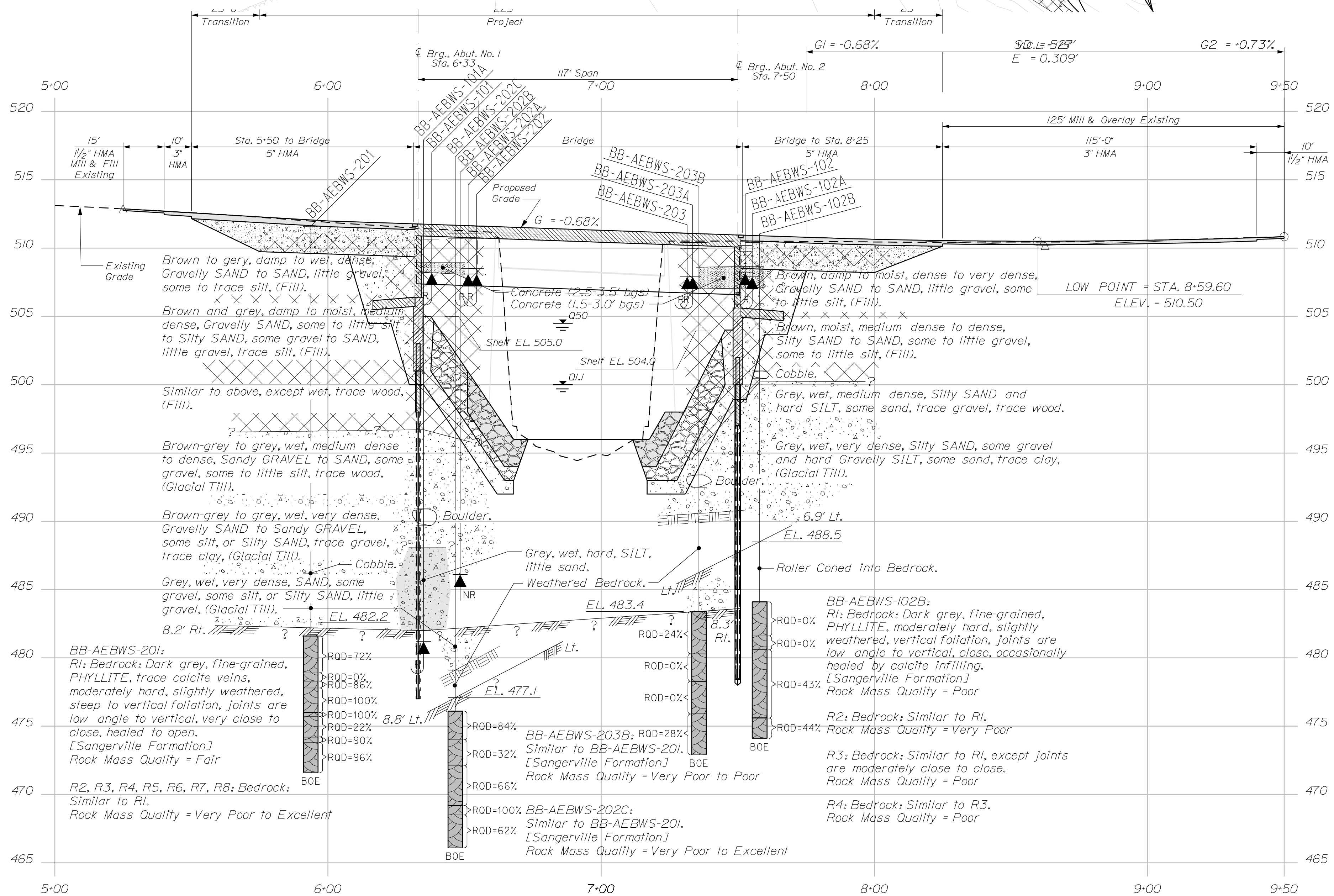
8. Add the attached Drivability Analyses to Appendix C – Calculations

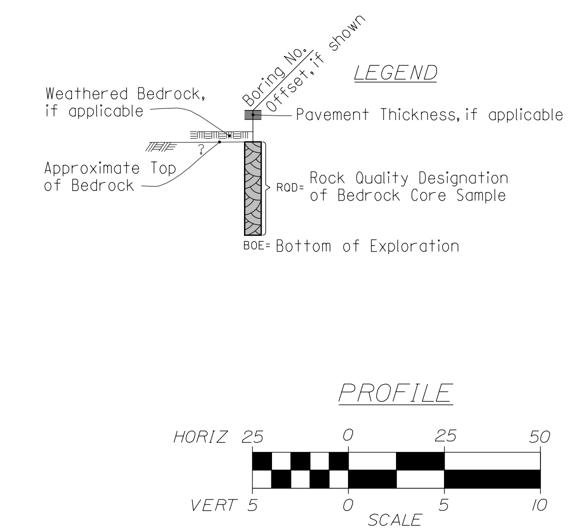
Attachments:

Sheet 2 – Boring Location Plan & Interpretive Subsurface Profile
Calculations - Drivability Analyses

Filename: ...\GEOTECH\MSTA\005_BLP&ISP1.dgn Division: GEOTECH Username: Terry.White Date:9/17/2019







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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18952.00

WIN 18952.00

BRIDGE PLANS

BRIDGE NO. 2313

		BY	DATE
PROJ. MANAGER	MIKE WIGHT		
DESIGN-DETAILED	M. Gray	W. Rohman	
CHECKED-REVIEWED	B. SLAVEN	T. WHITE	AUG 2017
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

GILMAN BRIDGE
EAST BRANCH WESSERUNSETT STREAM
ATHENS SOMERSET COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

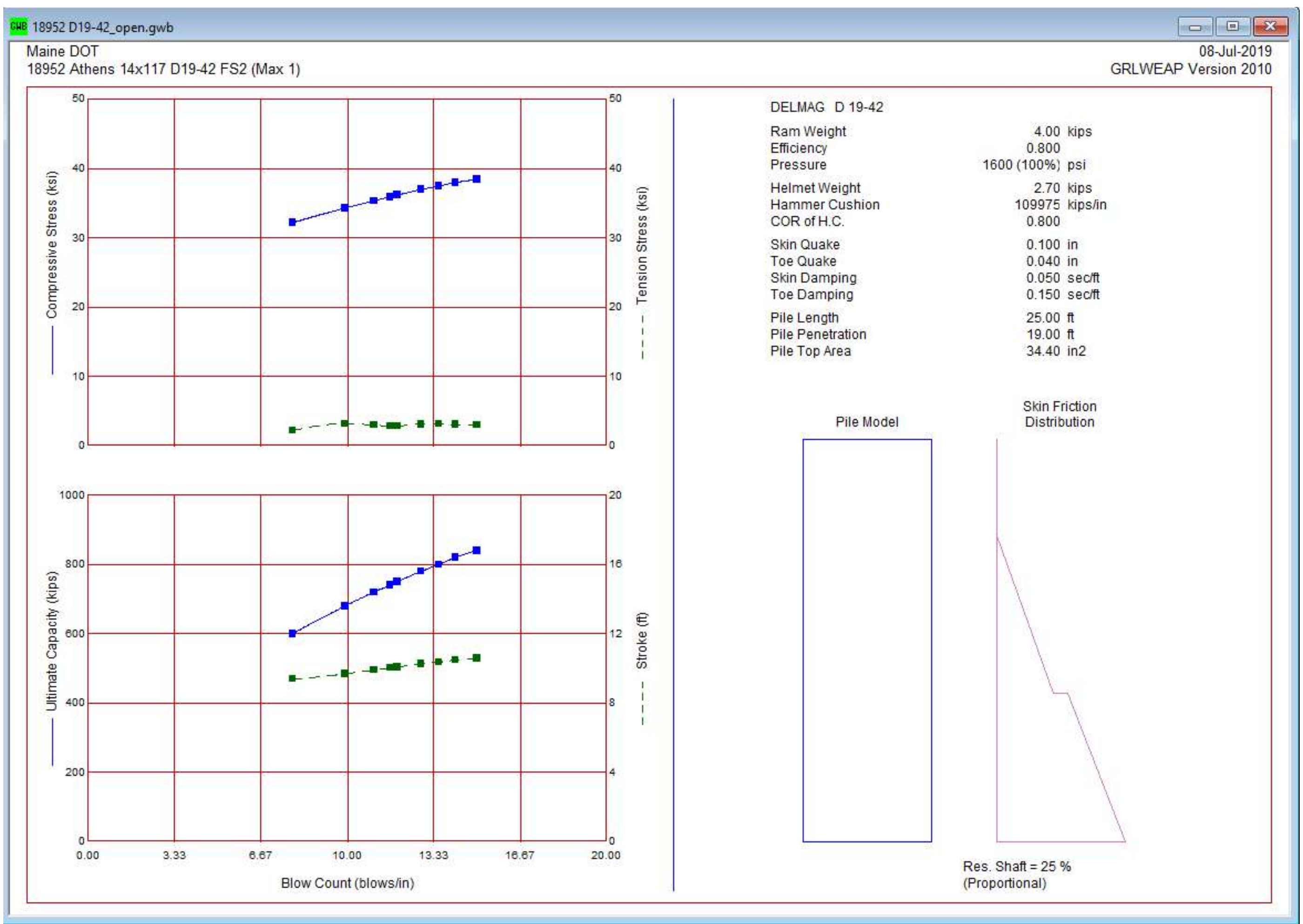
OF 5

Delmag D19-42 – open fuel – limit driving to 12 bpi

Maine DOT
18952 Athens 14x117 D19-42 FS2 (Max 1)

08-Jul-2019
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
600.0	32.14	2.17	7.9	9.39	17.61
680.0	34.26	3.17	9.9	9.69	18.21
720.0	35.30	2.99	11.0	9.91	18.68
740.0	35.87	2.82	11.6	10.03	18.96
750.0	36.12	2.81	11.9	10.08	19.09
780.0	36.94	3.10	12.8	10.26	19.49
800.0	37.42	3.13	13.5	10.37	19.72
820.0	37.95	3.05	14.2	10.48	19.98
840.0	38.38	3.01	15.0	10.57	20.16



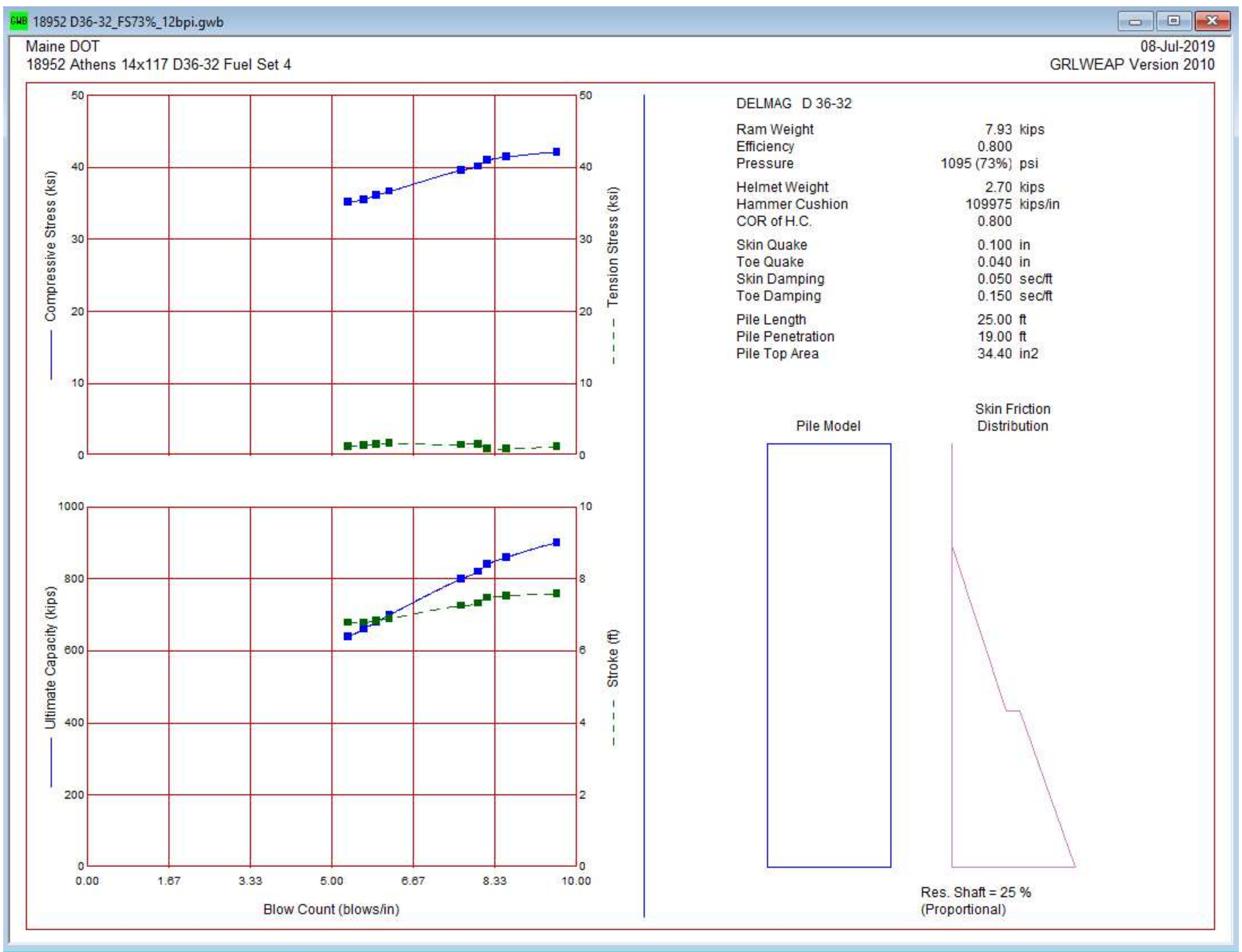
Nominal pile resistance = 750 kips
Factored pile resistance = 750 x 0.65 = 487 kip

D36-32 Fuel Setting 4 (1095 psi – lowest) target: max stress around 40 ksi

Maine DOT
18952 Athens 14x117 D36-32 Fuel Set 4

08-Jul-2019
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
640.0	35.16	1.25	5.3	6.78	23.34
660.0	35.53	1.39	5.6	6.78	23.10
680.0	36.12	1.56	5.9	6.84	23.20
700.0	36.64	1.70	6.2	6.90	23.35
800.0	39.59	1.47	7.6	7.26	24.22
820.0	40.13	1.53	8.0	7.32	24.39
840.0	40.97	0.95	8.2	7.48	24.94
860.0	41.44	0.88	8.6	7.53	25.05
900.0	42.11	1.20	9.6	7.59	25.03



Nominal pile resistance = 800 kip
Factored pile resistance = 800 x 0.65 = 520 kip (one dynamic load test required)

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

GILMAN BRIDGE
ROUTE 150 OVER EAST BRANCH WESSERUNSETT STREAM
ATHENS, MAINE

Prepared by:
Brandon Slaven
Assistant Geotechnical Engineer

Reviewed by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Somerset County
WIN 18952.00

Soils Report 2017-38
Bridge No. 2313

August 24, 2017

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Tables

Sheets

Appendices

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Gilman Bridge which carries Route 150 over East Branch of Wesserunsett Stream in Athens, Maine. This report presents the subsurface information obtained at the site during the subsurface investigations, geotechnical design parameters, and construction recommendations for the new bridge substructures.

The existing structure was constructed in 1963 utilizing the upstream half of the existing Abutment No. 2 and the upstream wingwall of the existing Abutment No. 1 from the previous bridge built in 1932. The existing structure is comprised of a 56-foot single span painted steel beam superstructure and mass concrete abutments with spread footings founded on native soils. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the structure is "Structurally Deficient", with a Sufficiency Rating of 81.4. The deck is considered in poor condition and rated as a 4 with advanced deterioration. The superstructure is considered in satisfactory condition and rated a 6 with minor deterioration. The substructure is considered in fair condition and rated a 5 with minor section loss. The existing structure is considered "Scour Critical" and the footings are undermined.

The replacement superstructure will consist of five weathering steel girders creating a 75-foot single span with a bridge deck width of 32 feet. The superstructure will be constructed with integral abutments founded on H-piles. As a result of the shallow bedrock surface, the H-piles will be installed in bedrock sockets. A temporary detour will accommodate traffic and allow complete removal of the existing structure while constructing the replacement structure. The new Gilman Bridge will have a similar horizontal alignment and vertical profile as the existing structure.

2.0 GEOLOGIC SETTING

The existing structure carries Route 150 over East Branch of Wesserunsett Stream in Athens, Maine approximately 150 feet northeast of Stickney Hill Road as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Skowhegan Quadrangle, Maine, Open-file No. 86-38 (1986), indicates the surficial soils in the vicinity of the bridge project consist of glacial till. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones. Glacial till includes two varieties; basil till and ablation till. Basal till is typically fine grained and very compact with low permeability and poor drainage. Ablation till is typically loose, sandy, and stony with moderate permeability and fair to good drainage. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

The Bedrock Geologic Map of the Skowhegan Quadrangle, Map Series GM-5 (1977), cites the bedrock at the project site as: thick ribbon limestone and variable bedded calcareous, dark grey, metapelite comprising the Sangerville Formation.

3.0 SUBSURFACE INVESTIGATION

Thirteen test borings explored subsurface conditions at the site. Borings BB-AEBWS-101, BB- AEBWS-101A, BB-AEBWS-201, BB-AEBWS-202, BB-AEBWS-202A, BB-AEBWS-202B, and BB-AEBWS-202C were drilled west of the existing structure. Borings BB-AEBWS-102, BB-AEBWS-102A, BB-AEBWS-102B, BB-AEBWS-203, BB-AEBWS-203A, and BB-AEBWS-203B were drilled east of the existing structure. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The 100-series test borings on September 20 and 21, 2016. The 200-series test borings were drilled between April 24 and 26, 2017. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 3, 4, and 5 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The drill rigs were equipped with an automatic hammer to drive the split spoon. The hammers were calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in April 2016 and March 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.9288 to the 100-series raw field N-values and 0.873 to the 200-series raw field N-values. The hammer efficiency factors (0.9288 and 0.873) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in test borings BB-AEBWS-102B, BB-AEBWS-201, BB-AEBWS-202C, and BB-AEBWS-203B using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core calculated. A geotechnical engineer logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs, and identified lab testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program and located by MaineDOT Region 3 Survey.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of nine standard grain size analyses with natural water content, one standard grain size analysis with hydrometer and natural water content, and one estimate of organic material by loss on ignition. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of fill material and glacial till underlain by metasedimentary bedrock. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The boring logs are provided in Appendix A – Boring Logs and on Sheets 3, 4, and 5 – Boring Logs. The following paragraphs discuss the subsurface conditions encountered:

5.1 Fill

Encountered in the borings was a layer of fill material. The thickness of the unit was approximately 14.5 to 15.0 feet at the boring locations. The fill unit encountered generally consisted of:

- Brown to grey, damp to moist, medium dense to very dense, gravelly sand, some to little silt;
- Brown, damp, very stiff, sandy silt, some gravel;
- Brown, moist to wet, medium dense to dense, sand, some to little gravel, some to trace silt;
- Brown to grey, moist to wet, medium dense, silty sand, some to trace gravel;
- Brown, wet, dense, silty gravel, little sand;
- Brown-grey, wet, dense, gravelly sand, little silt;
- Dark grey, wet, medium dense, silt, some sand, trace organics; and
- Dark grey, wet, medium dense, sand, some gravel, little silt, trace organics.

Borings BB-AEBWS-101, BB-AEBWS-102, BB-AEBWS-102A, BB-AEBWS-102B, BB-AEBWS-202, BB-AEBWS-202A, BB-AEBWS-202B, BB-AEBWS-203, BB-AEBWS-203A, and BB-AEBWS-203B encountered concrete ranging from 1.8 to 3.8 ft below ground surface (bgs) indicating the presence of an approach slab or waste concrete buried within the fill unit. BB-AEBWS-102B encountered cobbles between 9.3 and 9.8 feet bgs indicating cobbles were used in the fill material and may be present at locations and elevations within the fill unit other than those encountered in the borings.

Corrected SPT N-values in the fill unit ranged from 12 to 45 blows per foot (bpf), indicating the density of the coarse grained fill is medium dense to dense. Seven SPT attempts refused on what is assumed to be the approach slab of the existing structure or waste concrete. Three grain size analyses of the fill material resulted in the soil being classified as A-1-a and A-4 under the AASHTO Soil Classification System and SM or SC-SM under the Unified Soil Classification System (USCS). The natural water content of the samples tested approximately ranged from approximately 5 to 27 percent.

5.2 Glacial Till

Glacial till was encountered beneath the fill material. The thickness ranged from approximately 3.8 to 17.0 feet. The deposit generally consisted of:

- Brown-grey, wet, very dense, gravelly sand, some silt;
- Brown-grey, wet, dense, sandy gravel, some silt;
- Grey, wet, very dense, sandy gravel, some to little silt, occasional cobble;
- Grey, wet, very dense, silty sand, some to trace gravel, trace clay;
- Grey, wet, hard, silt, little fine sand;
- Grey, wet, medium dense to very dense, sand, some silt to little silt, some to trace gravel;
- Grey, wet, dense, sand, some gravel, some silt; occasional cobble;
- Grey, wet, hard, gravelly silt, some sand, trace clay; and
- Cobbles.

A boulder was encountered between 20.3 and 22 ft bgs in BB-AEBWS-101A when trying to advance casing. A boulder was also encountered between 17 and 17.9 ft bgs in BB-AEBWS-202B and BB-AEBWS-203B.

Corrected SPT N-values ranged from 31 to 90 bpf indicating the density of the coarse grained soil glacial till deposit is medium dense to very dense. Corrected SPT N-values measured in the fine grained glacial till deposit ranged from 90 to 101 bpf indicating the fine grained soil is hard in consistency. Gravel was recovered in the tip of the SPT sampler after two SPT tests and is noted on the boring logs. Gravel embedded in the tip of the SPT sampler could affect SPT N-values. Six grain size analyses conducted on samples from the glacial till deposit resulted in the samples being classified as A-1a, A-1-b, A-2-4, and A-4, under the AASHTO Soil Classification System and SM under the USCS. The moisture content of the tested samples ranged from approximately 9 to 16 percent.

5.3 Bedrock

Bedrock was encountered and cored in borings BB-AEBWS-102B, BB-AEBWS-201, BB-AEBWS-202C, and BB-AEBWS-203B. Table 1 summarizes inferred top of bedrock elevations at these boring locations:

Boring No.	Station	Offset	Inferred Depth to Bedrock (feet)	Inferred Top of Bedrock Elevation (feet)
BB-AEBWS-201	5+94	8.2 ft Rt	29.4	482.2
BB-AEBWS-202C	6+47	8.8 ft Lt	32.0 [1]	479.1[1]
BB-AEBWS-203B	7+36	8.3 ft Rt	19.8[1]	490.6[1]
BB-AEBWS-102B	7+58	6.9 ft Lt.	21.8	488.5

Table 1 – Summary Inferred Bedrock Depths and Elevations

Table 2 summarizes approximate depth to each bedrock core, corresponding approximate top of the bedrock core run and RQD. It is likely that the intact bedrock surface at the boring location begins at some elevation above the elevation in Table 2. Observed drilling behavior indicates that the roller cone penetrated bedrock prior to beginning the recovered core run.

[1] Top of highly weathered bedrock surface penetrated by rollercone.

Boring		Station	Offset	Approximate Depth to Core (feet)	Approximate Elevation of Top of Core (feet)	RQD (%)
BB-AEBWS-201		5+94	8.2 ft Rt			
Core Run	R1			30.0	481.6	72
	R2			32.7	478.9	0
	R3			33.3	478.4	86
	R4			33.8	477.8	100
	R5			35.6	476.0	100
	R6			35.9	475.7	22
	R7			37.4	474.2	90
	R8			37.8	473.8	96
BB-AEBWS-202C		6+47	8.8 ft Lt			
Core Run	R1			35.0	476.1	84
	R2			37.1	474.0	32
	R3			39.0	472.1	66
	R4			41.9	469.2	100
	R5			42.6	468.5	62
BB-AEBWS-203B		7+36	8.3 ft Rt			
Core Run	R1			27.0	483.4	24
	R2			30.1	480.3	0
	R3			32.1	478.3	0
	R4			34.5	475.9	28
BB-AEBWS-102B		7+58	6.9 ft Lt			
Core Run	R1			26.2	484.1	0
	R2			28.7	481.6	0
	R3			29.7	480.6	43
	R4			34.7	475.6	44

Table 2 – Summary of Approximate Bedrock Core Depths, Elevations, and Bedrock RQD

The bedrock recovered is identified as dark grey, fine grained, phyllite, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to moderately close, occasionally healed by calcite infilling to open. The RQD of the bedrock cores ranged from 0 to 100 percent correlating to a rock mass quality of very poor to excellent. Low RQD's are related to breaks along foliation. Detailed bedrock descriptions and RQD's are provided on the boring logs in Appendix A – Boring Logs and on Sheets 3, 4, and 5 – Boring Logs.

5.4 Groundwater

Groundwater depths measured in the test borings ranged from 11.9 to 12.7 ft bgs. The measurements were recorded after completion of the test borings. Note that water was introduced into the borehole during drilling operations and may not represent stabilized groundwater levels. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

The February 2017 Preliminary Design Report considered (1) bridge rehabilitation with reuse of the existing substructures and (2) bridge replacement with multiple superstructure types supported by pile-supported integral abutments. The rehabilitation and reuse alternative was eliminated because of the limited life remaining in the older portions of the abutments constructed in 1932. As a result, full bridge replacement with pile-supported integral abutments was identified as the most cost effective and preferred alternative.

Recommended pile types are H-piles and pipe piles. Due to the relatively shallow bedrock surface, H-pile lengths would be insufficient to control moments and insufficient to develop enough lateral soil support to create a fixed or pinned condition at the pile tip. To create sufficient pile length to reduce moments in the pile, and to prevent translation at the pile tip, H-piles should be installed in bedrock sockets. Due to the relatively shallow bedrock surface pipe piles shall be spun into bedrock to develop fixity.

A spun pipe pile is essentially a micropile with no central steel reinforcement. The bottom of the spun pipe pile bears in the bottom of the bedrock socket. Spun pipe piles typically utilize 80 ksi steel, therefore the pile can resist higher bending moments. The spun pipe pile gains axial compressive resistance through end bearing on the bedrock surface at the bottom of the casing, which requires that the casing be filled with grout to provide end bearing resistance over the entire tip area, similar to a bedrock-socketed drilled shaft. Preliminary bedrock tip resistance estimates for spun pipe piles utilizing drilled shaft methodology determined that a larger quantity of spun pipe piles are required to provide the required resistance, compared to the bedrock-socketed H-pile. Calculations are provided in Appendix C – Calculations.

The expense of installing a larger quantity of spun pipe piles resulted in the selection of bedrock-socketed H-piles as the preferred foundation system. The following sections provide geotechnical design considerations and recommendations for design and construction of bedrock-socketed H-pile supported integral bridge abutments.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.1 Integral Abutment H-Piles

Abutments No. 1 and No. 2 will be integral abutments founded on a single row of H-piles. The piles should be end bearing within bedrock. Piles may be HP 12x53, 12x74, 14x73, 14x89, or 14x117 depending on the factored design axial loads. H-piles shall be 50 ksi, Grade A572 steel with a steel plate welded to the pile web and flanges at the pile tip. Abutment No. 1 and Abutment No. 2 piles will require a bedrock socket to satisfy pile tip fixity requirements and control bending moments and stresses.

Pile lengths at the proposed abutments may be estimated based on Table 3:

Location (Boring)	Approximate Bottom Elevation of Proposed Abutment (feet)	Approximate Top of Bedrock Elevation (feet)	Approximate Tip Elevation of Pile in Socket (feet)	Estimated Pile Lengths (feet)
Abutment No. 1	496.6	482.0	471.5	25.1
Abutment No. 2	496.6	488.5	478.0	18.6

Table 3 – Estimated Pile Lengths for Integral Abutments No. 1 and No. 2

The estimated pile lengths in

Table 3 do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, or additional pile length needed for embedment in the abutment or pile cap.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on or within bedrock at the strength limit state shall consider;

- compressive axial geotechnical resistance of individual piles bearing on bedrock,
- structural resistance of individual piles in axial compression, and
- structural resistance of individual piles in combined axial loading and flexure.

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Since the H-piles will be subjected to lateral loading, the piles shall be checked for resistance against combined axial compression and flexure as prescribed in LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.70$ and the flexural resistance factor $\phi_f = 1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

Abutment H-piles should be analyzed for the determination of unbraced lengths by considering soil-structure interaction effects. The calculated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance as prescribed in LRFD Articles 6.9.2.2 and 6.15.2.

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (ℓ) determined from lateral pile analyses and effective length factors (K).

Geotechnical Resistance. The nominal static geotechnical resistance was calculated for five H-pile sections modeled with an oversized (extending beyond the flanges) steel plate welded across the pile tip to provide an increased bearing area. Static tip resistance was estimated using both the Intact Rock Method, based on Rowe and Armitage (1987) and cited in MaineDOT Transportation Research Division Technical Report 14-01 and the methodology for steeply dipping jointed rock presented in National Cooperative Highway Research Program (NCHRP) Synthesis 360, Equation 44. A resistance factor, φ_{static}, of 0.45 was selected from LRFD Table 10.5.5.2.3-1 for static analysis methods to determine end bearing in rock. The resulting factored static geotechnical resistances for five pile sections with steel plates are provided in Table 4.Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Strength Limit State	
	Plate Dimension d x b (in)	Factored Static Geotechnical Pile Resistance φ_{static} = 0.45 (kips)
HP 12 x 53	13x14	474
HP 12 x 74	14x14	512
HP 14 x 73	15x16	631
HP 14 x 89	15x16	631
HP 14 x 117	16x16	674

Table 4– Static Factored Geotechnical Resistance for H-Piles with Oversized (*extending beyond the flanges*) Steel Plate Installed in Bedrock Sockets at Strength Limit States

The maximum applied factored axial pile load for Strength Limit States should not exceed the factored pile resistances shown in Table 4 above. In no instance should the H-piles with plates be loaded beyond their structural capacity. This evaluation is the responsibility of the structural engineer.

7.1.2 Service and Extreme Limit State Design

The design of H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles and pile group movements/stability considering changes in soil conditions due to scour due to the design flood (Q_{100}). For the service limit state, resistance factors of ϕ = 1.0 should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design checks for the H-piles shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and certain hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as $\phi = 1.0$ with the exception of uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

The nominal static geotechnical resistance was calculated for five H-pile sections modeled with an oversized (extending beyond the flanges) steel plate welded across the pile tip to provide an increased bearing area. Static base resistance was estimated using both the Intact Rock Method, based on Rowe and Armitage (1987) and cited in MaineDOT Transportation Research Division Technical Report 14-01 and the methodology for steeply dipping joints presented in National Cooperative Highway Research Program (NCHRP) Synthesis 360, Equation 44. A resistance factor, ϕ, of 1.0 was applied per LRFD Article 10.5.5.3 for extreme limit state analysis. The resulting factored static geotechnical resistances for five pile sections with steel plates are provided in Table 5. Supporting calculations are provided in Appendix C – Calculations.

