

PRELIMINARY DESIGN REPORT

17-0079

September 1, 2017

Explorations and Geotechnical Engineering Services

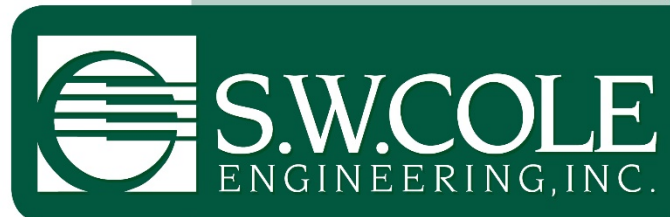
Half Mile Pond Brook Bridge #6246
Replacement
Route 9 over Half Mile Brook
Amherst, Maine
WIN 21870.00

PREPARED FOR:

Maine Department of Transportation
Attention: Laura Krusinski, P.E.
State House Station 16
Augusta, ME 04333-0016

PREPARED BY:

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- *Geotechnical Engineering*
- *Construction Materials Testing and Special Inspections*
- *GeoEnvironmental Services*
- *Test Boring Explorations*

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Dear Laura:

In accordance with our Proposal dated January 31, 2017 and project specific Assignment Letter #10 dated February 10, 2017, we have made the requested subsurface explorations for the subject project. The purpose of our services was to obtain subsurface information in order to provide geotechnical parameters and recommendations for foundations and earthwork associated with the proposed construction. The services provided by S. W. Cole Engineering, Inc. (S.W.COLE) were conducted in accordance with our Multi-PIN Agreement with the Maine Department of Transportation (MaineDOT), No. 20150720000000000085, dated July 20, 2015. The contents of this report are subject to the limitations in Appendix A.

1.0 INTRODUCTION

1.1 Site Conditions

The site is Half Mile Pond Brook Bridge (Bridge #6246) carrying Route 9 over Half Mile Brook in Amherst, Maine. The site location is shown on the "Site Location Map" attached in Appendix B.

Based on our review of the historic plans provided by MaineDOT, we understand the existing crossing was constructed in 1967. The existing structure consists of a structural plate pipe arch culvert an 16'-7" span and 10'-1" rise.

1.2 Proposed Construction

Based on discussions with MaineDOT, we understand the existing structure will be replaced; replacement alternatives under consideration include:

- Precast concrete box culvert (box culvert alternative);
- Precast concrete three-sided frame supported on strip footings or driven piles (three-sided frame alternative); and
- Single-span bridge supported on spread footings or driven piles.

We understand current planning is considering a clear span of about 23 feet (e.g. 1.2 times bank full width of 19 feet) with a 0 degree skew. We understand the horizontal and vertical alignment will remain the same.

2.0 EXPLORATIONS AND TESTING

2.1 Explorations

Two test borings (BB-AHMB-101 and BB-AHMB-102) were made at the site on February 23, 24 and 28, 2017 by S. W. Cole Explorations, LLC, a subsidiary of S.W.COLE. The as-drilled exploration locations were selected by MaineDOT and established in the field by S.W.COLE using taped measurements from existing site features. The ground surface elevations of the test borings were provided by MaineDOT. The approximate exploration locations are shown on the "Boring Location Plan" attached in Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix C.

2.2 Testing

The test borings were drilled using a combination of solid-stem auger, cased wash boring and rock core drilling techniques. The soils were sampled at 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) techniques using a calibrated automatic hammer. Upon encountering refusal on bedrock, borings BB-AHMB-101 and BB-AHMB-102 were advanced about 10 feet into bedrock using a

NQ2 wireline rock coring. The uncorrected SPT blow counts, uncorrected and corrected SPT N-values and rock core intervals are shown on the logs.

Soils samples recovered from the test borings were visually classified in our laboratory and transported to the MaineDOT Laboratory in Bangor, Maine for testing to assist soil classification and identification. Laboratory testing was performed on disturbed SPT samples obtained during the explorations. Laboratory testing was performed by the MaineDOT Materials Testing and Exploration Central Laboratory in Bangor, Maine in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) testing procedures. Laboratory testing included seven natural water content tests and seven grain size analyses (with and without hydrometer). Laboratory testing are provided in Appendix D.

3.0 SUBSURFACE CONDITIONS

3.1 Surficial and Bedrock Geology

According to the Maine Geological Survey's (MGS's) mapping of the Great Pond Quadrangle, Maine (Open-File No. 86-62, MGS 1986), surficial geologic units within the site vicinity consists of glacial stream deposits consisting of sand and gravel which may include glacial till. The subsurface conditions encountered were consistent with the mapped surficial geology; however, the explorations also encountered a surface deposit of fill soils from previous site development.

Bedrock in the site vicinity is mapped as Devonian age Lucerne Granite (Geologic Map of the Lucerne Granite, Hancock and Penobscot Counties, Maine, U.S. Geological Survey Miscellaneous Investigations Series Map I-2360, USGS 1993). The Lucerne Granite is generally composed of coarse-grained, quartz-feldspar-biotite-hornblende granite. The observed bedrock is generally consistent with the mapped bedrock geology.

Two historic strike-slip faults have been identified within about 5 miles of the site (USGS, 1993). The southwest-northeast trending Norumbega Fault Zone, located about 3.9 miles northwest of the site and the generally east-west trending Sunnyside Fault, located about 4.6 miles south of the site. Neither of these faults are mapped as intersecting the site.

3.2 Subsurface Conditions

The test borings encountered a soils profile generally consisting of a surface layer of pavement overlying fill overlying glacial till overlying bedrock. The principal strata encountered in the explorations are summarized below. An “Interpretive Subsurface Profile” is attached in Appendix B. Refer to the attached logs for more detailed information of the subsurface findings at the exploration locations.

Fill: Below an approximate 9 inch layer of pavement, the borings encountered fill extending to depths of about 15 to 17.1 feet below ground surface (bgs), corresponding to Elevation (El.) 198.7 to 201.1 feet. The fill generally consisted of:

- Brown, SAND, some to little gravel, trace silt;
- Brown, Clayey SILT, trace sand; and
- Brown, Gravelly SAND, trace silt.

The fill was generally dense to very dense with SPT N_{60} values ranging from 20 to 70 blows per foot (bpf).

Glacial Till: Below the fill, the borings generally encountered glacial till to depths of about 33 feet bgs, corresponding to Elevation (El.) 183.2 to 183.6 feet. The glacial till generally consisted of:

- Brown, SAND, little gravel, little silt;
- Brown, Gravelly SAND, little silt;
- Grey, SAND, some gravel, little silt.

Cobbles and occasional boulders were noted throughout the deposit. The glacial till was generally medium dense to very dense with SPT N_{60} values ranging from 22 bpf to 100 blows for 2 inches (sampler refusal).

Bedrock: Bedrock was encountered and sampled in borings BB-AHMB-101 and BB-AHMB-102. The top of bedrock was about 33 feet bgs. The bedrock consisted of grey, very hard, fresh, quartz-plagioclase-biotite-hornblende granite of the Lucerne Granite. Joints were generally low angle, tight with remobilized quartz mineralization along foliation stress planes.

Rock quality designation (RQD) values for the bedrock generally ranged from 75 to 100 percent correlating to a Rock Mass Quality (RMQ) of fair to excellent. The RQD for a 6 and 3.5-inch long core runs in BB-AHMB-102 were 0 percent due to plugging of the core barrel.

3.3 Groundwater Conditions

The soils encountered at the test borings were damp to moist from the ground surface. The measured water levels during drilling ranged from 16.5 to 22.00 feet below ground surface. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate seasonally, particularly in response to periods of snowmelt and precipitation, as well as changes in site use and the water level of Half Mile Brook.

4.0 PRELIMINARY GEOTECHNICAL EVALUATION

S.W.COLE conducted geotechnical engineering evaluations in accordance with 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 interim revisions (AASHTO) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through August 2014 (MaineDOT BDG) and offers the following:

4.1 Foundation Options

Proposed replacement structure alternatives include:

- Precast concrete box culvert (box culvert alternative);
- Precast concrete three-sided frame supported on strip footings or driven piles (three-sided frame alternative); and
- Single-span bridge on pile-supported integral abutments

We understand the proposed structure span will be about 23 feet and the horizontal alignment and vertical profile will not change.

The site is underlain by medium dense to very dense glacial till consisting of sand with variable amounts of gravel and silt with occasional cobbles and boulders overlying bedrock at a depth of 32.5 to 33.0 feet bgs.

4.2 Precast Concrete Box Culvert

The precast box culvert option would consist of a 23-foot span precast concrete box culvert with a 0 degree skew. The slopes on the inlet and outlet ends of the culvert shall have 1.75:1(H:V) or flatter riprap slopes. The invert of the box culvert will be embedded approximately 3 feet into the stream bed and about 2 feet of “special fill” will be placed inside the bottom of the culvert to create a natural streambed.

Based on the MaineDOT BDG Section 8.3.1, the precast box culvert shall include toe walls at the inlet and outlet ends to prevent undermining. The toe walls should extend at least 1 foot below the maximum scour depth.

