# GEOTECHNICAL DESIGN REPORT

16-1454.1

September 21, 2017

## Geotechnical Engineering Services

Billings Bridge #2979 Rehabilitation
WIN 022618.00
Routes 117 and 119 over Little Androscoggin River
Paris, Maine

PREPARED FOR:
VHB, Inc.
Attention: Steven Hodgdon, P.E.
2 Bedford Farms Drive #200
Bedford, NH 03110

S.W.COLE ENGINEERING, INC.

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


PREPARED BY:
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555 Eastern Avenue
Augusta, ME 04330
T: (207) 626-0600


*www.swcole.com*

## TABLE OF CONTENTS

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VHB, Inc.
Attention: Steven Hodgdon, P.E.
2 Bedford Farms Drive #200
Bedford, NH 03110

Subject: Geotechnical Design Report
Geotechnical Engineering Services
Billings Bridge #2979 Rehabilitation
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Paris, Maine

Dear Steve:

In accordance with our Proposal dated May 16, 2017, we have reviewed the previous subsurface explorations made at the site as well as laboratory test results and geotechnical evaluations made by the Maine Department of Transportation (MaineDOT). This information was used in order to provide geotechnical design parameters and recommendations for foundations and earthwork associated with re-use of existing substructure and foundations. Our services were provided to support of the development and submission of the 100 percent Plans, Specifications and Estimate (PS&E) package to MaineDOT. The contents of this report are subject to the limitations in Appendix A.

**1.0 INTRODUCTION**

This Geotechnical Design Report (GDR) presents a summary of the subsurface explorations, laboratory testing (completed by others), results of our engineering evaluations, and geotechnical design recommendations for the proposed rehabilitation of the Billings Bridge in Paris, Maine (see *Site Location Map* attached in Appendix B).

**1.1 Site Conditions**

The existing Billings Bridge is located on East Main Street (Routes 117 and 119) at the crossing of Little Androscoggin River in Paris, Maine.

Based on the provided Historic Bridge Plans, we understand the existing structure was constructed in 1938 and consists of a ±162-foot long (end-to-end) by ±36-foot wide (out-to-out) two-span bridge with rigid frame approach/abutment. We understand the two main spans are painted steel girders with a concrete deck and the 20 foot west approach span consists of a buried concrete rigid frame. We understand the existing substructure consists of the concrete rigid frame at the west abutment (Abutment No. 1), concrete wall pier (Pier No. 1) and 30 degree skewed concrete gravity abutment at the east abutment (Abutment No. 2). Based on the provided Historic Construction Diary dated November-December 1938, we understand the existing foundations are constructed on bedrock. We understand bedrock excavation was made by blasting for the existing abutments. We understand, bedrock excavation was not performed for the existing pier.

## 1.2 Proposed Construction

Based on the Preliminary Design Report (PDR), we understand replacement and rehabilitation alternatives under consideration included:

- Replace existing structure with a 114-foot single-span bridge with steel plate girders supported on cast-in-place concrete abutments founded on bedrock. We understand the new abutments would be cast perpendicular to centerline (zero skew) and in front of the existing rigid frame (west abutment) and east abutment.
- Replace the superstructure and rehabilitate the substructure. We understand the rigid frame, pier and gravity abutment foundations would be widened upstream (north). The superstructure will be replaced with new steel girders and concrete deck with integral wearing surface.

We understand superstructure replacement and substructure rehabilitation was selected as the preferred option. We understand the horizontal alignment will shift about 1 foot upstream and vertical alignment will be within 1 foot of the existing alignment. We understand the proposed superstructure rehabilitation will include reuse and widening of the existing foundations. We understand widening will require the following modifications to the existing abutments, wingwalls and pier:

- Abutment No. 1: Widen the existing buried concrete rigid frame approximately 9 feet to the north and reinforce the rigid frame slab, as needed, with a composite reinforced concrete topping.

- NW Wingwall: Modify existing gravity wingwall with new toe extension, structural facing, and variable cantilever slab for the sidewalk
- Abutment No. 2: Widen the existing full height concrete gravity abutment approximately 7 feet to the north.
- NE Wingwall: Modify the existing gravity wingwall with new toe extension, structural facing, and raise the top of wall.
- Pier: Widen the existing pier footing, stem, and cap approximately 5 to 7 feet to the north and reconstruct the existing pier cap seat to accommodate new superstructure.

## 2.0 EXPLORATIONS AND TESTING

### 2.1 Explorations

Subsurface conditions were explored by drilling four test borings. Boring BB-PLAR-101 was drilled through the existing concrete bridge pier. Boring BB-PLAR-102 was drilled through the bridge deck about 2 feet in front of the face of the west abutment (Abutment No. 1). Borings BB-PLAR-103 and BB-PLAR-103A were drilled from the roadway about 6.5 to 8.5 feet behind the face of the existing east abutment (Abutment No. 2). The exploration locations are shown on the *Boring Location Plan* attached in Appendix B.

Test boring BB-PLAR-101 was drilled on November 7, 2016 by MaineDOT using a CME 45C drill rig. Test borings BB-PLAR-102, BB-PLAR-103 and BB-PLAR-103A were drilled between February 20 and 21, 2017 by S. W. Cole Explorations, LLC (S.W.COLEX) a division of S.W.COLE using a CME 850 drill rig.

The exploration locations were selected by MaineDOT and established in the field by S.W.COLE using taped measurements from existing site features. The 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

Boring BB-PLAR-101 was drilled through the existing concrete pier using rock coring techniques. Boring BB-PLAR-102 was drilled through the bridge deck in front of Abutment No. 1 using cased-wash boring and rock coring techniques. Borings BB-PLAR-103 and BB-PLAR-103A were drilled in the roadway behind Abutment No. 2

using a combination of solid-stem auger, cased-wash boring and rock coring techniques.

Soil samples were typically obtained at 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) methods with rope and cathead with safety hammer. The N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.6 to the raw field N-values. The hammer efficiency factor (0.6) and both the raw field N-value and corrected N-value ($N_{60}$) are shown on the boring logs.

Soil 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 with soil classification and identification. Laboratory testing was performed on disturbed SPT samples obtained during the explorations. Soil 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. Rock core laboratory testing was performed by GeoTesting Express, Inc. in Acton, Massachusetts. Laboratory testing included two standard grain size analyses (AASHTO T27/T11), two natural water content tests (AASHTO T265) and two unconfined compressive strength tests (ASTM D7012 Method D). Laboratory test results are shown on the boring logs in Appendix B and 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 Norway Quadrangle, Maine (Open-File No. 08-74, MGS 2008), surficial geologic units within the site vicinity consists of stream alluvium (sand, gravel, silt and organic sediment) and river outwash (sand and gravel). 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 carboniferous muscovite granite with abundant metasedimentary inclusions (Bedrock Geologic Map of Maine, MGS 1985). The observed bedrock is generally consistent with the mapped bedrock geology.

## 3.2 Soil and Bedrock

The test borings encountered a soils profile generally consisting of fill overlying alluvium/river outwash sand and gravel overlying bedrock. An *Interpretive Subsurface Profile* is attached in Appendix B. The principal strata encountered at each substructure are summarized below. Refer to the attached logs for more detailed information regarding the subsurface findings at the exploration locations.

### 3.2.1 Abutment No. 1

Boring BB-PLAR-102 was made through the bridge deck about 2 feet in front of the face of Abutment No. 1 and encountered stream alluvium or river outwash to a depth of about 5.5 feet (±Elevation (El.) 325.6 feet) overlying bedrock. The bedrock was cored 10 feet at BB-PLAR-102.

Based on observed soil cuttings, the native soils consisted of brown, GRAVEL, little sand with cobbles. No sampling was performed in this deposit.

### 3.2.2 Pier

Boring BB-PLAR-101 was cored through the existing concrete pier overlying bedrock. The bedrock was cored 3.7 feet at BB-PLAR-101.

### 3.2.3 Abutment No. 2

Borings BB-PLAR-103 and BB-PLAR-103A were made from the roadway behind Abutment No. 2 and encountered fill material overlying concrete gravity abutment overlying bedrock. Boring BB-PLAR-103 was terminated at a depth of about 19 feet when the rock core cutting shoe broke. Wood (possibly old wooden concrete formwork) and wood fragments were encountered in boring BB-PLAR-103 at a depth of about 15.5 to 17 feet bgs. Below the fill, BB-PLAR-103A encountered abutment concrete overlying bedrock. The bedrock surface at boring BB-PLAR-103A was at approximately El. 331.9 feet. The bedrock was cored 11.4 feet at BB-PLAR-103A.

<u>Fill</u>: The fill extended to depths of about 18.5 to 19 feet below ground surface (bgs), corresponding to about Elevation (El.) 331.5 to 331.9 feet. The fill generally consisted of:

- Brown, SAND, little gravel, trace to little silt, and
- Black, SAND, some gravel, trace silt with wood fragments

The fill was generally medium dense to very dense with SPT $N_{60}$ values ranging from 19 blows per foot (bpf) to 50 blows for 5 inches (sampler refusal).

### 3.2.4 Bedrock

The bedrock was generally classified as white to light grey, hard, fresh to slightly weathered, gneiss with varying amounts of quartz, feldspar, garnet and bands of biotite-mica. Joints were generally low angle to moderately dipping, very close to moderately close spacing and tight. The following table summarizes the approximate depths to bedrock, corresponding top of bedrock elevations and Rock Quality Designation (RQD) at the boring location were encountered.

| Boring Number (Substructure) | Approximate Depth to Bedrock (feet) | Approximate Bedrock Elevation (feet) | RQD (RMQ) |
|---|---|---|---|
| BB-PLAR-102 (Abutment No. 1) | 5.5 (weathered) 6.5 (intact) | 325.6 (weathered) 324.6 (intact) | 66 to 85% (Fair to Good) |
| BB-PLAR-101 (Pier) | 21.3 | 329.2 | 65% (Fair) |
| BB-PLAR-103A (Abutment No. 2) | 18.6 | 331.9 | 50 to 97% (Poor to Excellent) |

RQD values for the bedrock generally ranged from 50 to 97 percent correlating to a Rock Mass Quality (RMQ) of poor to excellent.

## 3.3 Groundwater

The soils encountered at the borings were moist to wet from the ground surface. The measured water levels within the borings immediately after drilling were about 15 feet at BB-PLAR-102, 19 feet at BB-PLAR-103 and 5 feet at BB-PLAR-103A. It should be noted that water was introduced during drilling therefore, water levels indicated may not represent stabilized ground water conditions. 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, changes in site use and the water level of Little Androscoggin River.

## 4.0 GEOTECHNICAL EVALUATION AND RECOMMENDATIONS

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

### 4.1 Bedrock Removal and Bedrock Subgrade Preparation

Construction activities will likely include construction of cofferdams and earth support systems to support the approach fills and control stream flow during construction of concrete seals and spread footings for the abutments, wingwalls and pier foundations. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for the foundation widening.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the abutments and pier are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil to expose sound, intact bedrock. The final bearing surface shall be solid. If the bedrock surface is observed to slope steeper than 4H:1V at the subgrade elevation in any direction, the bedrock shall be benched to create level steps or excavated to be completely level. Excavation of highly sloped and loose fractured bedrock material shall be made using all conventional excavation methods (digging bucket, ripper tooth, hoe-ramming) possible in attempt to create a level steps or be completely level. Based on the proximity to existing foundations and structures, we recommend bedrock excavation by blasting be avoided. Anchors or dowels may also be designed and employed to improve sliding resistance where the prepared bedrock surface is steeper than 4H:1V in any direction. The bottom of footing or concrete seal elevation may vary based on the presence of fractured bedrock and the variability of the bedrock surface.

We anticipate portions of the abutment and pier excavations will be submerged. The contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Engineer in accordance with project plans and specifications.

The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete. The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete or concrete seal.

It is anticipated that groundwater will seep from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

## 4.2 Abutment, Wingwall and Pier Reuse and Design

The abutments, wingwalls and pier shall be evaluated for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and designed for all relevant strength, service and extreme limit states. In addition, the pier shall be designed to transmit the loads on the superstructure and the loads acting on the pier itself into the foundation.

### 4.2.1 Strength Limit State Design

The design of abutments, wingwalls and concrete piers founded on spread footings bearing on bedrock or on concrete seals overlying bedrock at the strength limit state shall consider bearing resistance, eccentricity (overturning) and failure by sliding and concrete structural failure. Additionally, a modified strength limit state analysis should be performed for the pier foundation that includes the ice pressures specified in MaineDOT BDG Section 3.9 Ice Loads.

For spread footings or concrete seals founded on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the base width.

### 4.2.2 Service Limit State Design

For the service limit state, a resistance factor, $\phi$, of 1.0 shall be used to assess spread footing design for settlement, horizontal movement and bearing resistance. The overall stability of foundations are typically investigated at the Service I Load Combination and a resistance factor, $\phi$, of 0.65. Shear failure along adversely oriented joint surfaces in the rock mass below the foundations is not anticipated, therefore, global stability was not evaluated.