Pile Section	Extreme and Service Limit State	
	Plate Dimension d x b (in)	Factored Static Geotechnical Pile Resistance $\phi = 1.0$ (kips)
HP 12 x 53	13x14	1054
HP 12 x 74	14x14	1138
HP 14 x 73	15x16	1402
HP 14 x 89	15x16	1402
HP 14 x 117	16x16	1498

Table 5 – Static Factored Geotechnical Resistance for H-Piles with Oversized (*extending beyond the flanges*) Steel Plate Installed in Bedrock Sockets at Service and Extreme Limit States

The maximum applied factored axial pile load for Service and Extreme Limit States should not exceed the geotechnical resistances shown in Table 5 above. In no instance shall the H-piles with plates be loaded beyond their structural capacity. This evaluation is the responsibility of the structural engineer.

7.1.3 Lateral Pile Resistance/Behavior

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.12.

A series of lateral pile resistance analyses should be performed to evaluate pile behavior at both abutments. The designer should utilize the results of the analyses to calculate axial compressive structural pile resistances based on unbraced pile segments and verify pile bending stresses do not exceed allowable stresses.

Geotechnical parameters for generation of soil-resistance (p-y) curves at Abutment No. 1 and Abutment No. 2 are provided in Table 6. Geotechnical parameters for generation of bedrock-resistance (p-y) curves at Abutment No. 1 and Abutment No. 2 are provided in Table 7. In general, the model developed should emulate the proposed subsurface conditions at the site by using the soil layers (referenced in Table 6 and Table 7 by elevations) and using appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed.

Soil Layer	Approx. Elevation of Soil Layer (feet)	Water Table Condition	Internal Angle of Friction	Effective Unit Weight (pcf)	Horizontal Soil Modulus, k_s (lb/in^3)	Specific Gravity, G_s	Void Ratio, e	At-rest Earth Pressure, K_o
Medium dense, SAND (Granular Borrow).	Ground Surface – 500.0	Above	32°	125	90	2.65	0.43	0.47
Medium dense, SAND (Granular Borrow).	500.0 – 496.6	Below	32°	80	60	2.65	0.43	0.47
Type C Underdrain (Pile Backfill)	496.6 – Top of Grout	Below	36°	91	60	2.65	0.32	0.47

Table 6 – Soil Parameters for Generation of Soil-Resistance (p-y) Curves

The parameters listed in Table 6 assume the backfill material within the structural backfill soil envelop conforms to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The parameters were also developed considering the annular space between the H-pile and temporary casing would be backfilled with MaineDOT Standard Specification 703.22 Underdrain Backfill Material Type C from the top of the grout to the bottom of the abutment or wingwall. Granular Borrow is listed as Soil Type 4 in Table 3-3 of the MaineDOT BDG and Type C is assumed to be similar to Soil Type 5.

Subsurface Material	Model Name[2]	Unconfied Compressive Strength (ksi)	k_{rm}[3]	E_r[4] (ksi)	RQD (%)	γ'[5] (pcf)
Bedrock	Weak Rock	6.8	.0005	337	23	108

Table 7 – Bedrock Parameters for Generation of Bedrock-Resistance (p-y) Curves

It is assumed that the free length of pile required to control bending moments and stresses results in minimal deflection of the pile at the pile tip. Therefore, the top elevation of the grout may be considered the point of fixity of the pile for structural design so long as the grout is fully entrained by bedrock. Table 7 provides bedrock properties should a refined analysis considering bedrock resistance be required for structural design.

7.1.4 Bedrock-Socket Design

A bedrock socket length should be selected such that: 1) the pile tips are installed 1 to 5 feet beyond the pile length required to achieve fixity and 2) the piles have adequate free length to control bending moments and limit stresses in the pile. The pile tip shall bear on a nominal 6-inch lift of 6,000 psi grout tremied into the bottom of the bedrock socket.

Bedrock socket diameter should be at least 0.5 inches greater than the diagonal H-pile section dimension selected. Minimum 6,000 psi grout placed by tremie shall fill the void space underneath the steel tip plate and the annular space between the pile and bedrock to the required elevation after the H-pile is placed into the socket. Underdrain Backfill Material Type C should be specified as fill material inside the casing between the top of the grout and abutment or wingwall to achieve the free length of pile required for adequate pile behavior. The socket bottom and sides should be cleaned of any residual drilling materials such as slurry and drilling spoils to an acceptable tolerance.

7.1.5 Rock-Socketed H-Pile Quality Control

The contract documents shall include a Special Provision 501 – Bedrock-Socketed H-pile Foundations which will require the contractor to submit a quality control plan detailing the equipment, methods, and materials for construction of the bedrock socketed H-piles. Equipment lists shall include; drilling equipment such as down-the-hole hammers, rotary percussive equipment, rock coring tools and drills, drill bits, augers, buckets, casing, tremies, and concrete pumps. Details of methods shall include; the construction sequence, excavations in soils and bedrock, cleaning the bedrock sockets and bearing surface, support of the piles in their final positions while ensuring plumbness, the grout placement, and welding. Materials

[2] See Reese, L. C., 1997. "Analysis of Piles in Weak Rock," *Journal of the Geotechnical and Geoenvironmental Engineering Division*, ASCE, pp. 1010-1017.
[3] Strain at 50 percent of the maximum uniaxial strength.
[4] Young's Modulus of Bedrock (adjusted for jointing in rock mass).
[5] Effective Unit Weight.

shall include; drilling slurry, grout, and grout additives required by the proposed grout mix design. The quality control plan shall also include provisions for a construction log where a record of the drilling method, position, resistance, obstructions, water seepage, and cleaning method shall be maintained.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the integral superstructure. The design of the integral abutments at the strength limit state shall consider reinforced-concrete structural design.

A resistance factor (ϕ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows: angle of internal friction (ϕ) of 32 degrees, total unit weight (γ) of 125 pcf, and a soil-concrete interface friction angle (δ) of 20 degrees.

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p, of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25. A load factor for passive earth pressure is not specified in LRFD. For purposes of the integral abutment backwall reinforcing steel design, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge load, is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 8:

Abutment Height (feet)	h_{eq} (feet)
5	4.0
10	3.0
≥20	2.0

Table 8 – Equivalent Height of Soil for Estimating Live Load Surcharge on Abutments

The abutment design shall include a drainage system behind the abutment to intercept any groundwater. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 – Granular Borrow for Underwater Backfill. The gradation of this material specifies 7 percent or less of the material passing the No. 200 sieve. Limiting the amount of fines is intended to minimize frost action behind the structure and facilitate drainage.

Slopes in front of the pile supported integral abutments should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V in accordance with MaineDOT Standard Detail 610(03).

7.3 Pile-Supported Return Wingwalls

Pile-supported return wingwalls may be used in conjunction with the integral abutments. The wingwalls shall be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The walls shall be designed to resist lateral earth pressures, vehicular loads, collision loads, as well as, creep, temperature, and shrinkage deformations. A chamfer, typically 1 foot, should be used between the abutment and the wingwalls to minimize concrete shrinkage cracking caused by the abrupt change in thickness at the connection. The wingwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) per LRFD Article 3.11.6.4.

For design of the wingwalls, two load cases shall be considered. The first load case is when the wingwall is subjected to passive earth pressure to account for the bridge moving laterally and pushing the wingwall into the fill. This load case is considered under strength limit state. Calculation of passive earth pressures may assume a Rankine passive earth pressure coefficient, K_p, of 3.25 assuming small wingwall movements. See Appendix C – Calculations for supporting documentation. A load factor for passive earth pressure is not specified in LRFD; use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

The second load case considers that the wingwall is free to rotate outwards and is subjected to active pressure. This load case is considered under the strength limit state. If collision loading is anticipated on wall-mounted bridge rail, this load case is considered under the extreme limit state. Calculation of active earth pressure shall use the Rankine active earth pressure coefficient, K_a, of 0.31 assuming a level backslope. See Appendix C – Calculations for supporting documentation.

There are no bearing resistance considerations for wingwalls that are pile-supported. However, the integral abutment pile design resistances and design recommendations in Section 7.1 apply to piles supporting wingwalls. Further, it is recommended that the geotechnical engineer be consulted should other earth retaining systems not discussed within this report be considered for design.

7.4 Settlement

Construction of the proposed abutments will require excavation to elevation of 496.6 at minimum resulting in backfill thicknesses of approximately 14.7 feet. Below elevation 496.6 the glacial till encountered is medium dense to dense and predominately coarse grained. These coarse-grained materials undergo elastic, immediate, compression in response to an increase of vertical overburden pressure. The project calls for the vertical alignment of the new structure to closely match the existing structure. No increase in overburden pressure is anticipated and any settlement that may take place should be small and occur relatively quickly. Construction loads could introduce elastic settlements and these settlements are also anticipated to be small and occur relatively quickly. Although trace organics were encountered in borings BB-AEBWS-201 and BB-AEBWS-202, these organics are significantly decomposed or of the type where long term secondary consolidation is unexpected. Post construction settlement should be negligible with proper compaction of any replaced fill materials used during construction.

Any settlement of the bridge abutments will be due to axial compression of the foundation piles and is anticipated to be minimal.

7.5 Frost Protection

Pile-supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Maine DOT BDG Figure 5-2.

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Athens has a design freezing index (DFI) of approximately 1850 F-degree days. The anticipated coarse grained fill soils were assigned a water content of 20%. These components correlate to a frost depth of 6.2 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Madison, Maine has a DFI from the Modberg database of approximately 1847 F-degree days. Madison was selected because it lies near the same isoline as Athens and Athens is not available in the Modberg database. A water content of 20% was assumed. These components correlate to a frost depth of approximately 6.9 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 6.2 feet for frost protection. See Appendix C – Calculations for supporting calculations.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.6 Scour and Riprap

Grain size analyses were performed on a soil sample of streambed material taken from the streambed and on selected samples obtained from the borings to generate grain size curves for determine parameters to be used in scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following grain size parameters can be used in abutment scour analyses:

Boring	Sample	Elevation (feet)	D_{50} (mm)
BB-AEBWS-101A	5D	501.7- 499.7	2.0
Streambed	Streambed	497.0 - 495.0	1.85
BB-AEBWS-12B	5D	495.3 – 492.3	0.8

Table 9 – Particle Size for Abutment Scour Calculations

The grain size distribution curves are included in Appendix B – Laboratory Test Results.

The consequences of changes in foundation conditions resulting from the design (Q_{100}) and check (Q_{500}) floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less that the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of the pile supported abutments, the PDR indicates the bridge approach slopes and the abutment slopes will be armored with heavy riprap. Refer to MaineDOT BDG Section 2.3.11.3 for information regarding scour countermeasure design. Typically the top of the riprap is located at the Q_{50} elevation. For this project, the top of the riprap will be located at a minimum elevation of $Q_{1.1}$ to minimize stream impacts.

Heavy riprap shall conform to MaineDOT Standard Specification 703.28 – Heavy Riprap. The maximum slope of the riprap shall not exceed 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming MaineDOT Standard Specification 703.19 and Class 1 nonwoven erosion control geotextile per MaineDOT Standard Details 610(02) and 610(03).

7.7 Seismic Design Considerations

The United States Geological Survey Seismic Design CD (Version 2.1) provided with the LRFD Manual, and LRFD Articles 3.10.3.1 and 3.10.6 were used to develop parameters for seismic design. Based on site coordinates, the software provided the recommended AASHTO Response Spectra for a 7 percent probability of exceedance in 75 years. These results are summarized in Table 10:

Parameter	Design Value
Peak Ground Acceleration (PGA)	0.074g
Acceleration Coefficient (A_S)	0.089g
SD_S (Period = 0.2 sec)	0.192g
SD_1 (Period = 1.0 sec)	0.081g
Site Class	C
Seismic Zone	1

Table 10 – Seismic Design Parameters

In conformance with LRFD Article 4.7.4 seismic analysis is not required for bridges in Seismic Zone 1 or single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9.2 and 4.7.4.4, respectively.

See Appendix C – Calculations for supporting documentation.

7.8 Construction Considerations

Construction of the abutments will require drilling into bedrock. Design of bedrock-sockets emphasizes bedrock of lower strength as the critical design consideration. Construction of bedrock-sockets should consider bedrock of the highest strength for estimating drilling costs and type of drilling tools.

Temporary lateral earth support systems may be required to permit construction of the abutments and temporary detour. The new integral abutments will be constructed behind the existing abutments. There is a potential that the existing abutments and their footings, if not removed entirely, may obstruct construction operations. The Contractor shall excavate those portions of the existing abutments and footings that conflict with construction activities. Excavation methods may include; conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Excavation by these methods shall be considered incidental to related pay items. The Contractor should utilize care to not unnecessarily disturb suitable materials during the excavation.

Concrete was encountered from approximately 1.8 to 3.8 feet bgs in the borings. Cobbles and boulders were encountered periodically in the fill unit and glacial till deposit. A thin layer containing trace organics and wood fiber immediately beneath the fill unit indicating the presence of wood debris or timber at the project location. There is a potential for these obstructions to impact construction activities. Impacts include an increased excavation difficulty, impediment of driving sheet piles for temporary earth support systems, and impediment of installing temporary casing for bedrock-socketed piles. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, spudding, use of rock chisels, or down-hole hammers. Alternative methods to clear obstructions may be employed as approved by the Resident.

Five feet of three-inch steel diameter casing was abandoned in BB-AEBWS-202B between 20 and 25 feet bgs after breaking though or kicking out on what is assumed to be a cobble. A casing shoe broke off from three inch diameter casing in BB-AEBWS-101A. Piles should be located to avoid conflict with the abandoned casing and shoe. If conflict is unavoidable, the casing may require removal by excavation methods.

Excavations for the proposed abutments will expose soils that may become saturated and water seepage may occur during construction. There may be localized sloughing and instability in some excavations and cut slopes. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for the specific application of the proposed replacement of Gilman Bridge in Athens, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

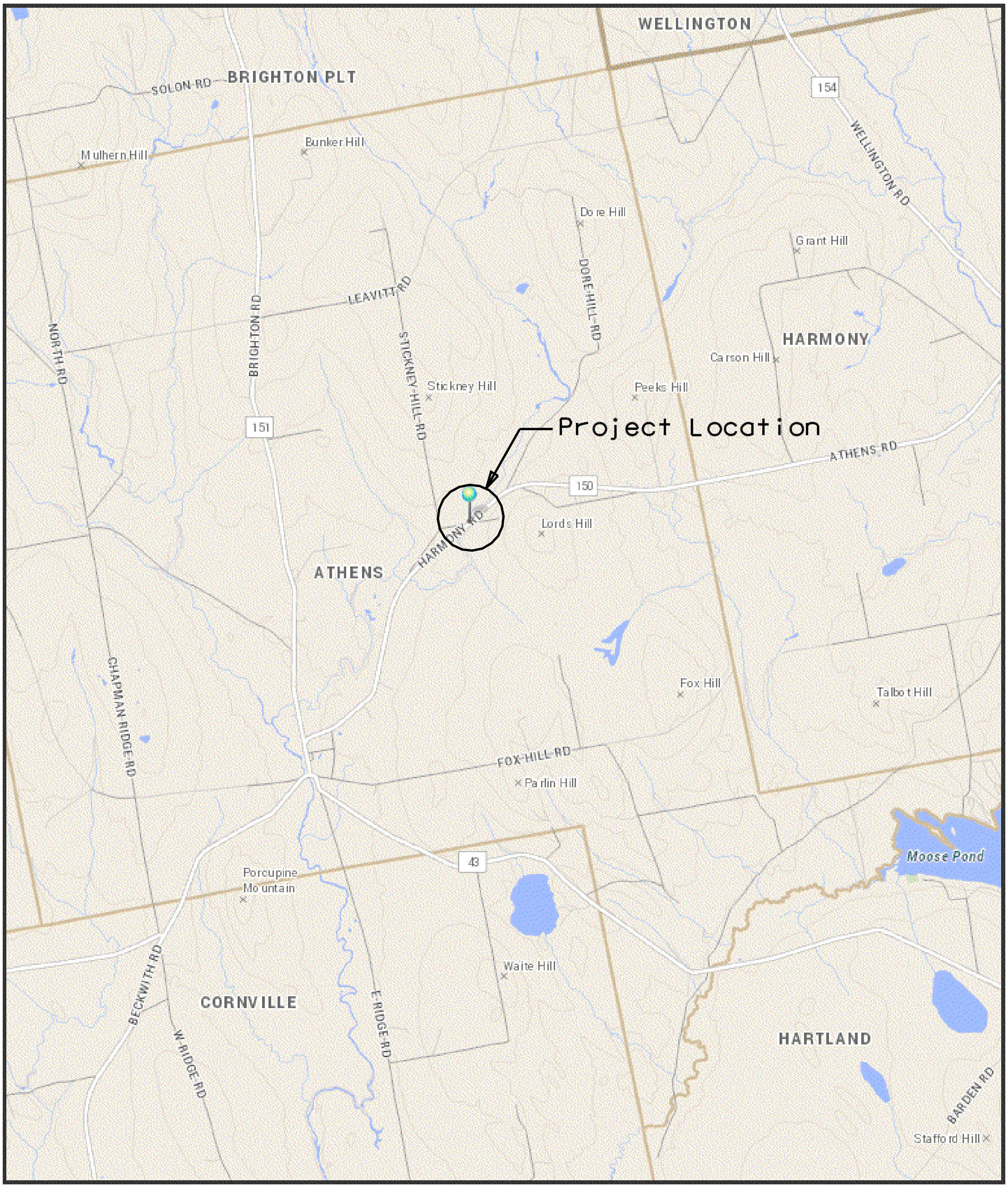
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is also recommend that the geotechnical engineer be provided an opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets

Username: Terry.White   Division: GEOTECH   Date:8/18/2017

ATHENS, MAINE



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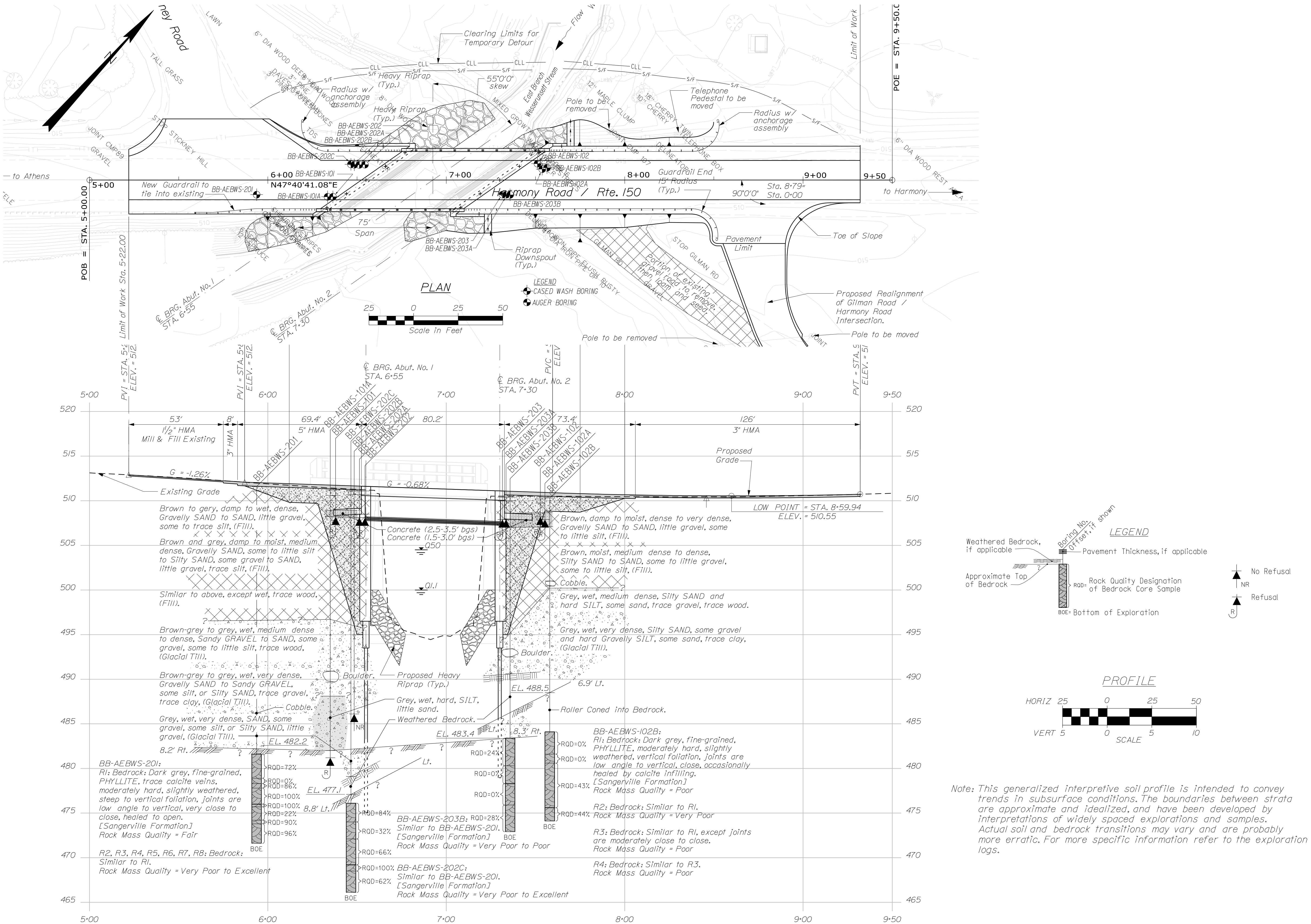
1 Miles
1 inch = 1.51 miles

Date: 6/12/2017
Time: 7:43:06 AM

SHEET NUMBER	GILMAN BRIDGE	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
1	EAST BRANCH WESSERUNSETT STREAM	18952.00
	ATHENS SOMERSET COUNTY	WIN
OF 5	LOCATION MAP	BRIDGE NO. 2313 18952.00 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.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

18952.00

WIN 18952.00

BRIDGE NO. 2313

BRIDGE PLANS

PROJ. MANAGER	MIKE WIGHT	BY	DATE
DESIGN-DETAILED	M. Gray	W. Rohman	
CHECKED-REVIEWED	B. SLAVEN	T. WHITE	AUG 2017
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GILMAN BRIDGE
EAST BRANCH WESSERUNSETT STREAM
ATHENS SOMERSET COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 5

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-201 WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.6	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/25/2017 / 04/26/2017	Drilling Method:	Cased Wash	Core Barrel:	NQ-2"
Boring Location:	5+93.7, 8.2 ft Rt.	Casing ID/OD:	HW 4"/4.5"	Water Level*:	12.0' (after drilling)
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
1D	24/16	1.00 - 3.00	15/13/11/11	24	35	
2D	24/10	5.00 - 7.00	4/4/5/5	9	13	26
						27
						41
						33
						42
3D	24/12	10.00 - 12.00	8/7/4/9	11	16	7
						36
						86
						119
						163
4D	24/13	15.00 - 17.00	51/13/9/9	22	32	61
						108
						208
						178
						192
MD	11/0	20.00 - 20.92	67/100-5"	- -		61
						68
5D	24/12	22.00 - 24.00	16/18/21/25	49	71	105
						116
						147
						103
MD	24/0	26.00 - 28.00	11/9/9/11	18	26	OPEN
6D	17/5	28.00 - 29.42	14/12/100-5"	- -		
R1	32/32	30.00 - 32.67	RQD = 72%			NQ-2
R2	7/7	32.67 - 33.25	RQD = 0%			
R3	7/7	33.25 - 33.83	RQD = 86%			
R4	21/21	33.83 - 35.58	RQD = 100%			
R5	4/4	35.58 - 35.91	RQD = 100%			
R6	18/18	35.91 - 37.41	RQD = 22%			
R7	5/5	37.41 - 37.83	RQD = 90%			
R8	26/24	37.83 - 40.00	RQD = 96%			

Elevation (ft.): 511.10, 496.60, 494.60, 482.20, 471.60

Visual Description and Remarks:

6" of Pavement — 0.50

Brown, moist, dense, SAND, little gravel, trace silt, (Fill).

Brown, moist, medium dense, Silty SAND, some gravel, (Fill).

Brown, wet, medium dense, Silty SAND, little gravel, (Fill).

Dark grey, wet, medium dense, SILT, some sand, trace organics (wood fiber). — 15.00

Dark grey, wet, medium dense, SAND, some gravel, little silt, trace organics (wood fiber). — 17.00

Laboratory Testing Results/AASHTO and Unified Class: G#304383 A-4, SC-SM WC=23.0% Ignition Loss 2.5%

Grey, wet, very dense, Sandy GRAVEL, some silt, (Glacial Till).

Cobble.

Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till). — 29.40

Top of Bedrock at Elev. 482.2 ft.
Advanced by rollercone through bedrock to 30 ft bgs. — 30.00

R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open. [Sangerville Formation]
Rock Mass Quality = Fair.
R1:Core Times (min:sec):
30.0-31.0 ft (2:28)
31.0-32.0 ft (2:45)
32.0-32.7 ft (1:08) Recovery = 100% — 32.67

R2:Bedrock: Similar to R1.
Rock Mass Quality = Very Poor.
R2:Core Times (min:sec)
32.7-33.0 ft (0:49)
33.0-33.3 ft (1:26) Recovery = 100% — 33.25

R3:Bedrock: Similar to R1.
Rock Mass Quality = Good
R3:Core Times (min:sec):
33.3-33.8 ft (2:15) Recovery = 100% — 33.83

R4:Bedrock: Similar to R1.
Rock Mass Quality = Excellent.
R4:Core Times (min:sec):
33.8-34.0 ft (0:22)
34.0-35.0 ft (3:23)
35.0-35.6 ft (2:10) Recovery = 100% — 35.58

R5:Bedrock: Similar to R1.
Rock Mass Quality = Excellent
R5:Core Times (min:sec):
35.6-35.9 ft (1:15) Recovery = 100% — 35.91

R6:Bedrock: Similar to R1.
Rock Mass Quality = Very Poor.
R6:Core Times (min:sec):
35.9-37.0 ft (5:08)
37.0-37.4 ft (2:04) Recovery = 100% — 37.41

R7:Bedrock: Similar to R1.
Rock Mass Quality = Good.
R7:Core Times (min:sec):
37.4-37.8 ft (2:59) Recovery = 100% — 37.83

R8:Bedrock: Similar to R1.
Rock Mass Quality = Excellent.
R8:Core Times (min:sec):
37.8-38.0 ft (0:21)
38.0-39.0 ft (2:44)
39.0-40.0 ft (2:46) Recovery = 92% — 40.00

Bottom of Exploration at 40.00 feet below ground surface.

Remarks:
Auto-hammer #362.
Casing driven with 140# Automatic Hammer.

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.

Page 1 of 1 Boring No.: BB-AEBWS-201

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-101A WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.2	Auger ID/OD:	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	Michael St. Pierre	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/20/2016	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	6+35, 9.4 ft RT.	Casing ID/OD:	HW 4"/4.5", NW 3"/3.5"	Water Level*:	11.9' after completion
Hammer Efficiency Factor: 0.9288		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: (as above)

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
3D	24/12	5.00 - 7.00	5/5/6/4	11	17	
4D	24/10	7.00 - 9.00	4/6/9/11	15	23	
5D	24/10	9.50 - 11.50	25/14/13/11	27	42	
						27
						49
						53
						44
6D	24/7	14.50 - 16.50	20/17/12/7	29	45	47
						62
						106
						193
						261
						204
7D	3/1	20.00 - 20.25	50-3"	50	77	147
						NW
8D	24/15	22.00 - 24.00	33/36/22/24	58	90	33
						46
						65
9D	24/9	25.00 - 27.00	10/13/22/18	35	54	27
						40
						73
						174
						100

Elevation (ft.): 496.70, 488.10, 482.00, 481.20

Visual Description and Remarks:

See BB-AEBWS-101 for description of strata from 0 to 3 ft bgs.

3D(A) 5-5.5 ft bgs: Brown, damp, very stiff, fine to coarse Sandy SILT, some gravel, (Fill).
3D(B) 5.5-7': Grey, damp, medium dense, Gravelly SAND, some silt, (Fill).

Grey and brown, moist, medium dense, Gravelly SAND, little silt, (Fill). — G#262832 A-1-a, SM WC=5.1%

Brown-grey, wet, dense, Gravelly SAND, little silt, (Fill). — G#262833 A-1-b, SM WC=12.5%

Brown-grey, wet, dense, Sandy Gravelly, some silt, (Glacial Till). — 14.50
Gravel fragment in end of sampler

Brown-grey, wet, very dense, Gravelly SAND, some silt, (Glacial Till).
Gravel fragment in sampler
HW casing refusal on probable boulder at ±20.3 ft bgs; Roller cone to 22 ft bgs and place NW casing.
8D(A) 22-23.1 ft bgs: Grey, wet, very dense, Sandy GRAVEL, little silt, occasional cobbles, (Glacial Till). — G#262834 A-1-a, SM WC=8.7% — 23.10
8D(B) 23.1-24': Grey, wet, hard, SILT, little fine sand.

Grey, wet, very dense, SAND, some silt, some platy gravel, (Glacial Till). — G#262835 A-2-4, SM WC=11.3%

Metal cutting in wash water. During driving NW casing shoe broke off. Casing shoe left in borehole. — 29.20
Probable Bedrock — 30.00

Bottom of Exploration at 30.00 feet below ground surface.

Remarks:
Auto-hammer #562

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.

Page 1 of 1 Boring No.: BB-AEBWS-101A

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-101 WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.2	Auger ID/OD:	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	Michael St. Pierre	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/20/2016	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	6+38, 9.4 ft Rt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.9288		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: (as above)

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
1D	24/17	0.70 - 2.70	14/12/10/15	22	34	SSA
2D	3.5/3	2.70 - 2.99	50-3"	100	155	

Elevation (ft.): 510.50, 508.20

Visual Description and Remarks:

7.25" of Pavement — 0.70
Brown, damp, dense, fine to coarse Gravelly SAND, some silt, (Fill).
2D(A) Similar to 1D
2D(B) Concrete; Offset to BB-AEBWS-101A — 3.00
Bottom of Exploration at 3.00 feet below ground surface.

Remarks:
Auto-hammer #562

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.