We recommend the box culvert be founded on a minimum 1-foot thick mat of MaineDOT Standard Specification 703.19 Granular Borrow for Underwater Backfill. The culvert backfill shall consist of MaineDOT Standard Specification 703.19 Granular Borrow for Underwater Backfill modified with a maximum particle size of 3 inches.

Based on the subsurface conditions, for a precast box culvert with a base width of 25 feet, the factored bearing resistance at the strength limit state shall not exceed the factored bearing resistance of 7.5 ksf. The factored bearing resistance at the service limit state shall not exceed 5.0 ksf. The service limit state may govern the design. The bearing stress shall not exceed the nominal structural resistance of the structural concrete of $0.3 \times f'_c$.

We recommend the following coefficients of subgrade reaction (k_v) for preliminary analysis for the box culvert bearing on a 1 foot layer of compacted Granular Borrow for Underwater Backfill overlying undisturbed native sands.

Location	Coefficient of Subgrade Reaction (k_v)
Corner of Box Culvert	93 pci
Center of Box Culvert	80 pci

We recommend the bearing resistance and coefficient of subgrade reaction be re-evaluated during the final design.

4.3 Precast Concrete Three-Sided Frame

The precast concrete three-sided frame option will consist of an approximate 23-foot span, precast three-sided rigid-frame supported on spread footings with a 0 degree skew. Considerations for scour and bearing resistance may preclude the use of individual spread footings for structure support. The slopes on the inlet and outlet ends of the three-sided frame will have 1.75:1(H:V) or flatter riprap slopes. Concrete headwalls may be included in the buried structure design to retain riprap slopes and prevent riprap from dropping or eroding into the waterway.

For scour protection, footings which are constructed on soil shall be embedded a minimum of 2 feet below the check flood (Q500) scour elevation: refer to AASHTO LRFD 2.4.4.2 and MaineDOT BDG Section 5.3.4.1 for guidance regarding scour protection. Furthermore, per the MaineDOT BDG Section 2.3.11.1, spread footings on soil within the stream channel shall be located a minimum of 6 feet below the thalweg of the waterway.

Individual spread footings for a three-sided frame shall be sized such that the applied contact footing pressures do not exceed the bearing resistances provided in the table below. The applied contact pressures may preclude the use of individual spread footings and may require a mat foundation. Footings and mat foundations founded on a minimum 1-foot thick mat of MaineDOT Standard Specification 703.19 Granular Borrow for Underwater Backfill. The backfill shall consist of MaineDOT Standard Specification 703.19 Granular Borrow for Underwater Backfill modified with a maximum particle size of 3 inches.

Based on the subsurface conditions and subgrade preparation described herein, the factored bearing resistance of footings/mat foundations at the strength and service limit states shall not exceed the factored bearing resistance shown in the following table.

Footing Width (feet)	Factored Bearing Resistance (ksf)	
	Strength Limit State	Service Limit State
10	4.4	5.0
12	5.0	5.0
14	5.6	5.0
16	6.3	5.0
18	6.9	5.0

The service limit state may govern the design. The bearing stress shall not exceed the nominal structural resistance of the structural concrete of $0.3 \times f'_c$.

Spread footings shall be designed for all relevant strength and service limit state load combinations per LRFD Article 10.6. Design of spread footings at the strength limit state shall consider:

- Bearing Resistance;
- Eccentricity;
- Lateral Sliding and;
- Reinforced-concrete structural failure.

4.4 Single-Span Bridge

The single-span bridge option will have a span if at least 23 feet with a 0 degree skew. We understand the bridge will be supported on pile-supported integral abutments. We anticipate the soils on the inlet and outlet ends of the bridge structure would be retained with short wing walls and 1.75:1(H:V) or flatter riprap slopes in accordance with Standard Detail 610(03)..

4.4.1 Driven Piles

Based on the overburden soils consisting of medium dense to very dense glacial till with occasional cobbles and boulders, we anticipate pile-supported integral abutments will be founded on a single row of steel H-piles. The piles shall be end bearing on or within bedrock and driven to the required resistance. Based on the MaineDOT BDG Section 5.4.2.1, H-piles may include HP 10x42, 12x53, 14x73 or 14x89. Additional pile sections may be considered depending on the factored design axial loads. S.W.COLE can provide additional input on pile size once pile loading has been developed for the proposed structure.

H-piles shall be 50 ksi, Grade A572 steel with cast driving points conforming to MaineDOT Standard Specification 711.10 to help reduce damage to the piles during driving and improve penetration.

Based on the depths to bedrock encountered at the borings, we estimated the following pile lengths:

Location	Assumed Abutment Subgrade Elevation (feet)	Approximate Top of Competent Bedrock Elevation (feet)	Estimated Pile Length (feet)
West Abutment	205.0	183.6	22
East Abutment	207.0	183.2	24

The estimated pile lengths do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment or pile cap.

Strength Limit State Design: 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;
- Drivability resistance of individual piles driven to bedrock;
- Structural resistance of individual piles in axial compression, and;
- Structural resistance of individual piles in combined axial loading and flexure.

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

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$, for severe driving conditions shall be applied to the structural compressive resistance of the pile. Piles subjected to lateral loading shall be evaluated for resistance against combined axial compression and flexure in accordance with LRFD Articles 6.9.2.2 and 6.15.2. 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).

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. The nominal axial structural compressive resistance (P_n) subject to the combined axial compression and flexure shall be evaluated based on unbraced lengths (l) and effective length factors (K) as determined from LPile once structural loads are

available. The nominal axial structural resistance should be evaluated based on combined axial compression and flexure.

Geotechnical Resistance: The nominal axial geotechnical resistance in the strength limit state should be calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions.

Drivability Analyses: Drivability analyses shall be performed to determine the pile resistance that might be achieved considering available diesel hammers. The drivability resistance of the piles typically controls the allowable axial pile loads. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances shall be calculated using the resistance factor, ϕ_{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.

Service and Extreme Limit State Design: The design of piles at the service limit state shall consider tolerable transverse and longitudinal movement of piles and pile group movement considering changes in soil conditions due to scour based on the design flood (Q_{100}). For the service limit state, resistance factors of $\phi = 1.0$ 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 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 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.

Lateral Pile Resistance/Behavior: In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

Lateral pile analyses shall be performed by the geotechnical engineer to evaluate pile behavior at both abutments using LPILE software with pile head deflections, moments, and axial loads supplied by the structural engineer.

Driven Pile Resistance and Pile Quality Control: The contract documents shall require the contractor to perform a wave equation analysis for the proposed pile-hammer system and conduct dynamic pile load tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Additional dynamic tests may be required as part of the pile field quality control program if:

- Pile behavior vary radically between adjacent piles;
- Pile behavior indicates pile refusal on a boulder or in a cobble layer above bedrock; or
- Pile is out of tolerance.

Piles should be driven to an acceptable penetration resistance based on the results of a wave equation analysis provided by the contractor and as approved by the design team. Pile load testing should be completed by PDA testing with signal matching including one pile at each abutment. Driving stresses in the pile determined in the drivability analysis and confirmed by PDA testing shall be less than 45 ksi (for 50 ksi steel), in accordance with LRFD Article 10.7.8. The pile hammer should be selected such that the required pile resistance when the penetration resistance for the final 3 to 6 inches is between 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. Termination criteria shall be confirmed and evaluated for the selected pile hammer.

4.5 Integral Abutment Design

Abutments 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. Integral 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 abutment at the strength limit state shall consider reinforced-concrete structural design. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

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
- Soil-concrete interface friction angle (δ) of 20 degrees.

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}) based on LRFD Table 3.11.6.4-1.

The abutment design shall include a drainage system behind the abutment to mitigate excessive hydrostatic pressures. 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.”

Slopes constructed in front of the pile supported abutments should be constructed with riprap and erosion control geotextile. The riprap slopes should not exceed 1.75:1(H:V) in accordance with MaineDOT Standard Detail 610(03). The 1.75H:1V riprap slopes shall “toe-in” behind the existing reinforced concrete mat foundation.

4.6 Frost Considerations

Based on the Maine Design Freezing Index Map¹, the design freezing index for the Amherst, Maine area is approximately 1,600 freezing degree-days. Based on Section 5.2.1 of the MaineDOT BDG and subsurface soils encountered, the maximum seasonal frost penetration is estimated to be on the order of about 7 feet. Considering this, we recommend foundations should have at least 7 feet of soil cover to provide frost protection.

4.7 Seismic Design Considerations

Seismic site class was evaluated in accordance with AASHTO Section 3.10.3.1 Method B (average N-value method). Based on the subsurface information, the average N-value was greater than 50 bpf corresponding to an AASHTO Site Class C as defined in AASHTO Table 3.10.3.1-1.

The USGS online Seismic Design Maps Tool was used to obtain the seismic design parameters for the site. Based on the assigned Site Class (AASHTO Site Class C) and site coordinates, the software provides the recommended AASHTO Response Spectrum for a 7 percent probability of exceedance in 75 years. The results for the project site are summarized below:

¹ Maine Department of Transportation, Bridge Design Guide (BDG), August 2003, with Revisions through 2014, Figure 5-1.