### 4.2.2 Extreme Limit State Design

Extreme limit state design checks for abutments, wingwalls and pier shall include bearing resistance, eccentricity (overturning), failure by sliding and structural failure with respect to extreme event load conditions relating to seismic forces, hydraulic events and ice. Resistance factors, $\phi$, for the extreme limit state shall be taken as 1.0 with the exception of bearing resistance for which a resistance factor of 0.8 shall be used. LRFD Figures C11.5.6-1 and C11.5.6-2 illustrate the typical load factors to produce the extreme factored effect for bearing resistance and sliding and eccentricity.

The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

For scour protection of spread footings or concrete seals, construct the spread footings or concrete seals directly on bedrock surfaces cleaned and free of all weathered, loose and potentially erodible or scourable rock. With these precautions, strength and extreme limit state designs do not need to consider rock scour for the proposed foundations.

## 4.3 Bearing Resistance and Eccentricity

The existing foundations shall be evaluated to ensure that they will continue to meet current LRFD standards against bearing capacity failure after superstructure replacement and substructure rehabilitation. The widened spread footings shall be proportioned to provide stability against bearing capacity failure.

Application of permanent and transient load combinations and applicable load factors are specified in LRFD Article 11.5.6. Based on LRFD Figure 11.6.3.2-2, the stress distribution at the abutments may be assumed to be a triangular or trapezoidal distribution over the effective base.

For abutment, wingwall and pier footings founded on competent, sound bedrock we recommend the following factored bearing resistances.

| Limit State | Bearing Resistance Factor $\varphi_b$ | Factored Bearing Resistance (ksf) | LRFD Reference |
|---|---|---|---|
| Service | 1.0 | 20.0 | Article 10.5.5.1 |
| Strength | 0.45 | 19.3 | Table 10.5.5.2.2-1 |
| Extreme | 0.8 | 34.2 | Article C11.5.8 |

LRFD Figures C11.5.6-2 and C11.5.6-4 (2016 Interim Revisions) illustrate the typical load factors to produce the strength and extreme factored conditions for evaluating eccentricity. Based on LRFD Article 11.6.3.3, the location of the resultant force for eccentricity evaluation shall fall within the middle nine-tenths (9/10) of the foundation base for foundations bearing on rock.

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as 0.3f'c. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

## 4.4 Sliding Resistance

The following table shows the resistance factors, $\varphi_\tau$, for sliding analyses of cast-in-place spread footings on bedrock.

| Limit State | Sliding Resistance Factor $\varphi_\tau$ | Reference |
|---|---|---|
| Strength | 0.8 | LRFD Article C10.5.5.2.2 |
| Service | 1.0 | LRFD Article 10.5.5.1 |
| Extreme | 1.0 | LRFD Article 10.5.5.3.3 |

Passive earth pressures due to the presence of soils in front of the abutments, wingwalls and pier shall be neglected in the sliding analysis.

Based on the provided Historic Construction Diary and borings BB-PLAR 101 and BB-PLAR-103A, the existing footings were cast directly on the bedrock. Sediment at the bedrock-concrete interface was not observed in the test borings. Therefore, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing.

For bedrock subgrade prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.7 at the bedrock-concrete seal interface.

Based on MaineDOT BDG Section 5.2.2, anchorage of the footing to a concrete seal, if used, is required. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. Anchorage of concrete seals to bedrock may also be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

## 4.5 Earth Pressure and Surcharge

### 4.5.1 Earth Pressure

The abutments and wingwalls should be designed for active earth pressure over the wall height unless restrained from movement. Walls restrained from movement should be designed for at-rest active earth pressure over the wall height. For design of gravity and semi-gravity walls backfilled with granular soil and drained (e.g. no hydrostatic pressures), we recommend the following earth pressure coefficients:

- Active Earth Pressure Coefficient, $k_a = 0.28$
- At-rest Earth Pressure Coefficient, $k_o = 0.47$

The resultant earth pressure is orientated at an angle δ of 21.33 degrees from a perpendicular line to the wall back-face, where δ is the angle of friction between the abutment backfill soil and the wall back-face.

Based on MaineDOT BDG Section 3.6.1, the designer may assume Soil Type 4 for the backfill material with the following soil properties:

- Internal Friction Angle, $\phi = 32$ degrees
- Total Unit Weight, $\gamma = 125$ pcf

### 4.5.2 Surcharge Pressure

Lateral earth pressure due to construction surcharge or live load surcharge is required per MaineDOT BDG Section 3.6.8 for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5.

The live load surcharge on wing walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($h_{eq}$) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ($h_{eq}$) based on the following:

| Abutment Height (feet) | Equivalent Height of Soil, $h_{eq}$ (feet) |
|---|---|
| 5 | 4.0 |
| 10 | 3.0 |
| ≥20 | 2.0 |

Abutment and wingwall modifications and design shall include a drainage system to ensure that drainage of water behind the structure is maintained. Drainage behind the structures shall be in accordance with MaineDOT BDG Section 5.4.1.4 Drainage.

## 4.6 Settlement

Proposed approach embankment widening at the bridge approaches will be constructed on granular soils overlying bedrock. Placement of the necessary fill will result in negligible densification of the underlying soils and elastic settlement of the embankments. Settlement is anticipated to occur during and immediately after construction of the embankments. Post-construction settlement will be minimal and anticipated to be less than ½ inch..

Any settlement of bridge abutments and pier will be due to elastic compression of the bedrock mass, and is anticipated to be less than ½ inch.

## 4.7 Frost

It is anticipated that the abutment, wingwall and pier spread footings will be founded directly on bedrock or mud slab on bedrock. For foundations on bedrock, heave due to frost is not a design concern therefore requirements for minimum depth of embedment are not necessary.

However, foundations placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. Based on the MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, the design freezing index for the Paris, Maine area is approximately 1,450 freezing degree-days. Based on Section 5.2.1 of the MaineDOT BDG and assuming a water content of 10% for the granular fills, the maximum seasonal frost penetration is estimated to be approximately 6.7 feet. Considering this, we recommend foundations constructed on granular fill be founded with least 6.7 feet of soil cover to provide frost protection.

## 4.8 Seismic Design

Seismic site class was evaluated in accordance with LRFD Section 3.10.3.1 Method B (average N-value method). An N-value of 100 bpf was assumed for the profile below the refusal surface. Based on the subsurface information, the average N-value fell between 50 and 100 bpf corresponding to a Site Class C as defined in LRFD 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 (Site Class C) and site coordinates, the software provides the recommended LRFD Response Spectrum for a 7% probability of exceedance in 75 years. The results for the project site are summarized below:

| **Recommended Seismic Design Parameters**[1] | |
|---|---|
| Site Class | C |
| PGA | 0.092 g |
| $S_s$ | 0.184 g |
| $S_1$ | 0.048 g |
| $F_{pga}$ | 1.2 |
| $F_a$ | 1.2 |
| $F_v$ | 1.7 |
| $A_s$ | 0.111 g |
| $S_{DS}$ | 0.221 g |
| $S_{D1}$ | 0.082 g |
| Seismic Zone (LRFD Table 3.10.6-1) | Zone 1 |

NOTE: Site Coordinates: N44.222389, W70.509500

[1] U.S. Geological Survey, Seismic Design Map, , accessed July 19, 2017
https://earthquake.usgs.gov/designmaps/us/application.php

**4.9 Scour and Riprap**

For scour protection of abutment, wingwall and pier footings, place the bottom of concrete seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock.

Bridge and channel soil slopes above the soil-bedrock interface shall be armored with 3 feet of riprap. Riprap shall conform to MaineDOT Standard Specification 703.26 “Plain and Hand Laid Riprap” and shall be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a 1 foot layer of MaineDOT Standard Specification 703.19 “Granular Borrow Material for Underwater Backfill” and a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

## 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.**

STATE OF MAINE
MICHAEL A. ST. PIERRE No. 12519
LICENSED
PROFESSIONAL ENGINEER

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

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

MAS:ejb/tjb

# APPENDIX A
# LIMITATIONS

This report has been prepared for the exclusive use of VHB, Inc. for specific application to the Billings Bridge #2979 Rehabilitation carrying Route 117 over Little Androscoggin River (MaineDOT WIN 022618.00) in Paris, 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.

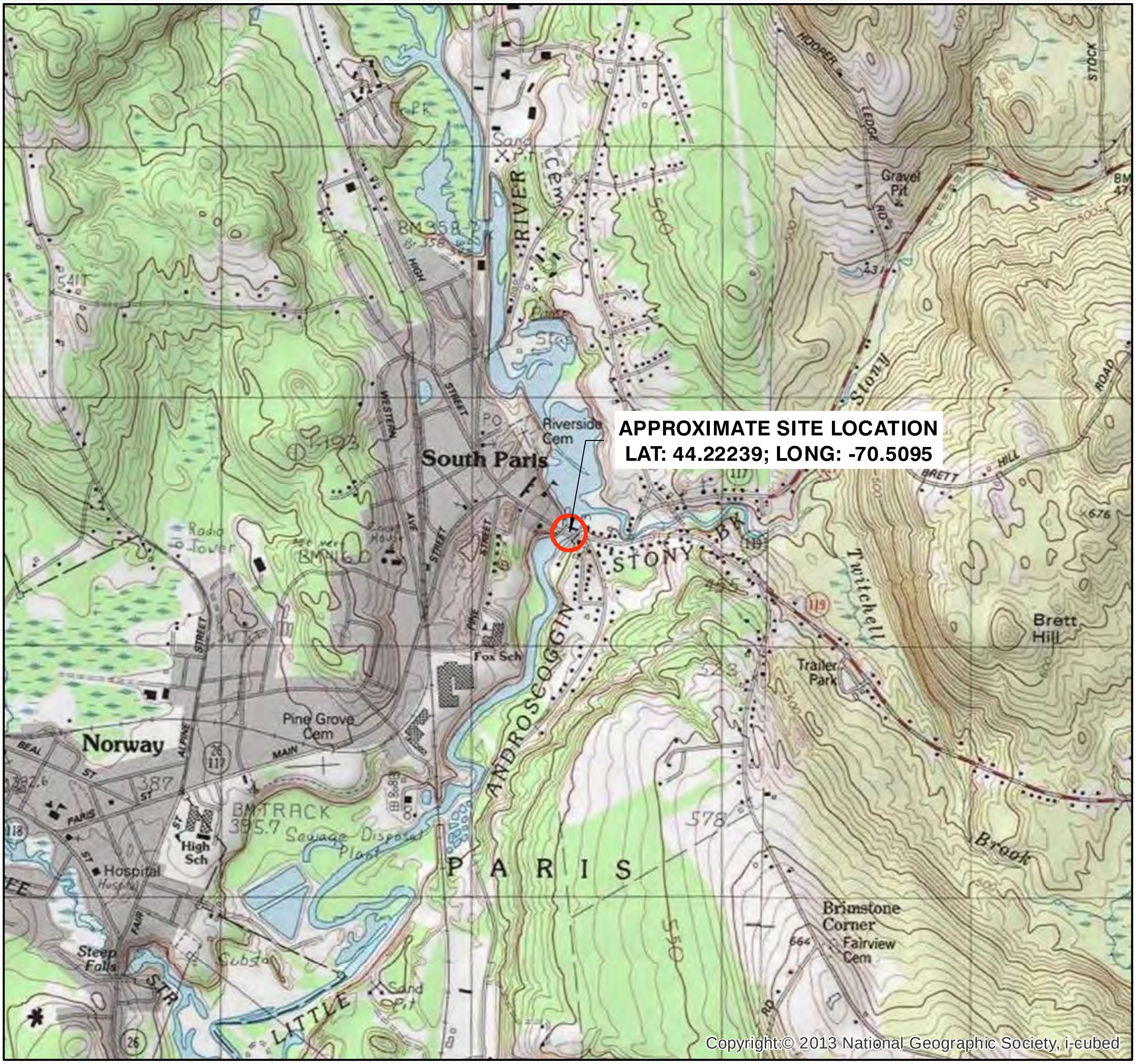
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations 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.

S.W.COLE ENGINEERING, INC.

# APPENDIX B
## Figures




2,000 0 2,000 4,000

Scale in Feet

S.W.COLE ENGINEERING, INC.

VHB, INC.