Page 1 of 1 Boring No.: BB-AEBWS-101

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-202C WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.1	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/24/2017 - 04/25/2017	Drilling Method:	Cased Wash	Core Barrel:	NQ-2
Boring Location:	6+46.6, 8.8 ft Lt.	Casing ID/OD:	HW 4"/4.5"	Water Level*:	12.7' on 04/25/2017
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions: (as above)

Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
						SSA
						SPUN
1D	24/15	26.00 - 28.00	12/12/13/14	25	36	
2D	17/14	30.00 - 31.42	39/46/100-5"	- -		
						OPEN
R1	25/23	35.00 - 37.08	RQD = 84%			NQ2
R2	23/22	37.08 - 39.00	RQD = 32%			
R3	35/34	39.00 - 41.92	RQD = 66%			
R4	8/8	41.92 - 42.59	RQD = 100%			
R5	29/19	42.59 - 45.01	RQD = 62%			

Elevation (ft.): 510.68, 496.10, 479.10, 477.10, 466.10

Visual Description and Remarks:

5" of Pavement — 0.42
See BB-AEBWS-202 and BB-AEBWS-202B for description of strata from 0 to 25 ft bgs. — 15.00

Grey, wet, dense, SAND, some gravel, some silt, occasional cobbles, (Glacial Till).

Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till). — 32.00

Weathered Bedrock — 34.00

Top of Bedrock at Elev 477.1 ft.
Advanced by rollercone through bedrock from 32 to 35 ft bgs. — 35.00

R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open. [Sangerville Formation]
Rock Mass Quality = Good.
R1:Core Times (min:sec):
35.0-36.0 ft (3:36)
36.0-37.0 ft (2:17)
37.0-37.1 ft (0:41) Recovery = 92% — 37.08

R2:Bedrock: Similar to R1.
Rock Mass Quality = Poor.
R2:Core Times (min:sec):
37.1-38.0 ft (3:32)
38.0-38.9 ft (3:03) Recovery = 96% — 39.00

R3:Bedrock: Similar to R1.
Rock Mass Quality = Fair.
R3:Core Times (min:sec):
39.0-40.0 ft (2:25)
40.0-41.0 ft (2:27)
41.0-41.9 ft (3:42) Recovery = 97% — 41.92

R4:Bedrock: Similar to R1.
Rock Mass Quality = Excellent.
R4:Core Times (min:sec):
41.9-42.0 ft (0:19)
42.0-42.6 ft (1:43) Recovery = 100% — 42.59

R5:Bedrock: Similar to R1.
Rock Mass Quality = Very Poor.
R5:Core Times (min:sec):
42.6-43.0 ft (1:08)
43.0-44.0 ft (2:13)
44.0-45.0 ft (2:18) Recovery = 66% — 45.00

Bottom of Exploration at 45.00 feet below ground surface.

Remarks:
Auto-hammer #562

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.

Page 1 of 1 Boring No.: BB-AEBWS-202C

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
18952.00
BRIDGE PLANS
WIN 18952.00
BRIDGE NO. 2313

	MIKE WIGHT	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	M Gray	W Rohman	
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	AUG 2017
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GILMAN BRIDGE
EAST BRANCH WESSERUNSETT STREAM
ATHENS SOMERSET COUNTY

BORING LOGS

SHEET NUMBER
3
OF 5

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-202B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.1	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/24/2017	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	6+48.4, 8.6 ft Lt.	Casing ID/OD:	HW 4"/4.5"	Water Level*:	12.7' (after drilling)
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	510.68	
								508.50	
								507.60	
5	1D	24/9	5.00 - 7.00	7/6/6/8	12	17	29		
							42		
							57		
							45		
							41		
10	2D	24/7	10.00 - 12.00	6/4/4/5	8	12	24		
							34		
							98		
							129		
							159		
15	3D	24/15	15.00 - 17.00	23/13/14/18	27	39	157	496.10	
							363		
							NW OPEN		
							144		
							163		
20	4D	24/14	20.00 - 22.00	21/22/22/42	44	64	40		G#304384 A-1-b, SM WC=11.1%
							45		
							211		
							203		
							117		
25								486.10	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- See BB-AEBWS-202 for description of strata from 0 to 2.6 ft bgs. — 2.60
- Concrete from 2.6 to 3.5 ft bgs. — 3.50
- Brown, moist, medium dense, SAND, little gravel, trace silt, (Fill).
- Similar to above except wet. — 15.00
- Grey, wet, dense, SAND, some gravel, some silt, (Glacial Till).
- Boulder from 17 to 17.9 ft bgs. Advanced by rollercone through cobble to 18 ft bgs. Place NW casing.
- Grey, wet, very dense, Silty SAND, trace gravel, trace clay, (Glacial Till).
- Metal shavings in wash water while advancing to 25 ft bgs. NW casing broke at ±20 ft bgs. 5 ft of casing abandoned. Bottom of Exploration at 25.00 feet below ground surface. — 25.00
- Bottom of Exploration at 25.00 feet below ground surface.

Remarks:
Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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.

Page 1 of 1
Boring No.: BB-AEBWS-202B

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-202A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.1	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/24/2017	Drilling Method:	Auger	Core Barrel:	
Boring Location:	6+51.3, 8.5 ft Lt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	510.68	
								508.50	
								508.10	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- See BB-AEBWS-202 for description of strata from 0 to 2.6 ft. — 2.60
- Concrete. Auger refusal at 3 ft bgs. Offset to BB-AEBWS-202B. — 3.00
- Bottom of Exploration at 3.00 feet below ground surface.

Remarks:
Auto-hammer #562

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.

Page 1 of 1
Boring No.: BB-AEBWS-202A

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-202
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	511.1	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/24/2017	Drilling Method:	Auger	Core Barrel:	
Boring Location:	6+54.4, 8.7 ft Lt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	510.68	
	1D	19/19	1.00 - 2.58	14/12/14/50-1"	26	38			
								508.50	
								508.10	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- Brown, moist, dense, SAND, some gravel, little silt, (Fill). — 2.60
- Concrete. Auger refusal at 3 ft bgs. Offset to BB-AEBWS-202A. — 3.00
- Bottom of Exploration at 3.00 feet below ground surface.

Remarks:
Auto-hammer #562
Casing driven with 140# Automatic Hammer.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AEBWS-202

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-203
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.4	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/26/2017	Drilling Method:	Auger	Core Barrel:	
Boring Location:	7+31.5, 8.1 ft Rt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	509.98	
	1D	15/12	1.00 - 2.25	12/14/100-3"	- -				
								508.20	
								507.90	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- Brown, moist, dense, SAND, little gravel, trace silt, (Fill). — 2.20
- Concrete. Auger refusal at 2.5 ft bgs. Offset to BB-AEBWS-203A. — 2.50
- Bottom of Exploration at 2.50 feet below ground surface.

Remarks:
Auto-hammer #562

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AEBWS-203

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-203A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.4	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/26/2017	Drilling Method:	Auger	Core Barrel:	
Boring Location:	7+33.5, 8.1 ft Rt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	509.98	
								508.20	
								507.90	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- See BB-AEBWS-203 for description of strata from 0 to 2.2 ft. — 2.20
- Concrete. Auger refusal at 2.5 ft bgs. Offset to BB-AEBWS-203B. — 2.50
- Bottom of Exploration at 2.50 feet below ground surface.

Remarks:
Auto-hammer #562

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.

Page 1 of 1
Boring No.: BB-AEBWS-203A

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-203B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.4	Auger ID/OD:	5" Solid-Stem Auger
Operator:	S. Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	N. Strout	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	04/24/2017	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	7+35.8, 8.3 ft Rt.	Casing ID/OD:	HW 4"/4.5"	Water Level*:	12.7' (after drilling)
Hammer Efficiency Factor: 0.873		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	509.98	
								508.20	
								506.60	
5	1D	24/14	5.00 - 7.00	8/6/4/4	10	15	26		
							30		
							23		
							32		
							131		
10	2D	24/12	10.00 - 12.00	27/20/12/12	32	47	61		
							60		
							107		
							143		
							169		
15	3D	24/16	15.00 - 17.00	19/28/28/29	56	81	89	495.40	
							147		
							123		
							163		
							213		
20	4D	24/16	20.00 - 22.00	38/38/41/39	79	115	OPEN	490.60	
	5D	16/13	22.00 - 23.33	25/35/100-4"	- -				
25	6D	7/6	25.00 - 25.58	39/50-1"					
	R1	37/24	27.00 - 30.08	RQD = 24%				483.40	
30	R2	24/23	30.08 - 32.08	RQD = 0%					
	R3	29/27	32.08 - 34.50	RQD = 0%					
	R4	36/18	34.50 - 37.50	RQD = 28%					
								472.90	

Visual Description and Remarks:
- 5" of Pavement — 0.42
- See BB-AEBWS-202 for description of strata from 0 to 2.2 ft bgs. — 2.20
- Concrete. — 3.80
- Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).
- 2D(A): Brown, wet, dense, Silty GRAVEL, little sand, (Fill).
- 2D(B): Dark grey, wet, hard, SILT, some sand, trace organics (wood fiber). — 15.00
- Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).
- Boulder from 17 to 17.9 ft bgs. Advanced by rollercone through cobble to 18 ft bgs. Place NW casing. — 19.80
- Highly weathered bedrock.
- Similar to above.
- Similar to above. — 27.00
- Top of Bedrock at Elev. 483.4 ft.
  R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open.
  [Sangerville Formation]
  Rock Mass Quality = Very Poor.
  R1:Core Times (min:sec):
  27.0-28.0 ft (6:56)
  28.0-29.0 ft (7:40)
  29.0-30.0 ft (3:25)
  30.0-30.1 ft (2:23) Recovery = 65% — 30.08
- R2:Bedrock: Similar to R1.
  Rock Mass Quality = Very Poor.
  R2:Core Times (min:sec):
  30.1-31.0 ft (5:15)
  31.0-32.0 ft (3:42)
  32.0-32.1 ft (2:14) Recovery = 96% — 32.08
- R3:Bedrock: Similar to R1.
  Rock Mass Quality = Very Poor.
  R3:Core Times (min:sec):
  32.1-33.0 ft (6:10)
  33.0-34.0 ft (5:39)
  34.0-34.5 ft (3:19) Recovery = 93% — 34.50
- R4:Bedrock: Similar to R1.
  Rock Mass Quality = Poor.
  R4:Core Times (min:sec):
  34.5-35.0 ft (3:43)
  35.0-36.0 ft (7:12)
  36.0-37.0 ft (4:47)
  37.0-37.5 ft (3:01) Recovery = 50% — 37.50
- Bottom of Exploration at 37.50 feet below ground surface.

Remarks:
Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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Page 1 of 1
Boring No.: BB-AEBWS-203B

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
18952.00
BRIDGE PLANS
WIN 18952.00
BRIDGE NO. 2313

		BY	DATE
PROJ. MANAGER	MIKE WIGHT		
DESIGN-DETAILED	M Gray	W Rohman	
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	AUG 2017
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE
P.E. NUMBER
DATE

GILMAN BRIDGE
EAST BRANCH WESSERUNSETT STREAM
ATHENS SOMERSET COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\MSTA\009_BORING LOGS3.dgn   Division: GEOTECH   Username: Brandon.Slaven   Date:8/24/2017

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-102
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.2	Auger ID/OD:	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	Michael St. Pierre	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/21/2016	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	7+52.7, 9.2 ft Lt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.9288		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	14/12	0.50 - 1.67	25/29/50-2"	79	122	SSA	509.70		5.5" of Pavement (0.50)	
								508.20		Brown, damp, very dense, Gravelly SAND, some silt, (Fill). Concrete at 1.7 ft bgs; Offset to BB-AEBWS-102A. (2.00)	
										Bottom of Exploration at 2.00 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:
Auto-hammer #562

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AEBWS-102

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-102A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.4	Auger ID/OD:	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	Michael St. Pierre	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/21/2016	Drilling Method:	Cased Wash	Core Barrel:	
Boring Location:	7+55.3, 6.1 ft Lt.	Casing ID/OD:		Water Level*:	Not Encountered
Hammer Efficiency Factor: 0.9288		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See BB-AEBWS-102 for description of strata from 0 to 2 ft bgs.	
										Concrete at 1.8 ft bgs; Offset to BB-AEBWS-102B.	
								507.90		(2.50) Bottom of Exploration at 2.50 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:
Auto-hammer #562

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.

Page 1 of 1
Boring No.: BB-AEBWS-102A

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

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett
Location: Athens, Maine

Boring No.: BB-AEBWS-102B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	Elevation (ft.)	510.3	Auger ID/OD:	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	Datum:	NAVD88	Sampler:	Standard Split-Spoon
Logged By:	Michael St. Pierre	Rig Type:	Diedrich D-50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	09/21/2016	Drilling Method:	Cased Wash	Core Barrel:	NQ-2"
Boring Location:	7+58, 6.9 ft Lt.	Casing ID/OD:		Water Level*:	12.3 afer completion
Hammer Efficiency Factor: 0.9288		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See BB-AEBWS-102 and BB-AEBWS-102A for description of strata from 0 to 2.5 ft bgs.	
								508.60		(1.70) Concrete from 1.8 to 3 ft bgs.	
								507.30		(3.00)	
5	2D	24/16	5.00 - 7.00	10/13/16/18	29	45				Brown, moist, dense, SAND, some gravel, some silt, (Fill).	
	3D	24/14	7.00 - 9.00	5/6/9/11	15	23				Brown, moist, medium dense, Silty SAND, some gravel, (Fill).	G#262836 A-4, SM WC=26.9%
										Cobble from 9.3 to 9.8 ft bgs.	
10	4D	24/15	10.00 - 12.00	19/12/7/11	19	29	102			Grey, wet, medium dense, Silty SAND, trace gravel, (Fill).	G#262837 A-4, SM WC=17.6%
							83				
							86				
							94				
							103				
15	5D	24/16	15.00 - 17.00	8/10/10/13	20	31	39	495.30		(15.00) Grey, wet, medium dense, SAND, little silt, trace gravel, (Glacial Till).	G#262838 A-1-b, SM WC=15.9%
							56				
							117				
							92				
							106				
20	6D	22/15	20.00 - 21.83	15/22/43/50-4"	65	101	69			Grey, wet, hard, platy Gravelly SILT, some sand, trace clay, (Glacial Till and Weathered Bedrock).	
							262				
							OPEN	488.50		(21.80) Top of Bedrock at Elev. 488.5 ft. Roller cone to 26.2 ft bgs and place NW casing	
25											
	R1	30/15	26.20 - 28.70	RQD = 0%			NQ2			(26.20) R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, close, healed to open. [SANGERVILLE FORMATION] Rock Mass Quality = Very Poor. R1 core times (min:sec): 26.2-27.2' (1:58) 27.2-28.2' (3:41) 28.2-28.7' (2:48) 50% Recovery	
	R2	12/12	28.70 - 29.70	RQD = 0%						(28.70) R2:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2 core time (min:sec): 28.7-29.7 ft (2:51) 100% Recovery	
30	R3	60/55	29.70 - 34.70	RQD = 43%						(29.70) R3:Bedrock: Similar to R1 above except joints are close to moderately close. Rock Mass Quality = Poor. R3 core times (min:sec): 29.7-30.7 ft (2:18) 30.7-31.7 ft (3:03) 31.7-32.7 ft (2:44) 32.7-33.7 ft (3:07) 33.7-34.7 ft (3:11) 92% Recovery	
	R4	18/18	34.70 - 36.20	RQD = 44%						(34.70) R4:Bedrock: similar to R3. Rock Mass Quality = Poor. R3 core times (min:sec): 34.7-35.7 ft (3:08) 35.7-36.2 ft (2:43) 100 % Recovery	
35								474.10		(36.20) Bottom of Exploration at 36.20 feet below ground surface.	
40											
45											
50											

Remarks:
Auto-hammer #562

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
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Page 1 of 1
Boring No.: BB-AEBWS-102B

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
18952.00
BRIDGE NO. 2313
WIN 18952.00
BRIDGE PLANS

PROJ. MANAGER	MIKE WIGHT	BY	DATE
DESIGN-DETAILED	M Gray	W Rohman	
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	AUG 2017
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GILMAN BRIDGE
EAST BRANCH WESSERUNSETT STREAM
ATHENS SOMERSET COUNTY

BORING LOGS

SHEET NUMBER
5
OF 5

Appendix A

Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-101
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.2	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/20/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	6+38, 9.4 ft Rt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor: 0.9288		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.70 - 2.70	14/12/10/15	22	34	SSA	510.50		7.25" of Pavement — 0.70 Brown, damp, dense, fine to coarse Gravelly SAND, some silt, (Fill).	
	2D	3.5/3	2.70 - 2.99	50-3"	100	155		508.20		2D(A) Similar to 1D 2D(B) Concrete; Offset to BB-AEBWS-101A — 3.00 **Bottom of Exploration at 3.00 feet below ground surface.**	
5											
10											
15											
20											
25											

Remarks:

Auto-hammer #562

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

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Boring No.: BB-AEBWS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-101A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.2	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/20/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	6+35, 9.4 ft RT.	**Casing ID/OD:**	HW 4"/4.5", NW 3"/3.5"	**Water Level*:**	11.9' after completion
Hammer Efficiency Factor:	0.9288	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
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WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See BB-AEBWS-101 for description of strata from 0 to 3 ft bgs.	
5	3D	24/12	5.00 - 7.00	5/5/6/4	11	17				3D(A) 5-5.5 ft bgs: Brown, damp, very stiff, fine to coarse Sandy SILT, some gravel, (Fill). 3D(B) 5.5-7': Grey, damp, medium dense, Gravelly SAND, some silt, (Fill).	
	4D	24/10	7.00 - 9.00	4/6/9/11	15	23				Grey and brown, moist, medium dense, Gravelly SAND, little silt, (Fill).	G#262832 A-1-a, SM WC=5.1%
	5D	24/10	9.50 - 11.50	25/14/13/11	27	42				Brown-grey, wet, dense, Gravelly SAND, little silt, (Fill).	G#262833 A-1-b, SM WC=12.5%
10							27				
							49				
							53				
							44				
	6D	24/7	14.50 - 16.50	20/17/12/7	29	45	47	496.70		14.50 Brown-grey, wet, dense, Sandy Gravelly, some silt, (Glacial Till). Gravel fragment in end of sampler	
15							62				
							106				
							193				
							261				
							204				
20	7D	3/1	20.00 - 20.25	50-3"	50	77	147			Brown-grey, wet, very dense, Gravelly SAND, some silt, (Glacial Till). Gravel fragment in sampler HW casing refusal on probable boulder at ±20.3 ft bgs; Roller cone to 22 ft bgs and place NW casing.	
							NW				
	8D	24/15	22.00 - 24.00	33/36/22/24	58	90	33			8D(A) 22-23.1 ft bgs: Grey, wet, very dense, Sandy GRAVEL, little silt, occasional cobbles, (Glacial Till).	G#262834 A-1-a, SM WC=8.7%
							46	488.10		23.10 8D(B) 23.1-24': Grey, wet, hard, SILT, little fine sand.	
							65				
25											

Remarks:

Auto-hammer #562

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Boring No.: BB-AEBWS-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-101A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.2	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/20/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	6+35, 9.4 ft RT.	**Casing ID/OD:**	HW 4"/4.5", NW 3"/3.5"	**Water Level*:**	11.9' after completion
Hammer Efficiency Factor: 0.9288		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	9D	24/9	25.00 - 27.00	10/13/22/18	35	54	27			Grey, wet, very dense, SAND, some silt, some platy gravel, (Glacial Till).	G#262835 A-2-4, SM WC=11.3%
							40				
							73				
							174				
							100	482.00		Metal cutting in wash water. During driving NW casing shoe broke off. Casing shoe left in borehole. 29.20	
30								481.20		Probable Bedrock 30.00	
										Bottom of Exploration at 30.00 feet below ground surface.	
35											
40											
45											
50											

Remarks:

Auto-hammer #562

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Boring No.: BB-AEBWS-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-102
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.2	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/21/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+52.7, 9.2 ft Lt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
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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
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WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	14/12	0.50 - 1.67	25/29/50-2"	79	122	SSA	509.70		5.5" of Pavement 0.50	
										Brown, damp, very dense, Gravelly SAND, some silt, (Fill).	
								508.20		Concrete at 1.7 ft bgs; Offset to BB-AEBWS-102A. 2.00	
										Bottom of Exploration at 2.00 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Auto-hammer #562

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Boring No.: BB-AEBWS-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-102A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.4	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/21/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+55.3, 6.1 ft Lt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See BB-AEBWS-102 for description of strata from 0 to 2 ft bgs.	
								507.90		Concrete at 1.8 ft bgs; Offset to BB-AEBWS-102B. 2.50	
										Bottom of Exploration at 2.50 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Auto-hammer #562

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Boring No.: BB-AEBWS-102A

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream

Location: Athens, Maine

Boring No.: BB-AEBWS-102B

WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.3	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/21/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+58, 6.9 ft Lt.	**Casing ID/OD:**		**Water Level*:**	12.3 afer completion
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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
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WC = Water Content, percent
LL = Liquid Limit
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			See BB-AEBWS-102 and BB-AEBWS-102A for description of strata from 0 to 2.5 ft bgs.	
								508.60		1.70	
										Concrete from 1.8 to 3 ft bgs.	
								507.30		3.00	
5	2D	24/16	5.00 - 7.00	10/13/16/18	29	45				Brown, moist, dense, SAND, some gravel, some silt, (Fill).	
	3D	24/14	7.00 - 9.00	5/6/9/11	15	23				Brown, moist, medium dense, Silty SAND, some gravel, (Fill).	G#262836 A-4, SM WC=26.9%
										Cobble from 9.3 to 9.8 ft bgs.	
10	4D	24/15	10.00 - 12.00	19/12/7/11	19	29	102			Grey, wet, medium dense, Silty SAND, trace gravel, (Fill).	G#262837 A-4, SM WC=17.6%
							83				
							86				
							94				
							103				
15	5D	24/16	15.00 - 17.00	8/10/10/13	20	31	39	495.30		15.00 Grey, wet, medium dense, SAND, little silt, trace gravel, (Glacial Till).	G#262838 A-1-b, SM WC=15.9%
							56				
							117				
							92				
							106				
20	6D	22/15	20.00 - 21.83	15/22/43/50-4"	65	101	69			Grey, wet, hard, platy Gravelly SILT, some sand, trace clay, (Glacial Till and Weathered Bedrock).	
							262	488.50		21.80 Top of Bedrock at Elev. 488.5 ft. Roller cone to 26.2 ft bgs and place NW casing	
							OPEN				
25											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-102B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.3	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	Scott Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	Michael St. Pierre	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	09/21/2016	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	7+58, 6.9 ft Lt.	**Casing ID/OD:**		**Water Level*:**	12.3 afer completion
Hammer Efficiency Factor:	0.9288	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
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T_v = Pocket Torvane Shear Strength (psf)
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
	R1	30/15	26.20 - 28.70	RQD = 0%			NQ2			R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, close, healed to open. [SANGERVILLE FORMATION] Rock Mass Quality = Very Poor. R1 core times (min:sec): 26.2-27.2' (1:58) 27.2-28.2' (3:41) 28.2-28.7' (2:48) 50% Recovery	
	R2	12/12	28.70 - 29.70	RQD = 0%						R2:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2 core time (min:sec): 28.7-29.7 ft (2:51) 100% Recovery	
	R3	60/55	29.70 - 34.70	RQD = 43%						R3:Bedrock: Similar to R1 above except joints are close to moderately close. Rock Mass Quality = Poor. R3 core times (min:sec): 29.7-30.7 ft (2:18) 30.7-31.7 ft (3:03) 31.7-32.7 ft (2:44) 32.7-33.7 ft (3:07) 33.7-34.7 ft (3:11) 92% Recovery	
30											
	R4	18/18	34.70 - 36.20	RQD = 44%						R4:Bedrock: similar to R3. Rock Mass Quality = Poor. R3 core times (min:sec): 34.7-35.7 ft (3:08) 35.7-36.2 ft (2:43) 100 % Recovery	
35											
								474.10		36.20	
										Bottom of Exploration at 36.20 feet below ground surface.	
40											
45											
50											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-201
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.6	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/25/2017 / 04/26/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	5+93.7, 8.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.0' (after drilling)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
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N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	511.10		6" of Pavement — 0.50	
	1D	24/16	1.00 - 3.00	15/13/11/11	24	35				Brown, moist, dense, SAND, little gravel, trace silt, (Fill).	
5	2D	24/10	5.00 - 7.00	4/4/5/5	9	13	26			Brown, moist, medium dense, Silty SAND, some gravel, (Fill).	
							27				
							41				
							33				
							42				
10	3D	24/12	10.00 - 12.00	8/7/4/9	11	16	7			Brown, wet, medium dense, Silty SAND, little gravel, (Fill).	
							36			Dark grey, wet, medium dense, SILT, some sand, trace organics (wood fiber).	
							86				
							119				
							163				
15	4D	24/13	15.00 - 17.00	51/13/9/9	22	32	61	496.60		15.00 — Dark grey, wet, medium dense, SAND, some gravel, little silt, trace organics (wood fiber).	G#304383 A-4, SC-SM WC=23.0% Ignition Loss 2.5%
							108				
							208	494.60		17.00	
							178				
							192				
20	MD	11/0	20.00 - 20.92	67/100-5"	- -		61				
							68				
	5D	24/12	22.00 - 24.00	16/18/21/25	49	71	105			Grey, wet, very dense, Sandy GRAVEL, some silt, (Glacial Till).	
							116				
							147				
25											

Remarks:

Auto-hammer #362.
Casing driven with 140# Automatic Hammer.

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Boring No.: BB-AEBWS-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-201
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.6	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/25/2017 / 04/26/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	5+93.7, 8.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.0' (after drilling)
Hammer Efficiency Factor:	0.873	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log
25							103		
	MD	24/0	26.00 - 28.00	11/9/9/11	18	26	OPEN		
	6D	17/5	28.00 - 29.42	14/12/100-5"	- -				
								482.20	
30	R1	32/32	30.00 - 32.67	RQD = 72%			NQ-2		
	R2	7/7	32.67 - 33.25	RQD = 0%					
	R3	7/7	33.25 - 33.83	RQD = 86%					
	R4	21/21	33.83 - 35.58	RQD = 100%					
35	R5	4/4	35.58 - 35.91	RQD = 100%					
	R6	18/18	35.91 - 37.41	RQD = 22%					
	R7	5/5	37.41 - 37.83	RQD = 90%					
	R8	26/24	37.83 - 40.00	RQD = 96%					
40								471.60	
45									
50									

Visual Description and Remarks

Cobble.

Grey, wet, very dense, SAND, some gravel, some silt, (Glacial Till).

29.40

Top of Bedrock at Elev. 482.2 ft.
Advanced by rollercone through bedrock to 30 ft bgs.
R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open.
[Sangerville Formation]
Rock Mass Quality = Fair.
R1:Core Times (min:sec):
30.0-31.0 ft (2:28)
31.0-32.0 ft (2:45)
32.0-32.7 ft (1:08) Recovery = 100%

R2:Bedrock: Similar to R1.
Rock Mass Quality = Very Poor.
R2:Core Times (min:sec)
32.7-33.0 ft (0:49)
33.0-33.3 ft (1:26) Recovery = 100%

R3:Bedrock: Similar to R1.
Rock Mass Quality = Good
R3:Core Times (min:sec):
33.3-33.8 ft (2:15) Recovery = 100%

R4:Bedrock: Similar to R1.
Rock Mass Quality = Excellent.
R4:Core Times (min:sec):
33.8-34.0 ft (0:22)
34.0-35.0 ft (3:23)
35.0-35.6 ft (2:10) Recovery = 100%

R5:Bedrock: Similar to R1.
Rock Mass Quality = Excellent
R5:Core Times (min:sec):
35.6-35.9 ft (1:15) Recovery = 100%

R6:Bedrock: Similar to R1.
Rock Mass Quality = Very Poor.
R6:Core Times (min:sec):
35.9-37.0 ft (5:08)
37.0-37.4 ft (2:04) Recovery = 100%

R7:Bedrock: Similar to R1.
Rock Mass Quality = Good.
R7:Core Times (min:sec):

Laboratory Testing Results/ AASHTO and Unified Class.

Remarks:

Auto-hammer #362.
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-201
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.6	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/25/2017 / 04/26/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2"
Boring Location:	5+93.7, 8.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.0' (after drilling)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOH = Weight of 140lb. Hammer
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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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LL = Liquid Limit
PL = Plastic Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										37.4-37.8 ft (2:59) Recovery = 100% R8:Bedrock: Similar to R1. Rock Mass Quality = Excellent. R8:Core Times (min:sec): 37.8-38.0 ft (0:21) 38.0-39.0 ft (2:44) 39.0-40.0 ft (2:46) Recovery = 92% 40.00 **Bottom of Exploration at 40.00 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Auto-hammer #362.
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Auger	**Core Barrel:**	
Boring Location:	6+54.4, 8.7 ft Lt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Depth	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	510.68		5" of Pavement	0.42	
	1D	19/19	1.00 - 2.58	14/12/14/50-1"	26	38				Brown, moist, dense, SAND, some gravel, little silt, (Fill).		
								508.50			2.60	
								508.10		Concrete. Auger refusal at 3 ft bgs. Offset to BB-AEBWS-202A.	3.00	
										Bottom of Exploration at 3.00 feet below ground surface.		
5												
10												
15												
20												
25												

Remarks:

Auto-hammer #562
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Auger	**Core Barrel:**	
Boring Location:	6+51.3, 8.5 ft Lt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	510.68		5" of Pavement — 0.42	
										See BB-AEBWS-202 for description of strata from 0 to 2.6 ft.	
								508.50		— 2.60	
								508.10		Concrete. Auger refusal at 3 ft bgs. Offset to BB-AEBWS-202B. — 3.00	
										Bottom of Exploration at 3.00 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Auto-hammer #562

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	6+48.4, 8.6 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' (after drilling)
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	510.68		5" of Pavement (0.42)	
										See BB-AEBWS-202 for description of strata from 0 to 2.6 ft bgs.	
								508.50		(2.60) Concrete from 2.6 to 3.5 ft bgs.	
								507.60		(3.50)	
5	1D	24/9	5.00 - 7.00	7/6/6/8	12	17	29			Brown, moist, medium dense, SAND, little gravel, trace silt, (Fill).	
							42				
							57				
							45				
							41				
10	2D	24/7	10.00 - 12.00	6/4/4/5	8	12	24			Similar to above except wet.	
							34				
							98				
							129				
							159				
15	3D	24/15	15.00 - 17.00	23/13/14/18	27	39	157	496.10		(15.00) Grey, wet, dense, SAND, some gravel, some silt, (Glacial Till).	
							363				
							NW OPEN			Boulder from 17 to 17.9 ft bgs. Advanced by rollercone through cobble to 18 ft bgs. Place NW casing.	
							144				
							163				
20	4D	24/14	20.00 - 22.00	21/22/22/42	44	64	40			Grey, wet, very dense, Silty SAND, trace gravel, trace clay, (Glacial Till).	G#304384 A-1-b, SM WC=11.1%
							45				
							211				
							203				
25							117			Metal shavings in wash water while advancing to 25 ft bgs. NW casing broke at ±20 ft bgs. 5 ft of casing abandoned.	