RECOMMENDED SEISMIC DESIGN PARAMETERS ²	
Site Class	C
PGA	0.066 g
S _s	0.142 g
S ₁	0.043 g
F _{pga}	1.2
F _a	1.2
F _v	1.7
A _s	0.079 g
S _{DS}	0.170 g
S _{D1}	0.072 g
Seismic Zone (based on S _{D1})	Zone 1

NOTE: Site Coordinates: N44.824312, W68.405759

4.8 Recommendations for Scour Evaluation

Based on the laboratory grain size analyses test results, we estimated the average grain diameter corresponding to 50 percent passing by weight (D₅₀) for use in scour evaluations ranges from 1.1 to 1.5 mm. Results of the grain size analyses tests are included in Appendix B and the estimated D₅₀ for samples near the channel elevation are summarized in the following table:

Boring No.	Sample No.	Depth (ft)	Sample Elevation (ft)	Estimated D ₅₀
BB-AHMB-101	4D	15.0	201.1	1.1 mm
BB-AHMB-102	4D(C)	17.5	198.7	1.5 mm

MaineDOT BDG Sections 2.3.11 and 5.3.4.4 requires that footings for bridges on soil be founded at least 2 feet below the scour elevation as determined from the check flood (Q500) event. Furthermore, spread footings supported on soil shall be located a minimum of 6 feet below the thalweg of the waterway.

² U.S. Geological Survey, Seismic Design Map, , accessed March 20, 2017
<http://earthquake.usgs.gov/designmaps/us/application.php>

5.0 CLOSURE

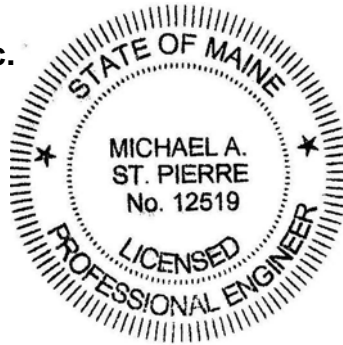
We trust this information meets your present needs. Please contact us if you have any questions or need further assistance.

Sincerely,

S. W. Cole Engineering, Inc.



Michael A. St. Pierre, P.E.
Geotechnical Engineer



Timothy J. Boyce, P.E.
Senior Geotechnical Engineer

MAS/tjb-rec

APPENDIX A LIMITATIONS

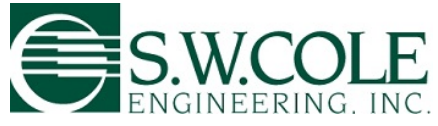
This report has been prepared for the exclusive use of the Maine Department of Transportation for specific application to the Half Mile Pond Brook Bridge #6246 Replacement carrying Route 9 over Half Mile Brook (MaineDOT WIN 021870.00) in Amherst, Maine. S. W. Cole Engineering, Inc. (S.W.COLE) has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

The soil profiles described in the report are intended to convey general trends in subsurface conditions. The boundaries between strata are approximate and are based upon interpretation of exploration data and samples.

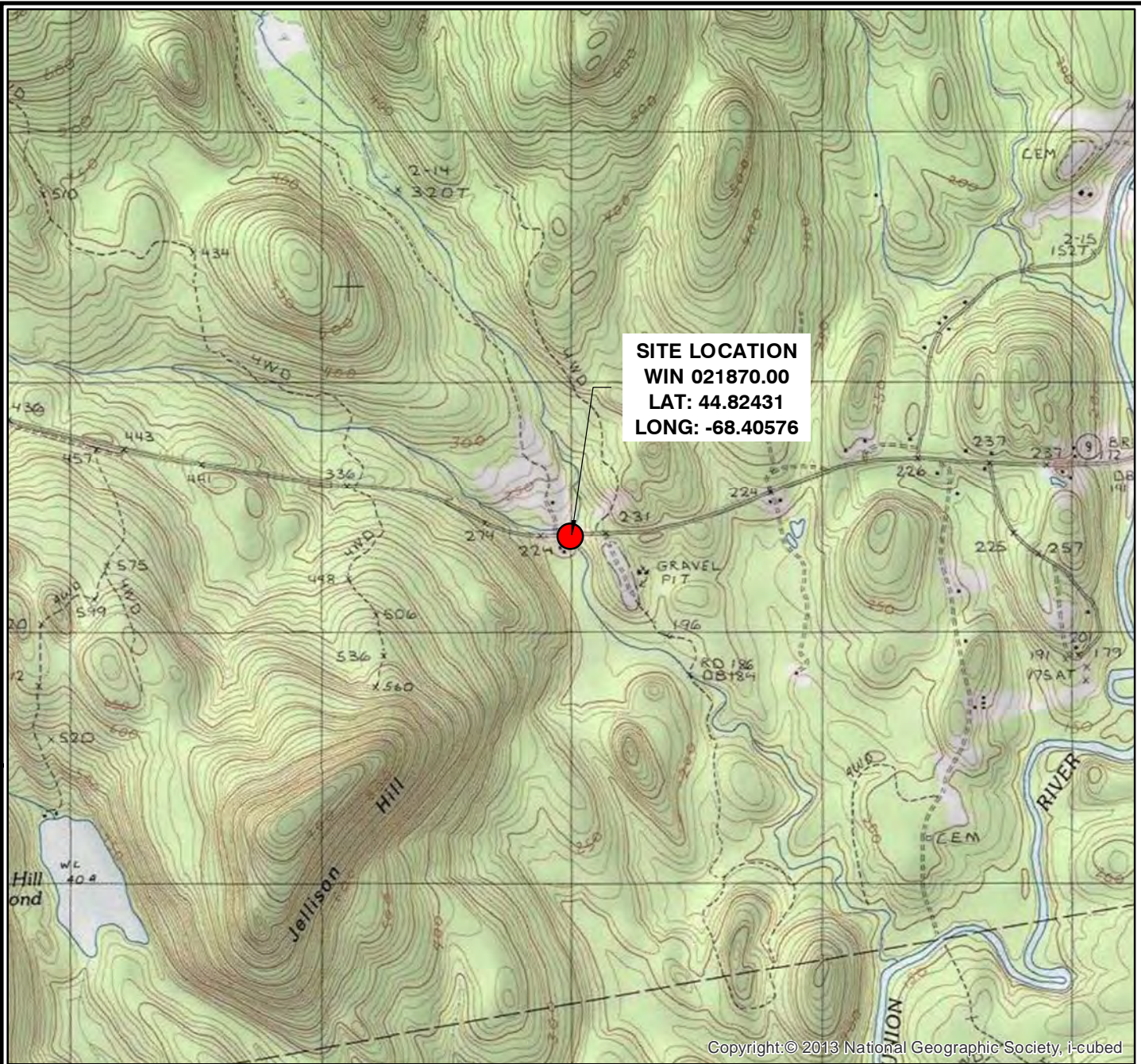
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and to review the recommendations of this report.

Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S.W.COLE should review such changes as they relate to analyses associated with this report. Recommendations contained in this report shall not be considered valid unless the changes are reviewed by S.W.COLE.

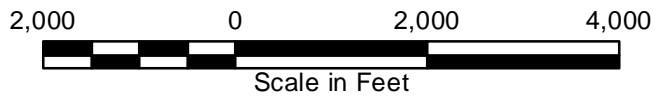


APPENDIX B
Figures



SITE LOCATION
WIN 021870.00
LAT: 44.82431
LONG: -68.40576

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MAINE DEPARTMENT OF TRANSPORTATION
SITE LOCATION MAP
 HALF MILE POND BROOK BRIDGE #6246 REPLACEMENT
 ROUTE 9 OVER HALF MILE POND BROOK
 AMHERST, MAINE
 WIN 021870.00

NOTE:
 SITE LOCATION MAP PREPARED FROM
 ESRI ArcGIS ONLINE AND DATA PARTNERS
 INCLUDING USGS AND © 2007 NATIONAL
 GEOGRAPHIC SOCIETY.