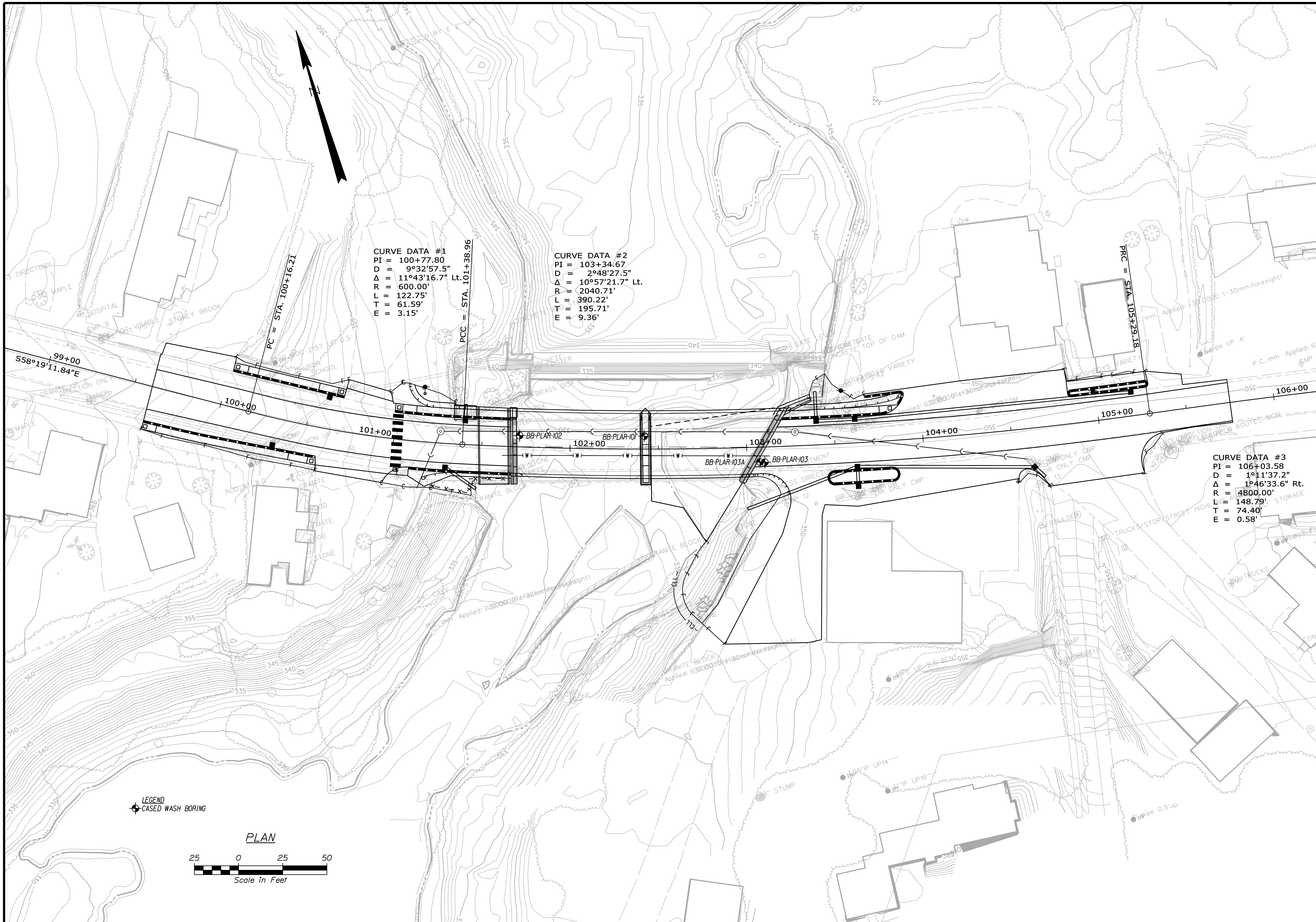
## SITE LOCATION MAP

BILLINGS BRIDGE #2979 REHABILITATION
WIN 022618.00
ROUTE 117 OVER LITTLE ANDROSCOGGIN RIVER
PARIS, MAINE

| | | | |
|---|---|---|---|
| Job No. | 16-1454.1 | Scale | 1:24000 |
| Date: | 07/25/2017 | Sheet | 1 |

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

R:\2016\16-1454.1\GIS\MXDs\16-1454.1 SLM.mxd, 7/25/2017 2:35:56 PM, 1:24,000, Filename: ...\00\geotech\msta\024_BLP1.dgn  Division: GEOTECH  Username: Terry.White  Date:9/21/2017




STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

022618.00

BRIDGE NO. 2979

WIN
22618.00

BRIDGE PLANS

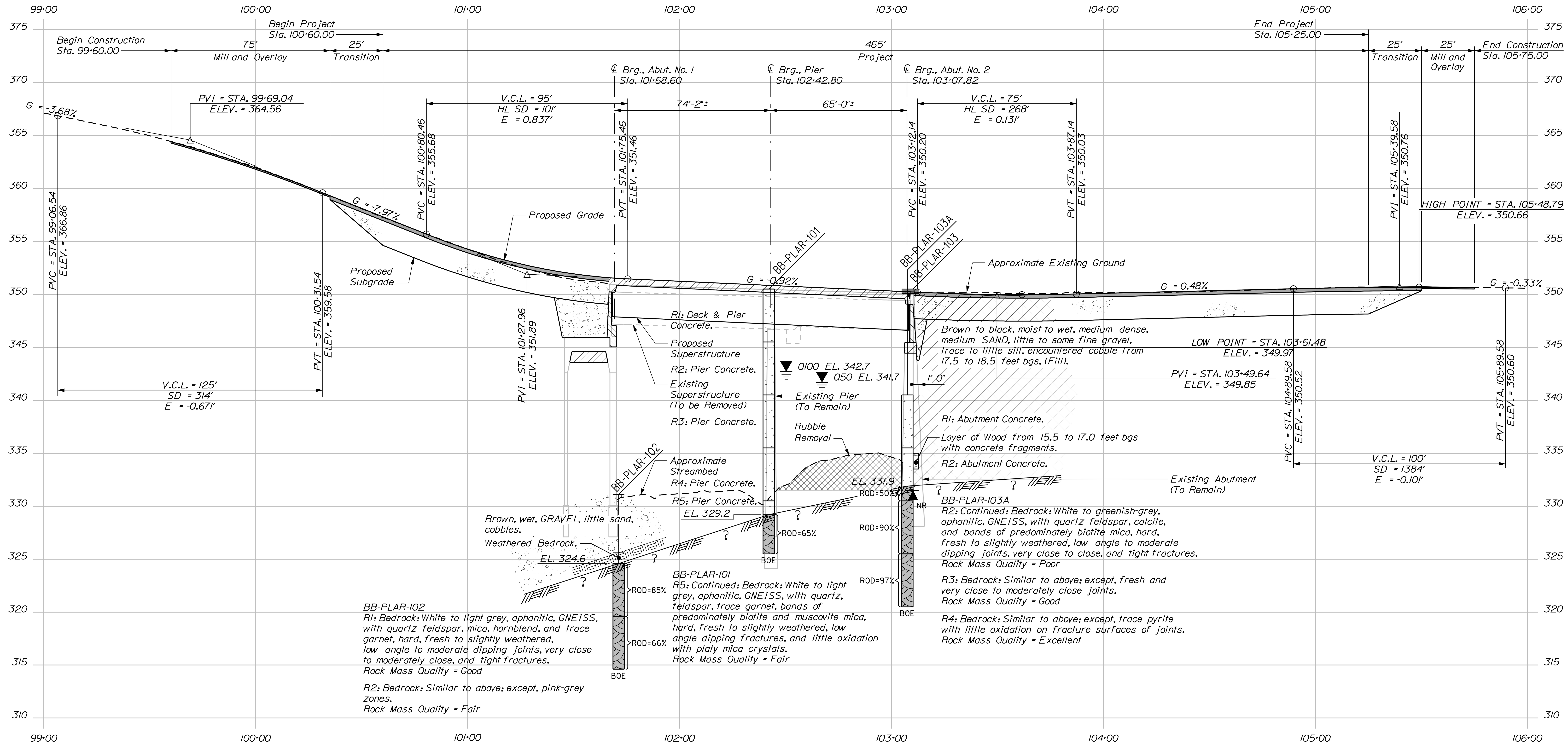
| | | BY | DATE | SIGNATURE |
|---|---|---|---|---|
| PROJ. MANAGER | | | | |
| DESIGN-DETAILED | | | | |
| CHECKED-REVIEWED | | | | |
| DESIGN2-DETAILED2 | | M.ST.PIERRE | T.WHITE | SEP 2017 |
| DESIGN3-DETAILED3 | | | | |
| REVISIONS 1 | | | | P.E. NUMBER |
| REVISIONS 2 | | | | |
| REVISIONS 3 | | | | |
| REVISIONS 4 | | | | DATE |
| FIELD CHANGES | | | | |

BILLINGS BRIDGE
LITTLE ANDROSCOGGIN RIVER
PARIS OXFORD COUNTY

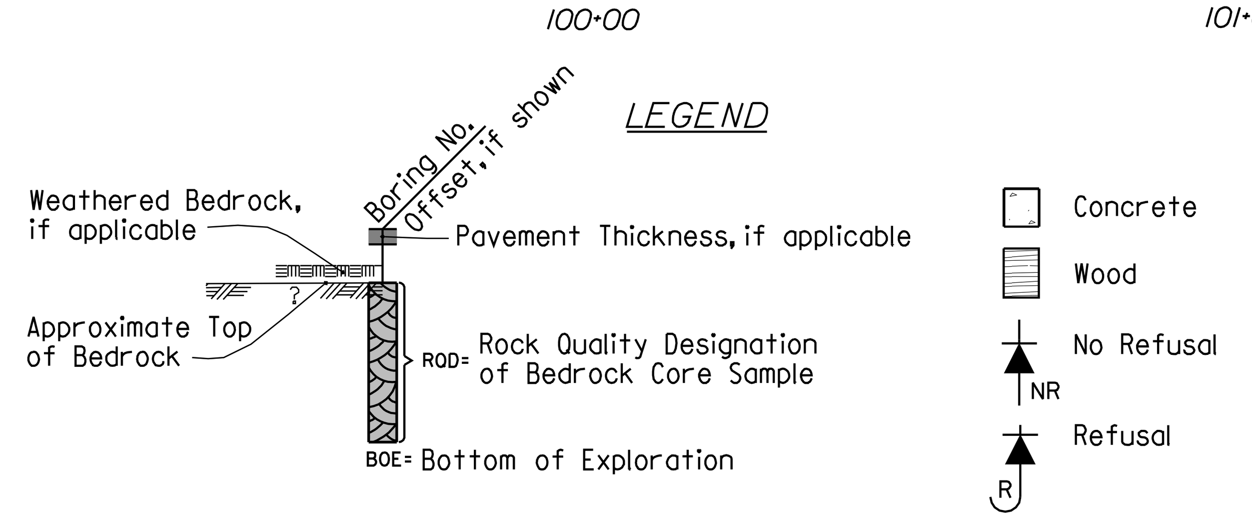
BORING LOCATION PLAN

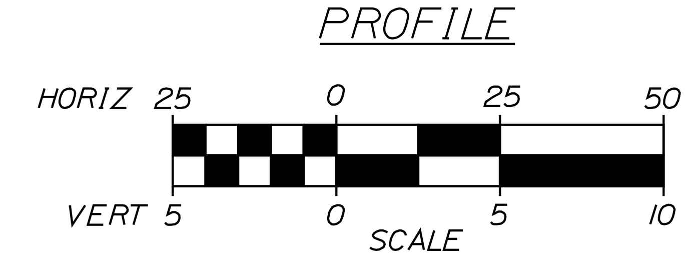
SHEET NUMBER
24
OF 46

Filename: ...\00\geotech\msta\025_ISP1.dgn Division: GEOTECH Username: Terry.White Date:9/21/2017




PROFILE




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

022618.00

WIN 22618.00

BRIDGE NO. 2979

BRIDGE PLANS

| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | | |
| DESIGN-DETAILED | | |
| CHECKED-REVIEWED | M.ST.PIERRE | |
| DESIGN2-DETAILED2 | T.WHITE | SEP 2017 |
| DESIGN3-DETAILED3 | | |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

SIGNATURE

P.E. NUMBER

DATE

BILLINGS BRIDGE
LITTLE ANDROSCOGGIN RIVER
PARIS OXFORD COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
25
OF 46

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

## UNIFIED SOIL CLASSIFICATION SYSTEM

| MAJOR DIVISIONS | | | GROUP SYMBOLS | TYPICAL NAMES |
|---|---|---|---|---|
| COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size) | GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size) | CLEAN GRAVELS (little or no fines) | GW | Well-graded gravels, gravel-sand mixtures, little or no fines. |
| | | | GP | Poorly-graded gravels, gravel sand mixtures, little or no fines. |
| | | GRAVEL WITH FINES (Appreciable amount of fines) | GM | Silty gravels, gravel-sand-silt mixtures. |
| | | | GC | Clayey gravels, gravel-sand-clay mixtures. |
| | SANDS (more than half of coarse fraction is smaller than No. 4 sieve size) | CLEAN SANDS (little or no fines) | SW | Well-graded sands, gravelly sands, little or no fines |
| | | | SP | Poorly-graded sands, gravelly sand, little or no fines. |
| | | SANDS WITH FINES (Appreciable amount of fines) | SM | Silty sands, sand-silt mixtures |
| | | | SC | Clayey sands, sand-clay mixtures. |
| FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size) | SILTS AND CLAYS (liquid limit less than 50) | | ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity. |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. |
| | | | OL | Organic silts and organic silty clays of low plasticity. |
| | SILTS AND CLAYS (liquid limit greater than 50) | | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | | | CH | Inorganic clays of high plasticity, fat clays. |
| | | | OH | Organic clays of medium to high plasticity, organic silts. |
| | HIGHLY ORGANIC SOILS | | Pt | Peat and other highly organic soils. |