Remarks:

Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	6+48.4, 8.6 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' (after drilling)
Hammer Efficiency Factor: 0.873		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								486.10		25.00 **Bottom of Exploration at 25.00 feet below ground surface.** **Bottom of Exploration at 25.00 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202C
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017 - 04/25/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2
Boring Location:	6+46.6, 8.8 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' on 04/25/2017
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	510.68		5" of Pavement 0.42	
										See BB-AEBWS-202 and BB-AEBWS-202B for description of strata from 0 to 25 ft bgs.	
5							SPUN				
10											
15								496.10		15.00	
20											
25											

Remarks:

Auto-hammer #362

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202C
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017 - 04/25/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2
Boring Location:	6+46.6, 8.8 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' on 04/25/2017
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
	1D	24/15	26.00 - 28.00	12/12/13/14	25	36				Grey, wet, dense, SAND, some gravel, some silt, occasional cobbles, (Glacial Till).	
30	2D	17/14	30.00 - 31.42	39/46/100-5"	- -					Grey, wet, very dense, Silty SAND, little gravel, (Glacial Till).	
							OPEN	479.10		32.00 Weathered Bedrock	
								477.10		34.00 Top of Bedrock at Elev 477.1 ft. Advanced by rollercone through bedrock from 32 to 35 ft bgs.	
35	R1	25/23	35.00 - 37.08	RQD = 84%			NQ2			R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open. [Sangerville Formation] Rock Mass Quality = Good. R1:Core Times (min:sec): 35.0-36.0 ft (3:36) 36.0-37.0 ft (2:17) 37.0-37.1 ft (0:41) Recovery = 92%	
	R2	23/22	37.08 - 39.00	RQD = 32%						R2:Bedrock: Similar to R1. Rock Mass Quality = Poor. R2:Core Times (min:sec): 37.1-38.0 ft (3:32) 38.0-38.9 ft (3:03) Recovery = 96%	
	R3	35/34	39.00 - 41.92	RQD = 66%						R3:Bedrock: Similar to R1. Rock Mass Quality = Fair. R3:Core Times (min:sec): 39.0-40.0 ft (2:25) 40.0-41.0 ft (2:27) 41.0-41.9 ft (3:42) Recovery = 97%	
40											
	R4	8/8	41.92 - 42.59	RQD = 100%						R4:Bedrock: Similar to R1. Rock Mass Quality = Excellent. R4:Core Times (min:sec): 41.9-42.0 ft (0:19) 42.0-42.6 ft (1:43) Recovery = 100%	
	R5	29/19	42.59 - 45.01	RQD = 62%						R5:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R5:Core Times (min:sec):	
45								466.10			
50											

Remarks:

Auto-hammer #362

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: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-202C
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	511.1	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017 - 04/25/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ-2
Boring Location:	6+46.6, 8.8 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' on 04/25/2017
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										R5:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R5:Core Times (min:sec): 42.6-43.0 ft (1:08) 43.0-44.0 ft (2:13) 44.0-45.0 ft (2:18) Recovery = 66%	
										45.00 **Bottom of Exploration at 45.00 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

Auto-hammer #362

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Boring No.: BB-AEBWS-202C

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-203
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.4	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/26/2017	**Drilling Method:**	Auger	**Core Barrel:**	
Boring Location:	7+31.5, 8.1 ft Rt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.873	**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
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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	depth	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	509.98		5" of Pavement	0.42	
	1D	15/12	1.00 - 2.25	12/14/100-3"	- -					Brown, moist, dense, SAND, little gravel, trace silt, (Fill).		
								508.20			2.20	
								507.90		Concrete. Auger refusal at 2.5 ft bgs. Offset to BB-AEBWS-203A.	2.50	
										Bottom of Exploration at 2.50 feet below ground surface.		
5												
10												
15												
20												
25												

Remarks:

Auto-hammer #562

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

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Boring No.: BB-AEBWS-203

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-203A
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.4	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/26/2017	**Drilling Method:**	Auger	**Core Barrel:**	
Boring Location:	7+33.5, 8.1 ft Rt.	**Casing ID/OD:**		**Water Level*:**	Not Encountered
Hammer Efficiency Factor:	0.873	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	509.98		5" of Pavement — 0.42	
										See BB-AEBWS-203 for description of strata from 0 to 2.2 ft.	
								508.20		— 2.20	
								507.90		Concrete. Auger refusal at 2.5 ft bgs. Offset to BB-AEBWS-203B. — 2.50	
										Bottom of Exploration at 2.50 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

Auto-hammer #562

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-203B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.4	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+35.8, 8.3 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' (after drilling)
Hammer Efficiency Factor:	0.873	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	509.98		5" of Pavement 0.42	
										See BB-AEBWS-202 for description of strata from 0 to 2.2 ft bgs.	
								508.20		2.20 Concrete.	
								506.60		3.80	
5	1D	24/14	5.00 - 7.00	8/6/4/4	10	15	26			Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).	
							30				
							23				
							32				
							131				
10	2D	24/12	10.00 - 12.00	27/20/12/12	32	47	61			2D(A): Brown, wet, dense, Silty GRAVEL, little sand, (Fill).	
							60			2D(B): Dark grey, wet, hard, SILT, some sand, trace organics (wood fiber).	
							107				
							143				
							169	495.40		15.00	
15	3D	24/16	15.00 - 17.00	19/28/28/29	56	81	89			Grey, wet, very dense, Silty SAND, some gravel, (Glacial Till).	
							147				
							123			Boulder from 17 to 17.9 ft bgs. Advanced by rollercone through cobble to 18 ft bgs. Place NW casing.	
							163				
							213	490.60		19.80	
20	4D	24/16	20.00 - 22.00	38/38/41/39	79	115	OPEN			Highly weathered bedrock.	
	5D	16/13	22.00 - 23.33	25/35/100-4"	- -					Similar to above.	
25											

Remarks:

Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Gilman Bridge #2313 carries Route 150 over East Branch Wesserunsett Stream
Location: Athens, Maine

Boring No.: BB-AEBWS-203B
WIN: 18952.00

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	510.4	**Auger ID/OD:**	5" Solid-Stem Auger
Operator:	S. Hollabaugh	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	04/24/2017	**Drilling Method:**	Cased Wash	**Core Barrel:**	
Boring Location:	7+35.8, 8.3 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	12.7' (after drilling)
Hammer Efficiency Factor:	0.873	**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
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	7/6	25.00 - 25.58	39/50-1"						Similar to above.	
								483.40		27.00	
	R1	37/24	27.00 - 30.08	RQD = 24%						Top of Bedrock at Elev. 483.4 ft. R1:Bedrock: Dark grey, fine-grained, PHYLLITE, trace calcite veins, moderately hard, slightly weathered, steep to vertical foliation, joints are low angle to vertical, very close to close, healed to open. [Sangerville Formation] Rock Mass Quality = Very Poor. R1:Core Times (min:sec): 27.0-28.0 ft (6:56) 28.0-29.0 ft (7:40) 29.0-30.0 ft (3:25) 30.0-30.1 ft (2:23) Recovery = 65%	
30	R2	24/23	30.08 - 32.08	RQD = 0%						R2:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2:Core Times (min:sec): 30.1-31.0 ft (5:15) 31.0-32.0 ft (3:42) 32.0-32.1 ft (2:14) Recovery = 96%	
	R3	29/27	32.08 - 34.50	RQD = 0%						R3:Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R3:Core Times (min:sec): 32.1-33.0 ft (6:10) 33.0-34.0 ft (5:39) 34.0-34.5 ft (3:19) Recovery = 93%	
35	R4	36/18	34.50 - 37.50	RQD = 28%						R4:Bedrock: Similar to R1. Rock Mass Quality = Poor. R4:Core Times (min:sec): 34.5-35.0 ft (3:43) 35.0-36.0 ft (7:12) 36.0-37.0 ft (4:47) 37.0-37.5 ft (3:01) Recovery = 50%	
								472.90		37.50	
										Bottom of Exploration at 37.50 feet below ground surface.	
40											
45											
50											

Remarks:

Auto-hammer #362
Casing driven with 140# Automatic Hammer.

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Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Athens   Work Number: 18952.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
Streambed				262839	1	28.1			SW-SM	A-1-a	0
BB-AEBWS-101A, 4D	6+35	9.4 Rt.	7.0-9.0	262832	2	5.1			SM	A-1-a	II
BB-AEBWS-101A, 5D	6+35	9.4 Rt.	9.5-11.5	262833	2	12.5			SM	A-1-b	II
BB-AEBWS-101A, 8D/A	6+35	9.4 Rt.	22.0-23.1	262834	2	8.7			SM	A-1-a	I
BB-AEBWS-101A, 9D	6+35	9.4 Rt.	25.0-27.0	262835	2	11.3			SM	A-2-4	II
BB-AEBWS-102B, 3D	7+58	6.9 Lt.	7.0-9.0	262836	3	26.9			SM	A-4	III
BB-AEBWS-102B, 4D	7+58	6.9 Lt.	10.0-12.0	262837	3	17.6			SM	A-4	III
BB-AEBWS-102B, 5D	7+58	6.9 Lt.	15.0-17.0	262838	3	15.9			SM	A-1-b	II
BB-AEBWS-201, 4D	5+93.7	8.2 Rt.	15.0-17.0	304383	4	23.0			SC-SM	A-4	III
BB-AEBWS-202B, 4D	6+48.4	8.6 Lt.	20.0-22.0	304384	4	11.1			SM	A-1-b	II
							Loss on Ignition, % (T 267)				
BB-AEBWS-201, 4D	5+93.7	8.2 Rt.	15.0-17.0	304383	4	23.0	2.5				

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

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

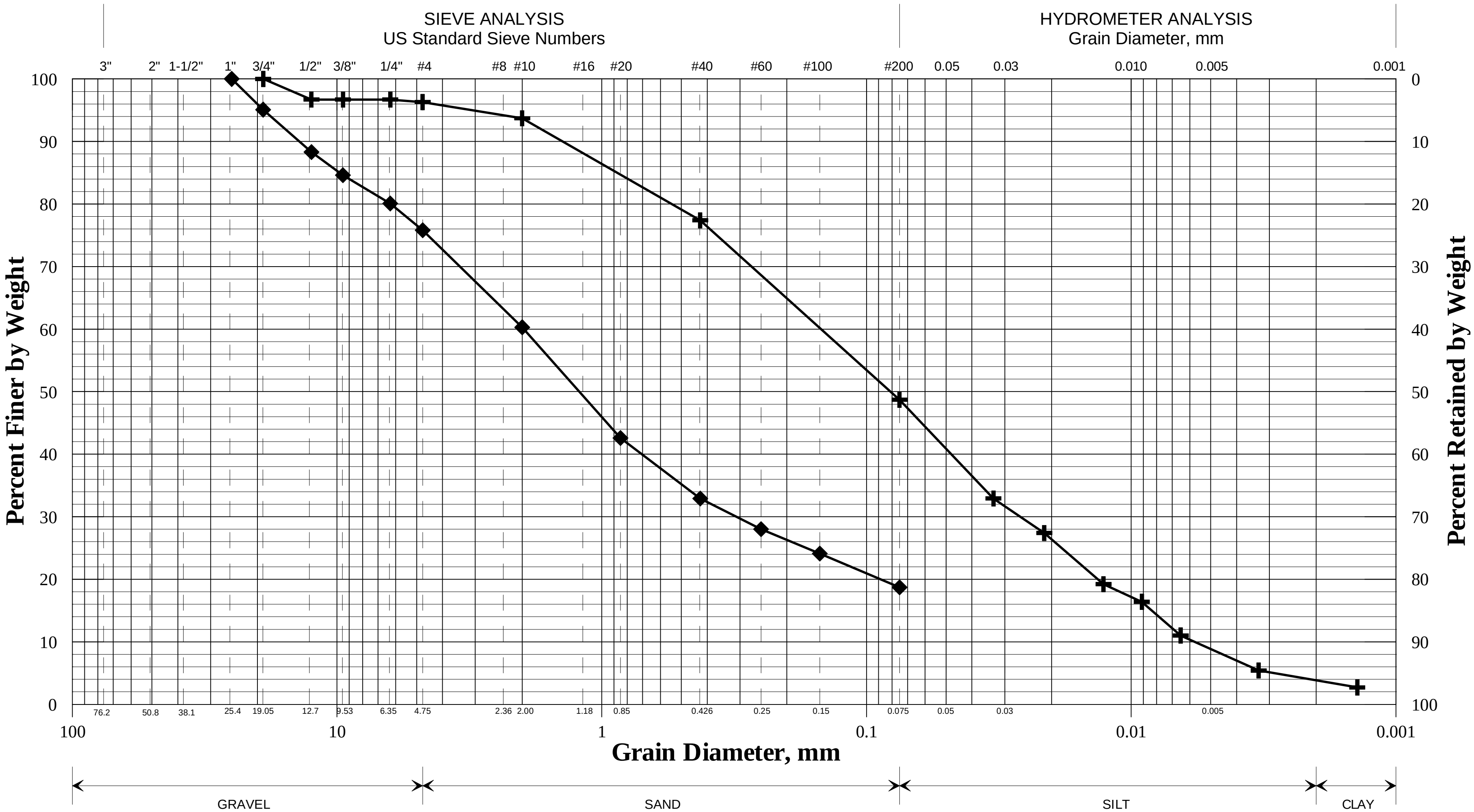
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98   NP = Non Plastic

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AEBWS-201/4D	5+93.7	8.2 RT	15.0-17.0	SAND, some gravel, little silt.	23.0			
◆	BB-AEBWS-202B/4D	6+48.4	8.6 LT	20.0-22.0	Silty SAND, trace clay, trace gravel.	11.1			
■									
●									
▲									
✖									

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WHITE, TERRY A 6/8/2017

SHEET 4

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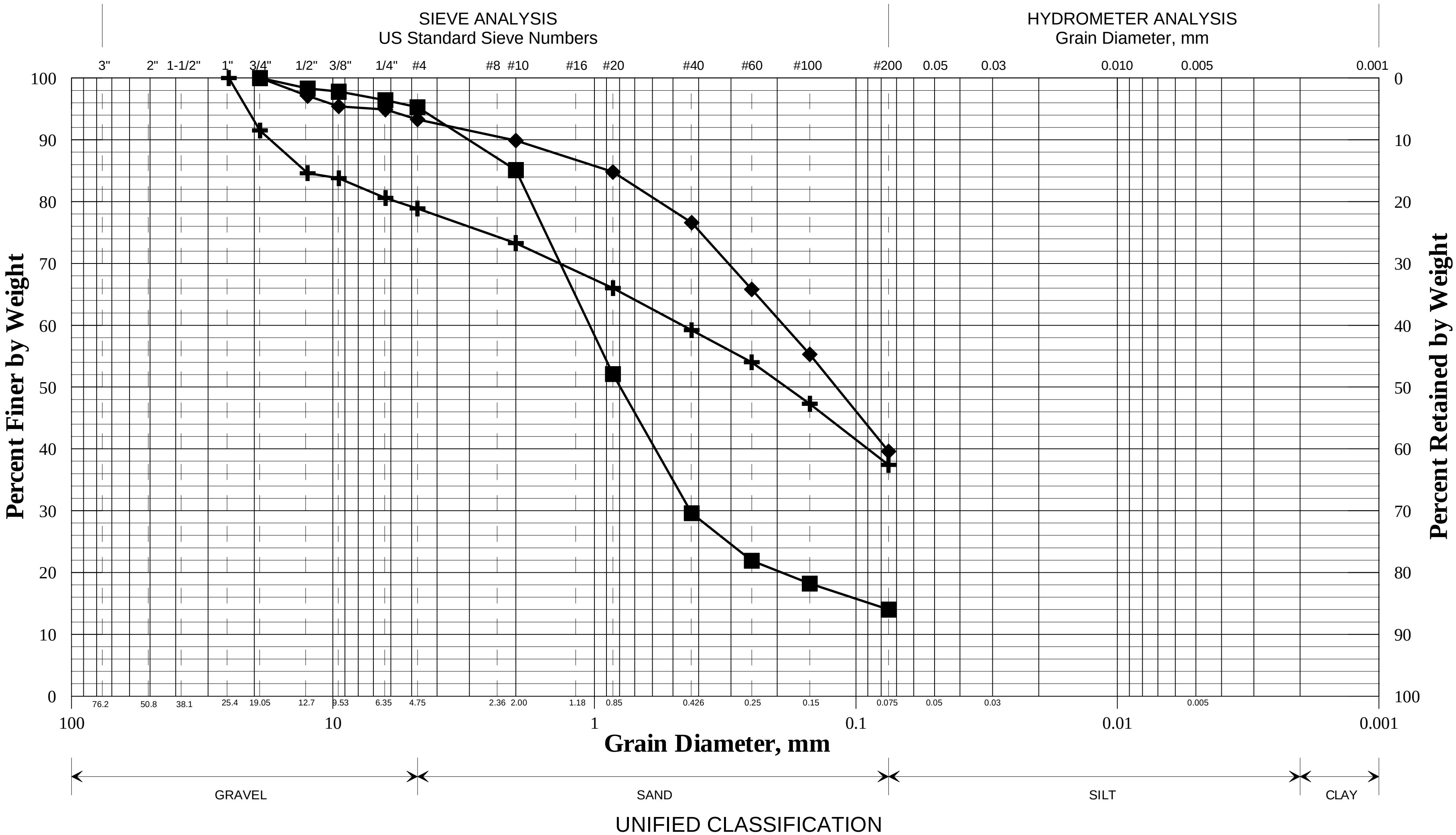
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AEBWS-102B/3D	7+58	6.9 LT	7.0-9.0	Silty SAND, some gravel.	26.9			
◆	BB-AEBWS-102B/4D	7+58	6.9 LT	10.0-12.0	Silty SAND, trace gravel.	17.6			
■	BB-AEBWS-102B/5D	7+58	6.9 LT	15.0-17.0	SAND, little silt, trace gravel.	15.9			
●									
▲									
✖									

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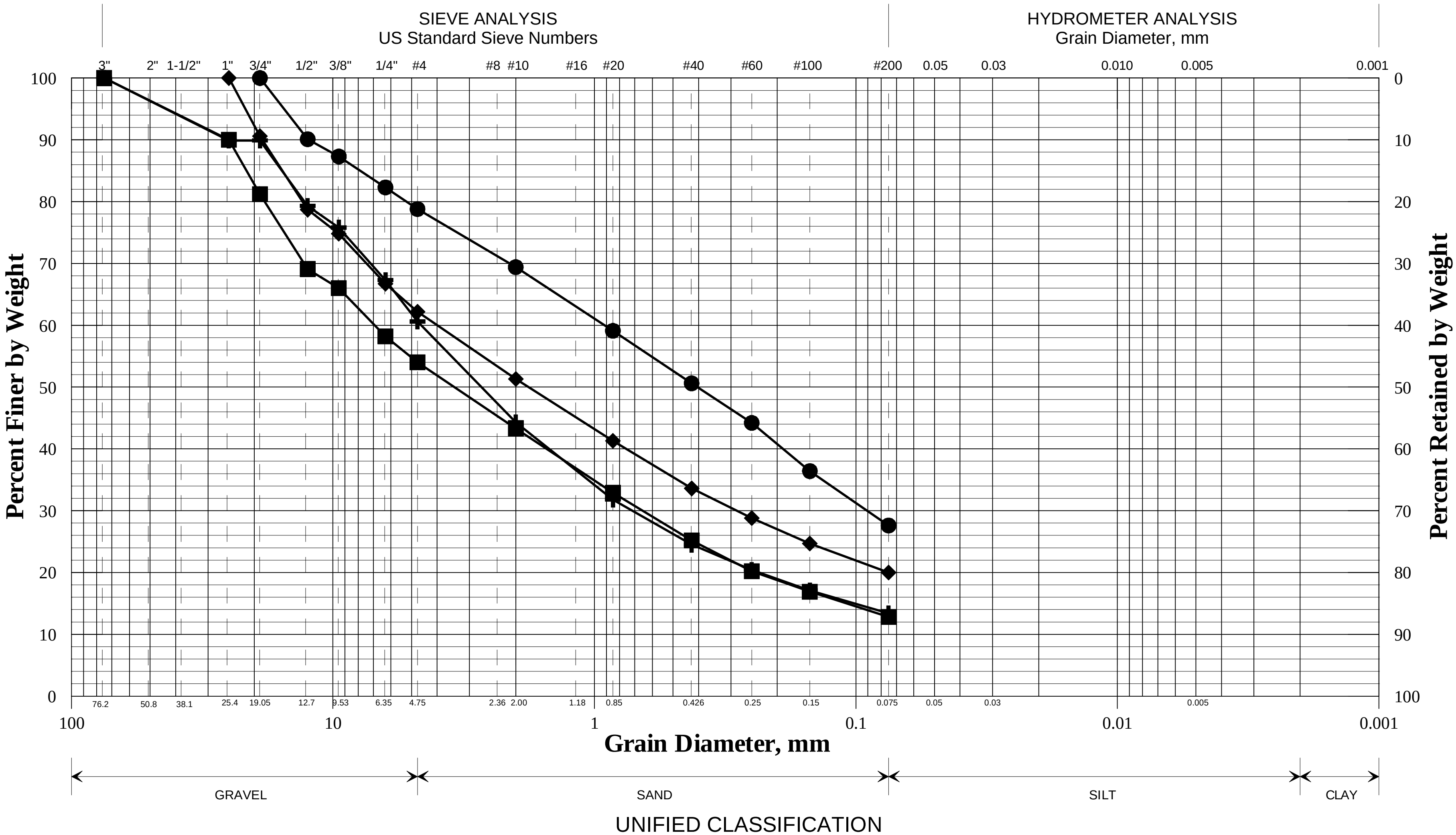
Reported by/Date	
WHITE, TERRY A	10/25/2016

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-AEBWS-101A/4D	6+35	9.4 RT	7.0-9.0	Gravelly SAND, little silt.	5.1			
◆	BB-AEBWS-101A/5D	6+35	9.4 RT	9.5-11.5	Gravelly SAND, little silt.	12.5			
■	BB-AEBWS-101A/8D(A)	6+35	9.4 RT	22.0-24.0	Sandy GRAVEL, little silt.	8.7			
●	BB-AEBWS-101A/9D	6+35	9.4 RT	25.0-27.0	SAND, some silt, some gravel.	11.3			
▲									
✖									

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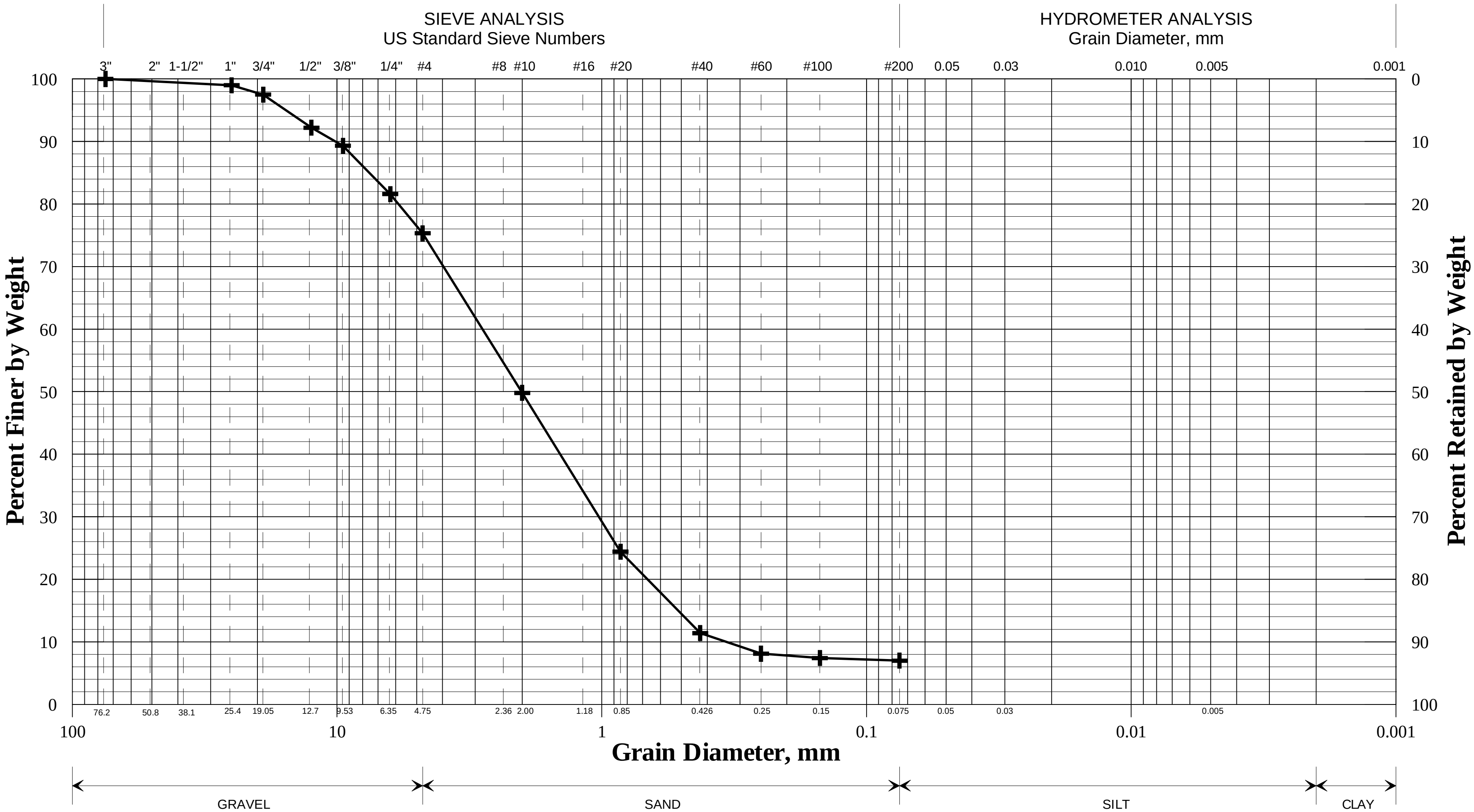
SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	STREAMBED				SAND, some gravel, trace silt.	28.1			
◆									
■									
●									
▲									
✖									

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SHEET 1

Appendix C

Calculations

Spun Pipe Pile Resistance

Geotechnical Axial Compressive Resistance for Spun Pipe Piles

Objective
Estimate the nominal and factored geotechnical axial compressive resistances for spun pipe piles at the strength and extreme limit states considering the piles are installed within bedrock.

Given
1) Lab data and boring logs

Assumptions
1) The piles are installed 5 feet below the top of intact bedrock at both abutments
2) Resistance will be supplied completely by end-bearing because the smooth outside of the permanent casing through low strength overburden materials (including weathered rock) into sound bedrock has minimal coefficient of friction and minimal interaction. Furthermore, the spinning shoe at the pipe tip is slightly larger than the pipe outerwall surface resulting in loose contact between the pipe and bedrock.

Bedrock tip resistance estimate

Unconfined uniaxial compressive strength, q_u:

$q_u := 6800\text{psi}$

Test result of Vassalboro Formation Phyllite from Howland-Enfield. Bedrock recovered at the Gilman Bridge is similar.

Geologic Strength Index, GSI:

$GSI := 33$

LRFD Figure 10.4.6.4-1 for laminated rock in fair condition. GSI range is between 30 and 38.

Disturbance Factor, D:

$D := 0$

LRFD C.10.4.6.4 (p. 10-26) states use of a down hole hammer likely results in a disturbance factor between 0 and 1. Rock coring techniques result in a disturbance factor of near 0.

Empirically determined parameter, s:

$$s := e^{\left(\frac{GSI-100}{9-3\cdot D}\right)}$$

LRFD Eq 10.4.6.4-2

$s = 5.847 \times 10^{-4}$

Empirically determined parameter, a:

$$a := \frac{1}{2} + \frac{1}{6}\left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}}\right)$$

LRFD Eq 10.4.6.4-3

$a = 0.518$

Constant, m_i:

$m_i := 7$

LRFD Table 10.4.6.4-1 m_i for Phyllite is 7 +/-3

Empirically determined parameter, m_b:

$$m_b := m_i \cdot e^{\left(\frac{GSI-100}{28-14D}\right)}$$

LRFD Eq 10.4.6.4-4

$m_b = 0.64$

Effective vertical overburden stress, σ'_{vb}, at tip (bearing) elevation:
Use Abutment No. 2 location for design resulting in less stress and a conservative fracturing coefficient.

Effective weights and thickness from Abutment No. 2 FEM Model:

Layer 1 (El. 510.3-500.0):	Layer 2 (El. 500.0-496.6):	Layer 3 (El. 496.6-488.5):
$\gamma_{1moist} := 125pcf$	$\gamma'_2 := 80pcf$	$\gamma'_3 := 93pcf$
$h_1 := 11.6ft$	$h_2 := 3.4ft$	$h_3 := 8.1ft$

Effective weight and thickness of bedrock

Layer 4 (El. 488.5-483.5):

Test result of Vassalboro Formation Phyllite from Howland-Enfield. Bedrock recovered at the Gilman Bridge is similar.