Job No.	17-0079	Scale	1:24000
Date:	09/01/2017	Sheet	1

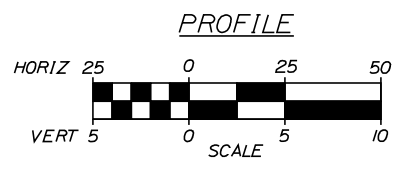
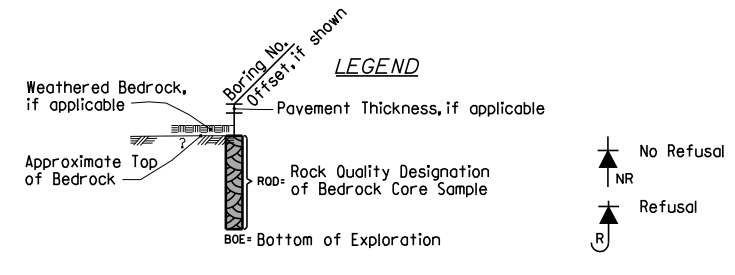
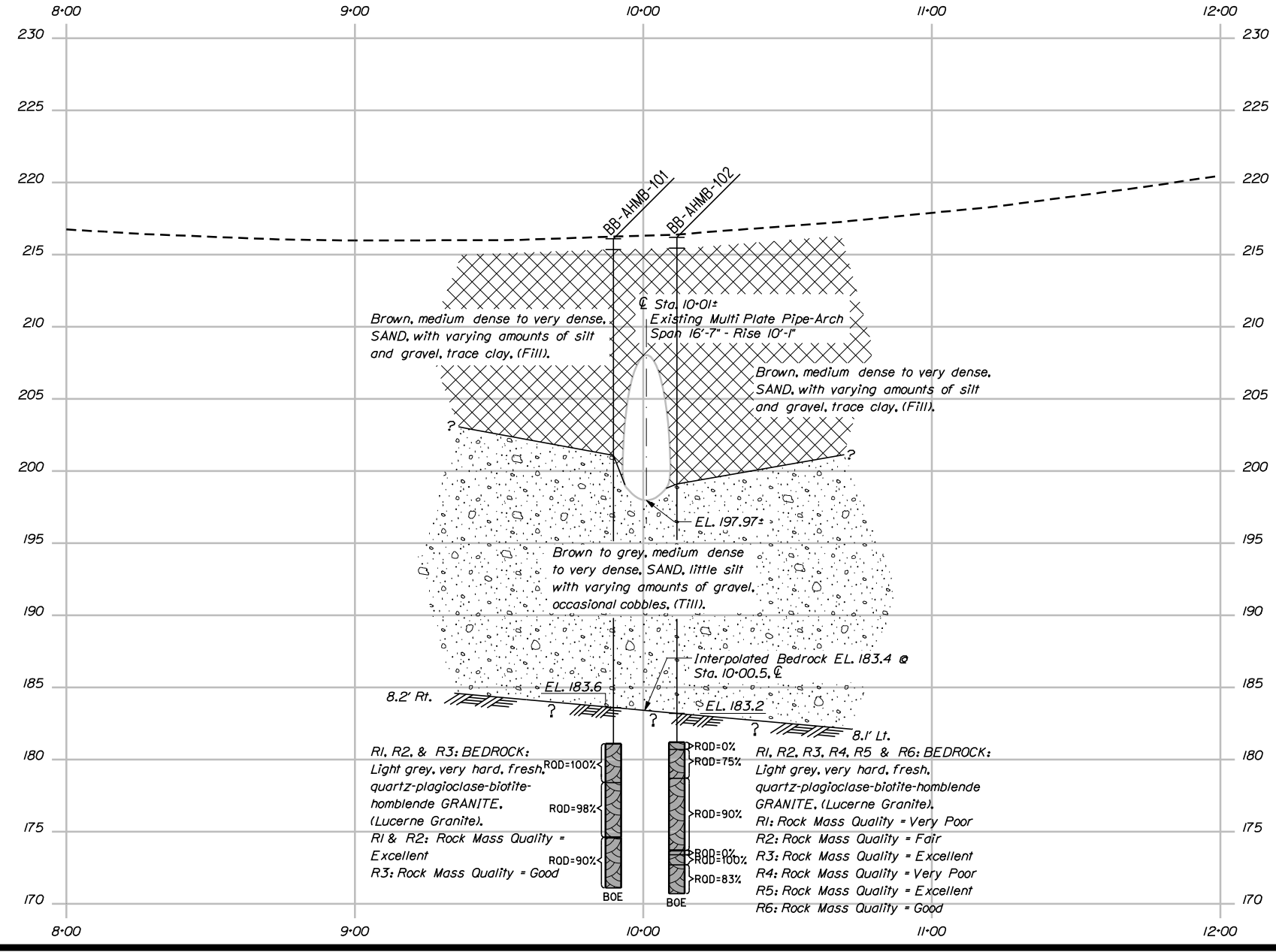
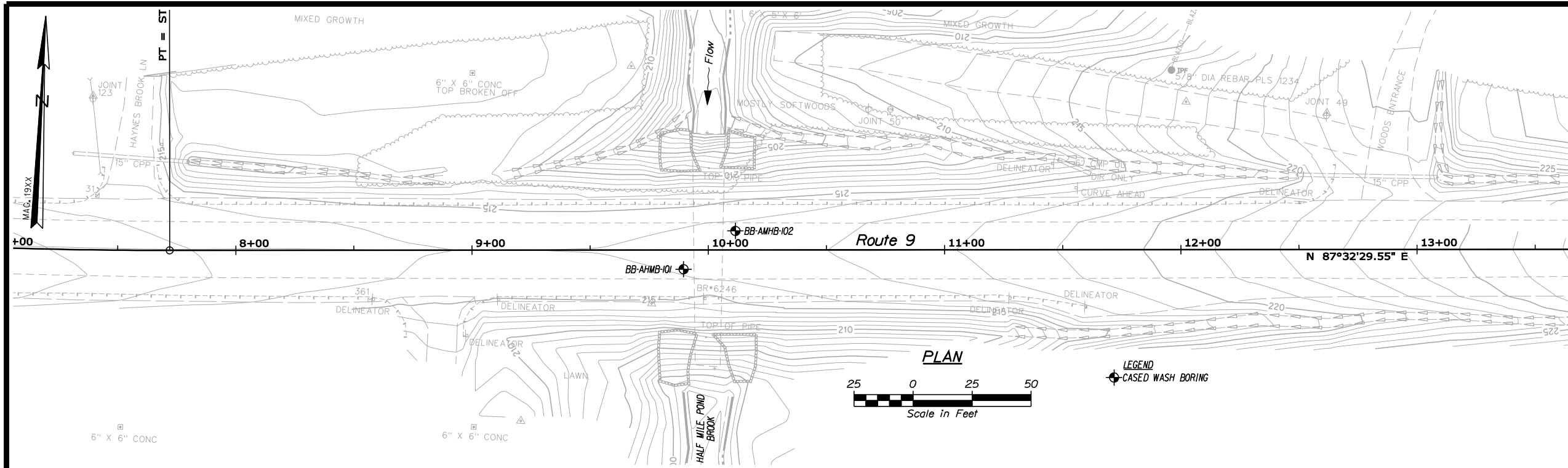
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Date: 5/17/2017

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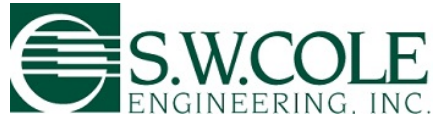
Division: GEOTECH

Filename: ... \GEOTECH\MSTA\006_BLP&ISPL.dgn



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	
STP-2187(000)		BRIDGE NO. 6246	
WIN		21870.00	
BRIDGE PLANS			
PROJ. MANAGER	M. WIGHT	BY	DATE
DESIGN-DETAILED	T. HELM	M.A.P.	
CHECKED-REVIEWED	M.S.T. PIERRE	T. WHITE	MAY 2017
DESIGNS-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
SIGNATURE		P.E. NUMBER	
DATE			
HALF MILE POND BROOK BRIDGE HALF MILE POND BROOK HANCOCK COUNTY AMHERST			
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE			
SHEET NUMBER			
6			
OF X			



APPENDIX C
Boring Logs & Key to Soil and Rock Descriptions and Terms

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																											
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)																										
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW Well-graded gravels, gravel-sand mixtures, little or no fines.	trace	0 - 10																										
		(little or no fines)	GP Poorly-graded gravels, gravel sand mixtures, little or no fines.	little	11 - 20																										
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM Silty gravels, gravel-sand-silt mixtures.	some	21 - 35																										
		CLEAN SANDS	SW Well-graded sands, gravelly sands, little or no fines	adjective (e.g. sandy, clayey)	36 - 50																										
		(little or no fines)	SP Poorly-graded sands, gravelly sand, little or no fines.	TERMS DESCRIBING DENSITY/CONSISTENCY Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value). <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr><td>Very loose</td><td>0 - 4</td></tr> <tr><td>Loose</td><td>5 - 10</td></tr> <tr><td>Medium Dense</td><td>11 - 30</td></tr> <tr><td>Dense</td><td>31 - 50</td></tr> <tr><td>Very Dense</td><td>> 50</td></tr> </tbody> </table>		Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50														
		Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)																												
Very loose	0 - 4																														
Loose	5 - 10																														
Medium Dense	11 - 30																														
Dense	31 - 50																														
Very Dense	> 50																														
SANDS WITH FINES (Appreciable amount of fines)	SM Silty sands, sand-silt mixtures	Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated. <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value (blows per foot)</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr> <tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr> <tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr> <tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr> <tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr> <tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr> </tbody> </table>		Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	Hard	>30	over 4000	Indented by thumbnail with difficulty
Consistency of Cohesive soils	SPT N-Value (blows per foot)			Approximate Undrained Shear Strength (psf)	Field Guidelines																										
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates																												
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Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort																												
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail																												
Hard	>30	over 4000	Indented by thumbnail with difficulty																												
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)	ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table border="1"> <thead> <tr> <th>Rock Mass Quality</th> <th>RQD (%)</th> </tr> </thead> <tbody> <tr><td>Very Poor</td><td>≤25</td></tr> <tr><td>Poor</td><td>26 - 50</td></tr> <tr><td>Fair</td><td>51 - 75</td></tr> <tr><td>Good</td><td>76 - 90</td></tr> <tr><td>Excellent</td><td>91 - 100</td></tr> </tbody> </table>			Rock Mass Quality	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100														
		Rock Mass Quality				RQD (%)																									
		Very Poor				≤25																									
	Poor	26 - 50																													
Fair	51 - 75																														
Good	76 - 90																														
Excellent	91 - 100																														
CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																															
OL Organic silts and organic silty clays of low plasticity.																															
SILTS AND CLAYS (liquid limit greater than 50)	MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Desired Rock Observations (in this order, if applicable): Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, etc.) Hardness (very hard, hard, mod. hard, etc.) Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.) Geologic discontinuities/jointing: -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -tightness (tight, open, or healed) -infilling (grain size, color, etc.) Formation (Waterville, Ellsworth, Cape Elizabeth, etc.) RQD and correlation to rock mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))																													
	CH Inorganic clays of high plasticity, fat clays.																														
	OH Organic clays of medium to high plasticity, organic silts.																														
HIGHLY ORGANIC SOILS	Pt Peat and other highly organic soils.																														
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (sand, silty sand, clay, etc., including portions - trace, little, etc.) Gradation (well-graded, poorly-graded, uniform, etc.) Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic) Structure (layering, fractures, cracks, etc.) Bonding (well, moderately, loosely, etc.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level				Sample Container Labeling Requirements: WIN Blow Counts Bridge Name / Town Sample Recovery Boring Number Date Sample Number Personnel Initials Sample Depth																											
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information																															