**Desired Soil Observations (in this order, if applicable):**

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

## MODIFIED BURMISTER SYSTEM

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

## TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

**Rock Quality Designation (RQD):**

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock 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))

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

**Sample Container Labeling Requirements:**

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

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-101
**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 350.5 | **Auger ID/OD:** | N/A |
| **Operator:** | T. Daggett | **Datum:** | NAVD88 | **Sampler:** | N/A |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | N/A |
| **Date Start/Finish:** | 11/7/2016; 09:00-14:30 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 102+42.0, 7.3 feet Lt. | **Casing ID/OD:** | NW (3/3.5 inches) | **Water Level*:** | Not Observed |
| **Hammer Efficiency Factor:** | 0.943 | **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 | N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | R1 | 60/60 | 0.00 - 5.00 | | | | NQ2 | | | R1: DECK AND PIER CONCRETE | |
| 5 | R2 | 60/60 | 5.00 - 10.00 | | | | | | | R2: PIER CONCRETE | |
| 10 | R3 | 60/60 | 10.00 - 15.00 | | | | | | | R3: PIER CONCRETE | |
| 15 | R4 | 60/60 | 15.00 - 20.00 | | | | | | | R4: PIER CONCRETE | |
| 20 | R5 | 60/60 | 20.00 - 25.00 | | | | | | | R5: PIER CONCRETE | |
| | | | | RQD = 65% | | | | 329.20 | | 21.30 Top of Bedrock at Elevation 329.2 feet. R5: Continued: Bedrock: White to light grey, aphanitic, GNEISS, with quartz, feldspar, trace garnet, bands of predominately biotite and muscovite mica, hard, fresh to slightly weathered, low angle dipping fractures, and little oxidation with platy mica crystals. Rock Mass Quality = Fair | UCT $q_p$ = 7, 409 psi |
| 25 | | | | | | | | | | | |

**Remarks:**

-drilled through 10-inch-thick concrete deck
-top of pier was approximately 4 feet beneath the top of the deck
-existing pier footing is founded on bedrock

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.



**Boring No.:** BB-PLAR-101

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-101
**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 350.5 | **Auger ID/OD:** | N/A |
| **Operator:** | T. Daggett | **Datum:** | NAVD88 | **Sampler:** | N/A |
| **Logged By:** | B. Wilder | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | N/A |
| **Date Start/Finish:** | 11/7/2016; 09:00-14:30 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 102+42.0, 7.3 feet Lt. | **Casing ID/OD:** | NW (3/3.5 inches) | **Water Level*:** | Not Observed |
| **Hammer Efficiency Factor:** | 0.943 | **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 | | | | | | | | 325.50 | | 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:**

-drilled through 10-inch-thick concrete deck
-top of pier was approximately 4 feet beneath the top of the deck
-existing pier footing is founded on bedrock

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:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-102
**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | S.W. Cole Explorations, LLC | **Elevation (ft.)** | 331.1 | **Auger ID/OD:** | N/A |
| **Operator:** | J. Lee | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | E. Walker | **Rig Type:** | CME 850 | **Hammer Wt./Fall:** | 140 lbs/30 inches |
| **Date Start/Finish:** | 2/20/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 101+71.2, 6.4 feet Lt. | **Casing ID/OD:** | HW (4/4.5 inches) | **Water Level*:** | 19 feet bgs |
| **Hammer Efficiency Factor:** | 0.60 | **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 | N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | 47/6" | | | Brown, wet, GRAVEL, little sand, cobbles. RC through cobble from 19.5 to 20.6 feet bgs. | |
| | | | | | | | 43 | | | | |
| | | | | | | | 58 | | | | |
| | | | | | | | 140 | | | | |
| | | | | | | | OPEN | | | | |
| 5 | 1D | 5/5 | 5.50 - 5.92 | 50-5" | - - | | | 325.60 | | 5.50 Weathered bedrock. RC through weathered bedrock from 24.5 to 25.5 feet bgs. | |
| | R1 | 60/58.5 | 6.50 - 11.50 | RQD = 85% | | | NQ2 | 324.60 | | 6.50 Top of Bedrock at Elevation 324.6 feet. R1: Bedrock: White to light grey, aphanitic, GNEISS, with quartz feldspar, mica, hornblend, and trace garnet, hard, fresh to slightly weathered, low angle to moderate dipping joints, very close to moderately close, and tight fractures. Rock Mass Quality = Good R1: Core Times (min:sec) 6.5-7.5 feet (4:08) 7.5-8.5 feet (4:21) 8.5-9.5 feet (4:11) 9.5-10.5 feet (4:12) 10.5-11.5 feet (4:18) | UCT $q_p$ = 17, 962 psi |
| 10 | | | | | | | | | | | |
| | R2 | 60/58 | 11.50 - 16.50 | RQD = 66% | | | | | | R2: Bedrock: Similar to R1 except with pink-grey zones. Rock Mass Quality = Fair R2: Core Times (min:sec) 11.5-12.5 feet (3:31) 12.5-13.5 feet (3:36) 13.5-14.5 feet (3:43) 14.5-15.5 feet (3:18) 15.5-16.5 feet (3:16) | |
| 15 | | | | | | | | | | | |
| | | | | | | | | 314.60 | | 16.50 **Bottom of Exploration at 16.50 feet below ground surface.** | |
| 20 | | | | | | | | | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

-bgs = below existing ground surface (bridge deck)
-Reinforced concrete deck was 10.5-inches-thick
-Unsupported HW casing through bridge deck to Elevation 331.1 feet (bottom of river)

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:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-103
**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | S.W. Cole Explorations, LLC | **Elevation (ft.)** | 350.5 | **Auger ID/OD:** | 5-inch-diameter Solid Stem |
| **Operator:** | J. Lee | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | E. Walker | **Rig Type:** | CME 850 | **Hammer Wt./Fall:** | 140 lbs/30 inches |
| **Date Start/Finish:** | 2/21/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 103+10.0, 8.7 feet Rt. | **Casing ID/OD:** | HW (4/4.5 inches) | **Water Level*:** | 15 feet bgs |
| **Hammer Efficiency Factor:** | 0.60 | **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 | N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 350.08 | | 5-inch-thick layer of pavement. 0.42 | |
| | 1D | 11/11 | 1.00 - 1.92 | 14/50-5" | - - | | | | | Brown, medium SAND, little gravel, trace silt, (Fill). | |
| 5 | 2D | 22/10 | 5.00 - 6.83 | 2/4/22/50-4" | 26 | 26 | | | | Brown, moist, medium dense, medium SAND, little fine gravel, little silt, (Fill). | G#271117 A-1-b, SW-SM WC=3.5% |
| 10 | 3D | 24/20 | 10.00 - 12.00 | 10/7/12/20 | 19 | 19 | 121 | | | Brown, moist, medium dense, medium SAND, little silt, little gravel, (Fill). | |
| | | | | | | | 118 | | | | |
| | | | | | | | 149 | | | | |
| | | | | | | | 166 | | | | |
| | | | | | | | 131 | | | | |
| 15 | 4D | 24/18 | 15.00 - 17.00 | 10/10/16/24 | 26 | 26 | 118 | | | 4D(A): 15-15.5 feet bgs: Black, wet, medium dense, medium SAND, some fine gravel, trace silt, (Fill). 4D(B):15.5-17 feet bgs: Wood | G#271118 A-1-b, SW WC=14.9% |
| | | | | | | | 122 | | | Mixed concrete and wood fragments. | |
| | 5D | 20/14 | 17.00 - 18.67 | 18/100/45/50-2" | - - | | NQ2 | | | Cobble from 17.5 to 18.5 feet bgs. | |
| | R1 | 12/12 | 17.50 - 18.50 | | | | | | | Broke RC drilling through cobble. Abandoned boring. | |
| | | | | | | | | 331.50 | | 19.00 **Bottom of Exploration at 19.00 feet below ground surface.** | |
| 20 | | | | | | | | | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

-bgs = below existing ground surface

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:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-103A

**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | S.W. Cole Explorations, LLC | **Elevation (ft.)** | 350.5 | **Auger ID/OD:** | 5-inch-diameter Solid Stem |
| **Operator:** | J. Lee | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | E. Walker | **Rig Type:** | CME 850 | **Hammer Wt./Fall:** | 140 lbs/30 inches |
| **Date Start/Finish:** | 2/21/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 103+07.3, 8.4 feet Rt. | **Casing ID/OD:** | NW (3/3.5"), HW (4/4.5") | **Water Level*:** | 5 feet bgs |
| **Hammer Efficiency Factor:** | 0.60 | **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 | N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SSA | 350.08 | | 5-inch-thick layer of pavement. 0.42 | |
| | | | | | | | | | | Augered to 5 feet bgs and set HW casing. Soil samples not retrieved. Soils similar to boring BB-PLAR-103 from 0 to 5.9 feet bgs, (Fill). | |
| 5 | | | | | | | NW | 344.60 | | 5.90 | |
| | | | | | | | | | | CONCRETE. Placed and spun NW casing to 10 feet bgs. | |
| 10 | R1 | 60/52 | 10.00 - 15.00 | | | | NQ2 | | | R1: ABUTMENT CONCRETE | |
| 15 | R2 | 60/59 | 15.00 - 20.00 | | | | | | | R2: ABUTMENT CONCRETE | |
| | | | | RQD = 50% | | | | 331.90 | | 18.60 | |
| | | | | | | | | | | Top of Bedrock at Elevation 331.9 feet.<br>R2: Bedrock: White to greenish-grey, aphanitic, GNEISS, with quartz feldspar, calcite, and bands of predominately biotite mica, hard, fresh to slightly weathered, low angle to moderate dipping joints, very close to close, and tight fractures.<br>Rock Mass Quality = Poor<br>R2: Core Times (min:sec)<br>18.6-19.0 feet (1:01)<br>19.0-20.0 feet (2:41) | |
| 20 | R3 | 60/60 | 20.00 - 25.00 | RQD = 90% | | | | | | R3: Bedrock: Similar to R2 except, fresh and very close to moderately close joints.<br>Rock Mass Quality = Good<br>R3: Core Times (min:sec) | |
| 25 | | | | | | | | | | | |

**Remarks:**

-bgs = below existing ground surface

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:** Billings Bridge (No. 2979) carries Routes 117/119 over Little Androscoggin River
**Location:** Paris, Maine

**Boring No.:** BB-PLAR-103A

**WIN:** 22618.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | S.W. Cole Explorations, LLC | **Elevation (ft.)** | 350.5 | **Auger ID/OD:** | 5-inch-diameter Solid Stem |
| **Operator:** | J. Lee | **Datum:** | NAVD88 | **Sampler:** | Standard Split Spoon |
| **Logged By:** | E. Walker | **Rig Type:** | CME 850 | **Hammer Wt./Fall:** | 140 lbs/30 inches |
| **Date Start/Finish:** | 2/21/2017 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ2 (2-inch-diameter) |
| **Boring Location:** | Sta 103+07.3, 8.4 feet Rt. | **Casing ID/OD:** | NW (3/3.5"), HW (4/4.5") | **Water Level*:** | 5 feet bgs |
| **Hammer Efficiency Factor:** | 0.60 | **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. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | R4 | 60/60 | 25.00 - 30.00 | RQD = 97% | | | | | | 20.0-21.0 feet (1:52)<br>21.0-22.0 feet (2:46)<br>22.0-23.0 feet (2:35)<br>23.0-24.0 feet (2:49)<br>22.0-25.0 feet (3:05)<br>R4: Bedrock: Similar to R2 except, trace pyrite with little oxidation on fracture surfaces of joints.<br>Rock Mass Quality = Excellent<br>R4: Core Times (min:sec)<br>25.0-26.0 feet (3:16)<br>26.0-27.0 feet (4:18)<br>27.0-28.0 feet (3:40)<br>28.0-29.0 feet (2:42)<br>29.0-30.0 feet (2:54) | |
| 30 | | | | | | | | 320.50 | | 30.00<br>**Bottom of Exploration at 30.00 feet below ground surface.** | |
| 35 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |
| 45 | | | | | | | | | | | |
| 50 | | | | | | | | | | | |

**Remarks:**

-bgs = below existing ground surface

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.

S.W.COLE ENGINEERING, INC.