$\gamma_4 := 170pcf$

$\gamma_w := 62.4pcf$

$\gamma'_4 := \gamma_4 - \gamma_w = 108 \cdot pcf$

$h_4 := 5ft$

$$\sigma'_{vb} := (\gamma_{1moist} \cdot h_1) + (\gamma'_2 \cdot h_2) + (\gamma'_3 \cdot h_3) + (\gamma'_4 \cdot h_4)$$

$$\sigma'_{vb} = 3013.3 \cdot psf$$

Fracturing Coefficient, A:

$$A := \sigma'_{vb} + q_u \cdot \left(m_b \cdot \frac{\sigma'_{vb}}{q_u} + s\right)^a$$

LRFD Eq 10.8.3.5.4c-3

$$A = 47.379 \cdot ksf$$

Nominal unit tip resistance of jointed rock mass, q_p:

$$q_p := A + q_u \cdot \left[m_b \cdot \left(\frac{A}{q_u}\right) + s\right]^a$$

LRFD Eq 10.8.3.5.4c-2

$$q_p = 210.6 \cdot ksf$$

Spun Pipe Pile Tip Resistance Estimate

Pile outside diameters

$OD_{9.625} := 9.625\,in$

$OD_{10.75} := 10.75\,in$

End bearing tip area

$A_{9.625} := \pi \cdot \left(\frac{OD_{9.625}}{2}\right)^2$ $\quad A_{9.625} = 0.505 \cdot ft^2$

$A_{10.75} := \pi \cdot \left(\frac{OD_{10.75}}{2}\right)^2$ $\quad A_{10.75} = 0.63 \cdot ft^2$

Nominal spun pipe pile tip resistance

$R_{p9.625} := q_p \cdot A_{9.625}$ $\quad R_{p9.625} = 106.4 \cdot kip$

$R_{p10.75} := q_p \cdot A_{10.75}$ $\quad R_{p10.75} = 132.8 \cdot kip$ $\quad$ LRFD Eq 10.83.5-2

Factored spun pipe pile tip resistance - Service and Extreme Limit States

$\phi_{qp} := 1.0$ $\quad$ LRFD 10.5.5.1

$R_{r9.625} := \phi_{qp} \cdot R_{p9.625}$ $\quad R_{r9.625} = 106 \cdot kip$

$R_{r10.75} := \phi_{qp} \cdot R_{p10.75}$ $\quad R_{r10.75} = 133 \cdot kip$ $\quad$ LRFD Eq 10.8.3.5-1

Factored spun pipe pile tip resistance - Strength Limit State

$\phi_{stat} := 0.5$ $\quad$ LRFD Table 10.5.5.2.4-1 for Drilled Shaft Tip Resistance in Rock

$R_{r9.625} := \phi_{stat} \cdot R_{p9.625}$ $\quad R_{r9.625} = 53 \cdot kip$

$R_{r10.75} := \phi_{stat} \cdot R_{p10.75}$ $\quad R_{r10.75} = 66 \cdot kip$ $\quad$ LRFD Eq 10.8.3.5-1

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Piscataquis Bridge #3040 Route 116/Coffin St. Location: Howland, Maine	Boring No.: BB-PR-201 PIN: 15635.00

Driller:	Maine Test Borings	**Elevation (ft.)**	134.10	**Auger ID/OD:**	-
Operator:	B. Enos	**Datum:**	NAVD 88	**Sampler:**	Split Spoon 1.375 in. ID
Logged By:	H. Hollauer	**Rig Type:**	Mobile B-24 Skid Mounted	**Hammer Wt./Fall:**	140/30 SS; 140/30 NW
Date Start/Finish:	1/21/10-1/22/10	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ - 2 in. ID
Boring Location:	N634399.9, E1760428.6	**Casing ID/OD:**	NW - 3 in.	**Water Level*:**	14.4 ft above mudline
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.00 - 2.00	1/7/5/5	12	12	39	133.85		Gray, wet, medium dense, fine to coarse SAND, trace silt. 0.25 Gray, wet, medium dense, sandy SILT, little gravel (Till).	A-4 (0), ML WC = 12.0% OC = 0.2% Non Plastic
							40				
							41				
							47				
							66				
5	2D	24/14	5.00 - 7.00	8/28/29/17	57	57	50			Gray, wet, very dense, silty SAND, trace gravel (Till).	A-4 (0), SM WC = 9.8% Non Plastic
							63	128.10		6.00 Gray, weathered rock.	
	R1	60/49.2	7.30 - 12.30	RQD = 63%			100/3 NQ-2	126.80		Casing refusal at 7.3 ft. Advance roller bit to 7.3 ft. Begin NQ rock core at 7.3 ft. 7.30 Top of bedrock at Elev. 126.8 ft. Bedrock: Light to medium greenish gray, fine grained to aphanitic, PHYLLITE, moderately fractured, moderately hard, fresh to slightly weathered; Color change to purplish gray from 13.9 to 15.0 ft-bml; highly fractured zone filled with clayey soil from 10.6 to 11.2 ft-bml; highly fractured zone filled with soil from 16.7 to 17.3 ft-bml [Vassalboro Formation]. R1: Recovery = 82% R1: Rock Mass Quality = Fair R1: Core Times (min:sec) 7.3-12.3' (8min/ft) R2: Recovery = 100% R2: Rock Mass Quality = Fair R2: Core Times (min:sec) 12.3-17.3' (6min/ft)	
10											124.17'-124.53' qt= 980 ksf 123.45'-124.82' qt= 1,587 ksf
	R2	60/60	12.30 - 17.30	RQD = 64%							
15											
								116.80		17.30 **Bottom of Exploration at 17.30 feet below ground surface.**	
20											
25											
30											

Remarks:

1. Coordinates of test boring determined by MaineDOT and provided in NAD83 (96) ME2000 Central Zone coordinate system.
2. Boring advanced through approximately 14 inches of ice.
3. At time of field exploration, water depth (including ice thickness) was 14.4 ft at boring location.

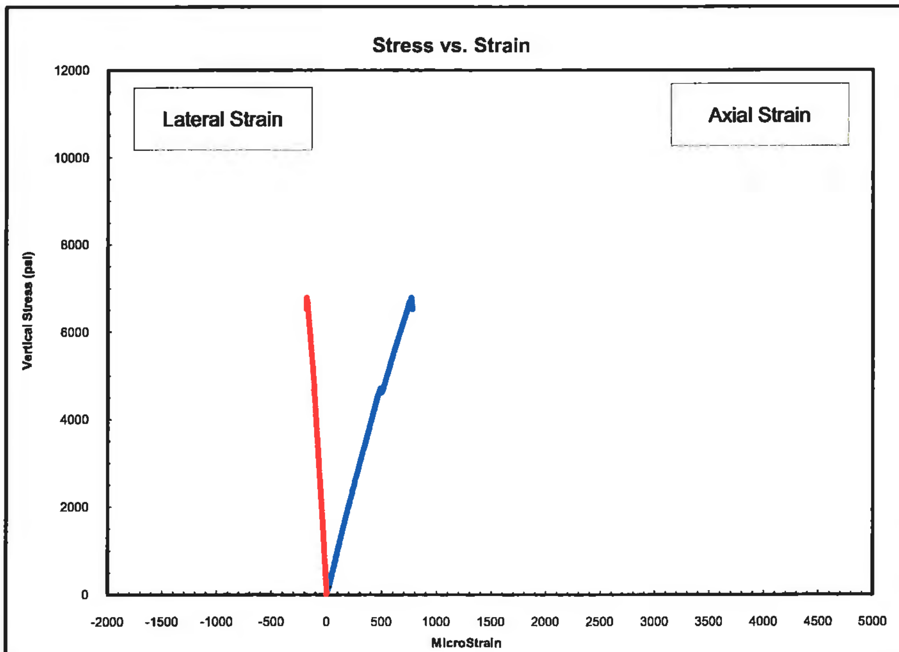
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* 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.

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Golder Associates
Project Name:	Howland Bridge
Project Location:	Howland, ME
GTX #:	9547
Test Date:	02/18/10
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-PR-201
Sample ID:	---
Depth, ft:	9.57-9.93
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 6,803 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-2000	10,300,000	0.23
2000-4000	9,410,000	0.25
4000-6000	7,370,000	0.21

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

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Client:	Golder Associates	Test Date:	2/16/2010
Project Name:	Howland Bridge	Tested By:	daa
Project Location:	Howland, ME	Checked By:	mpd
GTX #:	9547		
Boring ID:	BB-PR-201		
Sample ID:	---		
Depth:	9.57-9.93 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.33	4.34	4.34
Specimen Diameter, in:	1.98	1.97	1.98
Specimen Mass, g:	593.27		
Bulk Density, lb/ft^3	170		
Length to Diameter Ratio:	2.2		

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap ≤ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? YES

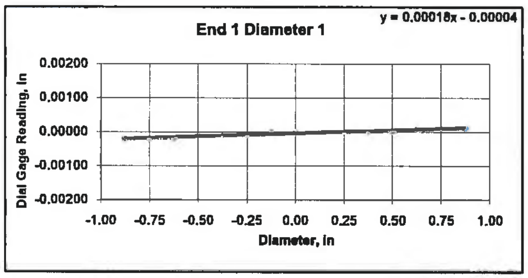
END FLATNESS AND PARALLELISM (Procedure FP1)

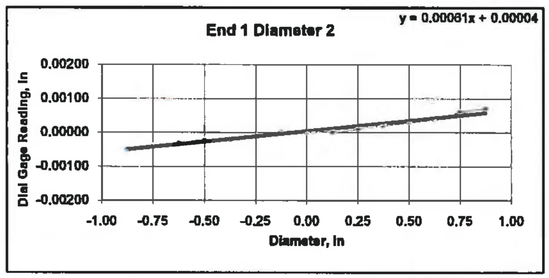
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00020	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00060	0.00070

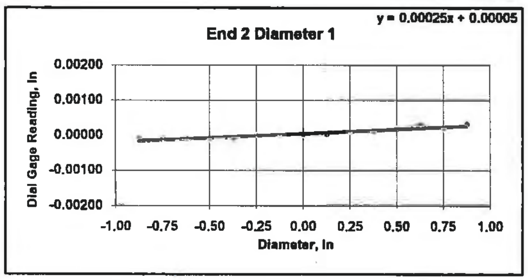
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00120

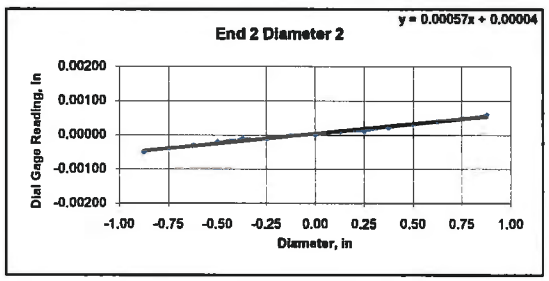
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030	0.00020	0.00030
Diameter 2, in (rotated 90°)	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060

Difference between max and min readings, in:
0° = 0.0004 90° = 0.0011
Maximum difference must be < 0.0020 in. Difference = ± 0.00060
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00018
Angle of Best Fit Line: 0.01031

End 2:
Slope of Best Fit Line 0.00025
Angle of Best Fit Line: 0.01432

Maximum Angular Difference: 0.00401

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00061
Angle of Best Fit Line: 0.03495

End 2:
Slope of Best Fit Line 0.00057
Angle of Best Fit Line: 0.03266

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? YES
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

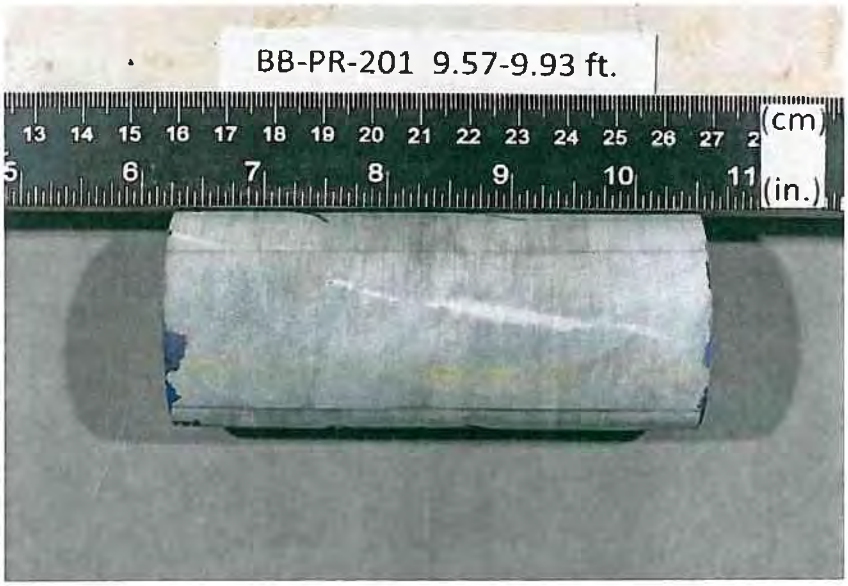
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00030	1.975	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00120	1.975	0.00061	0.035	YES
END 2					
Diameter 1, in	0.00040	1.975	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00110	1.975	0.00056	0.032	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

GeoTesting express
a subsidiary of Geocomp Corporation

Client:	Golder Associates
Project Name:	Howland Bridge
Project Location:	Howland, ME
GTX #:	9547
Test Date:	02/18/10
Tested By:	daa
Checked By:	mpd
Boring ID:	BB-PR-201
Sample ID:	---
Depth, ft:	9.57-9.93



After cutting and grinding

After break

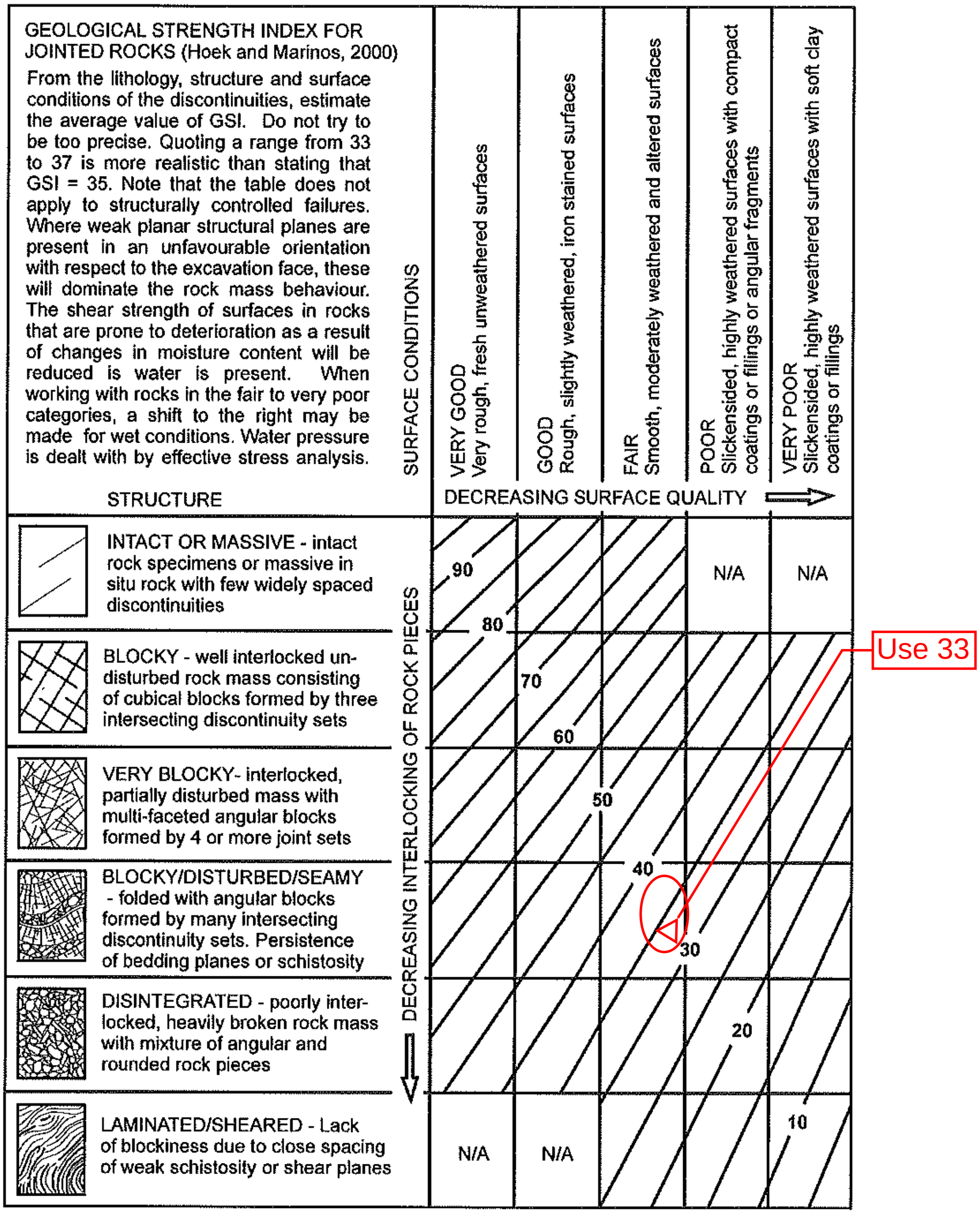


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

q_u = average unconfined compressive strength of rock core (ksf)

m_b, s, and a = empirically determined parameters

The value of the constant m_i should be estimated from Table 10.4.6.4-1, based on lithology. Relationships between GSI and the parameters m_b, s, and a, according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI-100}{28-14D}\right)} \qquad (10.4.6.4\text{-}4)$$

Table 10.4.6.4-1—Values of the Constant m_i by Rock Group (after Marinos and Hoek 2000; with updated values from Rocscience, Inc., 2007)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3) Breccia (19 ± 5)	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke (18 ± 3)	Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)) Metasandstone (19 ± 3)	Quartzite 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
	Foliated*			Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3)) Basalt (25 ± 5)	
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, φ_{stat}	Side resistance in clay	α-method (Brown et al., 2010)	0.45
	Tip resistance in clay	Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand	β-method (Brown et al., 2010)	0.55
	Tip resistance in sand	Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010)	0.60
	Tip resistance in cohesive IGMs	Brown et al. (2010)	0.55
	Side resistance in rock	Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985)	0.50
		Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	
Block Failure, φ_{bl}	Clay		0.55
Uplift Resistance of Single-Drilled Shafts, φ_{up}	Clay	α-method (Brown et al., 2010)	0.35
	Sand	β-method (Brown et al., 2010)	0.45
	Rock	Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, φ_{ug}	Sand and clay		0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials		1.0
Static Load Test (compression), φ_{load}	All Materials		0.70
Static Load Test (uplift), φ_{upload}	All Materials		0.60

Bedrock-Socket H-Pile Resistance

Nominal and Factored Axial Geotechnical Resistance of Rock-Socketed H-piles with Oversized (extending beyond the flanges) Steel Plate

Bedrock Properties

ref: boring logs

BB-AEBWS-102B;
R_1=0%, R_2=0%, R_3=43%, R_4=44%
BB-AEBWS-201;
R_1=72%, R_2=0%, R_3=86%, R_4=100%, R_5=100%, R_6=22%, R_7=90%, R_8=96%
BB-AEBWS-202C;
R_1=84%, R_2=32%, R_3=66%, R_4=100%

Rock Type: fine grained phyllite, moderately hard, slightly weathered, vertically foliated, joints are low angle to vertical, very close to close, tight to occasionally healed by calcite.

ϕ = 27-34 (AASHTO LRFD Table C.10.4.6.4-1);
Phylilite C_o= 3,500 - 35,000 psi
ref: AASHTO Standard Specifications for Bridges 17th Edition, Table 4.4.8.1.2B

For Design Purposes, use bedrock data from BB-AEBWS-203B;R1: RQD = 24% and an assumed Unconfined Compressive Strength of 6,800 psi based on lab testing of similar bedrock. Assume compressive strength of grout 6,000 psi.

Pile Properties

Use the following piles: 12x53, 12x74, 14x73, 14x89, 14x117

Flange depth

Plate Depth:
Flange depth +1 inch rounded up to nearest whole inch.

$$d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.6 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in \qquad d_1 := \begin{pmatrix} 13 \\ 14 \\ 15 \\ 15 \\ 16 \end{pmatrix} \cdot in$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Pile yield strength

$$F_y := 50 \cdot ksi$$

Web width

Plate Width:
Web width +1 inch rounded up to nearest whole inch.

$$b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in \qquad b_1 := \begin{pmatrix} 14 \\ 14 \\ 16 \\ 16 \\ 16 \end{pmatrix} \cdot in$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Steel Square Area

$$A_{plate} := \overrightarrow{(d_1 \cdot b_1)} = \begin{pmatrix} 182 \\ 196 \\ 240 \\ 240 \\ 256 \end{pmatrix} \cdot in^2$$

Pile Size, Plate Dim.
12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Diagonal Dimension of Plate

$$Diag_{plate} := \sqrt{d_1^{\,2} + b_1^{\,2}} = \begin{pmatrix} 19.1 \\ 19.8 \\ 21.9 \\ 21.9 \\ 22.6 \end{pmatrix} \cdot in$$

Pile Size, Plate Dim.
12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Grout tube Properties

Assume two 1" O.D. holes to accommodate grout tube

$r := 1in$

$A_{tube} := \pi \cdot r^2 \qquad A_{tube} = 3.142 \cdot in^2$

$$A_{square} := A_{plate} - 2 \cdot A_{tube} = \begin{pmatrix} 175.72 \\ 189.72 \\ 233.72 \\ 233.72 \\ 249.72 \end{pmatrix} \cdot in^2$$

Pile Size, Plate Dim.
12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Method A. Canadian Geotechnical Society Method

Geotechncial axial pile resistance for pile end bearing on rock as determined by CGS method and outlined in Canadian Foundation Engineering Manual, 4th Edition 2006, and FHWA LRFD Pile Foundation Design Example in FHWA-NHI-05-094.

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of grout $\quad f'_c := 6000 \cdot psi$

Spacing of discontinuities $\quad s_d := 4 \cdot in$

Width of discontinuities. Joints are healed to open $\quad t_d := .075 \cdot in$

Pile width is b - matrix $\quad D := b$

Embedment depth of pile in socket $\quad H_s := 5 \cdot ft$

Diameter of socket: $\quad D_s := 24 \cdot in$

Depth factor

$$dd := 1 + 0.4 \cdot \frac{H_s}{D_s} \qquad dd = 2$$

and dd < 3 OK

K_{sp}

$$K_{sp} := \frac{3 + \dfrac{s_d}{D}}{10 \cdot \left(1 + 300 \cdot \dfrac{t_d}{s_d}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.129 \\ 0.129 \\ 0.127 \\ 0.127 \\ 0.127 \end{pmatrix}$$

Ksp has a factor of safety of 3.0 in the CGS method. Remove in calculation of pile tip resistance, below.

Geotechnical tip resistance.

$$q_{p\_a} := 3 \cdot f'_c \cdot K_{sp} \cdot dd$$

$$q_{p\_a} = \begin{pmatrix} 671 \\ 670 \\ 659 \\ 659 \\ 658 \end{pmatrix} \cdot ksf$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Nominal geotechnical tip resistance, Rp -

$$R_{p\_a} := \overrightarrow{\left(q_{p\_a} \cdot A_{square}\right)}$$

$$R_{p\_a} = \begin{pmatrix} 819 \\ 883 \\ 1070 \\ 1070 \\ 1142 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$$\phi_{stat} := 0.45$$

LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance ($R_{strength}$)

$$R_{strength} := \phi_{stat} \cdot R_{p\_a}$$

$$R_{strength} = \begin{pmatrix} 369 \\ 397 \\ 482 \\ 481 \\ 514 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

CGS method is superceded by the Intact Rock or Jointed Rock Mass Method

Factored Axial Geotechnical Compressive Resistance - *Service Limit States*

Resistance factor, Service Limit States

$$\phi_{service} := 1.0$$

LRFD Article 10.5.5.1

Factored Geotechnical Tip Resistance ($R_{service}$)

$$R_{service} := \phi_{service} \cdot R_{p\_a}$$

$$R_{service} = \begin{pmatrix} 819 \\ 883 \\ 1070 \\ 1070 \\ 1142 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Extreme Limit States*

Resistance factor, Extreme Limit States

$\phi_{ee} := 1.0$

LRFD Article 10.5.5.3.2, 10.5.5.3.3

Factored Geotechnical Tip Resistance (R_{ee})

$R_{ee} := \phi_{ee} \cdot R_{p\_a}$

$$\boxed{R_{ee} = \begin{pmatrix} 819 \\ 883 \\ 1070 \\ 1070 \\ 1142 \end{pmatrix} \cdot kip}$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Method B. Intact Rock Method

Geotechncial axial pile resistance for pile end bearing on rock as determined by the Intact Rock Method, proposed by Sandford, MaineDOT Transportation Research Division Technical Report 14-01, Phase 2 (January 2014), based on Rowe and Armitage (1987) equation cited by NCHRP Synthesis 360, Turner, (2006) as an upper limit of bearing resistance.

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of grout: $f'_c := 6000psi$

Geotechnical tip resistance in massive rock:

$q_{p\_b} := 2.5 \cdot f'_c$ $\boxed{q_{p\_b} = 2160 \cdot ksf}$

Nominal geotechnical tip resistance, Rp

$R_{p\_b} := \overrightarrow{\left(q_{p\_b} \cdot A_{square}\right)}$

$$\boxed{R_{p\_b} = \begin{pmatrix} 2636 \\ 2846 \\ 3506 \\ 3506 \\ 3746 \end{pmatrix} \cdot kip}$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$R_{r\_b} := \phi_{stat} \cdot R_{p\_b}$

$$\boxed{R_{r\_b} = \begin{pmatrix} 1186 \\ 1281 \\ 1578 \\ 1578 \\ 1686 \end{pmatrix} \cdot kip}$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Service Limit States*

Resistance factor, Service Limit States

$\phi_{service} := 1.0$ LRFD Article 10.5.5.1

Factored Geotechnical Tip Resistance ($R_{service}$)

$R_{service} := \phi_{service} \cdot R_{p\_b}$

$$R_{service} = \begin{pmatrix} 2636 \\ 2846 \\ 3506 \\ 3506 \\ 3746 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Extreme Limit States*

Resistance factor, Extreme Limit States

$\phi_{ee} := 1.0$ LRFD Article 10.5.5.3.2, 10.5.5.3.3

Factored Geotechnical Tip Resistance (R_{ee})

$R_{ee} := \phi_{ee} \cdot R_{p\_b}$

$$R_{ee} = \begin{pmatrix} 2636 \\ 2846 \\ 3506 \\ 3506 \\ 3746 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Method C. Jointed Rock Mass Method

Geotechncial axial pile resistance for pile end bearing on rock as determined by the Jointed Rock Mass Method cited by NCHRP Synthesis 360, Turner, (2006) when discontinuities are vertical or nearly vertical and open joints are present with a spacing less than the socket diameter.

Nominal unit bearing resistance of pile point, q_p

Design value of compressive strength of grout: $f'_c := 6000psi$

Geotechnical tip resistance in jointed rock:

$q_{p\_c} := f'_c$ NCHRP Synthesis 360, Eq. 44 and Figure 28, pg. 43

$\boxed{q_{p\_c} = 864 \cdot ksf}$

Nominal geotechnical tip resistance, R_p -

$R_{p\_c} := \overrightarrow{\left(q_{p\_c} \cdot A_{square}\right)}$

$$R_{p\_c} = \begin{pmatrix} 1054 \\ 1138 \\ 1402 \\ 1402 \\ 1498 \end{pmatrix} \cdot kip$$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

Factored Axial Geotechnical Compressive Resistance - *Strength Limit States*

Resistance factor, end bearing on rock Canadian Geotechnical Society method

$\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

Factored Geotechnical Tip Resistance (R_r)

$R_{r\_c} := \phi_{stat} \cdot R_{p\_c}$ $R_{r\_c} = \begin{pmatrix} 474 \\ 512 \\ 631 \\ 631 \\ 674 \end{pmatrix} \cdot kip$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

These factored tip resistances will govern Geotechnical Resistance for Strength Limit States with a ϕ = 0.45

Factored Axial Geotechnical Compressive Resistance - *Service Limit States*

Resistance factor, Service Limit States

$\phi_{service} := 1.0$ LRFD Article 10.5.5.1

$R_{p\_c} := \phi_{service} \cdot R_{p\_c}$ $R_{p\_c} = \begin{pmatrix} 1054 \\ 1138 \\ 1402 \\ 1402 \\ 1498 \end{pmatrix} \cdot kip$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

These nominal tip resistances will govern Geotechnical Resistance for Service Limit States with a ϕ = 1.0

Factored Axial Geotechnical Compressive Resistance - *Extreme Limit States*

Resistance factor, Extreme Limit States

$\phi_{ee} := 1.0$ LRFD Article 10.5.5.3.2, 10.5.5.3.3

Factored Geotechnical Tip Resistance (R_{ee})

$R_{ee} := \phi_{ee} \cdot R_{p\_c}$ $R_{ee} = \begin{pmatrix} 1054 \\ 1138 \\ 1402 \\ 1402 \\ 1498 \end{pmatrix} \cdot kip$

Pile Size, Plate Dim.

12x53, 13x14
12x74, 14x14
14x73, 15x16
14x89, 15x16
14x117, 16x16

These nominal tip resistances will govern Geotechnical Resistance for Extreme Limit States with a ϕ = 1.0

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[1] (ksf)	C_o[1] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[2]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

[1]Range of Uniaxial Compressive Strength values reported by various investigations.
[2]Not including oil shale.

$$\rho = q_o (1 - \nu^2) B I_\rho / E_m, \text{ with } I_\rho = (L/B)^{1/2}/\beta_z \qquad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \qquad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(\text{RQD}) - 1.32 \geq 0.15 \qquad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

Table 10.5.5.2.3-1—Resistance Factors for Driven Piles (continued)

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile—Static Analysis Methods, φ_{stat}	Side Resistance and End Bearing: Clay and Mixed Soils	
	α-method (Tomlinson, 1987; Skempton, 1951)	0.35
	β-method (Esrig & Kirby, 1979; Skempton, 1951)	0.25
	λ-method (Vijayvergiya & Focht, 1972; Skempton, 1951)	0.40
	Side Resistance and End Bearing: Sand	
	Nordlund/Thurman Method (Hannigan et al., 2005)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (Canadian Geotech. Society, 1985)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Static load test	0.60
	Dynamic test with signal matching	0.50
Group Uplift Resistance, φ_{ug}	All soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
Pile Drivability Analysis, φ_{da}	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2
	In all three Articles identified above, use φ identified as "resistance during pile driving"	

bearing capacity of intact rock to the compressive strength of the bedrock. The equation is presented below:

$$q_p = 2.5 q_u \quad [3.15]$$

Where:

q_p = end bearing capacity of the bedrock

q_u = unconfined compressive strength of the bedrock

3.5. Tip Capacity for Piles Bearing in Till

There are a few piles included in the study that were designed to obtain support from soils without bearing on bedrock. There are also some piles that fetched up in the till or other overlying strata. There were not any piles that experienced end bearing in cohesive strata, so tip capacity in cohesive soils will not be considered in this report. The methods for determining end bearing on piles above bedrock are described in this section.

3.5.1. Nordlund Method

The Nordlund method (Nordlund 1963) comprised a bearing capacity relation from Berezantzes et al (1961) which did not have a limiting value. Since the Nordlund (1963) paper, the bearing capacity relation has been changed and a limiting value from Meyerhof (1976) has been added by Hannigan et al (2006a) based on Bowles (1977). Subsequent editions (Bowles 1982; Bowles 1988) do not use this method. The end bearing capacity of the soil now associated with the Nordlund method is detailed below (Hannigan et al, 2006a):

$$q_p = \alpha_t N'_q \sigma'_v \leq q_L \quad [3.16]$$

Where:

α_t = coefficient determined from Figure 3-9.

at the head, can vary over a wide range at typical working loads. Several authors suggest a typical range of 10% to 20% of the head load (Williams et al. 1980; Carter and Kulhawy 1988), and some authors suggest that base resistance should be neglected entirely for rock-socket design (Amir 1986). Elasticity solutions show that base load transfer depends on the embedment ratio (L/B) and the modulus ratio (E_c/E_r). The ratio of base load to applied load (Q_b/Q_c) decreases with increasing L/B (see Figure 21) and increases with increasing modular ratio. As discussed previously, there is ample evidence that base resistance should not be discounted in most cases (Figure 23), and that construction and inspection methods are available to control base quality. Load tests, described in chapter five, provide a means to determine the effects of construction on base load transfer.