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS		Project: Half Mile Pond Brook Bridge #6246 carries Route 9 over Half Mile Pond Brook	Boring No.: BB-AHMB-101
		Location: Amherst, Maine	WIN: 21870.00
Driller: S. W. Cole Explorations, LLC	Elevation (ft.): 216.1 ft	Auger ID/OD: 5" Solid Stem	
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon	
Logged By: N. Strout	Rig Type: Diedrich D50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 2/23/2017-2/24/2017	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"	
Boring Location: Sta. 9+89.6, 8.2 ft RT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 22.0' (on 2/24/2017)	

Hammer Efficiency Factor: 0.873	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) q_u = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	215.35	9" of Pavement		
	1D	24/22	1.00 - 3.00	25/29/30/64	59	86				1D(A) 1.0-2.2 ft: Brown, frozen, very dense, SAND, some gravel, little silt, trace clay, (Fill). 1D(B) 2.2-3.0 ft: Brown, frozen, very dense, Silty SAND, trace gravel, (Fill).	#271087 A-1-b, SC-SM WC=7.2%	
5	2D	24/18	5.00 - 7.00	11/16/8/9	24	35	65			2D(A) 5.0-6.0 ft: Similar to above except moist and dense. 2D(B) 6.0-7.0 ft: Brown, moist, dense, SAND, some silt, little gravel, (Fill).	#271088 A-2-4, SC-SM WC=11.8%	
10	3D	24/17	10.00 - 12.00	17/18/17/19	35	51	85			Brown, moist, hard, SILT, little sand, (Fill).	#271089 A-4, CL WC=18.6%	
15	4D	24/15	15.00 - 17.00	21/31/30/19	61	89	74		201.10	Brown, moist, very dense, SAND, little gravel, little silt, (Glacial Till).	#271087 A-1-b, SM WC=7.9%	
20	MD	5/0	20.00 - 20.42	100/5"	--		OPEN			Boulder from 20.3 to 22.9 ft bgs.		
25										Place spun NW casing.		

Remarks:
 Auto-hammer SN #362
 Casing driven using auto-hammer (140#/30" drop)
 Water level recorded at start of day on 2/24/2017 within casing to 32.0 ft bgs.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Half Mile Pond Brook Bridge #6246 carries Route 9 over Half Mile Pond Brook Location: Amherst, Maine	Boring No.: BB-AHMB-101 WIN: 21870.00
--	---	--

Driller: S. W. Cole Explorations, LLC	Elevation (ft.): 216.1 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/23/2017-2/24/2017	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: Sta. 9+89.6, 8.2 ft RT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 22.0' (on 2/24/2017)

Hammer Efficiency Factor: 0.873	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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Definitions: R = Rock Core Sample, SSA = Solid Stem Auger, S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf), T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample, HSA = Hollow Stem Auger, S_u(lab) = Lab Vane Undrained Shear Strength (psf), WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt, RC = Roller Cone, q_p = Unconfined Compressive Strength (ksf), LL = Liquid Limit
 U = Thin Wall Tube Sample, WOH = Weight of 140lb. Hammer, N-uncorrected = Raw Field SPT N-value, PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt, Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 V = Field Vane Shear Test, PP = Pocket Penetrometer, N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 MV = Unsuccessful Field Vane Shear Test Attempt, WO1P = Weight of One Person, N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/(6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	5D	24/8	25.00 - 27.00	11/16/39/29	55	80					Brown, wet, very dense, Gravelly SAND, little silt, (Glacial Till). Frequent cobbles from 27 to 29 ft bgs.	
30	6D	11/5	30.00 - 30.92	29/100-5"	--						Similar to above.	
								183.60			Top of Bedrock at Elev. 183.6 ft. Advanced by rollercone through bedrock from 32.5 to 35 ft bgs.	
35	R1	32/32	35.00 - 37.67	RQD = 100%							R1:Bedrock: Light grey, phaneritic, pegmatitic, GRANITE, predominately quartz-plagioclase-biotite-hornblende, very hard, fresh, weakly foliated (gneissic) steep (70 to 80 degrees), joints are rough, low angle (20 degree), and tight, with remobilized quartz mineralization infilling along foliation stress planes, (Lucerne Granite). Rock Mass Quality = Excellent. R1:Core Times (min:sec) 35.0-36.0 ft (4:59) 36.0-37.0 ft (4:56) 37.0-37.7 ft (3:50) 100% Recovery	
	R2	46/45	37.70 - 41.53	RQD = 98%							R2:Bedrock: Similar to R1. Rock Mass Quality = Excellent. R2:Core Times (min:sec) 37.7-38.0 ft (1:17) 38.0-39.0 ft (4:59) 39.0-40.0 ft (5:34) 40.0-41.0 ft (5:02) 41.0-41.5 ft (2:37) 100% Recovery	
40	R3	42/43	41.50 - 45.00	RQD = 90%							R3:Bedrock: Similar to R1. Rock Mass Quality = Good. R3:Core Times (min:sec) 41.5-42.0 ft (2:31) 42.0-43.0 ft (4:42) 43.0-44.0 ft (5:00) 44.0-45.0 ft (5:03) 100% Recovery	
45								171.10			Bottom of Exploration at 45.00 feet below ground surface.	
50												

Remarks:
 Auto-hammer SN #362
 Casing driven using auto-hammer (140#/30" drop)
 Water level recorded at start of day on 2/24/2017 within casing to 32.0 ft bgs.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Half Mile Pond Brook Bridge #6246 carries Route 9 over Half Mile Pond Brook	Boring No.: BB-AHMB-102
	Location: Amherst, Maine	WIN: 21870.00

Driller: S. W. Cole Explorations, LLC	Elevation (ft.): 216.2 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/24/2017-2/28/2017	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: Sta. 10+11.6, 8.1 ft LT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 16.5' (on 2/28/2017)

Hammer Efficiency Factor: 0.873	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
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Definitions:
 D = Split Spoon Sample
 MD = Unsuccessful Split Spoon Sample Attempt
 U = Thin Wall Tube Sample
 MU = Unsuccessful Thin Wall Tube Sample Attempt
 V = Field Vane Shear Test, PP = Pocket Penetrometer
 MV = Unsuccessful Field Vane Shear Test Attempt
 R = Rock Core Sample
 SSA = Solid Stem Auger
 HSA = Hollow Stem Auger
 RC = Roller Cone
 WOH = Weight of 140lb. Hammer
 WOR/C = Weight of Rods or Casing
 WO1P = Weight of One Person
 S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
 S_{u(lab)} = Lab Vane Undrained Shear Strength (psf)
 q_p = Unconfined Compressive Strength (ksf)
 N-uncorrected = Raw Field SPT N-value
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value
 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
 N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
 T_v = Pocket Torvane Shear Strength (psf)
 WC = Water Content, percent
 LL = Liquid Limit
 PL = Plastic Limit
 PI = Plasticity Index
 G = Grain Size Analysis
 C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/(6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25						52/6"						
						155 SPUN						
30	MD	2/0	30.00 - 30.17	100-2"	--							
								183.20				
						OPEN						
35	R1	6/2.5	35.00 - 35.50	RQD = 0%								
	R2	24/24	35.50 - 37.50	RQD = 75%								
	R3	60/61	37.50 - 42.50	RQD = 90%								
40												
	R4	3.5/3.5	42.50 - 42.79	RQD = 0%								
	R5	8.5/8	42.80 - 43.51	RQD = 100%								
	R6	24/22	43.50 - 45.50	RQD = 83%								
45												
								170.70				
50												