# APPENDIX D
# Laboratory Test Results

State of Maine - Department of Transportation

## Laboratory Testing Summary Sheet

# Town(s): Paris Work Number: 22618.00

| Boring & Sample Identification Number | Station (Feet) | Offset (Feet) | Depth (Feet) | Reference Number | G.S.D.C. Sheet | W.C. % | L.L. | P.I. | Classification | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Unified | AASHTO | Frost |
| BB-PLAR-103, 2D | 103+10 | 8.7 Rt. | 5.0-6.83 | 271117 | 1 | 3.5 | | | SW-SM | A-1-b | 0 |
| BB-PLAR-103, 4D/A | 103+10 | 8.7 Rt. | 15.0-15.5 | 271118 | 1 | 14.9 | | | SW | A-1-b | 0 |

**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.**

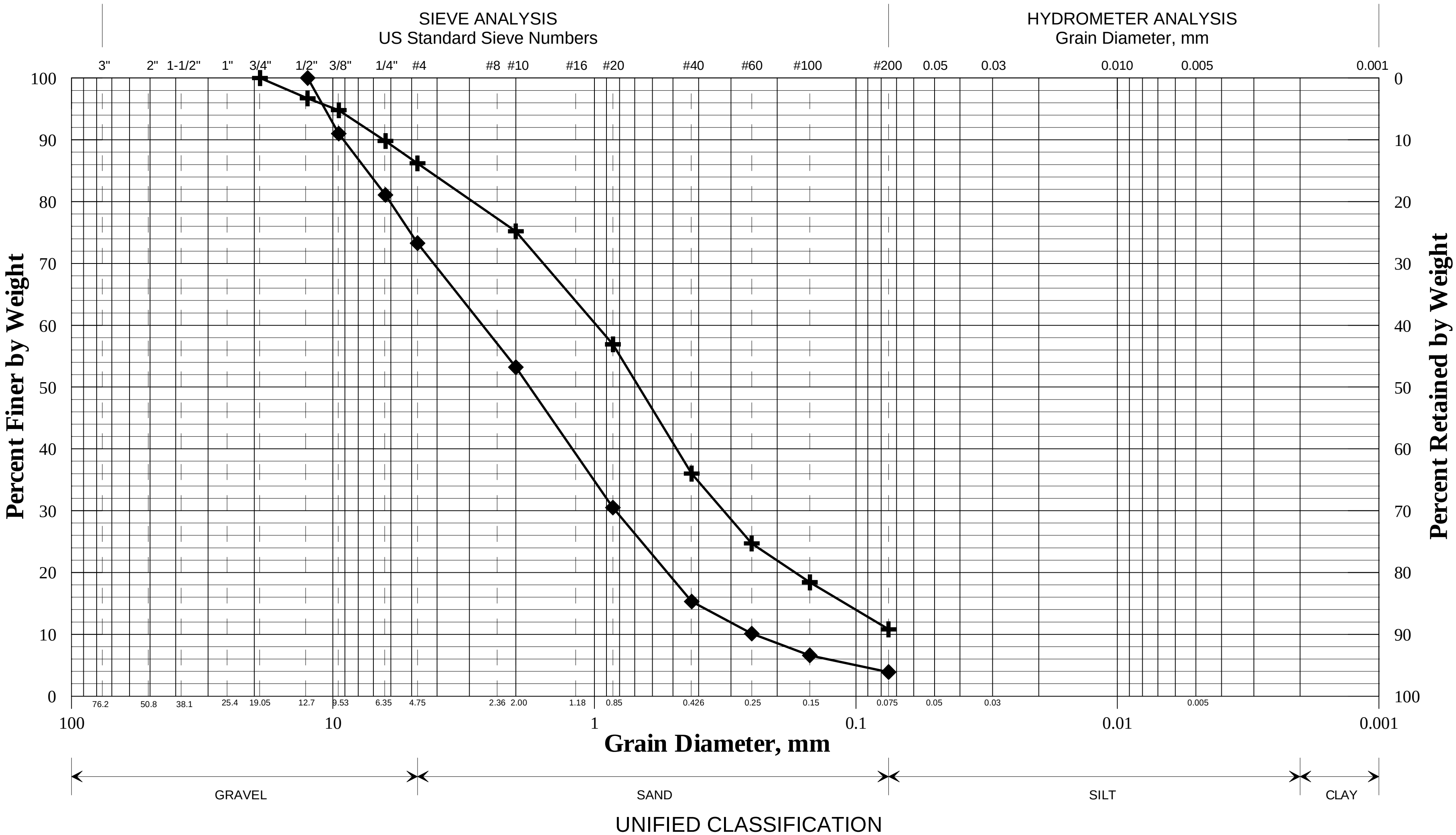
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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




| | Boring/Sample No. | Station | Offset, ft | Depth, ft | Description | W, % | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|---|
| ✚ | BB-PLAR-103/2D | 103+10 | 8.7 RT | 5.0-6.83 | SAND, little gravel, little silt. | 3.5 | | | |
| ◆ | BB-PLAR-103/4D(A) | 103+10 | 8.7 RT | 15.0-15.5 | SAND, some gravel, trace silt. | 14.9 | | | |
| ■ | | | | | | | | | |
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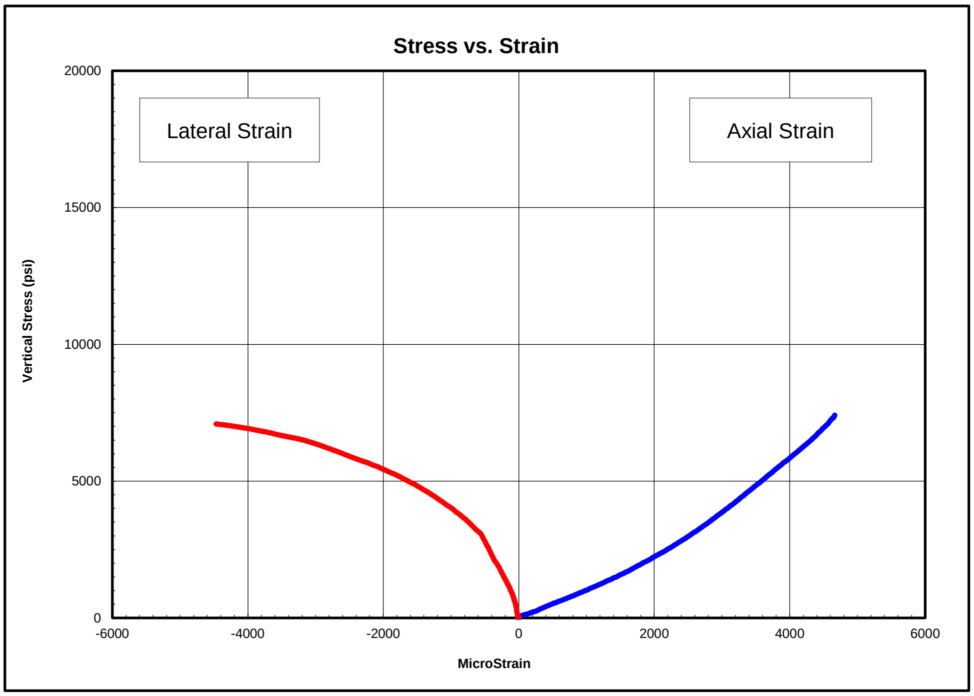
| ✖ | | | | | | | | | |

| WIN |
|---|
| 022618.00 |
| Town |
| Paris |
| Reported by/Date |
| WHITE, TERRY A 4/5/2017 |

**SHEET 1**

| | |
|---|---|
| Client: | Maine DOT |
| Project Name: | Billings Bridge |
| Project Location: | Paris, ME |
| GTX #: | 305648 |
| Test Date: | 11/16/2016 |
| Tested By: | daa |
| Checked By: | jsc |
| Boring ID: | BB-PLAR-101 |
| Sample ID: | R1 |
| Depth, ft: | 25.59-25.96 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Intact material failure<br>Diameter < ten times maximum particle size |

# Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D




Peak Compressive Stress: 7,409 psi

One axial strain gauge failed to record meaningful data. Young's Modulus and Poisson's Ratio reported based on results of a single axial strain gauge.

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 700-2700 | 1,220,000 | 0.26 |
| 2700-4700 | 1,820,000 | --- |
| 4700-6700 | 2,050,000 | --- |

Notes:
Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
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.

GeoTesting EXPRESS

| | | | |
|---|---|---|---|
| Client: | Maine DOT | Test Date: | 11/15/2016 |
| Project Name: | Billings Bridge | Tested By: | daa |
| Project Location: | Paris, ME | Checked By: | jsc |
| GTX #: | 305648 | | |
| Boring ID: | BB-PLAR-101 | | |
| Sample ID: | R1 | | |
| Depth: | 25.59-25.96 ft | | |
| Visual Description: | See photographs | | |

# UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

**BULK DENSITY**

| | 1 | 2 | Average | | |
|---|---|---|---|---|---|
| Specimen Length, in: | 4.27 | 4.27 | 4.27 | | |
| Specimen Diameter, in: | 1.97 | 1.97 | 1.97 | | |
| Specimen Mass, g: | 550.84 | | | | |
| Bulk Density, lb/ft$^3$ | 161 | | | **Minimum Diameter Tolerence Met?** | YES |
| Length to Diameter Ratio: | 2.2 | | | **Length to Diameter Ratio Tolerance Met?** | YES |

**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 | -0.00020 | -0.00020 | -0.00020 | -0.00010 | 0.00000 | 0.00010 | 0.00020 | 0.00030 | 0.00030 | 0.00030 | 0.00030 | 0.00030 |
| Diameter 2, in (rotated 90°) | -0.00030 | -0.00020 | -0.00020 | -0.00020 | -0.00020 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000 | 0.00010 | 0.00010 | 0.00010 | 0.00010 | 0.00020 |

Difference between max and min readings, in:

0° = 0.00050 90° = 0.00050

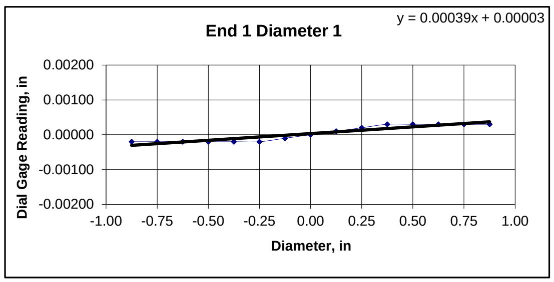
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00020 | -0.00020 | -0.00020 | -0.00010 | -0.00010 | -0.00010 | -0.00010 | 0.00000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

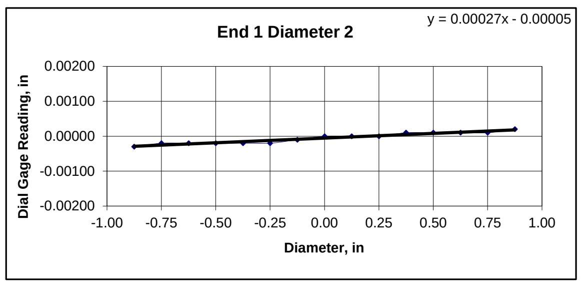
Difference between max and min readings, in:

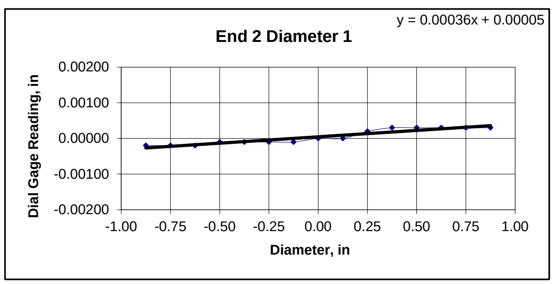
0° = 0.0005 90° = 0.0004

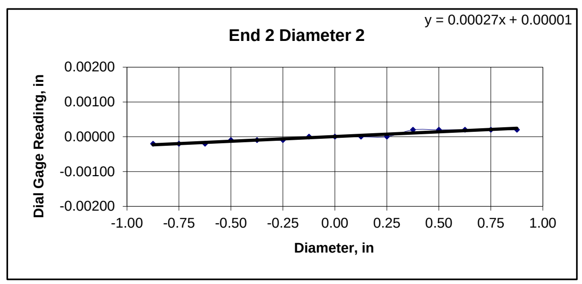
*Maximum difference must be < 0.0020 in.* Difference = ± 0.00025

**Flatness Tolerance Met?** YES




DIAMETER 1

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00039 |
| Angle of Best Fit Line: | 0.02235 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00036 |
| Angle of Best Fit Line: | 0.02063 |

Maximum Angular Difference: 0.00172

**Parallelism Tolerance Met?** YES

Spherically Seated

DIAMETER 2

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00027 |
| Angle of Best Fit Line: | 0.01547 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00027 |
| Angle of Best Fit Line: | 0.01547 |

Maximum Angular Difference: 0.00000

**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 | 0.00050 | 1.970 | 0.00025 | 0.015 | YES |
| Diameter 2, in (rotated 90°) | 0.00050 | 1.970 | 0.00025 | 0.015 | YES |
| **END 2** | | | | | |
| Diameter 1, in | 0.00050 | 1.970 | 0.00025 | 0.015 | YES |
| Diameter 2, in (rotated 90°) | 0.00040 | 1.970 | 0.00020 | 0.012 | YES |