The ultimate base resistance of a rock-socketed drilled shaft, Q_b, is the product of the limiting normal stress, or bearing capacity, q_{ult}, at the base and the cross-sectional area of the shaft base (A_b):

$$Q_b = q_{ult} A_b = q_{ult} [1/4 \pi B^2] \qquad (42)$$

Analytical solutions for bearing capacity of rock are based on the general bearing capacity equation developed for soil, with appropriate modifications to account for rock mass characteristics such as spacing and orientation of discontinuities, condition of the discontinuities, and strength of the rock mass. Typical failure modes for foundations bearing on rock are shown in Figure 28. The failure modes depicted were intended to address shallow foundations bearing on rock (Sowers 1976); however, the general concepts should be applicable to bearing capacity of deep foundations. The cases shown can be placed into four categories: massive, jointed, layered, and fractured rock.

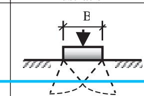
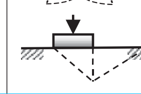
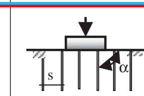
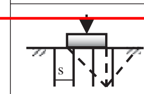
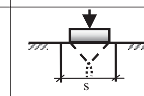
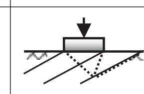
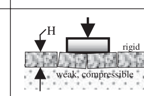
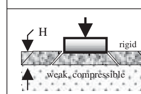
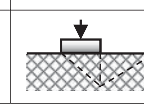
Rock Mass Conditions	Joint Dip Angle, from horizontal	Joint Spacing	Failure: Illustration	Failure: Mode	Bearing Capacity Equation No.
INTACT / MASSIVE	N/A	S>>B		(a) Brittle Rock: Local shear failure caused by localized brittle fracture	Eq. 43
				(b) Ductile Rock: General shear failure along well-defined shear surfaces	Eq. 43
STEEPLY DIPPING JOINTS	70° < α < 90°	S < B		(c) Open Joints: Compression failure of individual rock columns	Eq. 44
				(d) Closed Joints General shear failure along well defined failure surfaces; near vertical joints	Eqs. 45–52
		S > B		(e) Open or Closed Joints: Failure initiated by splitting leading to general shear failure; near vertical joints	Eqs. 53–54
JOINTED	20° < α < 70°	S < B or S > B if failure wedge can develop along joints		(f) General shear failure with potential for failure along joints; moderately dipping joint sets.	Eqs. 45–52
LAYERED	0 < α < 20°	Limiting value of H w/re to B is dependent upon material properties		(g) Rigid layer over weak compressible layer: Failure is initiated by tensile failure caused by flexure of rigid upper layer	N/A
				(h) Thin rigid layer over weak compressible layer: Failure is by punching shear through upper layer	N/A
FRACTURED	N/A	S << B		(g) General shear failure with irregular failure surface through fractured rock mass; two or more closely spaced joint sets	Eq. 57

FIGURE 28 Bearing capacity failure modes in rock (after *Rock Foundations* 1994).

Massive Rock

For this case, the ultimate bearing capacity will be limited to the bearing stress that causes fracturing in the rock. An intact rock mass can be defined, for purposes of bearing capacity analysis, as one for which the effects of discontinuities are insignificant. Practically, if joint spacing is more than four to five times the shaft diameter, the rock is massive. If the base is embedded in rock to a depth of at least one diameter, the failure mode is expected to be by punching shear (Figure 28, mode a). In this case, Rowe and Armitage (1987b) stated that rock fracturing can be expected to occur when the bearing stress is approximately 2.7 times the rock uniaxial compressive strength. For design, the following is recommended:

$$q_{ult} = 2.5\, q_u \qquad (43)$$

Other conditions that must be verified are that the rock to a depth of at least one diameter below the base of the socket is either intact or tightly jointed (no compressible or gouge-filled seams) and there are no solution cavities or voids below the base of the pier. O'Neill and Reese (1999) recommend limiting base resistance to $2q_u$ if the embedment into rock is less than one diameter. In rock with high compressive strength, the designer also must determine the structural capacity of the shaft, which may govern the allowable normal stress at the base.

Jointed Rock Mass

When discontinuities are vertical or nearly vertical ($\alpha > 70°$), and open joints are present with a spacing less than the socket diameter ($S < B$, Figure 28, mode c), failure can occur (theoretically) by unconfined compression of the poorly constrained columns (Sowers 1979). Bearing capacity can be estimated from

$$q_{ult} = q_u = 2c \tan(45° + \tfrac{1}{2}\phi) \qquad (44)$$

where q_u = uniaxial compressive strength and c and ϕ are Mohr–Coulomb strength properties of the rock mass. If the nearly vertical joints are closed (Figure 28, mode d), a gen-

Lateral Pile Resistance Soil-Parameters

Development of soil parameters for abutment soils springs and p-y curve generation

OBJECTIVE
Estimate soil parameters for software to generate soil springs and p-y curves

Given:
1) Boring logs and lab data

Assumptions:
1) Assume the groundwater table is at Elevation 500.0, or approximately 12' bgs.
2) MaineDOT Bridge Design Guide (BDG) Soil Type 4 will be used for integral abutment backfill.
3) Underdrain Backfill Material Type C will be used to fill annular space between temporary casing and H-pile
4) Piles shall be fixed within bedrock.

Abutment Soil Model

Soil Layer No. 1 (Granular Borrow) Finish Grade - El. 500.0

Internal Angle of Friction $\boxed{\phi_1 := 32deg}$

Soil Moist Unit Weight $\gamma_{1moist} := 125pcf$

MaineDOT BDG Table 3-3
Soil Type 4

Specific Gravity of quartz sand

$\boxed{G_{s1} := 2.65}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 35:
Specific Gravity of Quartz

Height of layer 1:

$h_1 := 511.6ft - 500ft \qquad h_1 = 11.6 \cdot ft$

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Based on density test results from nearby projects, optimum moisture content of granular borrow occurs between 6 and 10 percent. Assume 8 percent.

$w_{1opt} := .08 \qquad \gamma_w := 62.4pcf$

Dry unit weight

$$\gamma_{1dry} := \frac{\gamma_{1moist}}{1 + w_{1opt}} \qquad \gamma_{1dry} = 115.7 \cdot pcf$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.2 Natural Moisture Content in a saturated state

$w_{1sat} := .23$

$\gamma_{1saturated} := \gamma_{1dry} \cdot (1 + w_{1sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit Weight Relationships

$\gamma_{1saturated} = 142 \cdot pcf$

Void Ratio

$e_1 := \frac{G_{s1} \cdot \gamma_w}{\gamma_{1dry}} - 1$ $\boxed{e_1 = 0.43}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 55: Eq. 3.17

Representative constant for calculation of P-Y curve, initial horizontal soil modulus, k_h: Medium dense sand above water table with static loading = 90 pci

Technical Manual LPile 2016 p. 96

At-rest Earth Pressure - Rankine Theory

$K_o := 1 - \sin(\phi_1)$ $\boxed{K_o = 0.47}$

Fang, Foundation Engineering Handbook 2nd ed. p. 224, Eq. 6.2

Soil response behind the abutment may be modeled by use of soil springs to represent the soil resistance. The soil springs may assume shear interaction between the springs to represent a frictional response to transverse movements of the wall (the wall sliding along the soil).

Stress-Strain Modulus, E_s

$E_s := 25MPa = 3626 \cdot psi$

Bowles 5th ed. p. 125 Table 2-8

Poisson's Ratio, μ

$\mu := 0.30$

Bowles 5th ed. p. 123 Table 2-7

Shear Modulus, G'_s

$G'_s := \frac{E_s}{2(1 + \mu)}$ $\boxed{G'_s = 1395 \cdot psi}$

Bowles 5th ed. p. 121 Eq. (a)

Soil Layer No. 2 (Submerged, Granular Borrow) El. 500.0 - 496.6

Internal Angle of Friction $\boxed{\phi_2 := 32deg}$

Soil Total Unit Weight $\gamma_{2moist} := 125pcf$

MaineDOT BDG Table 3-3 Soil Type 4

Specific Gravity of quartz sand

$G_{s2} := 2.65$

Das, Principles of Geotechnical Eng. 7th Ed. p. 35: Specific Gravity of Quartz

Height of layer 2:

$h_2 := 500.0ft - 496.6ft$ $h_2 = 3.4 \cdot ft$

Assume the total (moist) unit weight of Soil Type 4 considers placement at the material's optimum moisture content. Based on density test results from nearby projects, optimum moisture content of granular borrow occurs between 6 and 10 percent. Assume 8 percent.

$w_{2opt} := .08$

Dry unit weight

$\gamma_{2dry} := \frac{\gamma_{2moist}}{1 + w_{2opt}}$ $\gamma_{2dry} = 115.7 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{2sat} := .23$

$\gamma_{2saturated} := \gamma_{2dry} \cdot (1 + w_{2sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{2saturated} = 142 \cdot pcf$

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_2 := \gamma_{2saturated} - \gamma_w$

Holtz and Kovacs, Intro to Geotechnical Eng.p. 18 Eq (2-11)

$\gamma'_2 = 80 \cdot pcf$

Void Ratio

$e_2 := \frac{G_{s2} \cdot \gamma_w}{\gamma_{2dry}} - 1$ $\boxed{e_2 = 0.43}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 55, Eq. 3.17

Representative constant for calculation of P-Y curve, initial horizontal soil modulus, k_h: Medium dense sand below water table with static loading = 60 pci

Technical Manual
LPile 2016 p. 96

At-rest Earth Pressure - Rankine Theory

$K_o := 1 - \sin(\phi_2)$ $K_o = 0.47$

Fang, Foundation Engineering Handbook 2nd ed. p. 224, Eq. 6.2

Soil response behind the abutment may be modeled by use of soil springs to represent the soil resistance. The soil springs may assume shear interaction between the springs to represent transverse continuity of the concrete between soil springs.

Stress-Strain Modulus, E_s

$E_s := 12MPa = 1740 \cdot psi$

Bowles 5th ed. p. 125
Table 2-8

Poisson's Ratio, μ

$\mu := 0.40$

Bowles 5th ed. p. 123
Table 2-7

Shear Modulus, G'_s

$G'_s := \frac{E_s}{2(1 + \mu)}$ $\boxed{G'_s = 622 \cdot psi}$

Bowles 5th ed. p. 121
Eq. (a)

Combine Soil Layer 1 and Soil Layer 2 El. 511.6 - 496.6 should LPile be utilized to develop p-y curves

LPile's layering correction to p-y curve algorithm returns an error when more than one soil layer is defined above the pile head. The recommended solution is to combine the soil layers above the pile head into a single layer with an effective unit weight that results in an equivalent effective vertical stress at the pile head and extend the combined layer to a minimum elevation of just below the top of the pile.

Effective Vertical stress at bottom of Soil Layer 1

$\sigma_{v1} := \gamma_{1moist} \cdot h_1$ $\quad \sigma_{v1} = 1450 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-14b).

Effective Vertical stress at bottom of Soil Layer 2

$\sigma_{v2} := \gamma'_2 \cdot h_2$ $\quad \sigma_{v2} = 272 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-15).

Total effective stress at top of pile

$\sigma_t := \sigma_{v1} + \sigma_{v2}$ $\quad \sigma_t = 1722 \cdot psf$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 213 Eq (7-14c).

Equivalent weight

$\gamma_{combined} := \dfrac{\sigma_t}{h_1 + h_2}$ $\quad \boxed{\gamma_{combined} = 114.8 \cdot pcf}$

H-pile Soil Model

Soil Layer No. 3 (Submerged, Glacial Till)

Internal Angle of Friction

Design N_{60} = 71 bpf $\quad \boxed{\phi_3 := 40deg}$

Kulawy and Mayne, Manual on Estimating Soil Properties p. 4-15: N vs. Phi

Dry Unit Weight

Dry, Dense Glacial Till = 134 pcf

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - dry unit weight

$\gamma_{3dry} := 134pcf$

Saturated Unit Weight

Natural water content at saturated state:
Glacial Till: 10%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{3sat} := .10$

$\gamma_{3saturated} := \gamma_{3dry} \cdot (1 + w_{3sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3saturated} = 147 \cdot pcf$

Soil Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_3 := \gamma_{3saturated} - \gamma_w$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 18 Eq (2-11). Multiply density by gravity to arrive at weights

$\boxed{\gamma'_3 = 85 \cdot pcf}$ $\quad 85pcf = 0.049 \cdot pci$

Representative constant for calculation of P-Y curve, initial horizontal soil modulus, k_h:
Dense sand below water table with static loading = 125 pci

Technical Manual
LPile 2016 p. 96

At-rest Earth Pressure - Rankine Theory

$K_o := 1 - \sin(\phi_3)$ $\boxed{K_o = 0.36}$

Fang, Foundation Engineering Handbook 2nd ed. p. 224, Eq. 6.2

Soil response behind the piles may be modeled by use of soil springs to represent the soil resistance. The soil springs should neglect any shear interaction between the pile springs because no transverse continuity of the pile between soil springs is likely to exist.

Replacement Soil layer for use when backfilling H-pile with Type C Underdrain Material

Replacement Soil Layer No. 3 (Submerged, Type C Underdrain Material) El. 496.6 - Top of Grout

Internal Angle of Friction $\boxed{\phi_{3r} := 36deg}$

Soil Total Unit Weight $\gamma_{3r\_moist} := 135pcf$

MaineDOT BDG Table 3-3
Soil Type 5

Specific Gravity of quartz sand

$\boxed{G_{s\_3r} := 2.65}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 35: Specific Gravity of Quartz

Optimum Moisture Content

$w_{3r\_opt} := .08$

Dry unit weight

$\gamma_{3r\_dry} := \frac{\gamma_{3r\_moist}}{1 + w_{3r\_opt}}$ $\gamma_{3r\_dry} = 125 \cdot pcf$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

Saturated Unit Weight

Natural water content at saturated state:
Loose uniform sand = 30%
Dense angular silty sand = 15%
Average Loose and Dense for Medium Dense:
Medium Dense angular silty sand: 23%

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2 - Natural Moisture Content in a saturated state

$w_{3r\_sat} := .23$

$\gamma_{3r\_saturated} := \gamma_{3r\_dry} \cdot (1 + w_{3r\_sat})$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.1 Unit Weight Relationships

$\gamma_{3r\_saturated} = 154 \cdot pcf$

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_{3r} := \gamma_{3r\_saturated} - \gamma_w$

Holtz and Kovacs, Intro to Geotechnical Eng. p. 18 Eq (2-11).

$\boxed{\gamma'_{3r} = 91 \cdot pcf}$

Void Ratio

$e_{3r} := \frac{G_{s\_3r} \cdot \gamma_w}{\gamma_{3r\_dry}} - 1$ $\boxed{e_{3r} = 0.32}$

Das, Principles of Geotechnical Eng. 7th Ed. p. 55, Eq. 3.17

Representative constant for calculation of P-Y curve, initial horizontal soil modulus, k_h:
Medium dense sand below water table with static loading = 60 pci

Technical Manual LPile 2016 p. 96

Soil Layer No. 4 (Submerged, Bedrock)

Dry Unit Weight

Phyllite = 170 pcf

$\gamma_{4moist} := 170pcf$

Lab test results from Phyllite of Vassalboro Formation, Sample BB-PR-201;R1. Test results provided in the Spun Pipe Pile Resistance calculations. Assume the voids are negligible and the bulk density (moist weight) is near the dry and saturated weight.

Effective Unit Weight

weight of water = γ_w =62.4 pcf

$\gamma_w := 62.4pcf$

$\gamma'_4 := \gamma_{4moist} - \gamma_w$

$\boxed{\gamma'_4 = 107.6 \cdot pcf}$ $108pcf = 0.063 \cdot pci$

Estimation of Rock Mass Modulus, E_m

Unconfined uniaxial compressive strength, q_u

$\boxed{q_u := 6800 \cdot psi = 46.884 \cdot MPa}$

Design compressive strength. Test results provided in the Spun Pipe Pile Resistance calculations.

Intact Rock Modulus

$E_i := 7370ksi$

Lab test results from Phyllite of Vassalboro Formation, Sample BB-PR-201;R1 at the design compressive strength.

Geologic Strength Index, GSI

$GSI := 33$

See the attached Geotechnical Axial Compressive Resistance for Spun Pipe Piles Calculations for GSI estimation

Rock Mass Modulus computed from GSI index-

$E_{m1} := \frac{E_i}{100} \cdot e^{\frac{GSI}{21.7}}$ $E_{m1} = 2.3 \cdot GPa$

2016 LRFD Table 10.4.6.5-1;
LPile Technical Manual Eq. 3-131

$E_{m1} = 337.2 \cdot ksi$

Joint Modification Factor, α_E

$\alpha_E := 0.45$

LRFD Table 10.8.3.5.4b-1 correlates a RQD of 20% to a joint modification factor (E_m/E_i ratio) of .05

Rock Mass Modulus adjusted for jointing method-

$E_{m2} := \alpha_E \cdot E_i$

$E_{m2} = 3316.5 \cdot ksi$

LPile User Manual p. 146 states the initial mass modulus for weak rock may be obtained from the product of the modulus reduction ratio and Young's modulus measured on intact rock specimens.

LRFD Article 10.4.6.5 recommends using the result of the lesser of the intact modulus of a sample of rock core and the modulus estimated based on GSI.

$\boxed{E_m := E_{m1} = 337.2 \cdot ksi}$

Rock Quality Designation, RQD

$\boxed{RQD := 23\%}$

Rock Quality and Core Descriptions are found on the boring logs. 23% selected for design.

Initial slope of p-y curve, K_{rm}

Strain at 50% of the maximum uniaxial strength of the sample is 0.0004. NCHRP 360 p. 58 cites the 2004 LPile manual which states that an upper bound of 0.0005 should be considered because of the limited experimental data comparing load tests to p-y curves.

$\boxed{K_{rm} := .0005}$

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29° *	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

Table 2.4 Specific Gravity of Common Minerals

Mineral	Specific gravity, G_s
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65–2.80
Halloysite	2.0–2.55
Potassium feldspar	2.57
Sodium and calcium feldspar	2.62–2.76
Chlorite	2.6–2.9
Biotite	2.8–3.2
Muscovite	2.76–3.1
Hornblende	3.0–3.47
Limonite	3.6–4.0
Olivine	3.27–3.7

2.5 Mechanical Analysis of Soil

Mechanical analysis is the determination of the size range of particles present in a soil, expressed as a percentage of the total dry weight. Two methods generally are used to find the particle-size distribution of soil: (1) *sieve analysis*—for particle sizes larger than 0.075 mm in diameter, and (2) *hydrometer analysis*—for particle sizes smaller than 0.075 mm in diameter. The basic principles of sieve analysis and hydrometer analysis are described briefly in the following two sections.

Sieve Analysis

Sieve analysis consists of shaking the soil sample through a set of sieves that have progressively smaller openings. U.S. standard sieve numbers and the sizes of openings are given in Table 2.5.

Table 2.5 U.S. Standard Sieve Sizes

Sieve no.	Opening (mm)	Sieve no.	Opening (mm)
4	4.75	35	0.500
5	4.00	40	0.425
6	3.35	50	0.355
7	2.80	60	0.250
8	2.36	70	0.212
10	2.00	80	0.180
12	1.70	100	0.150
14	1.40	120	0.125
16	1.18	140	0.106
18	1.00	170	0.090
20	0.850	200	0.075
25	0.710	270	0.053
30	0.600		

Das, Principles of Geotechnical Engineering 7th ed.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ, γ_d, and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft^3	kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Now, using the definitions of unit weight and dry unit weight [Eqs. (3.9) and (3.11)], we can write

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e} \tag{3.15}$$

and

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w}{1+e} \tag{3.16}$$

or

$$e = \frac{G_s\gamma_w}{\gamma_d} - 1 \tag{3.17}$$

Because the weight of water for the soil element under consideration is $wG_s\gamma_w$, the volume occupied by water is

$$V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w}{\gamma_w} = wG_s$$

Hence, from the definition of degree of saturation [Eq. (3.5)],

$$S = \frac{V_w}{V_v} = \frac{wG_s}{e}$$

or

$$Se = wG_s \tag{3.18}$$

This equation is useful for solving problems involving three-phase relationships.

If the soil sample is *saturated*—that is, the void spaces are completely filled with water (Figure 3.3)—the relationship for saturated unit weight (γ_{sat}) can be derived in a similar manner:

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + e\gamma_w}{1+e} = \frac{(G_s+e)\gamma_w}{1+e} \tag{3.19}$$

Also, from Eq. (3.18) with $S = 1$,

$$e = wG_s \tag{3.20}$$

As mentioned before, due to the convenience of working with densities in the SI system, the following equations, similar to unit–weight relationships given in Eqs. (3.15), (3.16), and (3.19), will be useful:

Table 3-6 Representative Values of *k* for Fine Sand Below the Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	5.4 (20.0)	16.3 (60.0)	34 (125.0)

Table 3-7 Representative Values of *k* for Fine Sand Above Water Table for Static and Cyclic Loading

Recommended *k*	Relative Density		
	Loose	Medium	Dense
MN/m^3 (pci)	6.8 (25.0)	24.4 (90.0)	61.0 (225.0)

If the sand profile is coarse or well-graded sand, the user may consider using a higher value of *k* that those suggested in the tables above. While experimental data for *k* in well-graded sands is poorly documented, use of values 10 to 50 percent higher may be appropriate in dense and very dense well-graded sands that do not contain any compressible minerals such as mica.

7. Fit the parabola between point *k* and point *m* as follows:

a. Compute the slope of the *p-y* curve between point *m* and point *u* using

$$m = \frac{p_u - p_m}{y_u - y_m} \quad \text{(3-62)}$$

b. Compute the power of the parabolic section using

$$n = \frac{p_m}{m y_m} \quad \text{(3-63)}$$

c. Compute the coefficient $\bar{C}$ using

$$\bar{C} = \frac{p_m}{y_m^{1/n}} \quad \text{(3-64)}$$

8. Compute the *y* value defining point *k* using

$$y_k = \left(\frac{\bar{C}}{kx}\right)^{\frac{n}{n-1}} \quad \text{(3-65)}$$

Compute the *p* value defining point *k* using

Fang, Foundation Engineering Handbook 2nd ed. p. 224

6.1 AT-REST LATERAL PRESSURES

At-rest pressures exist in level ground, and develop under long-term conditions as the soil is deposited and acted upon by changes in the loading environment as caused by erosion, glaciers, and physicochemical processes. At-rest pressures rigorously only apply for walls that are placed into the ground with a minimum of disturbance and that remain unmoved during loading, or for unmoving, frictionless walls with a backfill placed with a minimum of compactive effort. In practice such conditions are rarely achieved. However, at-rest pressures are still useful in design as either a baseline against which other pressure states can be judged or as an assumed conservative choice for the design loading.

At-rest effective lateral pressures are often assumed to follow a linear distribution (Fig. 6.2), with the effective lateral pressure σ'_x taken as a simple multiple of the vertical effective pressure σ'_z:

$$\sigma'_x = K_0(\sigma'_z) \qquad (6.1)$$

In homogeneous, dry soil with a constant K_0 and unit weight, both the vertical and lateral pressures are linearly distributed. With the presence of a water table, the at-rest pressure distribution exhibits a break in slope at the water table, reflecting the use of submerged unit weights to determine vertical effective stresses (Fig. 6.2).

Our early concepts of the parameter K_0 were formed on the basis of normally consolidated soils. Jaky (1944) proposed a relationship between K_0 and the drained friction angle ϕ' for normally consolidated soils:

$$\boxed{K_0 = 1 - \sin\phi'} \qquad (6.2)$$

Numerous studies have confirmed the general validity of this empirical equation (Brooker and Ireland, 1965; Mayne and Kulhawy, 1982). However, results from laboratory experiments and in-situ tests have shown that the K_0 value also varies as a function of overconsolidation ratio (OCR) and stress history. For the case of a soil that has been subjected to one or more cycles of unloading, Schmidt (1966) proposed that K_0 can be determined as a function of its value in the normally consolidated state using the relationship

$$K_{0u} = K_{0nc}(\mathrm{OCR})^{\alpha} \qquad (6.3)$$

in which K_{0u} is the coefficient for unloading, K_{0nc} is the coefficient for the normally consolidated soil, and α is a dimensionless coefficient. Experimental data have confirmed this relationship, and Mayne and Kulhawy (1982) showed that, for most soils, α can be taken as $\sin\phi'$.

Soils that are overconsolidated and are in the process of being reloaded pose a difficulty in that Equation 6.3 does not apply. For this condition, a more complex equation is needed as well as a full knowledge of the stress history of the soil (Mayne and Kulhawy, 1982). For practical purposes, it may be enough to know that the K_0 during reloading falls about halfway between that for unloading and normally consolidated conditions. Also, K_0 might be directly determined through in-situ testing methods.

TABLE 6.1 TYPICAL COEFFICIENTS OF LATERAL EARTH PRESSURE AT REST.

	Coefficient of Lateral Earth Pressure			
Soil type	*OCR = 1*	*OCR = 2*[a]	*OCR = 5*[a]	*OCR = 10*[a]
Loose sand	0.45	0.65	1.10	1.50
Medium sand	0.40	0.60	1.05	1.55
Dense sand	0.35	0.55	1.00	1.50
Silt	0.50	0.70	1.10	1.60
Lean clay, CL	0.60	0.80	1.20	1.65
Highly plastic clay, CH	0.65	0.80	1.10	1.40

[a] Unloading cycle.

Table 6.1 presents typical values for K_0 for a subset of soils. For other conditions, K_0 values can be determined directly from Equations 6.2 and 6.3, and/or using in-situ testing techniques.

Because the K_0 value in a given soil often varies with depth, and the soil types themselves may change with depth, the at-rest lateral pressure distribution is typically not linear as shown in Figure 6.2. Self-boring pressuremeter tests in clays with overconsolidated profiles induced by desiccation have demonstrated that the K_0 under such conditions decreases with depth in the soil deposit and reaches a steady state where the desiccation effects are no longer present (Clough and Denby, 1980).

6.2 ACTIVE AND PASSIVE LATERAL EARTH PRESSURES

Most walls move, either by global shifting or by local deformations. These movements cause adjustments to occur in the earth loads and the pressure distributions. Conventional means for assessing the effects of system movements are to set them into the context of extreme conditions. These are referred to as the active and passive earth pressure loadings.

6.2.1 Active Pressure

Assuming that a gravity wall with no friction on its face is translated away from a soil mass that is initially at the at-rest condition, then the soil mass adjacent to the wall will pass into a failure state as shown in Figure 6.3. At this stage, the

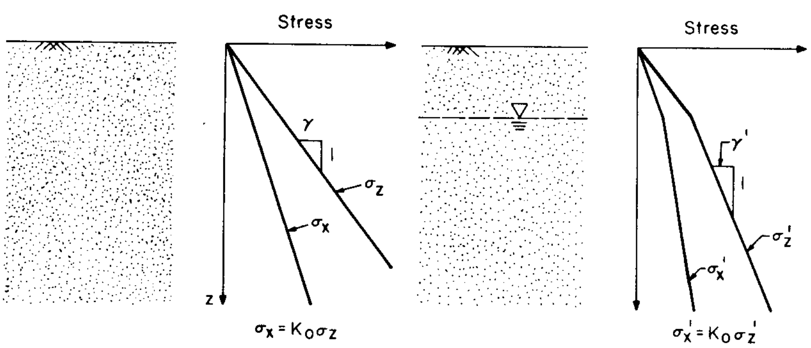


Fig. 6.2 At-rest earth pressure distribution—homogeneous soil.

Bowles Foundation Analysis and Design, 5th ed.

2-14 ELASTIC PROPERTIES OF SOIL

Hooke's generalized stress-strain law is commonly used in solving geotechnical problems of stress and settlement. In equation form Hooke's stress-strain law for any homogeneous, isotropic, elastic material is

$$\begin{aligned} \epsilon_x &= \frac{1}{E_s}(\sigma_x - \mu\sigma_y - \mu\sigma_z) \\ \epsilon_y &= \frac{1}{E_s}(\sigma_y - \mu\sigma_x - \mu\sigma_z) \\ \epsilon_z &= \frac{1}{E_s}(\sigma_z - \mu\sigma_x - \mu\sigma_y) \end{aligned} \qquad (2\text{-}64)$$

The signs here are based on using (+) μ of Eq. (*b*) following.

In matrix notation Eq. (2-64) can be written as

$$\boldsymbol{\epsilon} = \mathbf{D}\boldsymbol{\sigma} \qquad (2\text{-}64a)$$

where the matrix **D** is the following

$\mathbf{D} =$

$\epsilon \backslash \sigma$	1	2	3
1	1	$-\mu$	$-\mu$
2	$-\mu$	1	$-\mu$
3	$-\mu$	$-\mu$	1

The *shear modulus* G' (which may be subscripted) is defined as the ratio of shear stress s_s to shear strain ϵ_s.[25] It is related to E_s and μ as

$$\boxed{G'_s = \frac{s_s}{\epsilon_s} = \frac{E_s}{2(1+\mu)}} \qquad (a)$$

Poisson's ratio μ is used in both pressure and settlement studies and is defined as the ratio of axial compression ϵ_v to lateral expansion ϵ_L strains, or

$$\mu = \frac{\epsilon_L}{\epsilon_v} \qquad (b)$$

The μ ratio has a (+) sign in this equation if ϵ_v is compressive strain and the lateral strain ϵ_L causes the lateral dimension to increase. In a tension test the sign is (+) if the sample ϵ_v produces elongation while the lateral dimension(s) decrease. The shearing strain ϵ_s is defined as the change in right angle of any corner of an element in compression as illustrated in

[25] The shear modulus in structural mechanics literature often uses the symbol G_i, where i may be s = steel; c = concrete; etc. Since G_s is used in geotechnical work for the specific gravity of the soil solids, the closest symbol to the "literature" is G'.

Bowles Foundation Analysis and Design, 5th ed.