Remarks:
 Auto-hammer SN #362
 Casing driven using auto-hammer (140#/30" drop).
 Water level recorded at start of day on 2/28/2017 within casing to 24.5 ft bgs.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Half Mile Pond Brook Bridge #6246 carries Route 9 over Half Mile Pond Brook Location: Amherst, Maine	Boring No.: BB-AHMB-102 WIN: 21870.00
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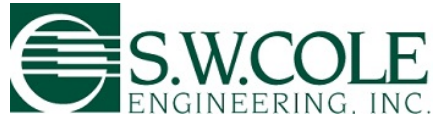
Driller: S. W. Cole Explorations, LLC	Elevation (ft.): 216.2 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/24/2017-2/28/2017	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: Sta. 10+11.6, 8.1 ft LT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 16.5' (on 2/28/2017)

Hammer Efficiency Factor: 0.873 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_u = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

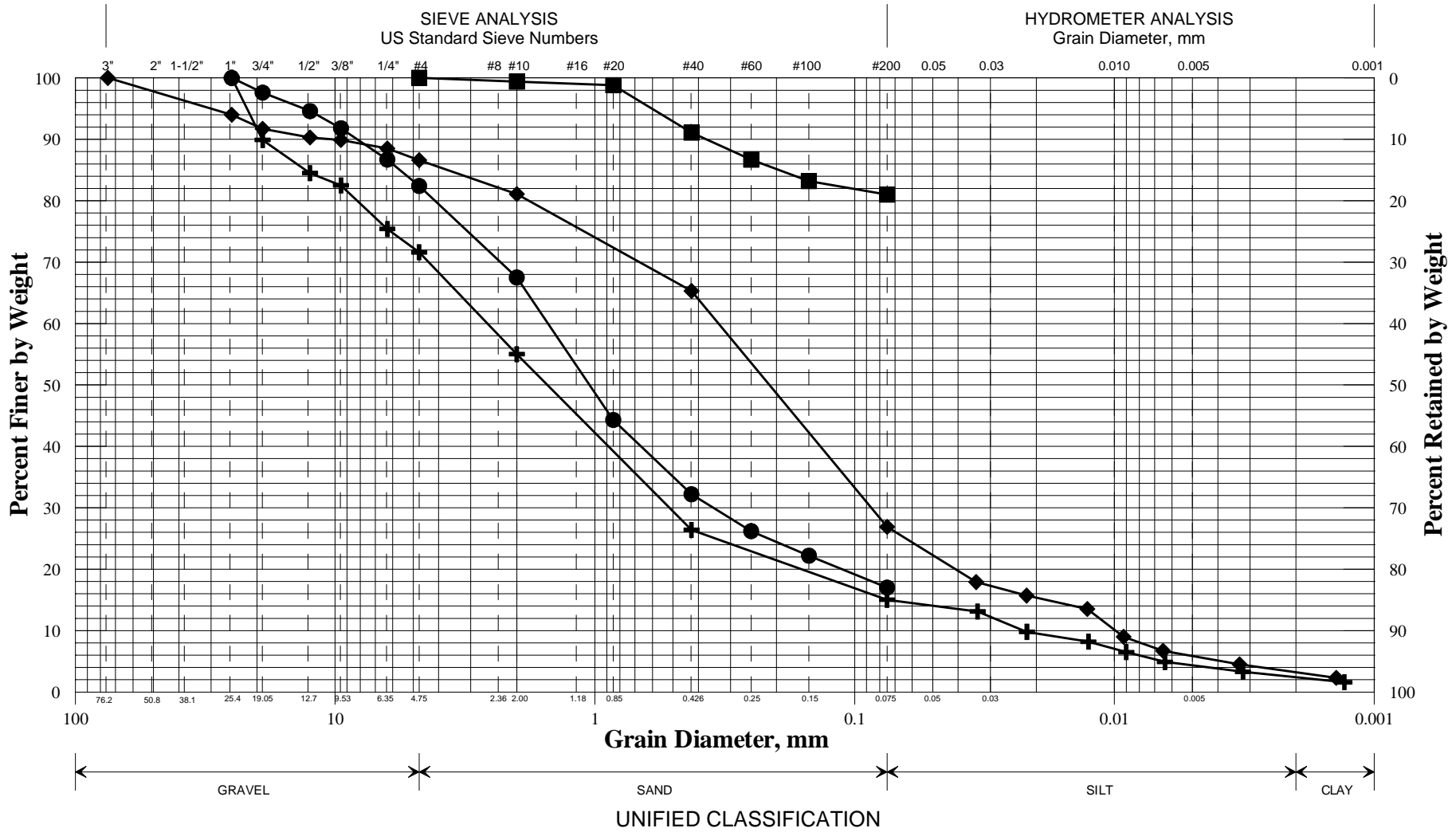
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50											R6:Core Times (min:sec) 43.5-44.5 ft (3:49) 44.5-45.5 ft (3:15) 92% Recovery _____45.50 Bottom of Exploration at 45.50 feet below ground surface.	
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												
63												
64												
65												
66												
67												
68												
69												
70												
71												
72												
73												
74												
75												

Remarks:
 Auto-hammer SN #362
 Casing driven using auto-hammer (140#/30" drop).
 Water level recorded at start of day on 2/28/2017 within casing to 24.5 ft bgs.



APPENDIX D
Laboratory Test Results

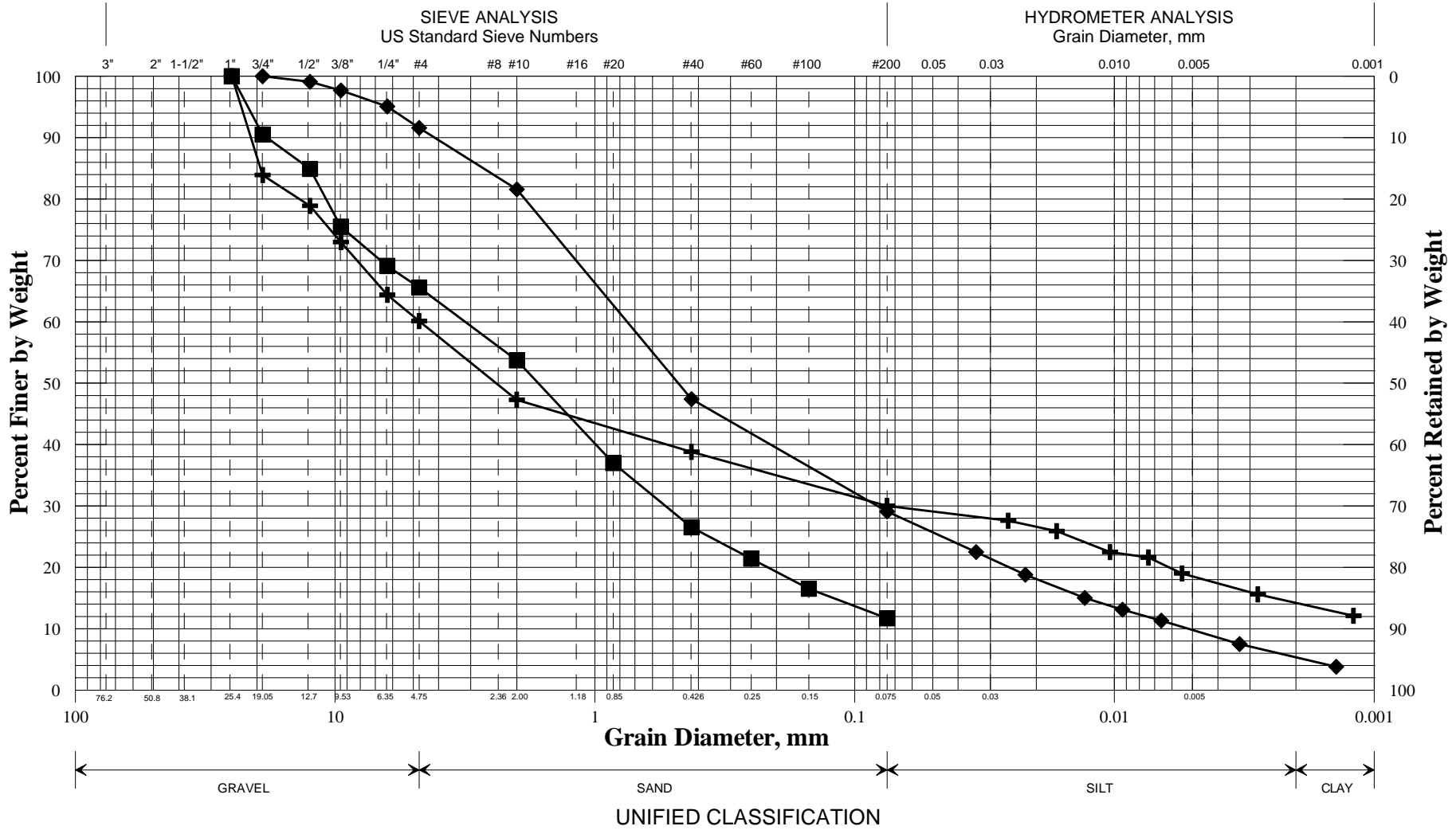
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AHMB-101/1DA	9+89.6	8.2 RT	1.0-2.2	SAND, some gravel, little silt, trace clay.	7.2			
◆	BB-AHMB-101/2DB	9+89.6	8.2 RT	6.0-7.0	SAND, some silt, little gravel, trace clay.	11.8			
■	BB-AHMB-101/3D	9+89.6	8.2 RT	10.0-12.0	SILT, little sand.	18.6			
●	BB-AHMB-101/4D	9+89.6	8.2 RT	15.0-17.0	SAND, little gravel, little silt.	7.9			
▲									
×									