*Maximum angle of departure must be ≤ 0.25°*

**Perpendicularity Tolerance Met?** YES

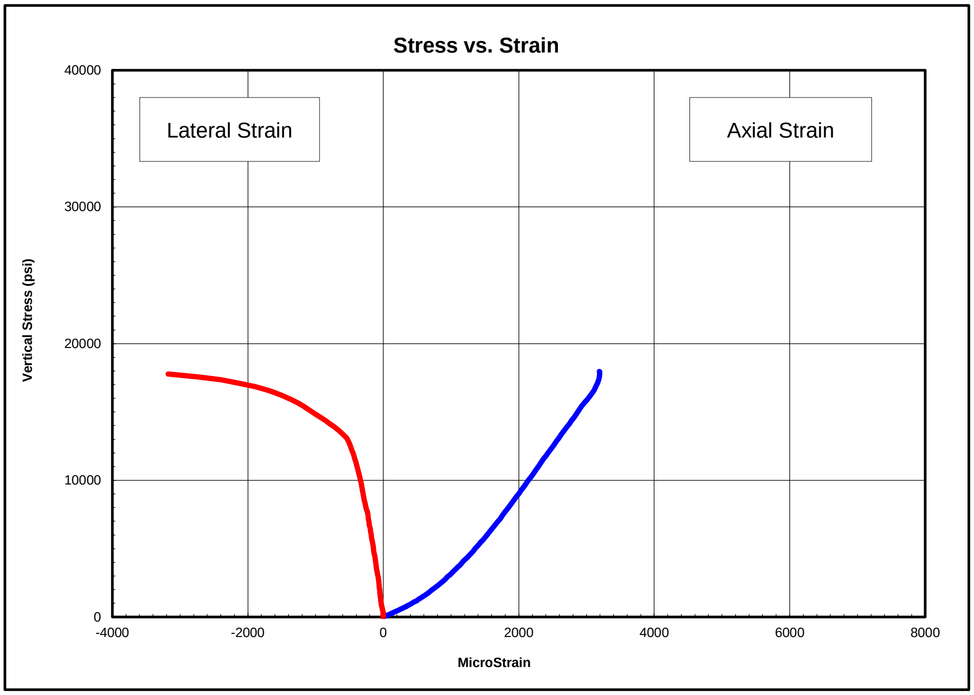
| | |
|---|---|
| Client: | Maine DOT |
| Project Name: | Billings Bridge |
| Project Location: | Paris, ME |
| GTX #: | 305648 |
| Test Date: | 11/16/2016 |
| Tested By: | daa |
| Checked By: | jsc |
| Boring ID: | BB-PLAR-101 |
| Sample ID: | R1 |
| Depth, ft: | 25.59-25.96 |


After cutting and grinding


After break

| | |
|---|---|
| Client: | Maine DOT |
| Project Name: | Billings Bridge |
| Project Location: | Paris, ME |
| GTX #: | 305648 |
| Test Date: | 3/24/2017 |
| Tested By: | trm/rlc |
| Checked By: | jsc |
| Boring ID: | BB-PLAR-102 |
| Sample ID: | R1 |
| Depth, ft: | 22.65-23.01 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Intact material failure |

# Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D




Peak Compressive Stress: 17,962 psi

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 1800-6600 | 5,010,000 | 0.16 |
| 6600-11400 | 6,720,000 | 0.29 |
| 11400-16200 | 6,820,000 | --- |

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
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.

GeoTesting EXPRESS

| | | | |
|---|---|---|---|
| Client: | Maine DOT | Test Date: | 3/23/2017 |
| Project Name: | Billings Bridge | Tested By: | trm/rlc |
| Project Location: | Paris, ME | Checked By: | jsc |
| GTX #: | 305648 | | |
| Boring ID: | BB-PLAR-102 | | |
| Sample ID: | R1 | | |
| Depth: | 22.65-23.01 ft | | |
| Visual Description: | See photographs | | |

# UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

**BULK DENSITY**

| | 1 | 2 | Average | | |
|---|---|---|---|---|---|
| Specimen Length, in: | 4.29 | 4.29 | 4.29 | | |
| Specimen Diameter, in: | 1.98 | 1.99 | 1.99 | | |
| Specimen Mass, g: | 565.89 | | | | |
| Bulk Density, lb/ft$^3$ | 162 | | | **Minimum Diameter Tolerence Met?** | YES |
| Length to Diameter Ratio: | 2.2 | | | **Length to Diameter Ratio Tolerance Met?** | YES |

**DEVIATION FROM STRAIGHTNESS (Procedure S1)**

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? NO

*Maximum difference must be < 0.020 in.*

**Straightness Tolerance Met?** NO

**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 | -0.00030 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 |
| Diameter 2, in (rotated 90°) | -0.00010 | -0.00020 | -0.00010 | -0.00010 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | -0.00010 | -0.00010 |

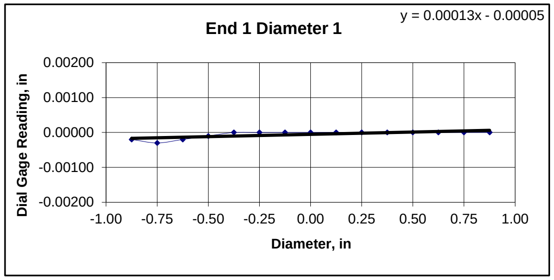
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00020

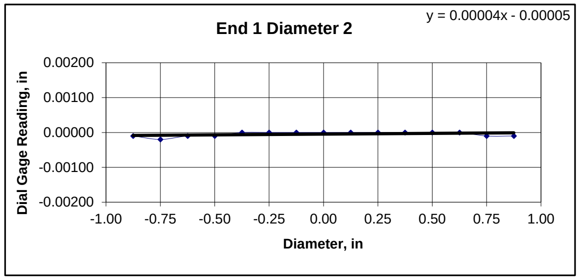
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00020 | -0.00030 | -0.00020 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000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

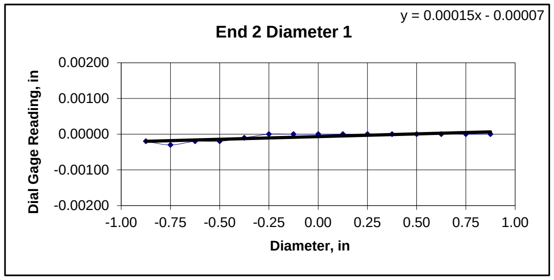
Difference between max and min readings, in:
0° = 0.0003 90° = 0

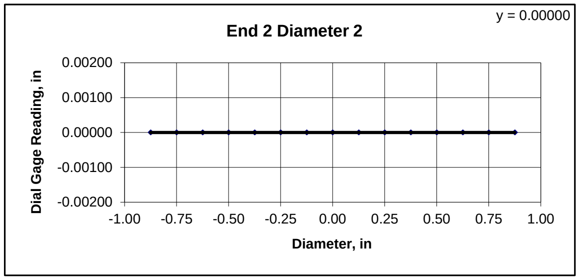
*Maximum difference must be < 0.0020 in.* Difference = $\pm$ 0.00015

**Flatness Tolerance Met?** YES




DIAMETER 1

End 1:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00745

End 2:
Slope of Best Fit Line 0.00015
Angle of Best Fit Line: 0.00859

Maximum Angular Difference: 0.00115

**Parallelism Tolerance Met?** YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00004
Angle of Best Fit Line: 0.00229

End 2:
Slope of Best Fit Line 0.00000
Angle of Best Fit Line: 0.00000

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 | 1.985 | 0.00015 | 0.009 | YES |
| Diameter 2, in (rotated 90°) | 0.00020 | 1.985 | 0.00010 | 0.006 | YES |
| END 2 | | | | | |
| Diameter 1, in | 0.00030 | 1.985 | 0.00015 | 0.009 | YES |
| Diameter 2, in (rotated 90°) | 0.00000 | 1.985 | 0.00000 | 0.000 | YES |

*Maximum angle of departure must be $\leq$ 0.25°*

**Perpendicularity Tolerance Met?** YES

| | |
|---|---|
| Client: | Maine DOT |
| Project Name: | Billings Bridge |
| Project Location: | Paris, ME |
| GTX #: | 305648 |
| Test Date: | 3/24/2017 |
| Tested By: | trm/rlc |
| Checked By: | jsc |
| Boring ID: | BB-PLAR-102 |
| Sample ID: | R1 |
| Depth, ft: | 22.65-23.01 |


After cutting and grinding


After break

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# APPENDIX E
# Calculations

## Evaluation of Nominal and Factored Bearing Resistance on Rock

### Service Limit State

From 2016 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: weathered or broken bedrock of any kind except shale

Consistency in Place: moderately hard to hard

Bearing Resistance Range: 16 to 24 ksf

Recommended Bearing Resistance: 20 ksf

Nominal Bearing Resistance $q_{nominal\_service} := 20\ \boldsymbol{ksf}$

Resistance Factor Service Limit $\phi_{bearing\_service} := 1.0$

Factored Bearing Resistance $q_{factored\_service} := \phi_{bearing\_service} \cdot q_{nominal\_service} = 20\ \boldsymbol{ksf}$

**Recommend Service Limit Nominal and Factored Bearing Resistance = 20 ksf**

From 2016 LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values the service limit bearing resistances are limited to 1 inch of settlement

### Strength and Extreme Limit States

Reference(s): Wyllie (2009) Foundations on Rock, 2nd Ed.
Hoek and Brown (1988) The Hoek-Brown Failure Criterion - A 1988 Update
AASHTO LRFD Bridge Design Specifications, 7th Ed. 2014 with 2016 Interims
AASHTO LRFD Bridge Design Specifications, 6th Ed. 2012

Establish Bedrock Properties

BB-PLAR-101, R1: GNEISS, hard, RQD = 65%, UCT qp = 7,409 psi

BB-PLAR-102, R1-R2: GNEISS, hard
R1 RQD = 85%, UCT qp = 17,962 psi, R2 RDQ = 66%

BB=PLAR-103A, R2-R4: GNEISS, hard
R2 RQD = 50%, R3 RDQ = 90%, R4 RQD = 97%

Calculated by: MAS
Date: July 21, 2017
Checked by: EJB
Date: July 24, 2017

Determine Rock Mass Rating (RMR)
Values based on 2012 LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Masses

1. Strength of Intact Rock Material
   Compressive Strengths (from laboratory testing): 7,410 and 17,960 psi

$q_{u1} := 7409\ psi$ $\quad q_{u1} = 1067\ ksf$

$q_{u2} := 17962\ psi$ $\quad q_{u2} = 2587\ ksf$

Use $q_{u\_design} := 7400\ psi$

For Uniaxial Compressive Strength = 520-1,080 ksf

$RR_1 := 4$

2. Drill Core Quality RQD
   RQD ranged from 50 to 97% (Poor to Excellent)
   RQD near bearing surface ranged from 50 to 85% (Poor to Good) with average of 67%

For RQD 50-75%

$RR_2 := 13$

3. Spacing of Joints
   Jointing near the bearing surface generally characterized as "close to moderately close." Assume bedrock joints close joint with spacing of 2 to 12 inches

For joint spacing of 2 in to 1 ft

$RR_3 := 10$

4. Condition of Joints
   Jointing generally characterized as tight with little oxidation

For joints with slightly rough surfaces, seperation of less than 0.05 inch and soft joint wall rock

$RR_4 := 12$

5. Groundwater Conditions
   Bedrock generally underwater.
   Assume "water under moderate pressure"

$RR_5 := 4$

Sum Relative Ratings 1 through 5 to develope Raw RMR

$RMR_{raw} := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 = 43$

Calculated by: MAS
Date: July 21, 2017
Checked by: EJB
Date: July 24, 2017

6. Strike and Dip Orientations

Jointing generally characterized as low angle to moderately dipping.

From 2012 LRFD Table 10.4.6.4-2 for Strike and Dip Orientations

For Foundations, assume rating of "Fair"

$RR_6 := -7$

Adjust RMR to accont for strike and dip

$RMR_{adjusted} := RMR_{raw} + RR_6 = 36$

Determine Rock Mass Class from Adjusted RMR

From 2012 LRDF Table 10.4.6.4-3 Geomechanics Rock Mass Classes

Adjusted RMR of 36 is indicative of Poor Rock - Class IV

Determine Rock Type

From 2012 LRDF Table 10.4.6.4-4

Rock Type E - Coarse grained polymineriallic igneous & metamorphic crystalline rocks - *amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*

Determine Rock Property Constants *s* and *m*

From Hoek and Brown (1988) Table 1, Calculate m and s

To evaluate the disturbed rock mass constants (m and s), the m and s values for "intact rock samples" are used.