TABLE 2-7
Values or value ranges for Poisson's ratio μ

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand, gravelly sand	−0.1–1.00
commonly used	0.3–0.4
Rock	0.1–0.4 (depends somewhat on type of rock)
Loess	0.1–0.3
Ice	0.36
Concrete	0.15
Steel	0.33

Another material property concept is the *bulk modulus* E_b, which is defined as the ratio of hydrostatic stress to the volumetric strain ϵ_v and is given as

$$E_b = \frac{2}{3}G'\frac{1+\mu}{1-2\mu} = \frac{E_s}{3(1-2\mu)} \tag{f}$$

For an *elastic* material the shear modulus G' cannot be ($-$), so Eq. (a) sets the lower limit of $\mu > -1$. Equation (f) sets the upper limit at $\mu < 0.5$. It appears that the range of μ for soils (that are not "elastic") is from about -0.1 to 1.00. Table 2-7 gives a range of values for select materials. It is very common to use the following values for soils:

μ	Soil type
0.4–0.5	Most clay soils
0.45–0.50	Saturated clay soils
0.3–0.4	Cohesionless—medium and dense
0.2–0.35	Cohesionless—loose to medium

Although it is common to use $\mu = 0.5$ for saturated clay soils, the reader should be aware that this represents a condition of no volume change under the applied stress σ_z. Over time, however, volume change does occur as the pore fluid drains. Equation (e) defines the Poisson's ratio that develops initially ($\epsilon_v = 0$) and also later when $\epsilon_v > 0$. Since the strain is produced from stress and Fig. 1-1 indicates a vertical variation, it necessarily follows that μ is stress-dependent from Eq. (e).

A special case in geotechnical work is that of *plane strain*. This arises where strains occur parallel to two of the coordinate axes (say the x and z) but the strain is zero perpendicular to the x-z plane (along the y axis). If we set $\epsilon_y = 0$ in the set of equations for Hooke's law [(Eqs. (2-64)] and solve for the resulting values of E_s and μ, we obtain the following:

$$E_s' = \frac{E_s}{1-\mu^2} \qquad \mu' = \frac{\mu}{1-\mu} \tag{2-65}$$

Bowles Foundation Analysis and Design, 5th ed.

TABLE 2-8
Value range* for the static stress-strain modulus E_s for selected soils (see also Table 5-6)

Field values depend on stress history, water content, density, and age of deposit

Soil	E_s, MPa	E_s, PSI
Clay		
Very soft	2–15	290.1 – 2175.6
Soft	5–25	725.2 – 3626.0
Medium	15–50	2175.6 – 7251.9
Hard	50–100	7251.9 – 14503.8
Sandy	25–250	3625.9 – 36259.4
Glacial till		
Loose	10–150	1450.4 – 21755.7
Dense	150–720	21755.7 – 104427
Very dense	500–1440	72518.9 – 208854.3
Loess	15–60	2175.6 – 8702.3
Sand		
Silty	5–20	725.2 – 2900.8
Loose	10–25	1450.4 – 3625.9
Dense	50–81	7251.9 – 11748.1
Sand and gravel		
Loose	50–150	7251.9 – 21755.7
Dense	100–200	14503.8 – 29007.5
Shale	150–5000	21755.7 – 725188.7
Silt	2–20	290.1 – 2900.8

*Value range is too large to use an "average" value for design.

in situ, it is reasonable for confined compression tests to produce better "elastic" parameters. Although it is difficult to compare laboratory and field E_s values, there is some evidence that field values are often four to five times larger than laboratory values from the unconfined compression test. For this reason, current practice tends to try to obtain "field" values from in situ testing whenever possible. This topic will be taken up in more detail in the next chapter.

Table 2-8 gives a range of E_s values that might be obtained. Note that the range is very large, owing to the foregoing factors as well as those factors given on the table. With this wide range of values the reader should not try to use "averaged" values from this table for design.

If laboratory test plots similar to Fig. 2-43*a* are used, it is most common to use the initial tangent modulus to compute the stress-strain modulus E_s for the following reasons:

1. Soil is elastic only near the origin.
2. There is less divergence between all plots in this region.
3. The largest values are obtained—often three to five times larger than a tangent or secant modulus from another point along the curve.

7.5 INTERGRANULAR OR EFFECTIVE STRESS

The concept of intergranular or *effective stress* was introduced in Sec. 6.2. By definition,

$$\sigma = \sigma' + u \tag{7-13}$$

where σ = total normal stress,
σ' = intergranular or effective normal stress, and
u = pore water or neutral pressure.

Both the total stress and pore water pressure may readily be estimated or calculated with knowledge of the densities and thicknesses of the soil layers and location of the ground water table. The effective stress cannot be measured; it can only be calculated!

The total vertical stress is called the *body stress* because it is generated by the mass (acted upon by gravity) in the body. To calculate the total vertical stress σ_v at a point in a soil mass, you simply sum up the densities of all the material (soil solids + water) above that point multiplied by the gravitational constant g, or

$$\sigma_v = \int_0^h \rho g dz \tag{7-14a}$$

If ρg is a constant throughout the depth, then

$$\boxed{\sigma_v = \rho g h \qquad (7\text{-}14b)}$$

Typically, we divide the soil mass into n layers and evaluate the total stress incrementally for each layer or

$$\boxed{\sigma_v = \sum_{i=1}^{n} \rho_i g z_i \qquad (7\text{-}14c)}$$

As an example, if a soil could have zero voids, then the total stress exerted on a particular plane would be the depth to the given point times the density of the material or, in this case, ρ_s times the gravitational constant g. If the soil were dry, then you would use ρ_d instead of ρ_s.

The neutral stress or pore water pressure is similarly calculated for static water conditions. It is simply the depth below the ground water table to the point in question, z_w, times the product of the density of water ρ_w and g, or

$$u = \rho_w g z_w \tag{7-15}$$

It is called the *neutral stress* because it has no shear component. Recall from fluid mechanics that by definition a liquid cannot support static shear stress. It has only normal stresses which act equally in all

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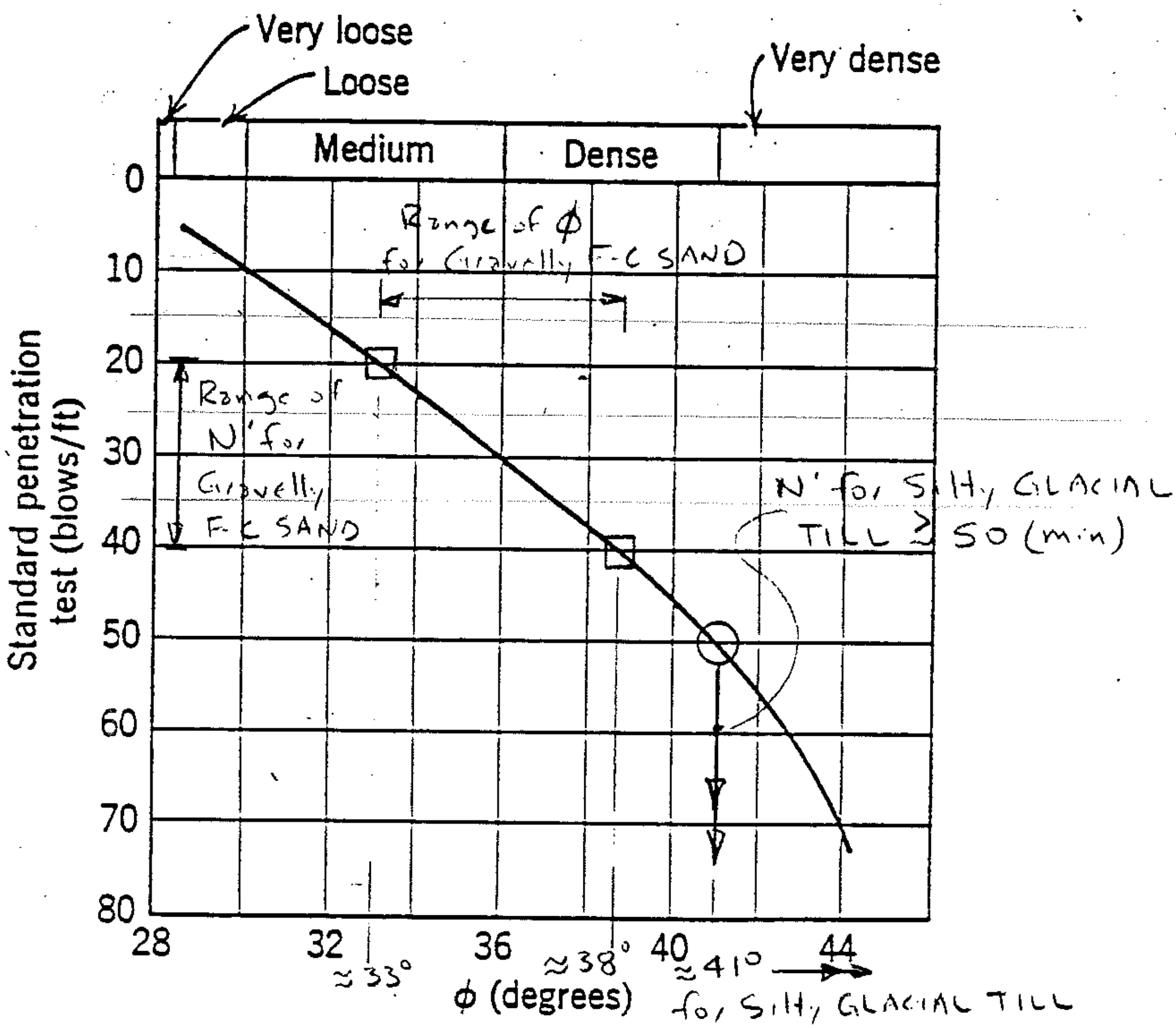

Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

Summary of Above

○ Silty GLACIAL TILL $\phi \gtrsim 41°$ for $N' > 50$

□ Gravelly F-C SAND $\phi \approx 33° \rightarrow 38°$ for $N' \approx 20 \rightarrow 40$

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If the uniaxial compressive strength of rock forming the sidewall of the socket exceeds the drilled shaft concrete compressive strength, the value of concrete compressive strength (f'_c) shall be substituted for q_u in Eq. 10.8.3.5.4b-1.

For fractured rock that caves and cannot be drilled without some type of artificial support, the unit side resistance shall be taken as:

$$\frac{q_s}{p_a} = 0.65\alpha_E\sqrt{\frac{q_u}{p_a}} \qquad (10.8.3.5.4b\text{-}2)$$

The joint modification factor, α_E is given in Table 10.8.3.5.4b-1 based on RQD and visual inspection of joint surfaces.

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

	Joint Modification Factor, α_E	
RDQ (%)	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

Eq. 10.8.3.5.4b-1 with $C = 1.0$. For additional guidance on assessing the magnitude of C, see Brown et al. (2010).

Shafts are sometimes constructed by supporting the hole with temporary casing or by grouting the rock ahead of the excavation. When using these construction methods, disturbance of the sidewall results in lower unit side resistances. Based on O'Neill and Reese (1999) and as discussed in Brown et al. (2010), the reduction in side resistance can be related empirically to the RQD and joint conditions.

10.8.3.5.4c—Tip Resistance

End-bearing for drilled shafts in rock may be taken as follows:

- If the rock below the base of the drilled shaft to a depth of 2.0B is either intact or tightly jointed, i.e., no compressible material or gouge-filled seams, and the depth of the socket is greater than 1.5B:

$$q_p = 2.5q_u \qquad (10.8.3.5.4c\text{-}1)$$

- If the rock below the base of the shaft to a depth of 2.0B is jointed, the joints have random orientation, and the condition of the joints can be evaluated as:

$$q_p = A + q_u\left[m_b\left(\frac{A}{q_u}\right) + s\right]^a \qquad (10.8.3.5.4c\text{-}2)$$

In which:

$$A = \sigma'_{vb} + q_u\left[m_b\frac{(\sigma'_{v,b})}{q_u} + s\right]^a \qquad (10.8.3.5.4c\text{-}3)$$

where:

C10.8.3.5.4c

If end bearing in the rock is to be relied upon, and wet construction methods are used, bottom clean-out procedures such as airlifts should be specified to ensure removal of loose material before concrete placement.

The use of Eq. 10.8.3.5.4c-1 also requires that there are no solution cavities or voids below the base of the drilled shaft.

For further information see Brown et al. (2010).

Bearing capacity theory provides a framework for evaluation of base resistance for cases where the bearing rock can be characterized by its *GSI*. Eq. 10.8.3.5.4c-2 (Turner and Ramey, 2010) is a lower bound solution for bearing resistance of a drilled shaft bearing on or socketed into a fractured rock mass. Fractured rock describes a rock mass intersected by multiple sets of intersecting joints such that the strength is controlled by the overall mass response and not by failure along pre-existing structural discontinuities. This generally applies to rock that can be characterized by the descriptive terms shown in Figure 10.4.6.4-1 (e.g., blocky, disintegrated, etc.).

10.4.6.5—Rock Mass Deformation

Revise Table 10.4.6.5-1 as follows:

Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m\left(GPa\right) = \sqrt{\frac{q_u}{100}}\,10^{\frac{GSI-10}{40}}$ for $q_u \le 100$ MPa ≤ 2,090 ksf ≤ 14500 psf $\cancel{E_m\left(GPa\right) = 10^{\frac{GSI-10}{40}}}$ ~~for~~ $\cancel{q_u \le 100}$ ~~MP~~a $E_m\left(GPa\right) = 10^{\frac{GSI-10}{40}}$ for $q_u > 100$ MPa	Accounts for rocks with $q_u < 100$ MPa; notes q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m = \frac{E_R}{100} e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)
Notes: E_r = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength, and 1 MPa = ~~2.09~~ 20.9 ksf		

Earth Pressure

Earth Pressure:

Backfill engineering strength parameters

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi' := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Integral Abutment and Wingwall - Passive Earth Pressure - Coulomb Theory

α = Angle of fill slope to the horizontal $\alpha := 0 \cdot deg$

$\phi_1 =$ Angle of internal friction $\phi' = 32 \cdot deg$

$\beta =$ Angle of back face of wall to the horizontal $\beta := 90 \cdot deg$

Use Coulomb for cases where interface friction is considered; typically gravity shaped structures, and integral abutments where the ratio of wall height to wall movement is .005 or greater. Coulomb should also be used when the fill slope is greater than horizontal.

For precast IAB abutment against clean sand, silty sand-gravel mixture use δ = 17 - 22, per LRFD Table 3.11.5.3-1

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$\delta' := 19.5 \cdot deg$

$$K_{p\_coulomb} := \frac{\sin(\beta - \phi')^2}{\sin(\beta)^2 \cdot \sin(\beta + \delta') \cdot \left(1 - \sqrt{\frac{\sin(\phi' + \delta') \cdot \sin(\phi' + \alpha)}{\sin(\beta + \delta') \cdot \sin(\beta + \alpha)}}\right)^2}$$

Das, Principles of Foundation Engineering 7th Ed. p. 366 Eq. 7.71

$K_{p\_coulomb} = 6.73$

Integral Abutment and Wingwall - Passive Earth Pressure - Rankine Theory

Use Rankine only if the ratio of wall height to wall movement is significantly less than .005 and the fill slope is horizontal to the top of the wall. Bowles does not recommend use of Rankine method for K_p when α > 0.

α = Angle of fill slope to the horizontal $\alpha := 0 \cdot deg$

$$K_{p\_rank} := \frac{\cos(\alpha) + \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}{\cos(\alpha) - \sqrt{\cos(\alpha)^2 - \cos(\phi')^2}}$$

Das, Principles of Foundation Engineering 7th Ed. p. 363 Eq. 7.67

$K_{p\_rank} = 3.25$

P_p is oriented at an angle of α to the vertical plane

At-Rest Earth Pressure - Rankine Theory

Abutment 1 and 2, soil layers 1 and 2.

$K_o := 1 - \sin(\phi')$

$\boxed{K_o = 0.47}$

Das, Principles of Geotechnical Engineering 7th Ed. p 427 Eq. 13.5

Wingwalls Active Earth Pressure Coefficient

If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb.

$$K_{ar} := \tan\left(45 \cdot deg - \frac{\phi'}{2}\right)^2$$

Das, Principles of Geotechnical Engineering 7th Ed. p 434 Eq. 13.19

$\boxed{K_{ar} = 0.31}$

Wingwalls Active Earth Pressure Coefficient - Rankine Theory

Recalculate for sloping backfill

β = Angle of fill slope to the horizontal $\beta := 30 \cdot deg$

$$K_{ar\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi')^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi')^2}}$$

Bowles, Foundation Analysis and Design 5th ed. p. 601 Eq. 11-7

$K_{ar\_slope} = 0.66$

Pa is oriented at an angle of β to the vertical plane

Wingwalls Active Earth Pressure Coefficient - Coulomb Theory

Angle of back face of wall to the horizontal, θ : $\theta := 90 \cdot deg$

Angle of fill slope to the horizontal $\beta := 0 \cdot deg$

Friction angle between fill and wall:

δ = 17 to 22 LRFD Table 3.11.5.3-1 for Clean sand against concrete

Use $\delta := 19.5$

Check angle is between recommended $\frac{\phi}{3}$ and $\frac{2\phi}{3}$ for Coulomb:

$\frac{\phi'}{3} = 10.7 \cdot deg$ $\frac{2\phi'}{3} = 21.3 \cdot deg$ OK

$$K_{ac} := \frac{\sin(\theta + \phi')^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi' + \delta) \cdot \sin(\phi' - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2}$$

Bowles, Foundation Analysis and Design 5th ed. p. 595 Eq. 11-3

$\boxed{K_{ac} = 0.28}$

Orientation of Coulomb Pa

- In the case of gravity shaped walls, Pa is oriented δ degrees up from a perpendicular line to the backface or 'pressure surface.'

Table 7.10 Values of K_p [from Eq. (7.71)] for $\beta = 90°$ and $\alpha = 0°$

	δ' (deg)				
ϕ' (deg)	0	5	10	15	20
15	1.698	1.900	2.130	2.405	2.735
20	2.040	2.313	2.636	3.030	3.525
25	2.464	2.830	3.286	3.855	4.597
30	3.000	3.506	4.143	4.977	6.105
35	3.690	4.390	5.310	6.854	8.324
40	4.600	5.590	6.946	8.870	11.772

Figure 7.25b shows the force triangle at equilibrium for the trial wedge ABC_1. From this force triangle, the value of P_p can be determined, because the direction of all three forces and the magnitude of one force are known.

Similar force triangles for several trial wedges, such as ABC_1, ABC_2, ABC_3, . . . , can be constructed, and the corresponding values of P_p can be determined. The top part of Figure 7.25a shows the nature of variation of the P_p values for different wedges. The *minimum value of* P_p in this diagram is *Coulomb's passive force,* mathematically expressed as

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.70}$$

where

$$K_p = \text{Coulomb's passive pressure coefficient}$$
$$= \frac{\sin^2(\beta - \phi')}{\sin^2\beta \sin(\beta + \delta')\left[1 - \sqrt{\dfrac{\sin(\phi' + \delta')\sin(\phi' + \alpha)}{\sin(\beta + \delta')\sin(\beta + \alpha)}}\right]^2} \tag{7.71}$$

The values of the passive pressure coefficient, K_p, for various values of ϕ' and δ' are given in Table 7.10 $(\beta = 90°, \alpha = 0°)$.

Note that the resultant passive force, P_p, will act at a distance $H/3$ from the bottom of the wall and will be inclined at an angle δ' to the normal drawn to the back face of the wall.

7.13 Comments on the Failure Surface Assumption for Coulomb's Pressure Calculations

Coulomb's pressure calculation methods for active and passive pressure have been discussed in Sections 7.5 and 7.12. The fundamental assumption in these analyses is the acceptance of *plane failure surface*. However, for walls with friction, this assumption does not hold in practice. The nature of *actual* failure surface in the soil mass for active and passive pressure is shown in Figure 7.26a and b, respectively (for a vertical wall with a horizontal backfill). Note that the failure surface BC is curved and that the failure surface CD is a plane.

Although the actual failure surface in soil for the case of active pressure is somewhat different from that assumed in the calculation of the Coulomb pressure, the results are not greatly different. However, in the case of passive pressure, as the value of δ' increases, Coulomb's

Table 7.9 (*Continued*)

		$c'/\gamma z$			
ϕ' (deg)	α (deg)	0.025	0.050	0.100	0.500
30	0	3.087	3.173	3.346	4.732
	5	3.042	3.129	3.303	4.674
	10	2.907	2.996	3.174	4.579
	15	2.684	2.777	2.961	4.394

7.12 Coulomb's Passive Earth Pressure

Coulomb (1776) also presented an analysis for determining the passive earth pressure (i.e., when the wall moves *into* the soil mass) for walls possessing friction (δ' = angle of wall friction) and retaining a granular backfill material similar to that discussed in Section 7.5.

To understand the determination of Coulomb's passive force, P_p, consider the wall shown in Figure 7.25a. As in the case of active pressure, Coulomb assumed that the potential failure surface in soil is a plane. For a trial failure wedge of soil, such as ABC_1, the forces per unit length of the wall acting on the wedge are

1. The weight of the wedge, W
2. The resultant, R, of the normal and shear forces on the plane BC_1, and
3. The passive force, P_p

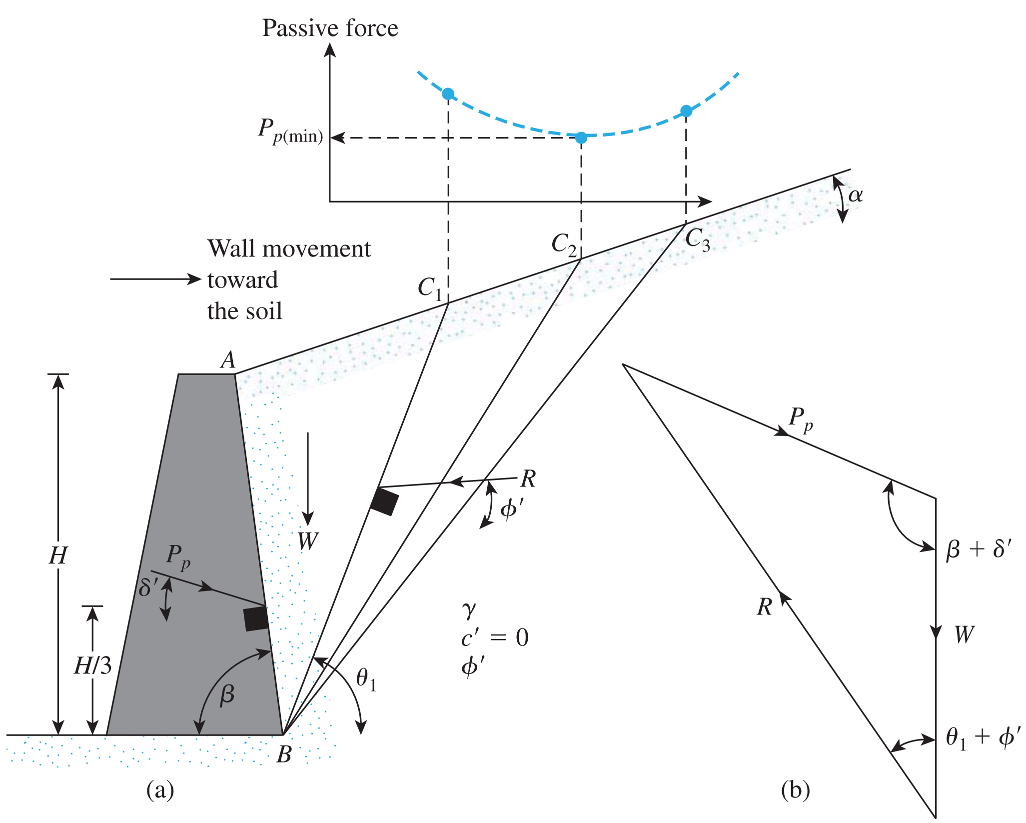


Figure 7.25 Coulomb's passive pressure

At this depth, that is $z = 2$ m, for the bottom soil layer

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2\sqrt{K_{p(2)}} = 31.44(2.56) + 2(10)\sqrt{2.56}$$
$$= 80.49 + 32 = 112.49 \text{ kN/m}^2$$

Again, at $z = 3$ m,

$$\sigma'_o = (15.72)(2) + (\gamma_{sat} - \gamma_w)(1)$$
$$= 31.44 + (18.86 - 9.81)(1) = 40.49 \text{ kN/m}^2$$

Hence,

$$\sigma'_p = \sigma'_o K_{p(2)} + 2c'_2\sqrt{K_{p(2)}} = 40.49(2.56) + (2)(10)(1.6)$$
$$= \mathbf{135.65\ kN/m^2}$$

Note that, because a water table is present, the hydrostatic stress, u, also has to be taken into consideration. For $z = 0$ to 2 m, $u = 0$; $z = 3$ m, $u = (1)(\gamma_w) = 9.81$ kN/m^2.

The passive pressure diagram is plotted in Figure 6.24b. The passive force per unit length of the wall can be determined from the area of the pressure diagram as follows:

Area no.	Area	
1	$(\frac{1}{2})$ (2)(94.32)	= 94.32
2	(112.49)(1)	= 112.49
3	$(\frac{1}{2})$ (1)(135.65 − 112.49)	= 11.58
4	$(\frac{1}{2})$ (9.81)(1)	= 4.905
		$P_P \approx$ 223.3 **kN/m**

7.11 Rankine Passive Earth Pressure: Vertical Backface and Inclined Backfill

Granular Soil

For a frictionless vertical retaining wall (Figure 7.10) with a *granular backfill* ($c' = 0$), the Rankine passive pressure at any depth can be determined in a manner similar to that done in the case of active pressure in Section 7.4. The pressure is

$$\sigma'_p = \gamma z K_p \tag{7.65}$$

and the passive force is

$$P_p = \tfrac{1}{2}\gamma H^2 K_p \tag{7.66}$$

where

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}} \tag{7.67}$$

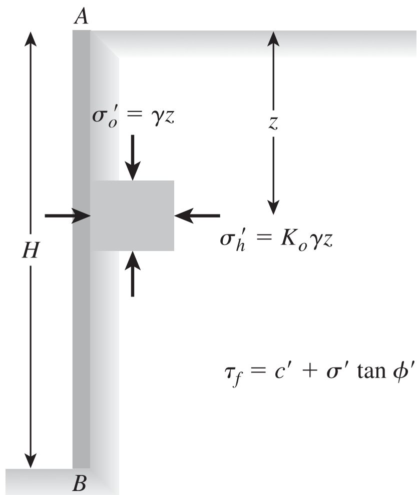


Figure 13.3
Earth pressure at rest

which shows a wall AB retaining a dry soil with a unit weight of γ. The wall is static. At a depth z,

$$\text{Vertical effective stress} = \sigma'_o = \gamma z$$
$$\text{Horizontal effective stress} = \sigma'_h = K_o \gamma z$$

So,

$$K_o = \frac{\sigma'_h}{\sigma'_o} = \text{at-rest earth pressure coefficient}$$

For coarse-grained soils, the coefficient of earth pressure at rest can be estimated by using the empirical relationship (Jaky, 1944)

$$\boxed{K_o = 1 - \sin \phi'} \tag{13.5}$$

where ϕ' = drained friction angle.

While designing a wall that may be subjected to lateral earth pressure at rest, one must take care in evaluating the value of K_o. Sherif, Fang, and Sherif (1984), on the basis of their laboratory tests, showed that Jaky's equation for K_o [Eq. (13.5)] gives good results when the backfill is loose sand. However, for a dense, compacted sand backfill, Eq. (13.5) may grossly underestimate the lateral earth pressure at rest. This underestimation results because of the process of compaction of backfill. For this reason, they recommended the design relationship

$$K_o = (1 - \sin \phi) + \left[\frac{\gamma_d}{\gamma_{d(\min)}} - 1\right]5.5 \tag{13.6}$$

where γ_d = actual compacted dry unit weight of the sand behind the wall
$\gamma_{d(\min)}$ = dry unit weight of the sand in the loosest state (Chapter 3)

Substituting the preceding values into Eq. (13.16), we get

$$\sigma'_a = \gamma z \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' \tan\left(45 - \frac{\phi'}{2}\right) \tag{13.17}$$

The variation of σ'_a with depth is shown in Figure 13.7c. For cohesionless soils, $c' = 0$ and

$$\sigma'_a = \sigma'_o \tan^2\left(45 - \frac{\phi'}{2}\right) \tag{13.18}$$

The ratio of σ'_a to σ'_o is called the *coefficient of Rankine's active earth pressure* and is given by

$$K_a = \frac{\sigma'_a}{\sigma'_o} = \tan^2\left(45 - \frac{\phi'}{2}\right) \tag{13.19}$$

Again, from Figure 13.7b we can see that the failure planes in the soil make $\pm(45 + \phi'/2)$-degree angles with the direction of the major principal plane—that is, the horizontal. These are called potential *slip planes* and are shown in Figure 13.7d.