WIN
021870.00
Town
Amherst
Reported by/Date
WHITE, TERRY A 4/7/2017

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AHMB-102/1D	10+11.6	8.1 LT	1.0-2.0	GRAVEL, little sand, little silt, little clay.	3.0			
◆	BB-AHMB-102/3D	10+11.6	8.1 LT	10.0-12.0	SAND, some silt, trace gravel, trace clay.	7.3			
■	BB-AHMB-102/4DC	10+11.6	8.1 LT	17.5-19.0	SAND, some gravel, little silt.	9.6			
●									
▲									
×									

WIN
021870.00
Town
Amherst
Reported by/Date
WHITE, TERRY A 4/10/2017



APPENDIX E
Calculations

Modulus of Subgrade Reaction

Reference: Bowles (1996) Foundaton Analysis and Design, 5th Ed.

Determine Fouldation and Soil Properties

Foundation Soil: med. dense to very dense, SAND, some gravel, little silt (SW-SM)

$\mu_s := 0.35$ Poisson's Ratio (Bowles Table 2-7, pg 123)
 typical value 0.3-0.4 - sand, gravelly sand

$N_{60} := 20$ N60 values near \pm EI 195 ft (anticipate subgrade elevation)
 range from 20, 23, 89 and refusal, use 20

$$N_{55} := N_{60} \cdot \frac{60}{55} = 22$$

Young's Modulus (from Bowles Table 5-6, pg 316)

$E_s := 600 \cdot \mathbf{kPa} \cdot (N_{55} + 6) + 2000 \mathbf{kPa}$ for Gravelly Sand with $N > 15$

$$E_s = 390.4 \mathbf{ksf}$$

$E_s := 500 \cdot \mathbf{kPa} \cdot (N_{55} + 15)$ for Sand (norm. consolidated)

$$E_s = 384.5 \mathbf{ksf}$$

Use $E_s := 385 \mathbf{ksf}$

Foundation Patameters:

$B := 25 \mathbf{ft}$ Foundation Width

$L_f := 98 \mathbf{ft}$ Assumed Length of Box Culvert

$D_f := 3 \mathbf{ft}$ Base Embedment Depth

$$B' := 0.5 \cdot B$$

$$L'_f := 0.5 \cdot L_f$$

Determine Steinbrenner Influence Factors

$H_s := 15 \mathbf{ft}$ Thickness of soil stratum

$\frac{D_f}{B} = 0.1$ Depth Ratio

$$M_{corner} := \frac{L_f}{B} = 3.9$$

$$M_{center} := \frac{L'_f}{B'} = 3.9$$

$$N_{corner} := \frac{H_s}{B} = 0.6$$

$$N_{center} := \frac{H_s}{B'} = 1.2$$

From Bowles Table 5-2 (pg 304 and 305)

$$I_{1_corner} := 0.051 \quad I_{1_center} := 0.148$$

$$I_{2_corner} := 0.097 \quad I_{2_center} := 0.124$$

$$I_{s_corner} := I_{1_corner} + \frac{1 - 2 \mu_s}{1 - \mu_s} \cdot I_{2_corner} = 0.096$$

$$I_{s_center} := I_{1_center} + \frac{1 - 2 \mu_s}{1 - \mu_s} \cdot I_{2_center} = 0.205$$

From Bowles Figure 5-7 (pg 303)

$$I_{F_corner} := 0.86$$

$$I_{F_center} := 0.93$$

$$E'_s := \frac{(1 - \mu_s^2)}{E_s}$$

$$k_{1_center} := \frac{1}{B' \cdot E'_s \cdot I_{s_center} \cdot I_{F_center}} = 106 \text{ pci}$$

$$k_{1_corner} := \frac{1}{B \cdot E'_s \cdot I_{s_corner} \cdot I_{F_corner}} = 123 \text{ pci}$$

From Das Principles of Foundation Engineering, 7th Ed. Eqn. 6.44

$$k_{s_center} := \frac{k_{1_center} \cdot \left(1 + 0.5 \frac{B'}{L'_f}\right)}{1.5} = 80 \text{ pci}$$

$$k_{s_corner} := \frac{k_{1_corner} \cdot \left(1 + 0.5 \frac{B}{L_f}\right)}{1.5} = 93 \text{ pci}$$

10.6.2.6—Bearing Resistance at the Service Limit State

10.6.2.6.1—Presumptive Values for Bearing Resistance C10.6.2.6.1

The use of presumptive values shall be based on knowledge of geological conditions at or near the structure site.

Unless more appropriate regional data are available, the presumptive values given in Table C10.6.2.6.1-1 may be used. These bearing resistances are settlement limited, e.g., 1.0 in., and apply only at the service limit state.

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	12–20	14
	Medium dense to dense	8–14	10
	Loose	4–12	6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense	8–12	8
	Medium dense to dense	4–8	6
	Loose	2–6	3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	6–10	6
	Medium dense to dense	4–8	5
	Loose	2–4	3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense	6–12	8
	Medium dense to dense	2–6	4
	Loose	1–2	1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	4–8	6
	Medium stiff to stiff	2–6	3
	Soft	1–2	1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the

Bearing Resistance of Precast Box Culvert

Foundation Soil Parameters: med. dense to very dense, SAND, some gravel, little silt (SW-SM)

$\gamma_{sat} := 125$ <i>pcf</i>	Saturated Unit Weight
$\gamma_{moist} := 120$ <i>pcf</i>	Moist Unit Weight
$\phi := 32$ <i>deg</i>	Undrained Friction Angle
$c_s := 0$ <i>psf</i>	Undrained Shear Strength
$\gamma_w := 62.4$ <i>pcf</i>	Unit Weight of Water

Foundation Parameters:

$B := 20$ <i>ft</i>	Foundation Width
$D_f := 2$ <i>ft</i>	Embedment Depth 2 ft below streambed or design scour elevation
$D_w := 0$ <i>ft</i>	Depth of Water Below Foundation

Nominal Bearing Resistance - Service Limit State

From AASHTO LRFD Table 10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material: fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)

Consistency in Place: medium dense to dense

Bearing Resistance Range: 4 to 8 ksf

Recommended Bear Resistance: 5 ksf

**For medium dense to very dense, SAND, some gravel, little silt (SW-SM)
Recommend Bearing Resistance of 5 ksf to limit settlement for Service Limit**

Nominal Bearing Resistance - Strength Limit State

Use Terzaghi Bearing Method for Strip Foundations since $L > B$

Reference: Bowles (1996) Foundaton Analysis and Design, 5th Ed.

Shape Factors for strip footing (From Table 4-1, page 220):

$$s_c := 1 \qquad s_\gamma := 1$$

Meyerhof Bearing Factors (From Table 4-4, page 223):

$$N_c := 35.47 \qquad N_q := 23.2 \qquad N_\gamma := 22.0$$

Calculated by: MAS
Date: May 17, 2017
Checked by: TJB
Date: May 18, 2017

Nominal Bearing Resistance (Terzaghi Equation, Bowles Table 4-1, page 220)

$$q := D_f \cdot (\gamma_{sat} - \gamma_w)$$

$$q = 125 \text{ psf}$$

$$q_n := c_s \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = 16.7 \text{ ksf} \quad \text{Nominal Bearing Resistance}$$

Factored Bearing Resistance

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

$$\varphi_b := 0.45$$

$$q_r := \varphi_b \cdot q_n$$

$$q_r = 7.5 \text{ ksf}$$

**Strength Limit Factored Bearing Resistance of 7.5 ksf
for 20-foot wide precast box foundation**

Calculated by: MAS
Date: May 17, 2017
Checked by: TJB
Date: May 18, 2017

Bearing Resistance of 3-sided Frame

Foundation Soil Parameters: med. dense to very dense, SAND, some gravel, little silt (SW-SM)

$\gamma_{sat} := 125$ *pcf* Saturated Unit Weight
 $\gamma_{moist} := 120$ *pcf* Moist Unit Weight
 $\phi := 32$ *deg* Undrained Friction Angle
 $c_s := 0$ *psf* Undrained Shear Strength
 $\gamma_w := 62.4$ *pcf* Unit Weight of Water

Foundation Parameters:

$B := \begin{bmatrix} 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{bmatrix}$ *ft* Foundation Width(s)

$D_f := 2$ *ft* Embedment Depth 2 ft below streambed or design scour elevation

$D_w := 0$ *ft* Depth of Water Below Foundation

Nominal Bearing Resistance - Service Limit State

From AASHTO LRFD Table 10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material: fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)

Consistency in Place: medium dense to dense

Bearing Resistance Range: 4 to 8 ksf

Recommended Bear Resistance: 5 ksf

**For medium dense to very dense, SAND, some gravel, little silt (SW-SM)
 Recommend Bearing Resistance of 5 ksf to limit settlement for Service Limit**

Nominal Bearing Resistance - Strength Limit State

Use Terzaghi Bearing Method for Strip Foundations since $L > B$

Reference: Bowles (1996) Foundaton Analysis and Design, 5th Ed.

Shape Factors for strip footing (From Table 4-1, page 220):

$s_c := 1$ $s_\gamma := 1$

Meyerhof Bearing Factors (From Table 4-4, page 223):

$N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Calculated by: MAS
 Date: May 17, 2017
 Checked by: TJB
 Date: May 18, 2017

Nominal Bearing Resistance (Terzaghi Equation, Bowles Table 4-1, page 220)

$$q := D_f \cdot (\gamma_{sat} - \gamma_w)$$

$$q = 125 \text{ psf}$$

$$q_n := c_s \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_n = \begin{bmatrix} 9.8 \\ 11.2 \\ 12.5 \\ 13.9 \\ 15.3 \end{bmatrix} \text{ ksf} \quad \text{for} \quad B = \begin{bmatrix} 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{bmatrix} \text{ ft} \quad \text{Nominal Bearing Resistance(s)}$$

Factored Bearing Resistance

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

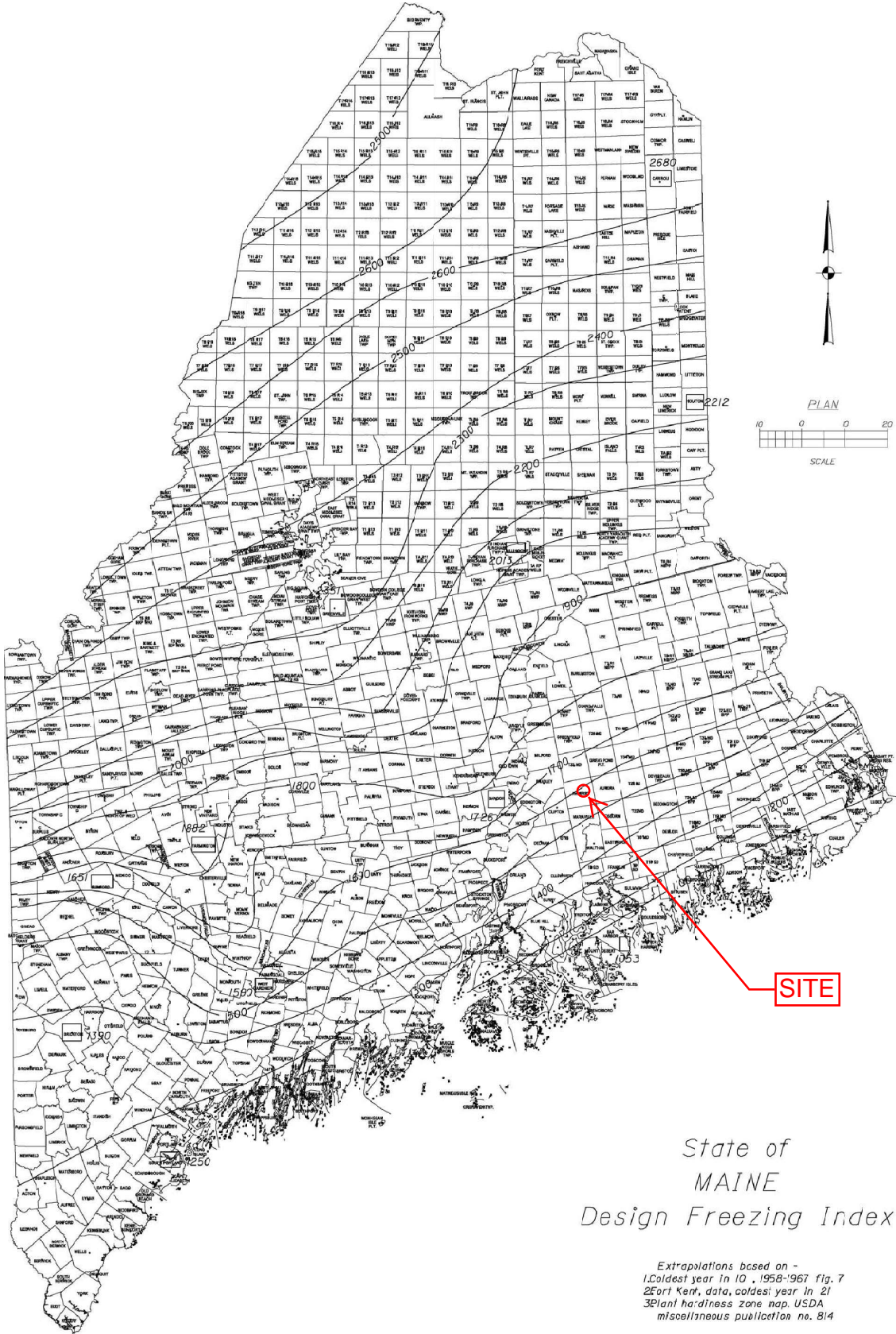
$$\varphi_b := 0.45$$

$$q_r := \varphi_b \cdot q_n$$

$$q_r = \begin{bmatrix} 4.4 \\ 5 \\ 5.6 \\ 6.3 \\ 6.9 \end{bmatrix} \text{ ksf} \quad \text{for} \quad B = \begin{bmatrix} 10 \\ 12 \\ 14 \\ 16 \\ 18 \end{bmatrix} \text{ ft}$$

Strength Limit Factored Bearing Resistance

Figure 5-1 Maine Design Freezing Index Map



State of
MAINE
Design Freezing Index

Extrapolations based on -
1) Coldest year in 10, 1958-1967 fig. 7
2) Fort Kent, data, coldest year in 21
3) Plant hardiness zone map, USDA
miscellaneous publication no. 814

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

CHAPTER 5 - SUBSTRUCTURES

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

Determine Seismic Site Classification per AASHTO LRFD Table C3.10.3.1-1 - Method B

Data From Boring BB-AHMB-101

Layer No.	Layer Description	Depth Range (ft)		N ₆₀ values recorded within layer								Average N ₆₀ value	Layer Thickness	d _r /N _i
		Top	End									N _i	d _i	
1	Fill	0	15	37	54							45.5	15	0.33
2	Till	15	32.5	94	100	85	100					94.8	17.5	0.18
3	Bedrock	32.5	100	100								100.0	67.5	0.68

- Notes:**
1. N₆₀ value for frozen soil ignored
 2. N₆₀ values > 100 taken as N=100
 3. N₆₀ value for bedrock taken as N=100

$\Sigma = 100 \quad 1.19$

N_{bar} = d_i/d_i/N_i = **84.1**
 Site Class **C**

Data From Boring BB-AHMB-102

Layer No.	Layer Description	Depth Range (ft)		N ₆₀ values recorded within layer								Average N ₆₀ value	Layer Thickness	d _r /N _i
		Top	End									N _i	d _i	
1	Fill	0	17.1	31	70	51						50.7	17.1	0.34
2	Till	17.1	33	25	22	100	100					61.8	15.9	0.26
4	Bedrock	33	100	100								100.0	67	0.67

- Notes:**
1. N₆₀ value for frozen soil ignored
 2. N₆₀ values > 100 taken as N=100
 3. N₆₀ value for bedrock taken as N=100

$\Sigma = 100 \quad 1.26$

N_{bar} = d_i/d_i/N_i = **79.1**
 Site Class **C**

USGS Design Maps Summary Report

User-Specified Input

Report Title AHMB
Thu May 18, 2017 15:39:24 UTC

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

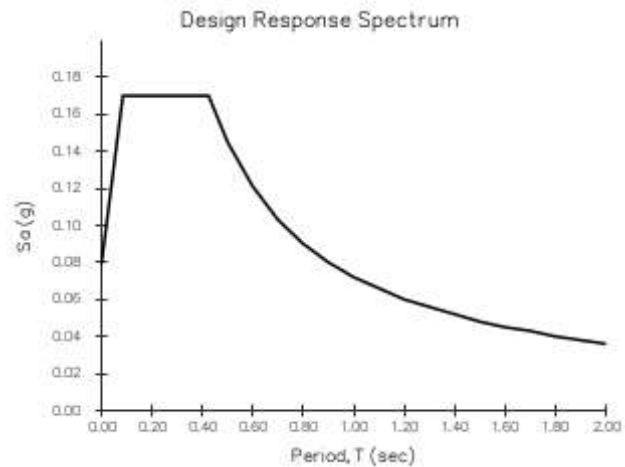
Site Coordinates 44.82431°N, 68.40576°W

Site Soil Classification Site Class C – “Very Dense Soil and Soft Rock”



USGS-Provided Output

PGA = 0.066 g	A_s = 0.079 g
S_s = 0.142 g	S_{DS} = 0.170 g
S₁ = 0.043 g	S_{D1} = 0.072 g



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.