For Rock Type E, Intact Rock Mass constants m (mi) and s (si):

$m_i := 25$

$s_i := 1$

For Disturbed rock mass use Hoek and Brown (1988)

Eqn 18 m/mi = exp((RMR-100)/14)
Eqn 19 s = exp((RMR-100)/6)

$m := m_i \cdot \exp\left(\frac{RMR_{adjusted} - 100}{14}\right)$ $m = 0.259$

$s := \exp\left(\frac{RMR_{adjusted} - 100}{6}\right)$ $s = 2.331 \cdot 10^{-5}$

Determine Correction Factor for Foundation Shape
From Wyllie (2009) Table 5.4 (Pg 138)

Evaluate abutments and pier foundation shape factors

Abutment No 1

$L_f := 37\ ft$ $B_f := 16\ ft$ $\frac{L_f}{B_f} = 2.3$ $C_{f1\_A1} := 1.12$

Abutment No 2

$L_f := 46\ ft$ $B_f := 9\ ft$ $\frac{L_f}{B_f} = 5.1$ $C_{f1\_A2} := 1.05$

Pier

$L_f := 44\ ft$ $B_f := 6\ ft$ $\frac{L_f}{B_f} = 7.3$ $C_{f1\_P} := 1.0$

Use $C_{f1\_design} := 1.0$

$$q_{nominal} := C_{f1\_design} \cdot \sqrt{s} \cdot q_{u\_design} \cdot \left(1 + \sqrt{m \cdot \left(s^{-0.5}\right) \cdot 1}\right) = 42.8\ ksf$$

**Recommend Strength & Extreme Limit Nominal Bearing Resistance = 42.8 ksf**

Factored Bearing Resistance - Strength I

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

$\varphi_b := 0.45$

$q_{factored\_strength} := \varphi_b \cdot q_{nominal} = 19.3\ ksf$

**Recommend Strength Limit Factored Bearing Resistance = 19.3 ksf**

Factored Bearing Resistance - Extreme I

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

$\varphi_b := 0.8$

$q_{factored\_extreme} := \varphi_b \cdot q_{nominal} = 34.2\ ksf$

**Recommend Extreme Limit Factored Bearing Resistance = 34.2 ksf**

**Table 10.4.6.4-1—Geomechanics Classification of Rock Masses**

| | Parameter | | Ranges of Values | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | Strength of intact rock material | Point load strength index | >175 ksf | 85–175 ksf | 45–85 ksf | 20–45 ksf | For this low range, uniaxial compressive test is preferred | | |
| | | Uniaxial compressive strength | >4320 ksf | 2160–4320 ksf | 1080–2160 ksf | 520–1080 ksf | 215–520 ksf | 70–215 ksf | 20–70 ksf |
| | Relative Rating | | 15 | 12 | 7 | 4 | 2 | 1 | 0 |
| 2 | Drill core quality RQD | | 90% to 100% | 75% to 90% | 50% to 75% | 25% to 50% | <25% | | |
| | Relative Rating | | 20 | 17 | 13 | 8 | 3 | | |
| 3 | Spacing of joints | | >10 ft | 3–10 ft | 1–3 ft | 2 in.–1 ft | <2 in. | | |
| | Relative Rating | | 30 | 25 | 20 | 10 | 5 | | |
| 4 | Condition of joints | | • Very rough surfaces<br>• Not continuous<br>• No separation<br>• Hard joint wall rock | • Slightly rough surfaces<br>• Separation <0.05 in.<br>• Hard joint wall rock | • Slightly rough surfaces<br>• Separation <0.05 in.<br>• Soft joint wall rock | • Slicken-sided surfaces<br>or<br>• Gouge <0.2 in. thick<br>or<br>• Joints open 0.05–0.2 in.<br>• Continuous joints | • Soft gouge >0.2 in. thick<br>or<br>• Joints open >0.2 in.<br>• Continuous joints | | |
| | Relative Rating | | 25 | 20 | 12 | 6 | 0 | | |
| 5 | Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration) | Inflow per 30 ft tunnel length | None | <400 gal./hr. | 400–2000 gal./hr. | >2000 gal./hr. | | | |
| | | Ratio = joint water pressure/ major principal stress | 0 | 0.0–0.2 | 0.2–0.5 | >0.5 | | | |
| | | General Conditions | Completely Dry | Moist only (interstitial water) | Water under moderate pressure | Severe water problems | | | |
| | Relative Rating | | 10 | 7 | 4 | 0 | | | |

**Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations**

| Strike and Dip Orientations of Joints | | Very Favorable | Favorable | Fair | Unfavorable | Very Unfavorable |
|---|---|---|---|---|---|---|
| Ratings | Tunnels | 0 | –2 | –5 | –10 | –12 |
| | Foundations | 0 | –2 | –7 | –15 | –25 |
| | Slopes | 0 | –5 | –25 | –50 | –60 |

**Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings**

| RMR Rating | 100–81 | 80–61 | 60–41 | 40–21 | <20 |
|---|---|---|---|---|---|
| Class No. | I | II | III | IV | V |
| Description | Very good rock | Good rock | Fair rock | Poor rock | Very poor rock |

The shear strength of fractured rock masses should be evaluated using the Hoek and Brown criteria, in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, $q_u$, and two dimensionless constants $m$ and $s$. The values of $m$ and $s$ as defined in Table 10.4.6.4-4 should be used.

The shear strength of the rock mass should be determined as:

$$\tau = \left(\cot\ \phi_i' - \cos\ \phi_i'\right) m\ \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

in which:

$$\phi_i' = \tan^{-1}\left\{4h\ \cos^2\left[30 + 0.33\ \sin^{-1}\left(h^{\frac{-3}{2}}\right)\right] - 1\right\}^{\frac{-1}{2}}$$

$$h = 1 + \frac{16\left(m\sigma_n' + sq_u\right)}{(3m^2 q_u)}$$

where:

$\tau$ = the shear strength of the rock mass (ksf)

$\phi'_i$ = the instantaneous friction angle of the rock mass (degrees)

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

$\sigma'_n$ = effective normal stress (ksf)

$m, s$ = constants from Table 10.4.6.4-4 (dim)

This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:

$$c_i = \tau - \sigma_n'\ \tan \phi_i' \qquad (C10.4.6.4\text{-}1)$$

The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.

**Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)**

| Rock Quality | Constants | Rock Type<br><br>A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble*<br>B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)*<br>C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*<br>D = Fine grained polymineralic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite*<br>E = Coarse grained polymineralic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite* | | | | |
|---|---|---|---|---|---|---|
| | | A | B | C | D | E |
| INTACT ROCK SAMPLES<br>Laboratory size specimens free from discontinuities.<br>CSIR rating: *RMR* = 100 | *m*<br>*s* | 7.00<br>1.00 | 10.00<br>1.00 | 15.00<br>1.00 | 17.00<br>1.00 | 25.00<br>1.00 |
| VERY GOOD QUALITY ROCK MASS<br>Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft<br>CSIR rating: *RMR* = 85 | *m*<br>*s* | 2.40<br>0.082 | 3.43<br>0.082 | 5.14<br>0.082 | 5.82<br>0.082 | 8.567<br>0.082 |
| GOOD QUALITY ROCK MASS<br>Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft<br>CSIR rating: *RMR* = 65 | *m*<br>*s* | 0.575<br>0.00293 | 0.821<br>0.00293 | 1.231<br>0.00293 | 1.395<br>0.00293 | 2.052<br>0.00293 |
| FAIR QUALITY ROCK MASS<br>Several sets of moderately weathered joints spaced at 1–3 ft<br>CSIR rating: *RMR* = 44 | *m*<br>*s* | 0.128<br>0.00009 | 0.183<br>0.00009 | 0.275<br>0.00009 | 0.311<br>0.00009 | 0.458<br>0.00009 |
| POOR QUALITY ROCK MASS<br>Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock.<br>CSIR rating: *RMR* = 23 | *m*<br>*s* | 0.029<br>$3 \times 10^{-6}$ | 0.041<br>$3 \times 10^{-6}$ | 0.061<br>$3 \times 10^{-6}$ | 0.069<br>$3 \times 10^{-6}$ | 0.102<br>$3 \times 10^{-6}$ |
| VERY POOR QUALITY ROCK MASS<br>Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines.<br>CSIR rating: *RMR* = 3 | *m*<br>*s* | 0.007<br>$1 \times 10^{-7}$ | 0.010<br>$1 \times 10^{-7}$ | 0.015<br>$1 \times 10^{-7}$ | 0.017<br>$1 \times 10^{-7}$ | 0.025<br>$1 \times 10^{-7}$ |

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

## Evaluation of Earth Pressure Coefficients for Substructure Design

### Assumed Backfill Values

MaineDOT BDG Section 3.6.1 - Soil Type 4

$\gamma_1 := 125\ pcf$ — Unit Weight

$\phi_1 := 32\ deg$ — Friction Angle

$c_1 := 0\ psf$ — Cohesion

### Wall Parameters

$\theta := 90\ deg$ — Angle of back face of wall (from horizontal)

$\delta := \frac{2}{3} \cdot \phi_1$ $\quad \delta = 21.3\ deg$ — Interface Friction between Fill and Wall
LRFD Table 3.11.5.3-1, δ = 19 to 24 deg

$\beta := 0\ deg$ — Continous Backslope Angle(s) (from horizontal)

### Coulomb Active Earth Pressure Coefficient (LRFD Eq. 3.11.5.3-1 and 3.11.5.3-2)

$$\Gamma_a := \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2$$

$$k_a := \frac{\sin(\theta + \phi_1)^2}{\Gamma_a \cdot (\sin(\theta))^2 \cdot (\sin(\theta - \delta))} \qquad k_a = 0.28$$

### Active Earth Pressure Coefficient (LRFD Eq. 3.11.5.2-1)

$$k_o := 1 - \sin(\phi_1) \qquad k_o = 0.47$$

Calculated by: MAS
Date: July 21, 2017
Checked by: EJB
Date: July 24, 2017

S.W.COLE ENGINEERING, INC.

**Estimated Frost Penetration Depth**

Based on MaineDOT Bridge Design Guide Section 5.2.1

Site Location: Paris, Maine

Soil Conditions: SAND, little gravel, little to trace silt (Coarse Grained)

**Step 1.** From Figure 5-1: Design Freezing Index = ±1450 freezing degree-days

**Step 2.** Soils moist; assume w = 10%

**Step 3.** From Table 5-1: Interpolate frost penetration for w = 10%

$$DFI := 1450$$

$$DFI_1 := 1400 \qquad d_1 := 79.2\ in$$

$$DFI_2 := 1500 \qquad d_2 := 82.1\ in$$

$$d_{frost} := d_1 + (d_2 - d_1) \cdot \left( \frac{DFI - DFI_1}{DFI_2 - DFI_1} \right)$$

$$d_{frost} = 80.7\ in$$

$$d_{frost} = 6.7\ ft$$

Calculated by: MAS
Date: July 19, 2017
Checked by: EJB
Date: July 24, 2017

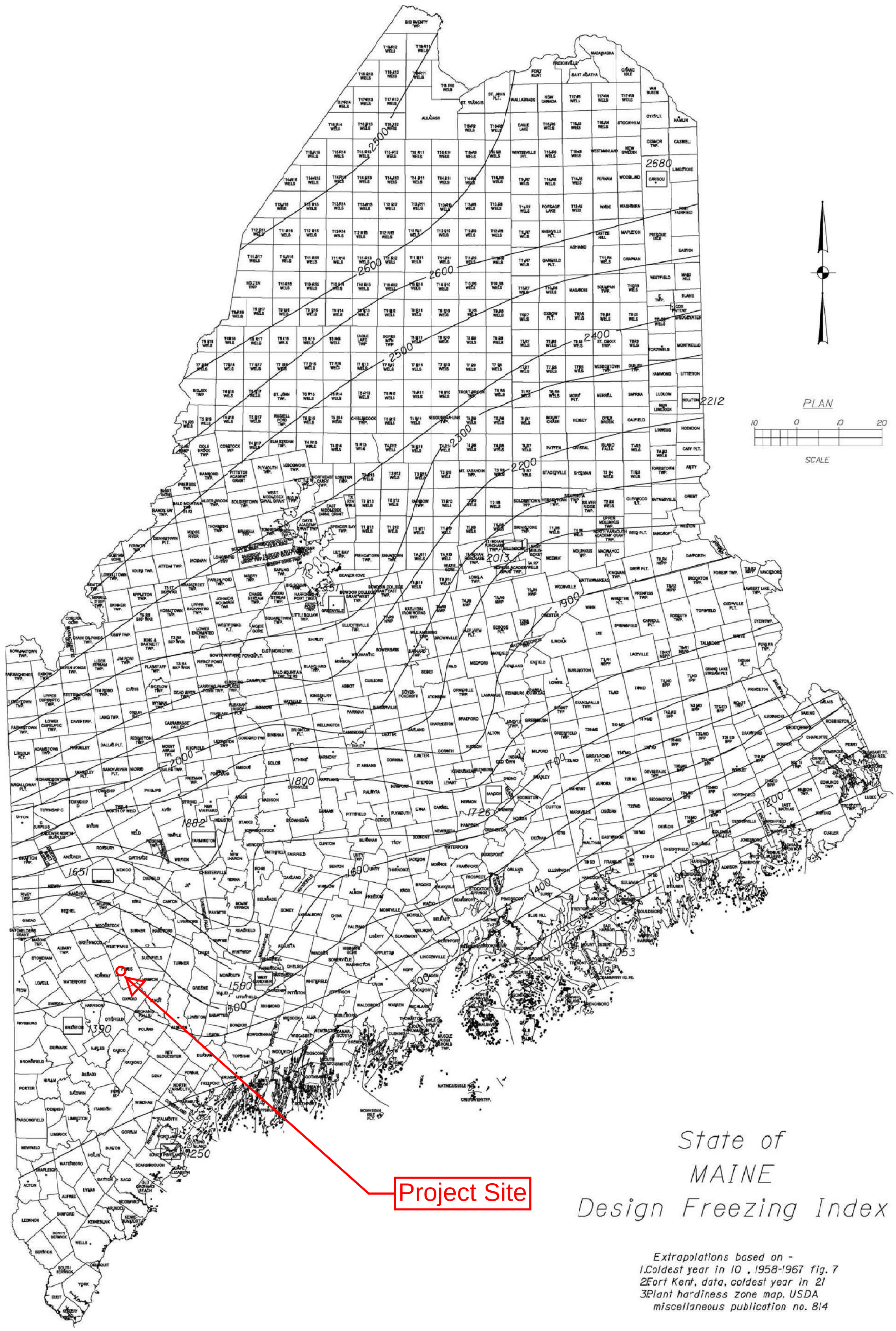



Figure 5-1 Maine Design Freezing Index Map

## 5.2 General

### *5.2.1 Frost*

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

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

**Table 5-1 Depth of Frost Penetration**

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

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-PLAR-102

| Layer No. | Layer Description | Depth Range (ft) | | $N_{60}$ values recorded within layer | | | | | | | | Average $N_{60}$ value | Layer Thickness | $d_i/N_i$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Top | End | | | | | | | | | Ni | di | |
| 1 | Overburden | 0 | 5.5 | 15 | | | | | | | | 15.0 | 5.5 | 0.37 |
| 3 | Bedrock | 5.5 | 100 | 100 | | | | | | | | 100.0 | 94.5 | 0.95 |
| | | | | | | | | | | | | Σ = | **100** | **1.31** |