It is important to realize that a similar equation for σ_a could be derived based on the total stress shear strength parameters—that is, $\tau_f = c + \sigma \tan \phi$. For this case,

$$\sigma_a = \gamma z \tan^2\left(45 - \frac{\phi}{2}\right) - 2c \tan\left(45 - \frac{\phi}{2}\right) \tag{13.20}$$

13.5 Theory of Rankine's Passive Pressure

Rankine's passive state can be explained with the aid of Figure 13.8. *AB* is a frictionless wall that extends to an infinite depth (Figure 13.8a). The initial stress condition on a soil element is represented by the Mohr's circle *a* in Figure 13.8b. If the wall gradually is *pushed into the soil mass,* the effective principal stress σ'_h will increase. Ultimately, the wall will reach a situation where the stress condition for the soil element can be expressed by the Mohr's circle *b*. At this time, failure of the soil will occur. This situation is referred to as *Rankine's passive state.* The lateral earth pressure σ'_p, which is the major principal stress, is called *Rankine's passive earth pressure.* From Figure 13.8b, it can be shown that

$$\begin{aligned}\sigma'_p &= \sigma'_o \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)\\ &= \gamma z \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)\end{aligned} \tag{13.21}$$

The water also contributes lateral pressure and has $K_a = K_p = 1$ since $\phi_w = 0°$. Thus,

$$p_w = \gamma_w z_w = 9.807(3.5) = \mathbf{34.3}\text{ kPa}$$

These pressure values are plotted on Fig. E11-2*b* so the several pressure areas can be numerically integrated to obtain the total wall force. By using triangles and rectangles as shown, the total wall force is the sum from the several areas and the forces act through the centroids of the areas as shown so that we can easily sum moments about the base to obtain

$$R\bar{y} = \sum P_i y_i$$

$$P_1 = 30.7(3.5) = 107.5\text{ kN} \qquad y_i = 3.5 + \frac{3.5}{2} = 5.25\text{ m}$$

$$P_2 = 17.7\left(\frac{3.5}{2}\right) = 31.0\text{ kN} \qquad y_2 = 3.5 + \frac{3.5}{3} = 4.67\text{ m}$$

$$P_3 = 52.5(3.5) = 183.8\text{ kN} \qquad y_3 = \frac{3.5}{2} = 1.75\text{ m}$$

Include water with P_4 since both areas are triangles:

$$P_4 = (34.3 + 11.0)\left(\frac{3.5}{2}\right) = 79.3\text{ kN} \qquad y_4 = \frac{3.5}{3} = 1.17$$

$$R = \sum P_i = 107.5 + 31.0 + 183.8 + 79.3 = \mathbf{401.6}\text{ kN}$$

Now sum the moments for $\bar{y}$:

$$401.6\bar{y} = 107.5(5.25) + 31.0(4.67) + 183.8(1.75) + 79.3(1.17)$$

$$\bar{y} = \frac{1123.6}{401.6} = \mathbf{2.80}\text{ m} \qquad \text{(above wall base)}$$

////

11-5 RANKINE EARTH PRESSURES

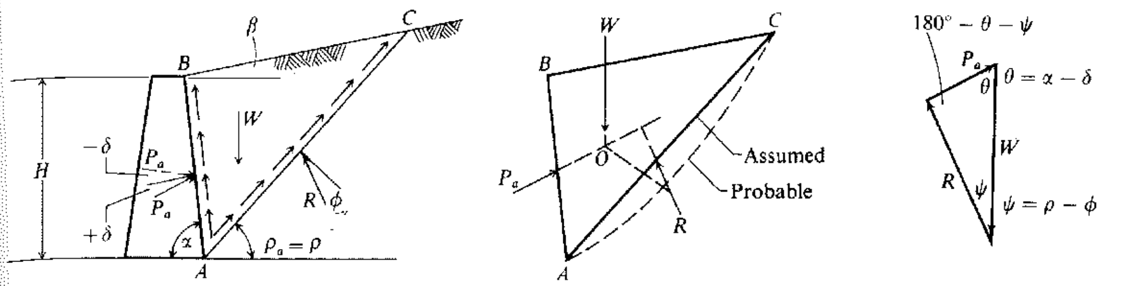
Rankine (ca. 1857) considered soil in a state of plastic equilibrium and used essentially the same assumptions as Coulomb, except that *he assumed no wall friction or soil cohesion.* The Rankine case is illustrated in Fig. 11-7 with a Mohr's construction for the general case shown in Fig. 11-8. From Fig. 11-8 we can develop the Rankine active and passive pressure cases by making substitution of the equation for r (shown on the figure) into the equations for EF (and FG) (also shown on the figure). Then substitution into the expression for K_a' (with OB canceling and using $\sin^2\beta = 1 - \cos^2\beta$) gives the pressure ratio acting parallel to backfill slope β as

$$\boxed{K_a' = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad (11\text{-}7)}$$

We note that the *horizontal component* of active earth pressure is obtained as

$$\sigma_{a,\text{hor}} = \sigma_a \cos\beta \qquad (= OE\cos\beta = OA \text{ of Fig. 11-8}b)$$

Bowles Foundation Analysis and Design 5th ed, 1996



(*a*) Assumed conditions for failure

(*b*) Indication that all force vectors may not pass through point *O*; hence static equilibrium is not satisfied

(*c*) Force triangle to establish P_a

Figure 11-5 Coulomb active pressure wedge.

The active force P_a is a component of the weight vector as illustrated in Fig. 11-5*c*. Applying the law of sines, we obtain

$$\frac{P_a}{\sin(\rho - \phi)} = \frac{W}{\sin(180^\circ - \alpha - \rho + \phi + \delta)}$$

or

$$P_a = \frac{W \sin(\rho - \phi)}{\sin(180^\circ - \alpha - \rho + \phi + \delta)} \tag{b}$$

From Eq. (*b*) we see that the value of P_a depends on angle ρ; that is, all other terms for a given problem are constant, and the value of P_a of primary interest is the largest possible value. Combining Eqs. (*a*) and (*b*), we obtain

$$P_a = \frac{\gamma H^2}{2 \sin^2 \alpha}\left[\sin(\alpha + \rho)\frac{\sin(\alpha + \beta)}{\sin(\rho - \beta)}\right]\frac{\sin(\rho - \phi)}{\sin(180^\circ - \alpha - \rho + \phi + \delta)} \tag{c}$$

The maximum active wall force P_a is found from setting $dP_a/d\rho = 0$ to give

$$P_a = \frac{\gamma H^2}{2}\frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}}\right]^2} \tag{11-1}$$

If $\beta = \delta = 0$ and $\alpha = 90^\circ$ (a smooth vertical wall with horizontal backfill), Eq. (11-1) simplifies to

$$P_a = \frac{\gamma H^2}{2}\frac{(1 - \sin\phi)}{(1 + \sin\phi)} = \frac{\gamma H^2}{2}\tan^2\left(45^\circ - \frac{\phi}{2}\right) \tag{11-2}$$

which is also the Rankine equation for the active earth pressure considered in the next section. Equation (11-2) takes the general form

$$P_a = \frac{\gamma H^2}{2} K_a$$

where

$$K_a = \frac{\sin^2(\alpha + \phi)}{\sin^2 \alpha \sin(\alpha - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\sin(\alpha - \delta)\sin(\alpha + \beta)}}\right]^2} \tag{11-3}$$

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
	22 to 26	0.40 to 0.49
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	$\boxed{17 \text{ to } 22}$	0.31 to 0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt		
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Frost Depth

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: **Athens, Maine**
DFI = 1850 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1800 $d1 := 74.5$

For DFI = 1900 $d2 := 76.6$

$$d := in \cdot \left(\frac{d2 - d1}{10} \cdot 2 + d1 \right)$$

Depth of Frost Penetration $d = 75 \cdot in$ $d = 6.2 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Madison lies along the same Maine Design Freezing Index contour - use Madison data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Madison, Maine
Air Design Freezing Index = 1847 F-days
N-Factor = 0.70
Surface Design Freezing Index = 1293 F-days
Mean Annual Temperature = 42.4 deg F
Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	83.2	20.0	125.0	34	46	3.8	1.9	3,600

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic f

Total Depth of Frost Penetration = 6.93 ft = 83.2 in.

Recommendation: 6.2 feet for design of foundations constructed on coarse grained soils

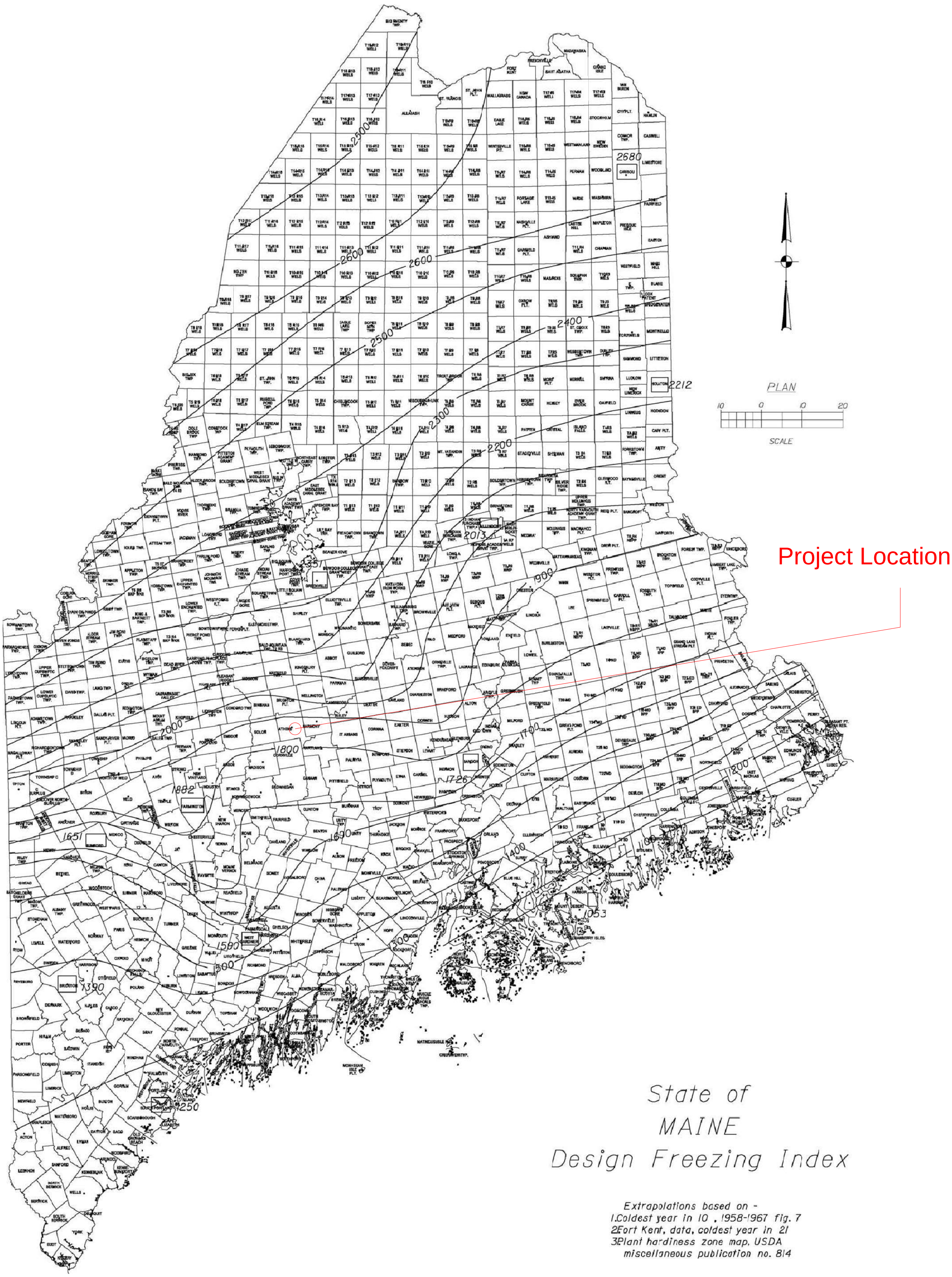


Figure 5-1 Maine Design Freezing Index Map

5.2 General

MaineDOT Bridge Design Guide

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

Seismic Design

BB-AEBWS-101, 101A					
Depth	**101 N_{60}**	**101A N_{60}**		**di**	**di/N**
1.7	34	**--**		1.7	0.05
2.8	--	**--**		**--**	**--**
6	--	17		4.3	0.25
8	--	23		2	0.09
10.5	--	42		2.5	0.06
15	--	45		4.5	0.10
20	**--**	100[1]		5	0.00
23	--	90		3	0.03
26	--	54	**Bedrock**	3	0.06
100		100		74	0.74
SUM				**100**	**1.38**

di/di/N **72.29**

BB-AEBWS-102B					
Depth	**102B N_{60}**			**di**	**di/N**
6	45			6	0.13
8	23			2	0.09
11	29			3	0.10
16	31			5	0.16
20.9	100[1]	**Bedrock**		4.9	0.00
100	100			79.1	0.79
				100	**1.15**

di/di/N **87.14**

BB-AEBWS-201					
Depth	**102B N_{60}**			**di**	**di/N**
2	35			2	0.06
6	13			4	0.31
11	16			5	0.31
16	32			5	0.16
20.5	100[1]			4.5	0.00
23	71			2.5	0.04
27	26			4	0.15
28.5	100	**Bedrock**		1.5	0.02
100	100			71.5	0.72
				100	**1.70**

di/di/N **58.82**

BB-AEBWS-202B, 202C					
Depth	**102B N_{60}**			**di**	**di/N**
6	17			6	0.35
11	12			5	0.42
16	39			5	0.13
21	64			5	0.08
27	36			6	0.17
31	100[1]			4	0.00
34	26	**Bedrock**		3	0.12
100	100			66	0.66
				100	**1.57**

di/di/N **63.73**

SUM **$N_{av.}$** **76.24**

$N_{av} > 50$

Conclusion: Site Class C

Site Classification per LRFD Table 3.10.3.1-1 Method B

Note:
Depths without measurements are indicated as --.
The value used for design is indicated in bold when two or more measurements are given at a depth.

[1]Refusal is assigned 100 blows/ft as reccomended by AASHTO Table C3.10.3.1-1 Method B stating Ni is not to exceed 100 blows/ft

BB-AEBWS-203B					
Depth	**102B N_{60}**			**di**	**di/N**
6	15			6	0.40
11	47			5	0.11
16	81			5	0.06
21	100	**Bedrock**		5	0.05
100	100			79	0.79
				100	**1.01**

di/di/N **99.20**

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.954690
Longitude = -069.645490
Site Class B
Data are based on a 0.05 deg grid spacing.

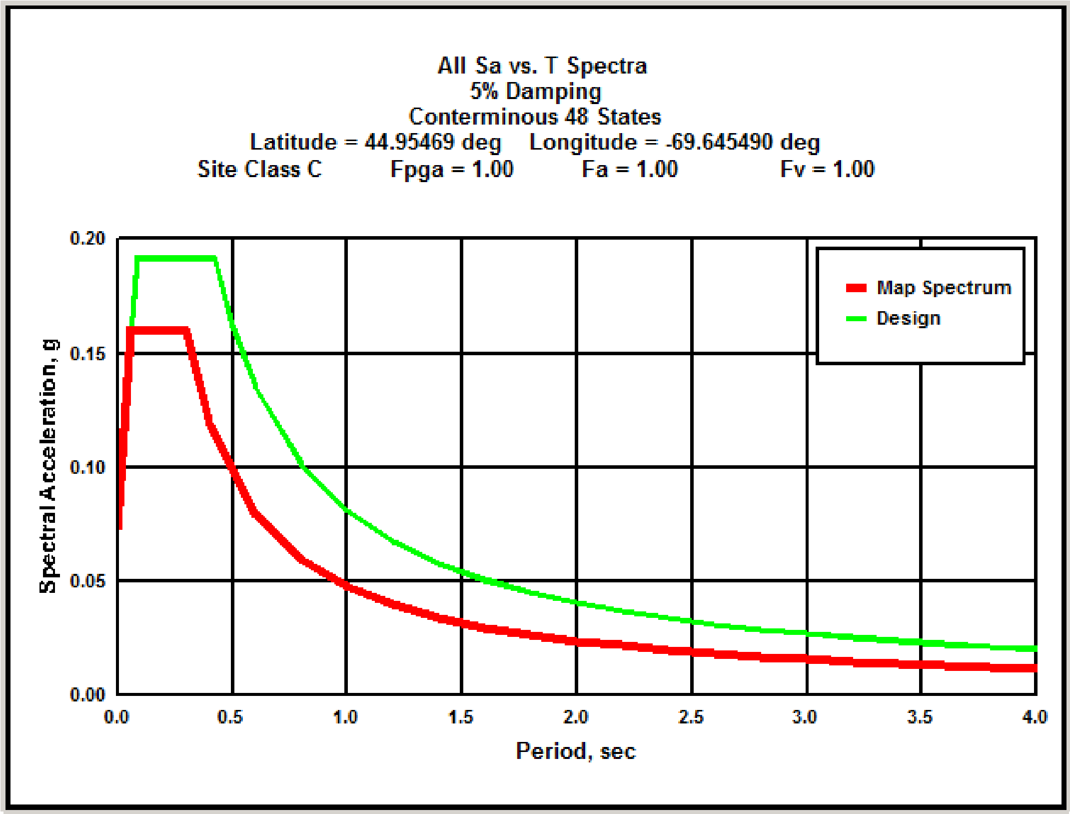
	Period (sec)	Sa (g)	
0.0		0.074	PGA - Site Class B
0.2		0.160	Ss - Site Class B
1.0		0.048	S1 - Site Class B

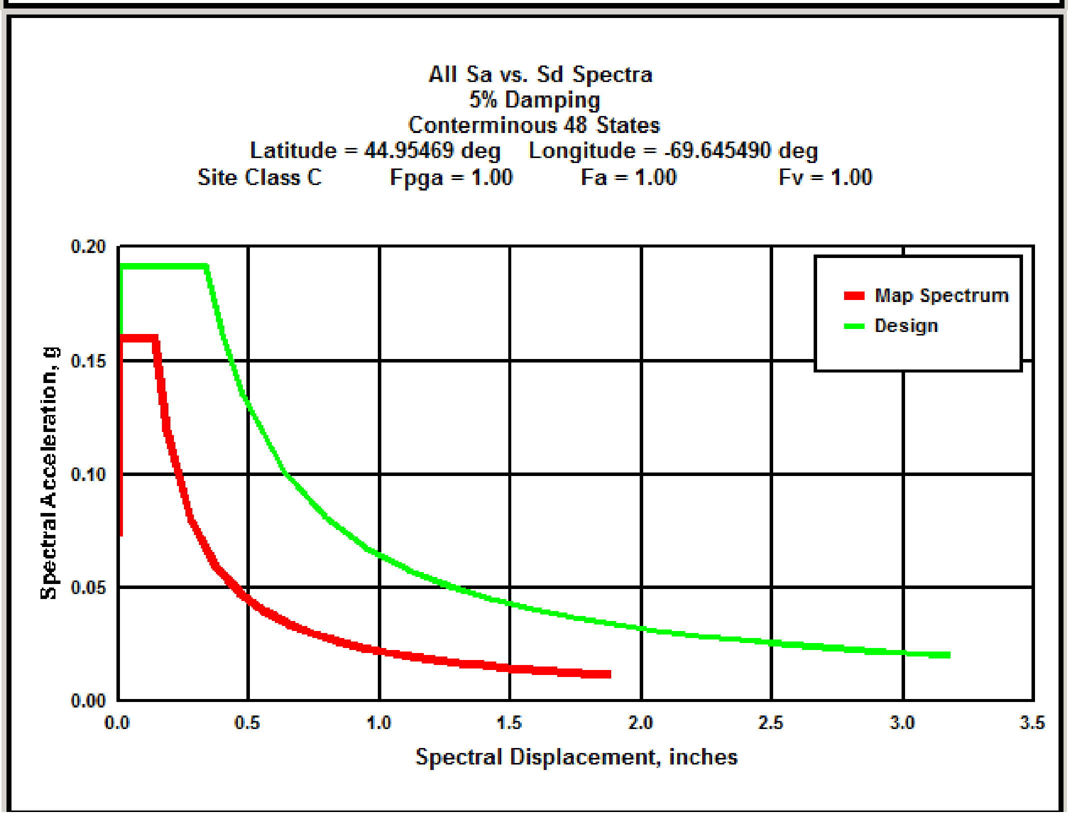
Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.954690
Longitude = -069.645490
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)	
0.0		0.089	As - Site Class C
0.2		0.192	SDs - Site Class C
1.0		0.081	SD1 - Site Class C

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Design Response Spectra for Site Class C
Latitude = 44.954690
Longitude = -069.645490
As = FpgaPGA, SDs = FaSs, SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

	Period (sec)	Sa (g)	Sd in.	
0.000	0.089	0.000		T = 0.0, Sa = As
	0.085	0.192	0.013	
0.200	0.192	0.075		T = 0.2, Sa = SDs
0.424	0.192	0.336		T = Ts, Sa = SDs
	0.500	0.162	0.397	
	0.600	0.135	0.476	
	0.800	0.101	0.634	
1.000	0.081	0.793		T = 1.0, Sa = SD1
	1.200	0.068	0.952	
	1.400	0.058	1.110	
	1.600	0.051	1.269	
	1.800	0.045	1.428	
	2.000	0.041	1.586	
	2.200	0.037	1.745	
	2.400	0.034	1.903	
	2.600	0.031	2.062	
	2.800	0.029	2.221	
	3.000	0.027	2.379	
	3.200	0.025	2.538	
	3.400	0.024	2.696	
	3.600	0.023	2.855	
	3.800	0.021	3.014	
	4.000	0.020	3.172	


All Sa vs. T Spectra
5% Damping
Conterminous 48 States
Latitude = 44.95469 deg   Longitude = -69.645490 deg
Site Class C   Fpga = 1.00   Fa = 1.00   Fv = 1.00
Map Spectrum
Design
Spectral Acceleration, g
0.20
0.15
0.10
0.05
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
Period, sec


All Sa vs. Sd Spectra
5% Damping
Conterminous 48 States
Latitude = 44.95469 deg   Longitude = -69.645490 deg
Site Class C   Fpga = 1.00   Fa = 1.00   Fv = 1.00
Map Spectrum
Design
Spectral Acceleration, g
0.20
0.15
0.10
0.05
0.00
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
Spectral Displacement, inches

<u>Appendix D</u>

Draft Special Provision

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock-Socketed H-Pile Foundation)

501.01 Description. The following is added to subsection 501.01:

This work shall consist of providing all materials, equipment and labor necessary for construction of Rock-Socketed H-pile Foundations for the replacement of Gilman Bridge, as shown on the Plans or as authorized by the Resident. Construction of Rock-Socketed H-Pile Foundation shall be as specified in Section 501 of the Standard Specifications, except as amended herein.

501.02 Materials. The following is added to subsection 501.02:

H-pile end plate material shall be ASTM A572 Grade 50 minimum. Fastening of the end plate to pile shall occur by welding the end plate to the pile web and flanges.

Non-shrink, or shrinkage compensating, grout and any grout admixtures shall be selected from the Maine Department of Transportation's Qualified Products List. The grout mix design shall be in accordance with the manufacturers and the responsibility of the Contractor. The water content and admixtures, if used, shall be the minimum necessary to achieve adequate strength and for proper placement. Final proportions of grout materials shall be based on the results of compressive testing of grout cubes made from sample mixtures of grout, or a valid compressive strength test performed within two years of the submittal date performed on the proposed mix design demonstrating the mix design achieves the required strength. The minimum seven-day compressive strength of 2-inch cubes, molded, cured, and tested in accordance with ASTM C-109 shall be 6,000 psi. The contractor shall be responsible for taking, curing, and breaking of grout test cubes prior to the start of work.

Aggregate for Rock-Socketed H-pile Foundation shall meet the requirements of 703.22 Underdrain Backfill Material, Type C.

501.03 Quality Control Plan. The following is added to subsection 501.03:

At least 21 days prior to beginning construction of the Rock-Socketed H-pile Foundation, the Contractor shall submit a Quality Control Plan for review by the Resident. Standard Specification 105.7 shall govern the review process.

The Quality Control Plan shall include the following:

A. List of proposed equipment to be used including drilling equipment, drills, drill bits, augers, buckets, casing, final cleaning equipment, rock coring equipment, tremies or grout pumps, etc.
B. Details and methods for welding end plate to the pile.
C. Details of overall Rock-Socketed H-Pile Foundation construction operation sequence.
D. Details of excavation methods in soils and bedrock, including methods of removing any obstructions such as boulders or cobbles.
E. Details of equipment and methods to clean bedrock-sockets.
F. Details of installing H-Piles and supporting H-piles laterally in their final positions until the abutment is complete and in place.
G. Details of grout mix design, demonstrated 7 day mix design compressive strength, placement methods, and grout placement quantity control.
H. Sample daily construction records.
I. Sample drilling and grout logs.

The Resident will evaluate the Rock-Socketed H-Pile Foundation Quality Control Plan, and all procedural approvals given by the Resident shall be subject to trial in the field and shall not relieve the Contractor of the responsibility to satisfactorily complete the work as detailed in the Plans and Specifications.

<u>501.042 Equipment.</u> The following is added to subsection 501.042:

Rock Sockets. Drilling of bedrock-sockets for Rock-Socketed H-Pile Foundation shall use cased-hole drilling methods.

The excavation and drilling equipment shall have adequate capacity including power, torque and down thrust to excavate a hole of both the diameter and to a depth of 20 percent beyond the depth indicated on the plans. When the material encountered cannot be drilled using conventional earth augers with soil or rock teeth and drill buckets, the Contractor shall provide drilling equipment including, but not limited to, rock core barrels, rock tools, air tools, and other equipment as necessary to construct the shaft excavation to the size and depth required.

Failure by the Contractor to demonstrate adequate methods and equipment shall be reason for the Resident to require alterations in equipment and/or method by the Contractor to eliminate unsatisfactory results. Any altered methods or construction equipment shall be at the Contractors expense and incidental to this item.

The Contractor shall perform the excavations required for bedrock-sockets as shown on the Plans, through whatever materials are encountered, to the dimensions and elevations shown on the Plans or otherwise required by the specifications and special provisions. The Contractor's methods and equipment shall be suitable for the intended purpose and materials encountered. Blasting is prohibited.

501.043 Location and Alignment Tolerances. The following is added to subsection 501.043:

The Contractor shall hold the piles in place to allowable tolerances. Piles may be braced or suspended, or both, to insure vertical and horizontal alignment.

The maximum elevation deviation between each quadrant of the bedrock-socket shall be 0.5 inches. The highest point of the unexcavated bedrock and residual material within the bedrock-socket footprint shall be considered the bedrock-socket bottom elevation and shall not be above the elevation shown on the Plans.

The contractor shall demonstrate that a minimum of 50 percent of the base of each socket shall have less than 0.5 inches of sediment at the time of placement of the concrete measured by weighted tape, solid rod, or other suitable method detailed in the Quality Control Plan and accepted by the Resident.

Pile tip plates shall be supported between 3 and 6 inches above the bedrock-socket bottom by the pile tip shoe plate. See the attached Pile Base Detail.

The maximum deviation of top of grout elevation from that shown on the plans is 2 inches.

The following sections are added to 501.04 Construction Requirements:

501.049 Drilling and Rock-Socket Excavation.

Bedrock excavations shall be made at locations and to the elevations and dimensions shown on the Plans. Bedrock-socket bottom elevations may be adjusted when the Resident determines that the material encountered during excavation is unsuitable or differs from that anticipated in the design of the Rock-Socketed H-Pile Foundation.

The Contractor shall maintain a construction method log during socket drilling and excavation. The log shall contain information such as: project, date, start time, stop time, personal, weather, pile identification indicated on the Plans, drilling methods, drilling resistance, cleaning methods, obstructions, seepage of groundwater through casing/bedrock seal, etc.

The Contractor shall maintain a grout placement log during grout placement. The log shall contain information such as: project, date, start time, stop time, personal, weather, pile identification indicated on the Plans, number of bags mixed and placed, final elevation of the placement top, number, type, and identification number of samples collected.

Excavated materials which are removed from socket excavations shall be disposed of by the Contractor in accordance with the applicable specifications for disposal of excavated materials. Cleaning of the bedrock-socket may be performed by cleanout auger or bucket, reverse circulation, air-lifting, or vacuum methods.

The Contractor shall perform the necessary excavation for the Rock-Socketed H-pile Foundation under this item. No separate payment will be made for either excavation of materials of different densities or employment of special tools and procedures necessary to accomplish the excavation in an acceptable fashion.

After removal of the overburden from within the casing, the casing shall be further advanced into bedrock, where shown on the plans or directed by the Resident, if necessary to achieve sealing against the entry of overburden and groundwater. Then the excavation shall continue into bedrock as an uncased or cased rock socket of the length and diameter indicated. The rock socket shall not be constructed until the casing is sealed in bedrock and until the casing has been checked for plumbness. A method of excavating the rock socket that is capable of providing a cylindrical opening of the specific diameter and to full-depth as shown on the plans or to the depth directed by the Resident shall be used. Overbreakage of the rock surface shall be avoided to not destroy the seal at the bottom of the casing. The rock socket shall be constructed to have a bottom meeting the allowable tolerances.

The Contractor shall keep a daily construction record. The Contractor shall provide access and equipment for checking the alignment of the casing and for checking the dimension, alignment and cleanliness of the rock socket. Final pile and socket depths shall be measured with suitable weighted tape or other approved methods after final cleaning. Socket cleanliness shall be demonstrated by the Contractor to the satisfaction of the Resident. Grout placement shall not begin until the Resident's approval has been obtained.

<u>501.050 Obstructions</u>

Surface and subsurface obstructions at the pile locations shall be removed by the Contractor. Such obstruction may include man made materials such as old concrete foundations and natural materials such as boulders. Obstructions may be cleared by conventional excavation methods, use of special procedures and/or tools by the Contractor after the casing cannot be advanced using conventional augers fitted with soil or rock teeth, or drilling buckets. Such special procedures/tools may include but are not limited to: chisels, boulder breakers, core barrels, air tools, etc. Materials removed by conventional excavation methods shall be replaced with suitable backfill material and compacted. Blasting is prohibited.

Drilling tools which are lost in the excavation shall not be considered obstructions and shall be promptly removed by the Contractor without compensation. All costs due to lost tool removal shall be borne by the Contractor.

<u>501.051 H-Pile and Grout Installation</u>

The casing shall be installed in a manner that will produce a positive seal at the bottom of the casing so that no piping of water or other materials occurs into the socket. Vertically split seamed break-away style casing is prohibited.

The H-pile shall be lowered into the cased-hole so as to be suspended 3 to 6 inches above the bottom of the bedrock-socket. The Contractor will be required to support the H-Piles laterally in their final positions until the abutment is complete and in place. The socket shall then be filled with grout, as specified in this Section, to the elevation shown on the Contract Plans. "Wet sticking", placing the pile into wet grout, is prohibited.

Rock-Socketed H-Pile Foundation grout and grout mix shall conform to the materials portion of this provision. Grout placement shall utilize tremie methods. An independent laboratory approved by the Resident shall conduct testing throughout the course of work at a frequency of no less than every other bedrock-socket or every other continuous grout placing operation.

After the grout has achieved the required strength, the cased hole shall be filled with aggregate. The aggregate shall be dropped. Casing may be withdrawn as aggregate is placed provided no damage to the grout column or pile occurs.

Where the Contractor is to excavate after rock-socketed H-pile installation, the cased hole does not need to be filled with aggregate. The Contractor is responsible for maintaining the integrity of the fixed, grouted bottom of the H-pile when excavating after installation, as determined by the Resident. Additional length of rock socket, bracing, and other incidentals associated with maintaining the integrity of the concrete bottom of the foundation and maintaining the lateral positions of the piles shall be incidental.

<u>501.047 Splicing Piles.</u> The following is added to subsection 501.047:

Splicing of H-piles for Rock-Socketed H-Pile Foundation is prohibited.

<u>501.05 Method of Measurement.</u> The following is added to subsection 501.05:

a) <u>Piles Furnished</u> Furnishing of H-piles for Rock-Socketed H-Pile Foundation shall be as outlined in subsection 501.05.

b) <u>Piles in Place.</u> Method of measurement for constructing Rock-Socketed H-pile Foundation as described in this Section shall be measured by the linear foot of piles in place. Method of measurement shall include all materials, excavation, construction methods, mobilization of equipment, and incidentals necessary to complete the work, as described herein.

<u>501.06 Basis of Payment.</u> The following is added to subsection 501.06:

The accepted quantities of Rock-Socketed H-piles will be paid for at the Contract Unit Price per linear foot, delivered, and complete, in place. Such payment will include full compensation for all material, excavation, construction methods, mobilization of equipment, and incidentals necessary to complete the work as specified herein.

Payment shall be under:

Pay Items		Pay Unit
501.xx	Steel H-beam Piles 117 lb/ft, delivered	Linear foot
501.xxx	Rock-Socketed H-Piles 117 lb/ft, in place	Linear foot
501.084	Drilling Equipment Mobilization, Rock-Socketed H-pile	Lump Sum