**Notes:** 1. N60 values in overburden soil assumed
2. N60 value for bedrock taken as N=100

**N_bar = di/di/Ni =** **76.24**
**Site Class** **C**

Data From Borings BB-PLAR-103 and -103A

| Layer No. | Layer Description | Depth Range (ft) | | $N_{60}$ values recorded within layer | | | | | | | | Average $N_{60}$ value | Layer Thickness | $d_i/N_i$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Top | End | | | | | | | | | Ni | di | |
| 1 | Fill | 0 | 19 | 100 | 26 | 19 | 26 | | | | | 42.8 | 19 | 0.44 |
| 3 | Bedrock | 19 | 100 | 100 | | | | | | | | 100.0 | 81 | 0.81 |
| | | | | | | | | | | | | Σ = | **100** | **1.25** |

**Notes:** 1. Refusal N60 values taken as N=100
2. N60 value for bedrock taken as N=100

**N_bar = di/di/Ni =** **79.72**
**Site Class** **C**

Calculated by: MAS 07/19/2017
Checked by: EJB 07/24/2017

# Design Maps Summary Report

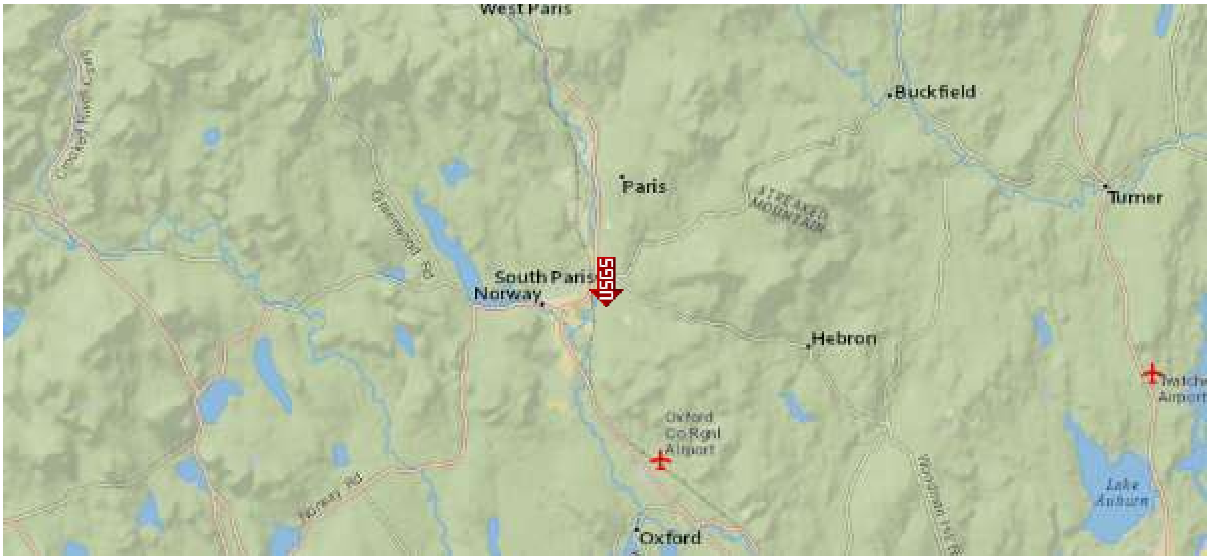
## User–Specified Input

**Report Title** 2979_BillingsBr
Wed July 19, 2017 14:29:01 UTC

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

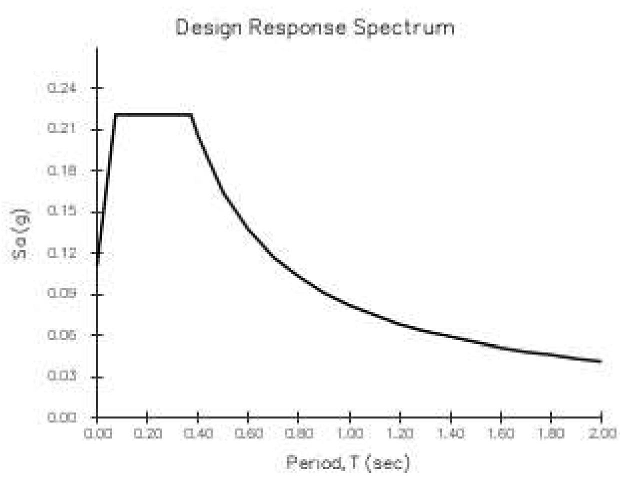
**Site Coordinates** 44.22239°N, 70.5095°W

**Site Soil Classification** Site Class C – "Very Dense Soil and Soft Rock"




## USGS–Provided Output

| | | | |
|---|---|---|---|
| **PGA =** | 0.092 g | **$A_s$ =** | 0.111 g |
| **$S_S$ =** | 0.184 g | **$S_{DS}$ =** | 0.221 g |
| **$S_1$ =** | 0.048 g | **$S_{D1}$ =** | 0.082 g |



# Design Maps Detailed Report

## 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.22239°N, 70.5095°W)

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

### Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

| | |
|---|---|
| **From Figure 3.4.1-2** [1] | PGA = 0.092 g |
| **From Figure 3.4.1-3** [2] | $S_S$ = 0.184 g |
| **From Figure 3.4.1-4** [3] | $S_1$ = 0.048 g |

## Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

| SITE CLASS | SOIL PROFILE NAME | Soil shear wave velocity, $\bar{v}_s$, (ft/s) | Standard penetration resistance, $\bar{N}$ | Soil undrained shear strength, $\bar{s}_u$, (psf) |
|---|---|---|---|---|
| A | Hard rock | $\bar{v}_S > 5{,}000$ | N/A | N/A |
| B | Rock | $2{,}500 < \bar{v}_S \leq 5{,}000$ | N/A | N/A |
| C | Very dense soil and soft rock | $1{,}200 < \bar{v}_S \leq 2{,}500$ | $\bar{N} > 50$ | >2,000 psf |
| D | Stiff soil profile | $600 \leq \bar{v}_S < 1{,}200$ | $15 \leq \bar{N} \leq 50$ | 1,000 to 2,000 psf |
| E | Stiff soil profile | $\bar{v}_S < 600$ | $\bar{N} < 15$ | <1,000 psf |
| E | — | Any profile with more than 10 ft of soil having the characteristics:<br>1. Plasticity index $PI > 20$,<br>2. Moisture content $w \geq 40\%$, and<br>3. Undrained shear strength $\bar{s}_u < 500$ psf | | |
| F | — | Any profile containing soils having one or more of the following characteristics:<br>1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.<br>2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where $H$ = thickness of soil)<br>3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$)<br>4. Very thick soft/medium stiff clays ($H > 120$ feet) | | |

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

## Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for $F_{pga}$)—Values of $F_{pga}$ as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

| Site Class | Mapped Peak Ground Acceleration | | | | |
|---|---|---|---|---|---|
| | PGA ≤ 0.10 | PGA = 0.20 | PGA = 0.30 | PGA = 0.40 | PGA ≥ 0.50 |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See AASHTO Article 3.4.3 | | | | |

Note: Use straight–line interpolation for intermediate values of PGA

**For Site Class = C and PGA = 0.092 g, $F_{PGA}$ = 1.200**

Table 3.4.2.3-1 (for $F_a$)—Values of $F_a$ as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

| Site Class | Spectral Response Acceleration Parameter at Short Periods | | | | |
|---|---|---|---|---|---|
| | $S_S \leq 0.25$ | $S_S = 0.50$ | $S_S = 0.75$ | $S_S = 1.00$ | $S_S \geq 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See AASHTO Article 3.4.3 | | | | |

Note: Use straight–line interpolation for intermediate values of $S_S$

**For Site Class = C and $S_S$ = 0.184 g, $F_a$ = 1.200**

Table 3.4.2.3-2—Values of $F_v$ as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

| Site Class | Mapped Spectral Response Acceleration Coefficient at 1-sec Periods | | | | |
|---|---|---|---|---|---|
| | $S_1 \le 0.10$ | $S_1 = 0.20$ | $S_1 = 0.30$ | $S_1 = 0.40$ | $S_1 \ge 0.50$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| F | See AASHTO Article 3.4.3 | | | | |

Note: Use straight–line interpolation for intermediate values of $S_1$

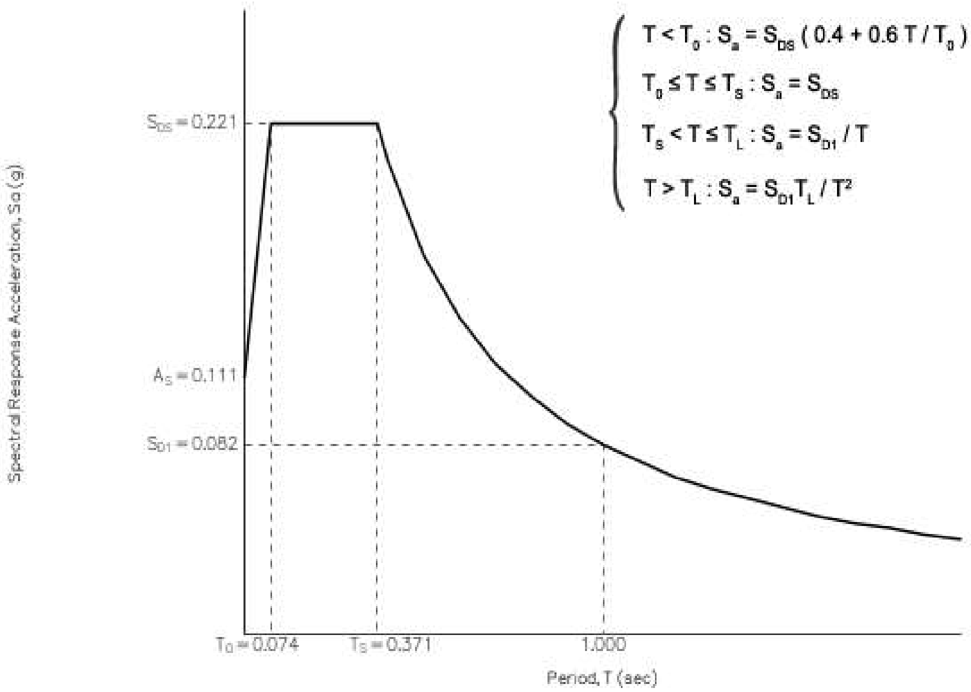
**For Site Class = C and $S_1$ = 0.048 g, $F_v$ = 1.700**

**Equation (3.4.1-1):** $A_S = F_{PGA}$ PGA = 1.200 x 0.092 = 0.111 g

**Equation (3.4.1-2):** $S_{DS} = F_a S_S$ = 1.200 x 0.184 = 0.221 g

**Equation (3.4.1-3):** $S_{D1} = F_v S_1$ = 1.700 x 0.048 = 0.082 g

Figure 3.4.1-1: Design Response Spectrum



## Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

| VALUE OF $S_{D1}$ | SDC |
|---|---|
| $S_{D1} < 0.15g$ | A |
| $0.15g \leq S_{D1} < 0.30g$ | B |
| $0.30g \leq S_{D1} < 0.50g$ | C |
| $0.50g \leq S_{D1}$ | D |

**For $S_{D1}$ = 0.082 g, Seismic Design Category = A**

Seismic Design Category ≡ "the design category in accordance with Table 3.5-1" = A

## References


1. *Figure 3.4.1-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf