STATE OF MAINE
MAINE DEPARTMENT OF TRANSPORTATION
Letter of Transmittal

To: Roger Naous, Bridge Program
Terry White, Highway Geotechnical Program
Kate Maguire, Bridge Program
Kristen Chamberlain, Environmental Office (Electronic Copy Only)
Project Resident, Bridge Program (Unknown as of 5/28/2014)

Author: Kate Maguire, Bridge Program

Subject: Geotechnical Design Report, Saco and Biddeford, Maine

Document Type: 24

Date: 5/28/2014

Soils Report No.: 2014-14

Bridge #: 3412

Route: N/A

WIN: 018233.00

Town: Saco and Biddeford

Attached is one (1) copy of Soils Report 2014-014, "GEOTECHNICAL DESIGN REPORT For the Replacement of: SOMESVILLE BRIDGE OVER SACO RIVER, SACO AND BIDDEFORD, MAINE" dated: May 28, 2014.

This report is available in TEDOCS as Document # 1372012.

att: 1 of 2014-14

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

SOMESVILLE BRIDGE
OVER SACO RIVER
SACO AND BIDDEFORD, MAINE

Prepared by:
Kathleen Maguire, P.E.
Geotechnical Engineer

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

York County
WIN 18233.00

Soils Report No. 2014-14
Bridge No. 3412

Fed No. BH-1823(300)X
May 28, 2014

Table of Contents

GEOTECHNICAL DESIGN SUMMARY **1**

1.0 INTRODUCTION **5**

2.0 GEOLOGIC SETTING **5**

3.0 SUBSURFACE INVESTIGATION **6**

4.0 LABORATORY TESTING **7**

5.0 SUBSURFACE CONDITIONS **7**

5.1 ABUTMENT NO. 1 (SACO) 7
5.2 PIER 8
5.3 ABUTMENT NO. 2 (BIDDEFORD) 8
5.4 BEDROCK 9
5.5 GROUNDWATER 9

6.0 FOUNDATION ALTERNATIVES **10**

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS **10**

7.1 GENERAL - SPREAD FOOTINGS OR CONCRETE SEALS ON BEDROCK 11
7.2 ABUTMENT AND WINGWALL DESIGN 11
7.3 REINFORCED CONCRETE PIER DESIGN 13
7.4 BEDROCK REMOVAL AND BEDROCK SUBGRADE PREPARATION 14
7.5 BEARING RESISTANCE FOR ABUTMENTS, WINGWALLS AND PIER 15
7.6 RESTACK EXISTING GRANITE BLOCK WALLS 16
7.7 SCOUR AND RIPRAP 16
7.8 SETTLEMENT 17
7.9 FROST PROTECTION 17
7.10 SEISMIC DESIGN CONSIDERATIONS 17
7.11 CONSTRUCTION CONSIDERATIONS 18

8.0 CLOSURE **19**

Tables

Table 1 - Summary of Bedrock Depths, Bedrock Elevations and RQD
Table 2 - Approximate Top of Bedrock Elevations
Table 3 - Equivalent Height of Soil for Vehicular Load on Abutments Perpendicular to Traffic

Sheets

Sheet 1 - Location Map
Sheet 2 - Boring Location Plan
Sheet 3 - Interpretive Subsurface Profile
Sheet 4 - Interpretive Subsurface Transverse Sections
Sheets 5 and 6 - Boring Logs

Appendices

Appendix A - Boring Logs
Appendix B - Laboratory Data
Appendix C - Calculations
Appendix D - Special Provision

GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of Somesville Bridge which carries Pine Street and Market Street over the Saco River on the Saco - Biddeford town line. The proposed bridge superstructure will be a 340 foot long two-span, steel girder superstructure. The proposed abutments will consist of a mass concrete abutments founded on spread footings on bedrock or on concrete seals on bedrock. The single center pier will consist of a mass concrete pier founded on a spread footing on bedrock or on a concrete seal on bedrock. The following design recommendations are discussed in detail in Section 7.0 of this report:

General - Spread Footings or Concrete Seals on Bedrock- The proposed abutments, wingwalls and pier will be founded on spread footings constructed directly on bedrock or on concrete seals cast on bedrock. The bedrock surface shall be cleared of all loose bedrock and loose, decomposed bedrock. The approximate top of bedrock elevations in the boring locations for the substructures were:

Substructure	Approximate Top of Bedrock Elevation
Abutment No. 1	20.5 to 30.2 feet
Pier	39.0 to 46.1 feet
Abutment No. 2	40.1 to 42.4 feet

Abutment and Wingwall Design - Abutments and wingwalls shall be designed for all applicable load combinations and for all relevant strength, service and extreme limit states. The design of abutments and wingwalls founded on spread footings or concrete seals at the strength limit state shall consider bearing resistance, eccentricity (overturning), failure by sliding and reinforced concrete structural failure. For the service limit state design, analyses shall consider settlement, horizontal movement, bearing resistance, sliding and eccentricity. For the extreme limit state design, analyses shall consider bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice and seismic forces. Additional lateral earth pressure due to construction surcharge or live load surcharge is required. Abutment and wingwall designs shall include a drainage system to intercept any groundwater. Anchoring of the spread footing to the concrete seal is required by the Maine Department of Transportation Bridge Design Guide.

Reinforced Concrete Pier Design - The reinforced concrete pier shall be designed for all applicable load combinations for all relevant strength, service and extreme limit states. Piers shall be designed to transmit the loads on the superstructure and the loads acting on the pier itself into the foundation. The design of the pier founded on a spread footing at the strength limit state shall consider bearing resistance, eccentricity (overturning), failure by sliding and reinforced concrete structural failure. For the service limit state design analyses shall consider settlement, horizontal movement, bearing resistance, sliding and eccentricity. For the extreme limit state design, analyses shall consider bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice,

vessel collision and seismic forces. Anchoring of the footing to the concrete seal is required by the MaineDOT Bridge Design Guide.

Bedrock Removal - Construction activities should not be permitted to disturb the bedrock mass or to create any rock falls or any open fissures. For spread footings constructed in-the-dry, any irregularities in the existing bedrock surface or irregularities created during the excavation process should be cleared of all loose fractured bedrock, loose decomposed bedrock and soil and the slope shall be less than 4 horizontal to 1 vertical (4H:1V). The irregularities shall then be backfilled with unreinforced concrete to the bearing elevation. Footings may be stepped for varying depths to bedrock along the centerline of bearing. The bottom of footing or concrete seal elevation may vary based on the presence of fractured bedrock. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The bedrock surface slope shall be less than 4H:1V or it shall be benched in level steps or excavated to be completely level. 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. For spread footings or concrete seals constructed in the dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing.

Bearing Resistance – When analyzing the service limit state load combination, a factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements. The bearing resistance for abutment, wingwall and pier foundations founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 19 ksf. For extreme limit state load combinations a factored bearing resistance of 33 ksf may be used for gravity and cantilever semi-gravity walls.

Restacking Existing Granite Block Walls - The existing granite block walls adjacent to Abutment No. 2 will be restacked as a part of this project. A Special Provision for restacking the granite block walls shall be included in the Contract Documents. The design of the restacked walls shall meet current AASHTO LRFD standards for stability. In order to reconstruct these walls it is recommended that the fill material behind the walls be removed and replaced using geosynthetic reinforcement layers within the backfill and between the blocks. Any voids in the wall facing shall be filled with grout. As the wall is reconstructed, the blocks shall be placed so that there are no continuous joint planes in the vertical direction. Each block should bear on at least two blocks below it. A geotextile fabric shall be placed between the back of the block wall and the backfill material to prevent the loss of material through any voids in the wall face. Surface drainage shall be intercepted and directed away from the wall.

Scour and Riprap – The consequences of changes in foundation conditions resulting from streambed material loss due to the design flood for scour shall be considered for any foundation constructed on granular soils at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments, wingwalls and the pier. For scour protection, any footings for wingwalls, which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Bridge approach slopes, slopes at wingwalls and slopes at the toes of any footings on granular soils shall be armored with 3 feet of riprap. Stone riprap shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed

elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material and Class "1" Erosion Control Geotextile.

Settlement - The proposed approach embankments at the bridge approaches will be constructed on granular soils. Placement of the necessary fill will result in negligible densification of the underlying soils and minimal settlement of the embankments. Any settlement will occur during and immediately after construction of the widened embankments. Post-construction settlement will be minimal. Any settlement of bridge abutments or pier will be due to elastic compression of the bedrock mass, and is anticipated to be less than 1.0 inch.

Frost Protection - It is anticipated that the abutment, wingwall and pier spread footings and concrete seals will be founded directly on bedrock. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary. Any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations - Seismic analysis is not required for multi-span bridges in Seismic Zone 1. The Somesville Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

Construction Considerations - Construction activities will 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 abutments, wingwalls and the pier. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for major structures. Plans should call for removal of the existing abutments and wingwalls in their entirety and the old piers to a minimum of 2 feet below streambed.

There is a potential for the existing abutment and wingwall foundations to interfere with the excavation activities for the abutments and wingwalls. Obstructions may be cleared by conventional excavation methods. The existing abutments and wingwalls shall be removed in their entirety. This condition should be noted on the plans and the work shall be considered incidental to bridge removal.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil.

The contractor may maintain the abutment, wingwall and pier excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete.

Where foundations are constructed in the dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. In-the-dry or underwater

excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps.

1.0 INTRODUCTION

The purpose this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for replacement of Somesville Bridge which carries Pine Street and Market Street over the Saco River on the Saco - Biddeford town line. This report presents the soils information obtained during the subsurface investigations, foundation recommendations, and geotechnical design parameters for the bridge replacement.

The existing Somesville Bridge was built in 1937 and is an approximately 344 foot long, five-span, steel girder superstructure. The bridge abutments were rehabilitated in 1971. The piers have not been rehabilitated over the life of the bridge and are showing signs of significant deterioration. The existing Biddeford abutment (southeast) on Springs Island is a mass concrete abutment founded on bedrock. The existing Saco abutment (northwest) is a concrete abutment founded partially on bedrock and partially on steel piles. The four (4) existing piers are mass concrete reinforced piers founded on bedrock. Year 2012 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports assign the substructures a condition rating of 4 – poor with a Bridge Sufficiency Rating of 37.8. The Inspection Notes state that the bridge is in overall poor condition with extensive deterioration of concrete elements.

The proposed replacement bridge will be a two-span, steel girder superstructure. Each span will be approximately 170 feet in length for an overall bridge length of 340 feet on the existing horizontal alignment. The Saco (northwest) abutment (Abutment No. 1) will consist of a full height, cantilever-type concrete abutment founded on a spread footing constructed directly on bedrock or on a concrete seal cast on bedrock. The single center pier will consist of a full height, cantilever-type concrete pier founded on a spread footing constructed directly on bedrock or on a concrete seal cast on bedrock. The Biddeford (southeast) abutment (Abutment No. 2) will consist of a full height, cantilever-type concrete abutment founded a spread footing constructed directly on bedrock or on a concrete seal cast on bedrock. The vertical alignment of the proposed bridge will be raised by approximately 4 feet at the proposed center pier. Existing granite block retaining walls adjacent to Abutment No. 2 will be rebuilt as a part of this project. The bridge will be closed to traffic during construction.

2.0 GEOLOGIC SETTING

Somesville Bridge on the Saco - Biddeford town line crosses the Saco River as shown on Sheet 1 - Location Map. The Saco River flows southeast into Casco Bay at Saco, Maine.

According to the Surficial Geology Map, Biddeford Quadrangle, Maine, Open-File No. 07-81, 2007 by the Maine Geological Survey (MGS) the surficial soils in the vicinity of the site consist of artificial fill and stream alluvium. Artificially emplaced fills can be of any composition. Stream alluvium deposits consist of fine sand, silt and clay with some gravel and organic matter in places. The unit is generally deposited in flood plains and modern streams. The extent of the alluvium generally approximates the area of potential flooding.

According to the Bedrock Geologic Map of Maine (1985) the bedrock at the site is identified as calcareous feldspathic sandstone of the Berwick Formation. A thrust fault is mapped north of the site. According to the Kittery Quadrangle, MGS, Geologic Map 08-78, 2008, the bedrock at the site is identified as thin-bedded grey, calcareous and ankeritic, quartz-biotite-chlorite phyllite and metasiltstone and dark grey biotite-chlorite-muscovite phyllite of the Eliot Formation.

3.0 SUBSURFACE INVESTIGATIONS

Subsurface conditions at the site were explored by drilling eight (8) test borings at the site. Test borings BB-SBSR-101, BB-SBSR-101A and BB-SBSR-102 were drilled behind the Saco (northwest) abutment (Abutment No. 1). Test borings BB-SBSR-103 and BB-SBSR-201 were drilled at the location of the proposed pier. Test borings BB-SBSR-104, BB-SBSR-104A and BB-SBSR-202 were drilled behind the Biddeford (southeast) abutment (Abutment No. 2) on Springs Island. The boring locations are shown in Sheet 2 - Boring Location Plan. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile. The interpretive subsurface soil stratigraphy at each proposed substructure is shown on Sheet 4 – Interpretive Subsurface Transverse Sections.

The 100-series borings were drilled between May 13 and 30, 2013 by the MaineDOT Materials Testing and Exploration drill crew using a truck mounted drill rig. The 200-series borings were drilled on April 9, 2014 by the MaineDOT Materials Testing and Exploration drill crew using a trailer mounted drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on Sheets 5 and 6 – Boring Logs.

All the borings were drilled using solid stem auger and cased wash boring techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rigs are equipped with automatic hammers to drive the split spoon. The hammers were calibrated per ASTM D 4633-05 "Standard Test Method for Energy Measurement for Dynamic Penetrometers". The truck rig mounted hammer was calibrated in August 2012 and was found to deliver approximately 26 percent more energy during driving than the standard rope and cathead system. The trailer rig mounted hammer was calibrated in July 2013 and was found to deliver approximately 44 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.756 for the truck mounted drill rig and 0.867 for the trailer mounted drill rig to the raw field N-values. These hammer efficiency factors (0.756 and 0.867) and both the raw field N-value and the corrected N-value (N_{60}) are shown on the boring logs.

Where bedrock was encountered, the bedrock was cored using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, and identified field and laboratory testing requirements. The MaineDOT Subsurface Inspector certified by the Northeast Transportation Technical Certification Program

(NETTCP) logged the subsurface conditions encountered at borings. The borings were located in the field by tape after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 5 and 6 – Boring Logs.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected samples recovered from test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site.

Laboratory testing consisted of six (6) standard washed grain size analyses with natural moisture content and six (6) grain size analyses with hydrometer and natural moisture content. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of this laboratory testing are provided in Appendix B - Laboratory Data. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 5 and 6 - Boring Logs.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the site consisted of a layer of fill material underlain by native sand overlying bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show on Sheet 3 - Interpretive Subsurface Profile. The interpretive subsurface soil stratigraphy at each proposed substructure is shown on Sheet 4 – Interpretive Subsurface Transverse Sections. A brief summary description of the strata encountered at each substructure is as follows:

5.1 Abutment No. 1 (Saco)

At the proposed Abutment No. 1 (Saco) location fill material was encountered overlying native sands overlying bedrock. The elevation of the bedrock surface at the boring locations varied from approximately elevation 20.5 feet at the southwest corner of the proposed abutment to approximately elevation 30.2 feet at the northeast corner of the proposed abutment. Cobbles boulders and concrete were encountered within boring BB-SBSR-102. Wood (possibly old wooden piles) and wood fragments were encountered in boring BB-SBSR-101A.

Fill. The thickness of the fill ranged from approximately 10.5 to 11.0 feet at the boring locations. The fill consisted of brown, moist to damp, fine to coarse sand with some gravel and little silt. Corrected SPT N-values in the fill ranged from 15 to 30 blows per foot (bpf) indicating that the soil is medium dense in consistency. Two (2) water contents from samples obtained within the fill were approximately 4 and 5%. Two (2) grain size analyses conducted on samples of fill indicated that the soil is classified as an A-1-b under the AASHTO Soil Classification System and as an SW-SM under the Unified Soil Classification System.

Native Sand. The thickness of the native sand ranged from approximately 19.3 to 29.5 feet at the boring locations. The native sand consisted of olive-brown and grey, wet, fine to coarse sand with trace to some gravel, little to some silt and trace clay. Corrected SPT N-values in the native sand ranged from 5 to 91 bpf indicating that the soil is loose to very dense in consistency. Seven (7) natural water contents from samples obtained within the native sand ranged from approximately 9 to 26%. Seven (7) grain size analyses conducted on samples of native sand indicated that the soil is classified as an A-1-b or A-2-4 under the AASHTO Soil Classification System and as an SC-SM or SM under the Unified Soil Classification System. Laboratory test results can be found in Appendix B - Laboratory Data.

5.2 Pier

Native soils were encountered overlying bedrock at the proposed pier location. The thickness of the native sediments ranged from approximately 0.5 to 2.8 feet at the boring locations. The native sediments consisted of brown, fine sand. One corrected SPT N-value in the native sediment was 3 bpf indicating that the soil is loose in consistency.

5.3 Abutment No. 2 (Biddeford)

At the proposed Abutment No. 2 (Biddeford) location fill material was encountered overlying native sands overlying bedrock. The elevation of the bedrock surface at the boring locations varied from approximately elevation 40.1 feet at the northeast corner of the proposed abutment to approximately elevation 42.4 feet at the southwest corner of the proposed abutment. Cobbles, boulders and granite blocks were encountered in both of the borings at this abutment.

Fill. The thickness of the fill ranged from approximately 9.0 to 14.1 feet at the boring locations. The fill consisted of brown, moist to wet, fine to coarse sand, with some gravel and trace to some silt with granite blocks and cobbles. Corrected SPT N-values in the fill ranged from 8 to 24 bpf indicating that the soil is loose to medium dense in consistency. One (1) water content from a sample obtained within the fill was approximately 4%. One (1) grain size analysis conducted on a sample of fill indicated that the soil is classified as an A-1-b under the AASHTO Soil Classification System and as an SW-SM under the Unified Soil Classification System.

Native Sand. The thickness of the native sand ranged from approximately 4.6 to 11.5 feet at the boring locations. The native sand consisted of brown and grey, wet, fine to coarse sand with some gravel, trace to little silt, with occasional cobbles, boulders and broken rock fragments. Corrected SPT N-values in the native sand ranged from 21 to 88 bpf indicating that the soil is medium dense to very dense in consistency. Two (2) natural water contents from samples obtained within the native sand were approximately 12 and 15%. Two (2) grain size analyses conducted on samples of native sand indicated that the soil is classified as an A-1-b under the AASHTO Soil Classification System and as an SW-SM or SM under the Unified Soil Classification System. Laboratory test results can be found in Appendix B - Laboratory Data.

5.4 Bedrock

The bedrock was cored at all three proposed substructure locations. Table 1 summarizes approximate depths to bedrock, corresponding top of bedrock elevations and RQD at the boring locations:

Boring Number Substructure	Approximate Depth to Bedrock (feet)	Approximate Bedrock Elevation (feet)	RQD
BB-SBSR-102 Abutment No 1 (northeast)	30.3	30.2	21 to 60%
BB-SBSR-101A Abutment No. 1 (southwest)	40.0	20.5	0%
BB-SBSR-201 Pier (northeast)	0.5	46.1	86%
BB-SBSR-103 Pier (southwest)	2.8	39.0 (weathered bedrock) 38.8 (intact bedrock)	57 to 80%
BB-SBSR-104A Abutment No. 2 (northeast)	20.5	40.1 (weathered bedrock) 39.6 (intact bedrock)	52 to 72%
BB-SBSR-202 Abutment No. 2 (southwest)	18.7	42.4	0%

Table 1 – Summary of Bedrock Depths, Bedrock Elevations and RQD

The bedrock in borings is identified as dark grey, fine grained, very hard, cemented sandstone with calcite and quartz veins and numerous open joints of the Berwick Formation. A black, intrusive, basalt dike, hard, with white, rectangular crystals was encountered in boring BB-SBSR-201. The RQD of the bedrock ranged from 0 to 86% indicating a Rock Mass Quality of very poor to good.

5.5 Groundwater

Groundwater was observed in boring BB-SBSR-101A at a depth of approximately 12.0 feet bgs (approximate elevation 48.5 feet) and in boring BB-SBSR-202 at a depth of approximately 9.0 feet bgs (approximate elevation 52.1 feet). The existing ordinary high water elevation (Q1.1) at the site is approximate El. 49.2 feet. No groundwater was observed in the remaining borings. Groundwater levels will fluctuate with precipitation, seasonal changes, runoff and construction activity. Water levels encountered during construction may differ from those observed in the test borings.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered at the site, the following foundation alternatives were considered feasible:

Abutment No. 1 - Bedrock was encountered at elevations of approximately 30.2 and 20.5 feet in the borings taken at the proposed Abutment No. 1 (Saco) location. Plans of the existing abutment indicate that it is founded partially on bedrock and partially on short, steel piles. Due to property constraints it is not feasible to place the proposed abutment behind the existing abutment. The MaineDOT design team considered constructing the proposed abutment partially on bedrock and partially on rock-socketed H-piles. Due to constructability issues this alternative was abandoned. Therefore, the abutment will consist of a mass concrete abutment founded on a spread footing directly on bedrock or on a concrete seal constructed on bedrock after removal of the existing abutment. The excavation necessary for this will be large and expensive especially on the southwest side of the abutment.

Abutment No. 2 - Bedrock was encountered at elevations of approximately 40.1 and 42.4 feet in the borings taken at the proposed Abutment No. 2 (Biddeford) location. Due to the relatively shallow bedrock encountered, the abutment will consist of a mass concrete abutment founded on a spread footing constructed directly on bedrock or on a concrete seal constructed on bedrock after removal of the existing abutment.

Center Pier - Bedrock was encountered at elevations of approximately 39.0 and 46.1 feet in the borings taken at proposed pier location. Due to the shallow bedrock encountered at the pier borings, the center pier will consist of a mass concrete pier founded on bedrock or on a concrete seal constructed on bedrock.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections will discuss the foundation considerations and recommendations for:

- Abutments - full height, cast-in-place, cantilever-type abutments founded on spread footings on bedrock or on a concrete seal constructed on bedrock, and
- Center Pier - a mass concrete pier founded a spread footing on bedrock or on a concrete seal constructed on bedrock.

Cast-in-place concrete wingwall retaining walls on spread footings on bedrock will be constructed at both abutments to retain the approach fills. Existing granite block retaining walls adjacent to Abutment No. 2 will be rebuilt as a part of this project. The design recommendations in this Section are in accordance with AASHTO LRFD Bridge Design Specifications, 6th Edition, 2012 (herein referred to as LRFD).

7.1 General - Spread Footings or Concrete Seals on Bedrock

The existing abutments will be removed in their entirety prior to construction of the proposed abutments. It is assumed that the abutment excavations will require cofferdams and temporary soil support systems.

The borings indicate that fractured bedrock may be encountered at the bearing elevations. Prior to construction of the concrete seals or spread footings, the bedrock surface shall be cleared of all loose and fractured bedrock to expose sound bedrock. Table 2 presents approximate top of bedrock elevations at the substructure locations. For footings constructed in the dry, fill concrete can be used to level the bearing area prior to placement of the footing.

Substructure	Approximate Top of Bedrock Elevation
Abutment No. 1	20.5 to 30.2 feet
Pier	39.0 to 46.1 feet
Abutment No. 2	40.1 to 42.4 feet

Table 2 – Approximate Top of Bedrock Elevations

These top of bedrock elevations are estimated based on the borings drilled at each proposed substructure location. 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.

7.2 Abutment and Wingwall Design

The proposed abutments and wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, service and extreme limit states.

The design of abutments and wingwalls founded on spread footings or on concrete seals at the strength limit state shall consider bearing resistance, eccentricity (overturning), failure by sliding and reinforced concrete structural failure. For spread footings or concrete seals 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.

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

Extreme limit state design checks for abutments and wingwalls shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice (if warranted by ice history or stream constriction by the

abutments) and seismic forces. Resistance factors, φ, 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.

For scour protection of abutment and wingwall spread footings or concrete seals, construct the spread footings or concrete seals directly on bedrock surfaces cleaned 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 due to the design or check floods for scour.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on concrete seals on bedrock assuming the bedrock subgrade will be prepared in-the-wet and some amount of sediment will remain on the bedrock surface. LRFD Table 11.5.7-1 allows a sliding resistance factor, φ_τ, of 1.0 for semigravity retaining walls regardless of subgrade material.

Assuming that the rock subgrade will be prepared in-the-wet, some amount of sediment is expected to remain on the rock surface and the sliding computations for resistance of abutment and wingwall spread footings to lateral loads shall assume a maximum friction coefficient of 0.60 at the bedrock-concrete seal interface. If the bedrock subgrade is 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 0.70 at the bedrock-concrete seal interface.

Anchorage of the footing concrete is required by MaineDOT Bridge Design Guide (BDG) Section 5.2.2. 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.

Cantilever-type abutments should be designed for active earth pressure over the abutment height. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.31 is recommended. Earth loads for wingwalls shall also be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory. After evaluation of the anticipated magnitude of lateral movement of the abutment, the designer may elect to use at-rest earth pressures. An at-rest earth pressure coefficient, K_o, of 0.47 is recommended. This recommendation assumes the granular soil behind abutments and wingwalls will be drained and no unbalanced hydrostatic pressures will develop behind abutments and wingwalls. See Appendix B – Calculations at the end of this report for supporting documentation.

The designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG 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 wingwalls

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}) taken from the Table 3 below:

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

Table 3 - Equivalent Height of Soil for Vehicular Load on Abutments Perpendicular to Traffic

Abutment and wingwall designs shall include a drainage system to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

7.3 Reinforced Concrete Pier Design

The solid shaft reinforced concrete pier shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant service, strength and extreme limit states. The pier shall be designed to transmit the loads on the superstructure and the loads acting on the pier itself into the foundation.

The design of reinforced concrete piers on spread footings on bedrock at the strength limit state shall consider bearing resistance, eccentricity, failure by sliding, and reinforced concrete structural failure. A modified strength limit state analysis should be performed that includes the ice pressures specified in MaineDOT BDG Section 3.9 - Ice Loads.

For pier spread footings or concrete seals 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.

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

Extreme limit state design checks for the pier shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events, ice, vessel collision and seismic forces. Resistance factors, φ, 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. 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 the pier footings, construct the footing or concrete seal directly on bedrock surface cleaned 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 due to the design or check floods for scour.

For sliding analyses at the strength limit state, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of the pier founded on a spread footing or concrete seal on bedrock assuming the bedrock subgrade will be prepared in-the-wet and some amount of sediment will remain on the bedrock surface. If the bedrock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete a sliding resistance factor, φ_τ, of 0.9 may be used.

Assuming that the rock subgrade will be prepared in-the-wet, some amount of sediment is expected to remain on the rock surface and the sliding computations for resistance of pier footings to lateral loads shall assume a maximum friction coefficient of 0.60 at the bedrock-concrete seal interface. If the bedrock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of pier footings to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

Anchorage of the footing to the concrete seal is required by MaineDOT BDG Section 5.2.2. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. Anchorage of the footing concrete or of the concrete seal to the bedrock may also be required to resist sliding forces and improve stability. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. 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.

Site conditions may warrant that the pier nose be designed to effectively break up or deflect floating ice or debris. Facing the pier nose with a steel plate/angle or facing the pier with granite should be considered.

7.4 Bedrock Removal and Bedrock Subgrade Preparation

Construction activities should not be permitted to disturb the bedrock mass or to create any rock falls or any open fissures. 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. If spread footings or concrete seals are constructed in-the-dry, the bedrock surface shall be

cleared of all loose and fractured bedrock to expose sound bedrock. If spread footings or concrete seals are constructed in-the-dry, any irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with unreinforced concrete to the bearing elevation. The bedrock surface may be stepped along the centerline of bearing to create a workable bearing surface. The bottom of footing or concrete seal elevation may vary based on the presence of fractured bedrock and the variability of the bedrock surface.

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. The final bearing surface shall be solid. The bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it shall be benched in level steps or excavated to be completely level. 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 contractor may maintain portions or all of the abutment, wingwall and pier excavations so that the foundations can be constructed in the dry. 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.

Portions of the abutment and pier excavations may be submerged. The contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Resident in accordance with Special Provision 511 found in Appendix D.

Where foundations are constructed in the dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. In-the-dry or underwater excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby structures in accordance with industry standards at the time of the blast.

It is anticipated that there will be seepage of water 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.

7.5 Bearing Resistance for Abutments, Wingwalls and Pier

Substructure spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.6. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2.

A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. The bearing

resistance for abutment, wingwall and pier footings founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 19 ksf. This assumes a bearing resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. For extreme limit state load combinations a factored bearing resistance of 33 ksf. This assumes a bearing resistance factor of 0.8 for gravity and semi-gravity walls in accordance with LRFD Article C11.5.8. See Appendix C – Calculations for supporting documentation.

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.

7.6 Restack Existing Granite Block Walls

The existing granite block walls adjacent to Abutment No. 2 will be restacked as a part of this project. A Special Provision for restacking the existing granite block walls shall be included in the Contract Documents. The design of the restacked walls shall meet current AASHTO LRFD standards for stability. In order to reconstruct these walls it is recommended that the fill material behind the walls be removed and replaced using geosynthetic reinforcement layers within the backfill and between the blocks. Any voids in the wall facing shall be filled with grout. As the wall is reconstructed, the blocks shall be placed so that there are no continuous joint planes in the vertical direction. Each block should bear on at least two blocks below it. A geotextile fabric shall be placed between the back of the block wall and the backfill material to prevent the loss of material through any voids in the wall face. The fabric shall be a geotextile meeting the requirements of MDOT Standard Specification Subsection 722.02, Drainage Geotextile. Surface drainage shall be intercepted and directed away from the wall. Wall backfill shall be compacted to 95% of the maximum density as determined by AASHTO T-180, Method C or D.

7.7 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.

The consequences of changes in foundation conditions resulting from streambed material loss due to the design flood for scour shall be considered for any foundation at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments, wingwalls and the pier. For scour protection, any footings for wingwalls, which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Bridge approach slopes, slopes at wingwalls and slopes at the toes of any footings on granular soils shall be armored with 3 feet of riprap. Stone riprap shall conform to item number 703.26 of MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap

section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04).

7.8 Settlement

The proposed approach embankments at the bridge approaches will be constructed on granular soils. Placement of the necessary fill will result in negligible densification of the underlying soils and minimal settlement of the embankments. Any settlement will occur during and immediately after construction of the embankments. Post-construction settlement will be minimal.

Any settlement of bridge abutments and pier will be due to elastic compression of the bedrock mass, and is anticipated to be less than 1.0 inch.

7.9 Frost Protection

It is anticipated that the abutment, wingwall and pier spread footings or concrete seals will be founded directly on bedrock. For foundations on bedrock, heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary.

In the event that any foundation is placed on granular subgrade soils, it should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1123 F-degree days. A granular soil with a water content of approximately 10% correlates to a frost depth of approximately 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on granular soils and not those founded on bedrock. See Appendix C - Calculations for supporting documentation.

7.10 Seismic Design Considerations

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD Manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak ground acceleration coefficient (PGA) = 0.091g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.286g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.107g
- Site Class D (stiff soil with 600 ft/s < average v_s < 1,200 ft/s or with either 15 < average N < 50 blows/foot, or 1.0 < average s_u < 2.0 ksf)
- Seismic Zone 1, based on a S_{D1} < 0.15g

In conformance with LRFD Table 4.7.4.3 seismic analysis is not required for multi-span bridges in Seismic Zone 1. According to Figure 2-2 of the MaineDOT BDG, Somesville Bridge is not

on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively. See Appendix C – Calculations for supporting documentation.

7.11 Construction Considerations

Construction activities will 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 abutments, wingwalls and the pier. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for major structures. Plans should call for removal of the existing abutments and wingwalls in their entirety and the old piers (there are four (4) existing piers) to a minimum of 2 feet below streambed.

There is a potential for the existing abutment and wingwall foundations to interfere with the excavation activities for the abutments and wingwalls. Obstructions may be cleared by conventional excavation methods. The existing abutments and wingwalls shall be removed in their entirety. This condition should be noted on the plans and the work shall be considered incidental to bridge removal. Clearing obstructions shall be specified as incidental to related pay items. Alternative methods to clear obstructions may be used as approved by the Resident.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The final bearing surface shall be solid. The bedrock surface slope shall be less than 4H:1V or it shall be benched in level steps or excavated to be completely level. Anchoring, doweling or other means of improving sliding resistance may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction.

Portions of the abutment and pier excavations may be submerged. The contractor shall prepare and submit a written procedure for cleaning and inspection of the bedrock subgrade to the Resident in accordance with Special Provision 511 found in Appendix D. Where practical, the contractor may maintain the abutment, wingwall or pier excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete.

Where foundations are constructed in the dry, the final bearing surface shall be washed with high pressure water and air prior to concrete being placed for the footing. In-the-dry or underwater excavation of highly sloped and loose fractured bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with Section 105.2.6 of the MaineDOT Standard Specifications. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring at nearby residences and bridge structures in accordance with industry standards at the time of the blast.

The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete or concrete seal.

It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps.

Using the excavated native soils as structural backfill should not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

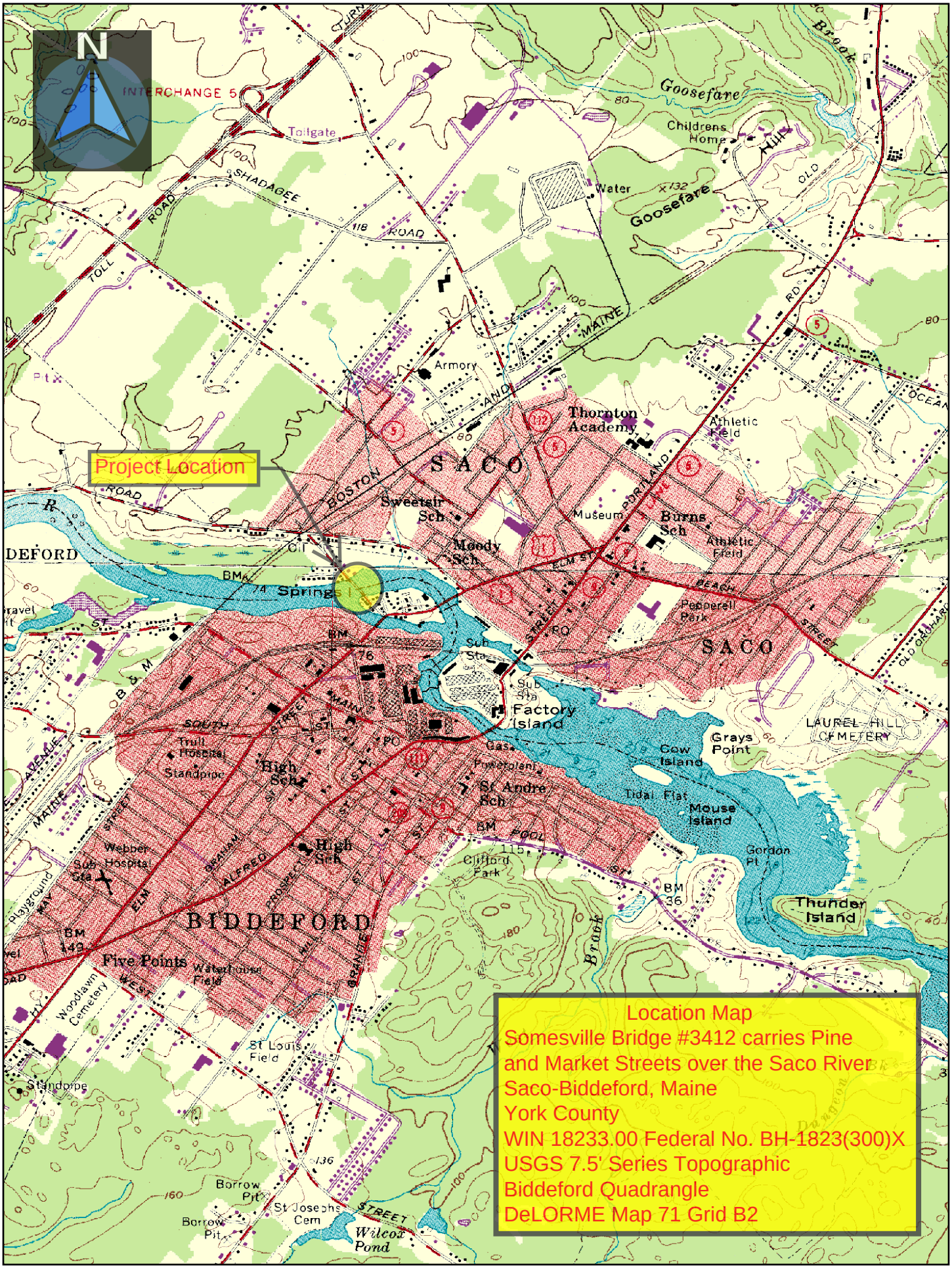
8.0 CLOSURE

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

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

We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

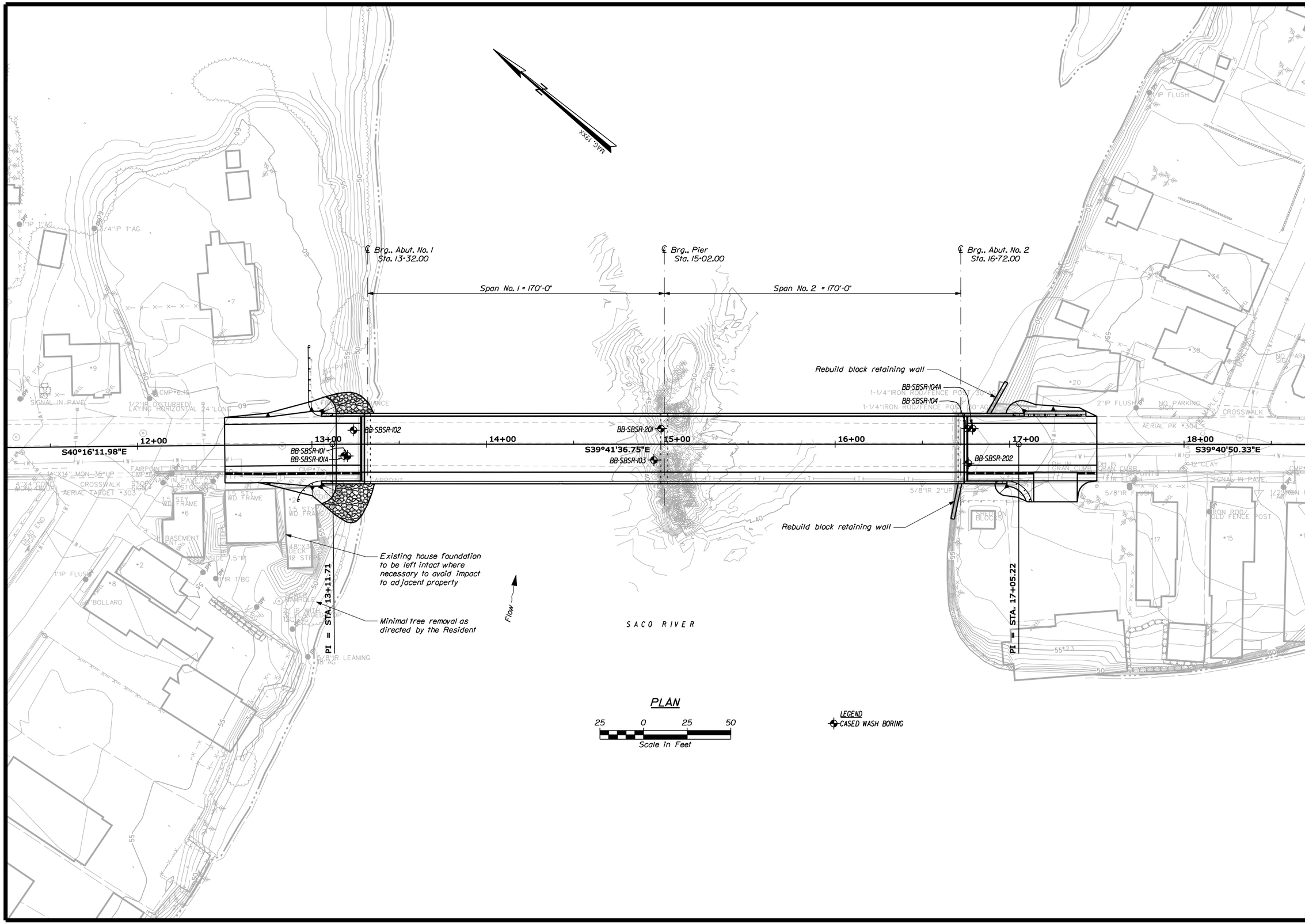
Sheets



Map Scale 1:24000

The Maine Department of Transportation provides this publication for information only. Reliance upon this information is at user risk. It is subject to revision and may be incomplete depending upon changing conditions. The Department assumes no liability if injuries or damages result from this information. This map is not intended to support emergency dispatch. Road names used on this map may not match official road names.

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: Terry.white Date:5/28/2014



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(300)X

WIN 18233.00

BRIDGE NO. 3412

BRIDGE PLANS

PROJ. MANAGER	N. BENOIT	BY	DATE
DESIGN-DETAILED	J. Velleux	D. Damren	-----
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	K.MACURE	T.WHITE	APR 2014
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

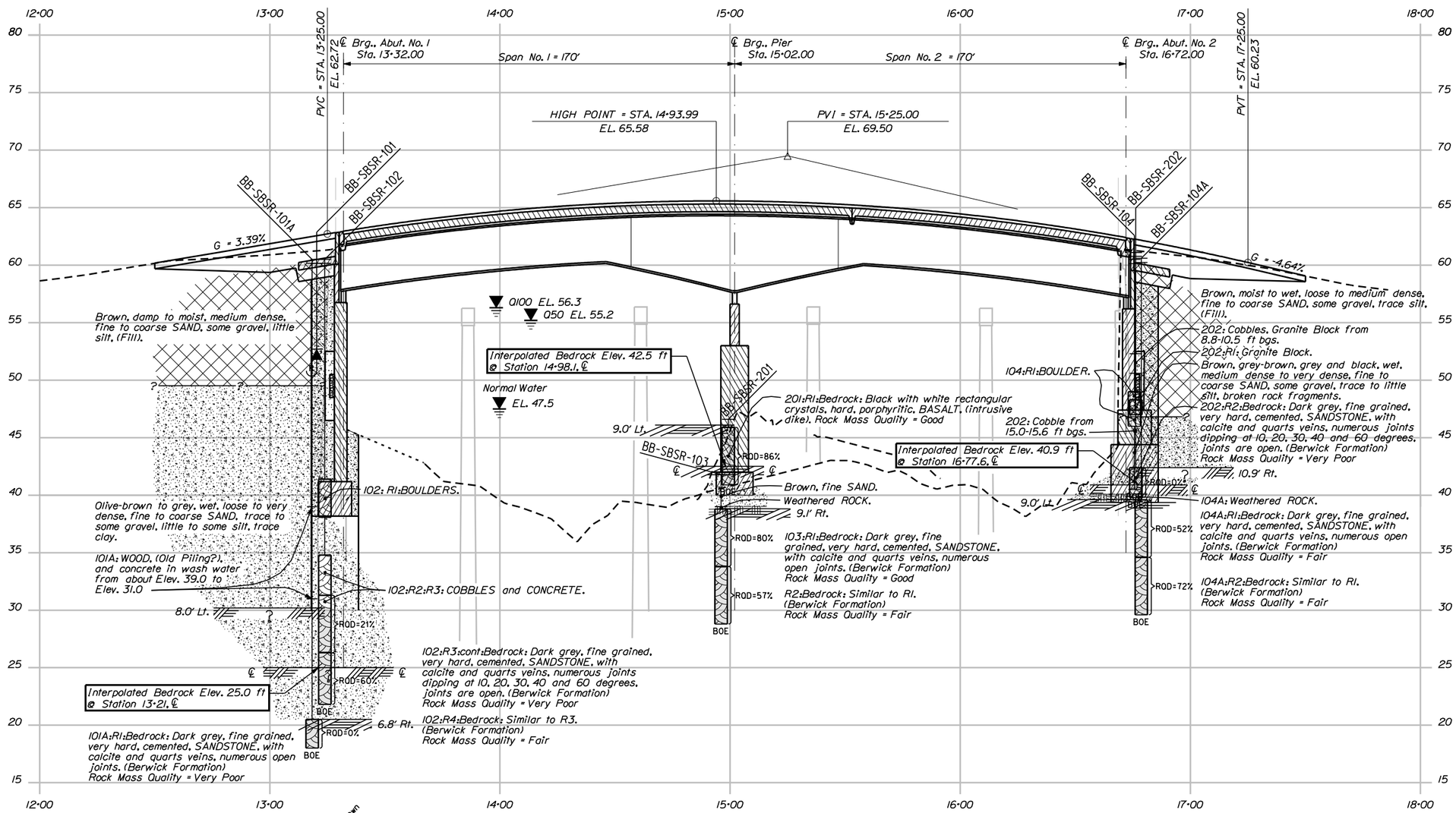
SOMESVILLE BRIDGE
SACO RIVER
SACO & BIDDEFORD YORK COUNTY

BORING LOCATION PLAN

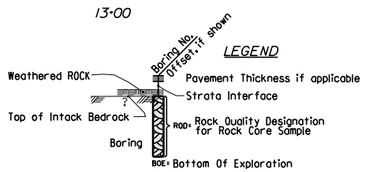
SHEET NUMBER

2

OF 6



LEGEND



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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(300)X

WIN 18233.00

BRIDGE NO. 3412

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. BENOIT		
DESIGN-DETAILED	J. Velleux	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2	K.MAGUIRE	T.WHITE	APR 2014
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

SOMESVILLE BRIDGE
SACO RIVER
SACO & BIDDEFORD YORK COUNTY

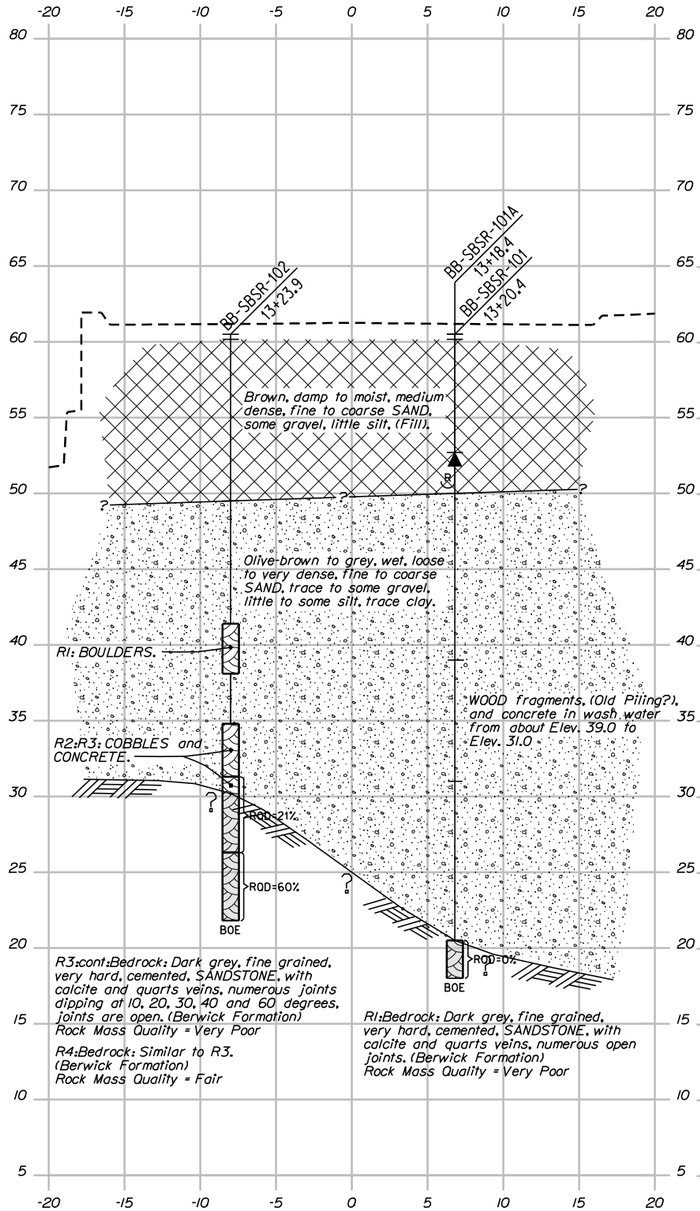
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

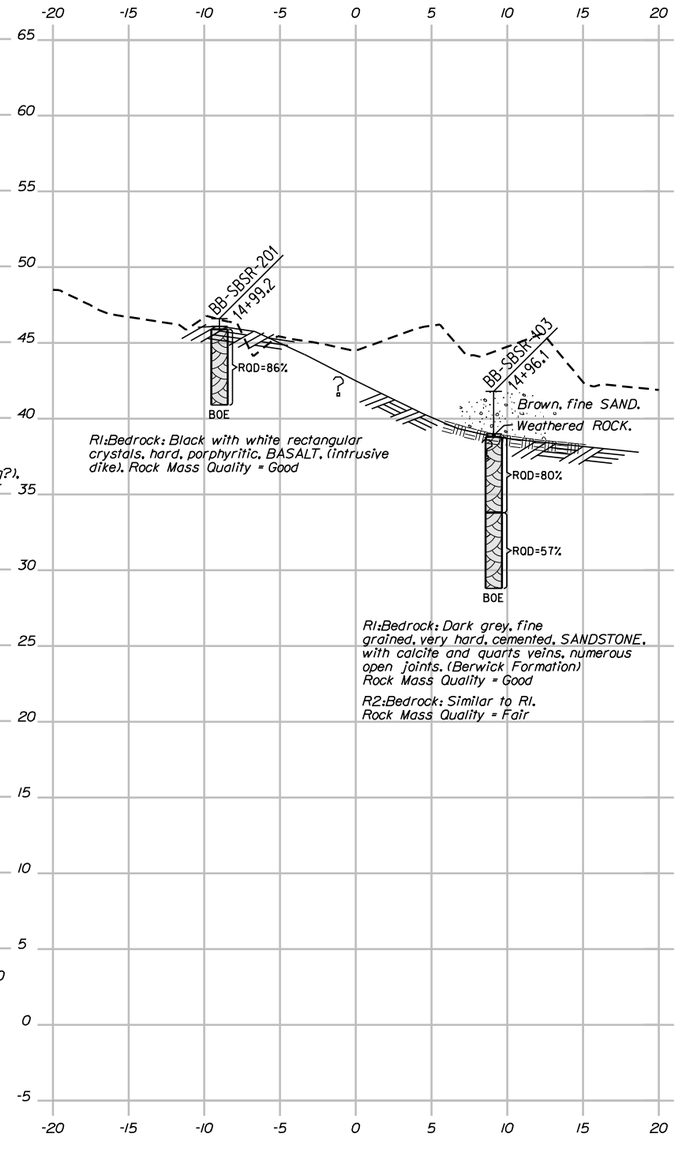
3

OF 6

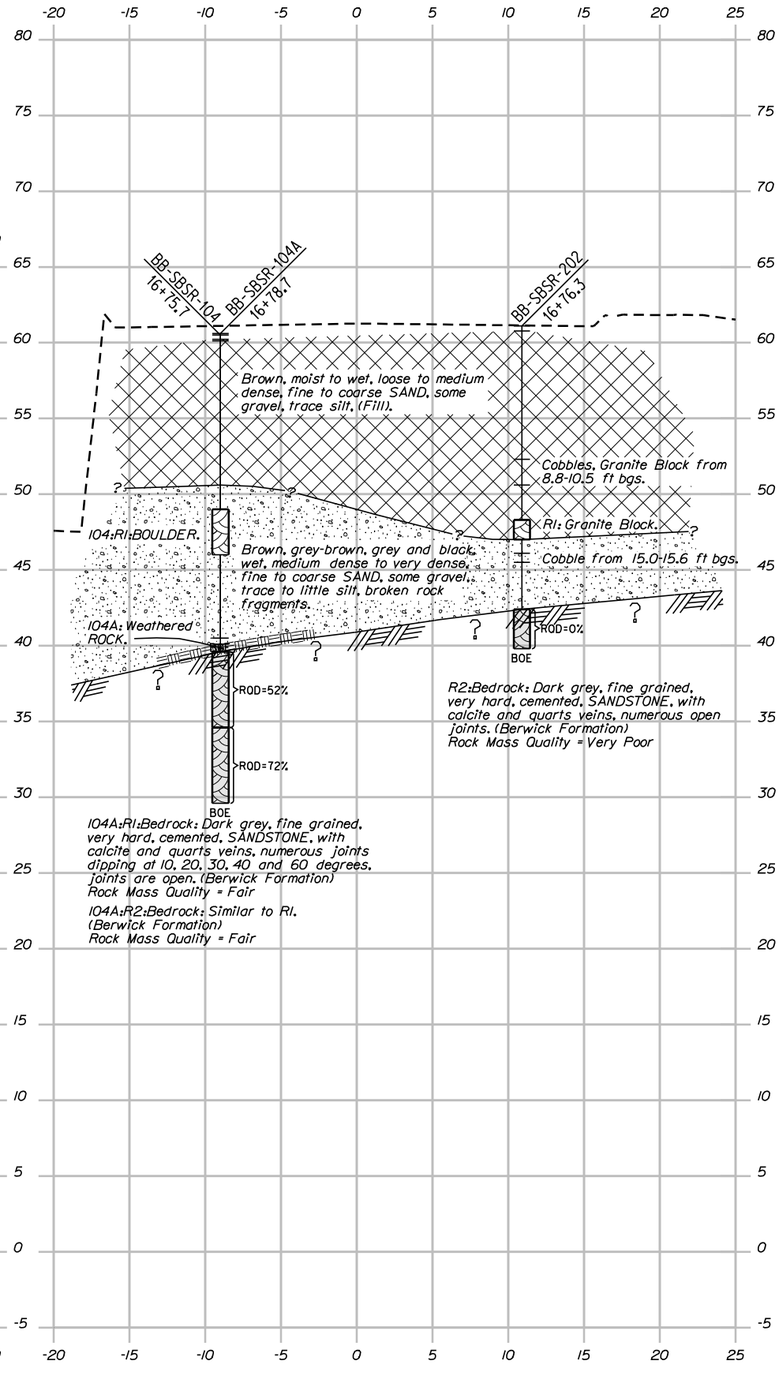
Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: Terry.white Date:5/28/2014



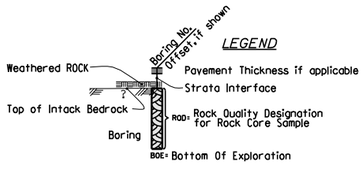
13+22.00



14+98.00



16+76.00



TRANSVERSE SECTION

HORIZ 5 0 5 10

VERT 5 0 5 10

SCALE

Note: This generalized interpretive soil transverse section 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(300)X

WIN 18233.00

BRIDGE NO. 3412

BRIDGE PLANS

PROJ. MANAGER	N. BENOIT	BY	DATE
DESIGN-DETAILED	J. Veilleux	D. Damren	-----
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	K.MAGUIRE	T.WHITE	APR 2014
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

SOMESVILLE BRIDGE
SACO RIVER
SACO & BIDDEFORD YORK COUNTY

INTERPRETIVE SUBSURFACE
TRANSVERSE SECTIONS

SHEET NUMBER

4

OF 6

Filename: ...\00\geotech\msto\008_ISTS1.dgn Division: GEOTECH Username: Terry.white Date:5/28/2014

Filename: ...\msta\009_BORING LOGS1.dgn Division: GEOTECH Username: Terry.white Date:5/28/2014

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-101A
WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	60.5	Auger ID/OD:	5" Dia. Solid Stem
Operator:	Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/30/13: 08:00-14:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+18.4, 6.8 ft Rt.	Casing ID/OD:	NW	Water Level*:	12.0 ft bgs.
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	60.17	4" Pavement (0.33); See BB-SBSR-101 for description of upper 0 to 7.8 feet of soil.	
10	1D/A	24/18	10.00 - 12.00	6/7/8/9	15	19	22	50.00	1D (10.0-10.5 ft) Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill). (10.50) 1D/A (10.5-12.0 ft) Olive-brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#266627 A-1-b, SC-SM WC=14.2%
							20			
							14			
							30			
							38	46.00	(14.50)	
15	2D	24/7	15.00 - 17.00	5/2/2/2	4	5	12		Grey, wet, loose, fine to coarse SAND, little silt, trace gravel, trace clay.	G#266628 A-2-4, SC-SM WC=22.2%
							14			
							12			
							8			
							10			
20	3D	24/9	20.00 - 22.00	2/1/3/3	4	5	8		Grey, wet, loose, fine to coarse SAND, some gravel, little silt, wood fragments.	
							20			
							24			
							84			
							53			
25	4D	24/6	25.00 - 27.00	5/4/3/3	7	9	42		WOOD, possible Bridge Pile. Bent Casing.	
							45			
							46			
							59			
							44		End of wood, concrete in wash water. Roller Coned ahead to 29.0-31.0 ft bgs.	
30							31			
	5D	24/14	31.00 - 33.00	3/5/5/11	10	13	38		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, wood fragments.	G#266629 A-2-4, SC-SM WC=12.1%
							81			
							60			
							137			
35							35			
	6D	24/18	36.00 - 38.00	11/54/18/11	72	91	120		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#266630 A-1-b, SC-SM WC=9.0%
							99			
							64			
							a150		a150 blows for 0.8 ft.	
40	R1	30/8	40.00 - 42.50	RQD = 0%			NQ-2	20.50	(40.00) Top of Bedrock at Elev. 20.5 ft. R1: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints, (Berwick Formation). Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 40.0-41.0 ft (4:25) 41.0-42.0 ft (9:00) 42.0-42.5 ft (5:00) Core Blocked. Bent casing, could not get core barrel back down hole. Core Run in sample cup. (42.50)	
								18.00	Bottom of Exploration at 42.50 feet below ground surface.	
45										
50										

Remarks:

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-SBSR-101A

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-101
WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	60.5	Auger ID/OD:	5" Dia.
Operator:	Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/30/13: 07:30-08:00	Drilling Method:	Solid Stem Auger	Core Barrel:	N/A
Boring Location:	13+20.4, 6.8 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	60.17	4" Pavement (0.33)	
5	1D	24/19	5.00 - 7.00	11/8/6/9	14	18			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#266626 A-1-b, SW-SM WC=4.4%
								52.70	(7.80) Bottom of Exploration at 7.80 feet below ground surface. REFUSAL on obstruction, moved to BB-SBSR-101A.	
10										
15										
20										
25										

Remarks:

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-SBSR-101

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-102
WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	60.5	Auger ID/OD:	5" Solid Stem
Operator:	Enos/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/15/13: 07:00-13:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	13+23.9, 8.0 ft Lt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	60.17	4" Pavement (0.33)	
	1D	24/16	1.00 - 3.00	7/13/11/10	24	30			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
5	2D	24/19	5.00 - 7.00	8/5/7/7	12	15			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#266631 A-1-b, SW-SM WC=4.7%
10	3D/A	24/20	10.00 - 12.00	7/8/13/15	21	26	62		3D (10.0-11.0 ft) Similar to above.	
	3D/A						79	49.50	(11.00) 3D/A (11.0-12.0 ft) Grey, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#266632 A-1-b, SC-SM WC=13.5%
							65			
							59			
							54			
15	4D	24/10	15.00 - 17.00	5/2/2/1	4	5	71		Grey, wet, loose, fine to coarse SAND, some gravel, little silt, trace clay.	G#266633 A-1-b, SC-SM WC=12.4%
							65			
							60			
							72	41.40	(19.10)	
20	R1	39.6/ 39.6	19.10 - 22.40				NQ-2		R1:BOULDER - Grey SANDSTONE. R1:Core Times (min:sec) 19.1-20.1 ft (7:10) 20.1-21.1 ft (4:15) 21.1-22.1 ft (4:14) 22.1-22.4 ft (2:20)	
								38.10	(22.40)	
25	5D R2	8.4/5 42/42	25.00 - 25.70 25.70 - 29.20	5/30(2.4")	---		NQ-2	34.80	Grey, wet, dense, fine to coarse SAND, little silt, little gravel. (25.70) R2:COBBLE and CONCRETE. R2:Core Times (min:sec) 25.7-26.7 ft (3:00) 26.7-27.7 ft (2:15) 27.7-28.7 ft (2:10) 28.7-29.2 ft (1:00) Core Blocked	G#266634 A-2-4, SM WC=26.1%
	R3	60/60	29.20 - 34.20	RQD = 21%					R3:CONCRETE. No Core Times taken.	
30								30.20	(30.30) Top of Bedrock at Elev. 30.2 ft. R3:Cont:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous joints dipping at 20, 30, 40 and 60 degrees, joints are open. (Berwick Formation.) Rock Mass Quality = Very Poor. 100% Recovery	
	R4	54/54	34.20 - 38.70	RQD = 60%					R4:Bedrock: Similar to R3 with minor iron staining and more low angle joints at 10 to 20 degrees. (Berwick Formation). Rock Mass Quality = Fair. R4:Core Times (min:sec) 34.2-35.2 ft (2:19) 35.2-36.2 ft (2:35) 36.2-37.2 ft (2:27) 37.2-38.2 ft (2:10) 38.2-38.7 ft (1:30) 100% Recovery Core Blocked	
								21.80	(38.70) Bottom of Exploration at 38.70 feet below ground surface.	
40										
45										
50										

Remarks:

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-SBSR-102

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-103
WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	41.8	Auger ID/OD:	N/A
Operator:	Enos/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/14/13: 10:00-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+96.1, 9.1 ft Rt.	Casing ID/OD:	HW	Water Level*:	Water Boring
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	MD	24/0	0.00 - 2.00	1/1/1/1	2	3	41		Failed sample attempt. Brown, fine SAND in wash water.	
							52			
							a210	39.00	a210 blows for 0.8 ft. (2.80) Weathered ROCK.	
	R1	60/57	3.00 - 8.00	RQD = 80%			NQ-2	38.80	(3.00) Top of Intact Bedrock at Elev. 38.8 ft. R1:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints, (Berwick Formation). Rock Mass Quality = Good. R1:Core Times (min:sec) 3.0-4.0 ft (7:00) 4.0-5.0 ft (3:50) 5.0-6.0 ft (5:25) 6.0-7.0 ft (4:45) 7.0-8.0 ft (4:32) 95% Recovery	
5										
	R2	60/60	8.00 - 13.00	RQD = 57%					R2:Bedrock: Similar to R1. Rock Mass Quality = Fair. R2:Core Times (min:sec) 8.0-9.0 ft (2:25) 9.0-10.0 ft (4:15) 10.0-11.0 ft (2:30) 11.0-12.0 ft (2:10) 12.0-13.0 ft (2:10) 100% Recovery	
10										
								28.80	(13.00) Bottom of Exploration at 13.00 feet below ground surface.	
15										
20										
25										

Remarks:

Bridge Deck: 4" Pavement, 9" Concrete.
19.7 ft from Bridge Deck to Ground.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1

Boring No.: BB-SBSR-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(300)X

WIN 18233.00

BRIDGE NO. 3412 — BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. BENOIT		
DESIGN-DETAILED	J. Veilleux	D. Damren	-----
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	K.MAGUIRE	T.WHITE	MAY 2014
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

SOMESVILLE BRIDGE
SACO RIVER
SACO & BIDDEFORD YORK COUNTY

BORING LOGS

SHEET NUMBER
5
OF 6

Filename: ...\msta\010_BORING LOGS2.dgn Division: GEOTECH Username: Terry.white Date:5/28/2014

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-201 | WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	46.6
Operator:	Giles/Daggett	Datum:	NAVD88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	4/8/14; 08:00-11:30	Drilling Method:	Cased Wash Boring
Boring Location:	14+93.2, 9.0 ft Lt.	Casing ID/OD:	NW
Hammer Efficiency Factor: 0.867		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Auger ID/OD:	N/A	
Sampler:	Standard Split Spoon	
Hammer Wt./Fall:	140#/30"	
Core Barrel:	NQ-2"	
Water Level*:	None Observed	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	$N_{uncorrected}$	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0								46.10		Roller Coned ahead to 0.7 ft bgs.	
										0.50 Top of Bedrock at Elev. 46.1 ft. R1:Bedrock:Block with white rectangular crystals. hard. porphyritic. BASALT. (Intrusive dike). Rock Mass Quality = Good. R1:Core Times (min:sec) 0.7-1.7 ft (5:10) 1.7-2.7 ft (5:00) 2.7-3.7 ft (3:30) 3.7-4.7 ft (3:20) 4.7-5.7 ft (3:15) 100% Recovery	
	R1	60/60	0.70 - 5.70	RQD = 86%			NQ-2				
5											
								40.90		5.70 Bottom of Exploration at 5.70 feet below ground surface.	
10											
15											
20											
25											

Remarks: 15.0 ft from Bridge Deck to Ground.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-SBSR-201

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-104 | WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	60.5
Operator:	Enos/Giles/Daggett	Datum:	NAVD88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	5/13/13; 08:30-13:00	Drilling Method:	Cased Wash Boring
Boring Location:	16+75.7, 9.0 ft Lt.	Casing ID/OD:	NW & NW
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐

Auger ID/OD:	5" Solid Stem	
Sampler:	Standard Split Spoon	
Hammer Wt./Fall:	140#/30"	
Core Barrel:	NQ-2"	
Water Level*:	None Observed	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	$N_{uncorrected}$	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.13		4.5" Pavement	
										0.38 Brown, moist, medium dense, fine to coarse SAND, trace silt, some gravel, (Fill).	
	10	24/15	1.00 - 3.00	7/9/10/8	19	24					GM266635 A-1-b, SW-SM WC=14.1%
5										Similar to above, except loose.	
	20	24/13	5.00 - 7.00	4/2/4/4	6	8	15				
							21				
							38				
							23				
							17	51.50		9.00 Brown, very wet, very dense, fine to coarse SAND, some gravel, little silt, occasional cobbles.	
10	30	24/12	10.00 - 12.00	24/56/14/13	70	88					GM266636 A-1-b, SM WC=11.6%
	R1	36/36	11.50 - 14.50				RC			11.50 R1:BOULDER, roller coned with large roller coned through boulder, then dropped in NW Casing.	
								46.00		14.50 Grey brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, with broken rock fragments.	
15	40	24/8	15.00 - 17.00	6/11/6/12	17	21	21				
							33				
							96				
							114				
							78	40.50		20.00 Bottom of Exploration at 20.00 feet below ground surface. Broke casing, left 5 ft NW Casing in bore hole. Moved to BB-SBSR-104A.	
20											
25											

Remarks:

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-SBSR-104

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-104A | WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	60.6
Operator:	Enos/Giles/Daggett	Datum:	NAVD88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	5/13/13-5/14/13	Drilling Method:	Cased Wash Boring
Boring Location:	16+78.7, 9.0 ft Lt.	Casing ID/OD:	NW
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Auger ID/OD:	5" Solid Stem	
Sampler:	Standard Split Spoon	
Hammer Wt./Fall:	140#/30"	
Core Barrel:	NQ-2"	
Water Level*:	None Observed	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	$N_{uncorrected}$	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.23		4.5" Pavement	
										0.38 See BB-SBSR-104 for description of upper 0 to 18.0 feet of soil.	
5											
10								51.60		9.00	
15											
							32				
							31				
							11				
							10				
							10				
20	10	12/10	20.00 - 21.00	8/40	---					Black, wet, dense, fine to coarse SAND, some gravel, little silt, weathered ROCK in spoon tip.	GM266637 A-1-b, SW-SM WC=14.7%
								40.10		20.50 Weathered ROCK.	
	R1	60/60	21.00 - 26.00	RQD = 52%			NQ-2	39.60		21.00 Top of Intact Bedrock at Elev. 39.6 ft. R1:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous joints dipping at 20, 30, 40 and 60 degrees, joints are open. (Berwick Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 21.0-22.0 ft (5:30) 22.0-23.0 ft (5:40) 23.0-24.0 ft (5:45) 24.0-25.0 ft (3:10) 25.0-26.0 ft (3:35) 100% Recovery	
25	R2	60/58	26.00 - 31.00	RQD = 72%						R2:Bedrock: Similar to R1. (Berwick Formation). Rock Mass Quality = Fair. R2:Core Times (min:sec) 26.0-27.0 ft (5:45) 27.0-28.0 ft (6:50) 28.0-29.0 ft (4:10) 29.0-30.0 ft (3:10) 30.0-31.0 ft (3:35) 97% Recovery	
30								29.60		31.00 Bottom of Exploration at 31.00 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-SBSR-104A

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

Project: Somesville Bridge #3412 carries Pine and Market Street over the Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-202 | WIN: 18233.00

Driller:	MaineDOT	Elevation (ft.)	61.1
Operator:	Giles/Daggett	Datum:	NAVD88
Logged By:	B. Wilder	Rig Type:	CME 45C
Date Start/Finish:	4/9/14; 08:00-11:30	Drilling Method:	Cased Wash Boring
Boring Location:	16+76.3, 10.9 ft Rt.	Casing ID/OD:	NW & HW
Hammer Efficiency Factor: 0.867		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Auger ID/OD:	5" Solid Stem	
Sampler:	Standard Split Spoon	
Hammer Wt./Fall:	140#/30"	
Core Barrel:	NQ-2"	
Water Level*:	9.0 ft bgs.	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	$N_{uncorrected}$	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.77		4" Pavement	
										0.33	
5										Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
	10	24/17	5.00 - 7.00	12/6/7/9	13	19					
								52.30		8.80 Cobbles or Granite Block, augered thru, set in HW Casing.	
10							WO	50.60		10.50 WO blows for 0.5 ft.	
							25			Brown, wet, medium dense, fine to coarse SAND, some silt, some gravel.	
	20	21.6/13	11.00 - 12.80	5/6/7/30(3.6")	13	19	66				
	R1	15.6/ 15.6	12.80 - 14.10				57	48.30		12.80 R1: Granite Block, cored thru with large Roller Coned. Set in NW Casing.	
							60	47.00		14.10 Grey, wet, dense, fine to coarse SAND, some gravel, trace silt. Cobble from 15.0-15.6 ft bgs.	
15	30	6/5	14.50 - 15.00	60	---						
							38				
							72				
							70				
	R2	31.2/ 31.2	18.70 - 21.30	RQD = 0%			D100 NQ-2	42.40		18.70 D100 blows for 0.7 ft. Top of Bedrock at Elev. 42.4 ft. R2:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints. (Berwick Formation). Rock Mass Quality = Very poor. R2:Core Times (min:sec) 18.7-19.7 ft (4:02) 19.7-20.7 ft (5:30) 20.7-21.3 ft (6:00) 100% Recovery	
20								39.80		21.30 Bottom of Exploration at 21.30 feet below ground surface.	
25											

Remarks: Granite block moved while coring, tore up core barrel, could not get back down.

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

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-SBSR-202

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1823(300)X

WIN 18233.00

BRIDGE NO. 3412

BRIDGE PLANS

PROJ. MANAGER	N. BENOIT	BY	DATE
DESIGN-DETAILED	J. Velleux	D. Damren	-----
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	K.MAGUIRE	T. WHITE	MAY 2014
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----	-----	-----
REVISIONS 2	-----	-----	-----
REVISIONS 3	-----	-----	-----
REVISIONS 4	-----	-----	-----
FIELD CHANGES	-----	-----	-----

SIGNATURE

P.E. NUMBER

DATE

SOMESVILLE BRIDGE
SACO RIVER
SACO & BIDDEFORD YORK COUNTY

BORING LOGS

SHEET NUMBER

6

OF 6

Appendix A

Boring Logs

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)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
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., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

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. Consistency is rated according to standard penetration resistance.

Modified Burmister System

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

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

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 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 thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*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)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, 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, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -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: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

PIN	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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-101
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Dia.
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/13; 07:30-08:00	**Drilling Method:**	Solid Stem Auger	**Core Barrel:**	N/A
Boring Location:	13+20.4, 6.8 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.17		4" Pavement — 0.33	
5	1D	24/19	5.00 - 7.00	11/8/6/9	14	18				Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#266626 A-1-b, SW-SM WC=4.4%
								52.70		7.80	
10										**Bottom of Exploration at 7.80 feet below ground surface.** REFUSAL on obstruction, moved to BB-SBSR-101A.	
15											
20											
25											

Remarks:

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-SBSR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-101A
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Dia. Solid Stem
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/13; 08:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+18.4, 6.8 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.17	4" Pavement (0.33)	
									See BB-SBSR-101 for description of upper 0 to 7.8 feet of soil.	
5										
10	1D/A	24/18	10.00 - 12.00	6/7/8/9	15	19	22	50.00	1D (10.0-10.5 ft) Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill). (10.50)	G#266627 A-1-b, SC-SM WC=14.2%
							20		1D/A (10.5-12.0 ft) Olive-brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay.	
							14			
							30			
							38	46.00	(14.50)	
15	2D	24/7	15.00 - 17.00	5/2/2/2	4	5	12		Grey, wet, loose, fine to coarse SAND, little silt, trace gravel, trace clay.	G#266628 A-2-4, SC-SM WC=22.2%
							14			
							12			
							8			
							10			
20	3D	24/9	20.00 - 22.00	2/1/3/3	4	5	8		Grey, wet, loose, fine to coarse SAND, some gravel, little silt, wood fragments.	
							20			
							24			
							84			
25							53			

Remarks:

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-SBSR-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-101A
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Dia. Solid Stem
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/30/13; 08:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+18.4, 6.8 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	12.0 ft bgs.
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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	4D	24/6	25.00 - 27.00	5/4/3/3	7	9	42			WOOD, possible Bridge Pile. Bent Casing.	
							45				
							46				
							59				
							44			End of wood, concrete in wash water. Roller Coned ahead to 29.0-31.0 ft bgs.	
30							31				
	5D	24/14	31.00 - 33.00	3/5/5/11	10	13	38			Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, wood fragments.	G#266629 A-2-4, SC-SM WC=12.1%
							81				
							60				
							137				
35							35				
	6D	24/18	36.00 - 38.00	11/54/18/11	72	91	120			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#266630 A-1-b, SC-SM WC=9.0%
							99				
							64				
							[a]150			[a]150 blows for 0.8 ft.	
40								20.50		40.00	
	R1	30/8	40.00 - 42.50	RQD = 0%			NQ-2			Top of Bedrock at Elev. 20.5 ft. R1: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints, (Berwick Formation). Rock Mass Quality = Very Poor.	
								18.00		R1:Core Times (min:sec) 40.0-41.0 ft (4:25) 41.0-42.0 ft (9:00) 42.0-42.5 ft (5:00) Core Blocked. Bent casing, could not get core barrel back down hole. Core Run in sample cup.	
45										42.50	
50										**Bottom of Exploration at 42.50 feet below ground surface.**	

Remarks:

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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-102
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/15/13; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+23.9, 8.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.17	4" Pavement (0.33)	
	1D	24/16	1.00 - 3.00	7/13/11/10	24	30			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	
5	2D	24/19	5.00 - 7.00	8/5/7/7	12	15			Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#266631 A-1-b, SW-SM WC=4.7%
10	3D/A	24/20	10.00 - 12.00	7/8/13/15	21	26	62		3D (10.0-11.0 ft) Similar to above.	
	3D/A						79	49.50	(11.00) 3D/A (11.0-12.0 ft) Grey, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#266632 A-1-b, SC-SM WC=13.5%
							65			
							59			
							54			
15	4D	24/10	15.00 - 17.00	5/2/2/1	4	5	71		Grey, wet, loose, fine to coarse SAND, some gravel, little silt, trace clay.	G#266633 A-1-b, SC-SM WC=12.4%
							65			
							60			
							72	41.40	(19.10)	
	R1	39.6/39.6	19.10 - 22.40				NQ-2		R1:BOULDER - Grey SANDSTONE. R1:Core Times (min:sec) 19.1-20.1 ft (7:10) 20.1-21.1 ft (4:15) 21.1-22.1 ft (4:14) 22.1-22.4 ft (2:20)	
20								38.10	(22.40)	
25										

Remarks:

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-SBSR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-102
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/15/13; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	13+23.9, 8.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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	5D	8.4/5	25.00 - 25.70	5/30(2.4")	---		NQ-2	34.80		Grey, wet, dense, fine to coarse SAND, little silt, little gravel. — 25.70	G#266634 A-2-4, SM WC=26.1%
	R2	42/42	25.70 - 29.20							R2:COBBLE and CONCRETE. R2:Core Times (min:sec) 25.7-26.7 ft (3:00) 26.7-27.7 ft (2:15) 27.7-28.7 ft (2:10) 28.7-29.2 ft (1:00) Core Blocked R3:CONCRETE. No Core Times taken.	
	R3	60/60	29.20 - 34.20	RQD = 21%							
30								30.20		— 30.30 Top of Bedrock at Elev. 30.2 ft. R3:Cont:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous joints dipping at 20, 30, 40 and 60 degrees, joints are open, (Berwick Formation.) Rock Mass Quality = Very Poor. 100% Recovery	
	R4	54/54	34.20 - 38.70	RQD = 60%						R4:Bedrock: Similar to R3 with minor iron staining and more low angle joints at 10 to 20 degrees, (Berwick Formation). Rock Mass Quality = Fair. R4:Core Times (min:sec) 34.2-35.2 ft (2:19) 35.2-36.2 ft (2:35) 36.2-37.2 ft (2:27) 37.2-38.2 ft (2:10) 38.2-38.7 ft (1:30) 100% Recovery Core Blocked	
35								21.80		— 38.70 **Bottom of Exploration at 38.70 feet below ground surface.**	
40											
45											
50											

Remarks:

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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-103
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	41.8	**Auger ID/OD:**	N/A
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/14/13; 10:00-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+96.1, 9.1 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Water Boring
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.00 - 2.00	1/1/1/1	2	3	41			Failed sample attempt. Brown, fine SAND in wash water.	
							52				
							[a]210			[a]210 blows for 0.8 ft.	
								39.00		2.80	
	R1	60/57	3.00 - 8.00	RQD = 80%			NQ-2	38.80		Weathered ROCK. 3.00	
										Top of Intact Bedrock at Elev. 38.8 ft. R1:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints, (Berwick Formation). Rock Mass Quality = Good. R1:Core Times (min:sec) 3.0-4.0 ft (7:00) 4.0-5.0 ft (3:50) 5.0-6.0 ft (5:25) 6.0-7.0 ft (4:45) 7.0-8.0 ft (4:32) 95% Recovery	
5											
	R2	60/60	8.00 - 13.00	RQD = 57%						R2:Bedrock: Similar to R1. Rock Mass Quality = Fair. R2:Core Times (min:sec) 8.0-9.0 ft (2:25) 9.0-10.0 ft (4:15) 10.0-11.0 ft (2:30) 11.0-12.0 ft (2:10) 12.0-13.0 ft (2:10) 100% Recovery	
10											
								28.80		13.00	
										Bottom of Exploration at 13.00 feet below ground surface.	
15											
20											
25											

Remarks:

Bridge Deck: 4" Pavement, 9" Concrete.
19.7 ft from Bridge Deck to Ground.

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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-104
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/13/13; 08:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+75.7, 9.0 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.13	4½" Pavement — 0.38	
	1D	24/15	1.00 - 3.00	7/9/10/8	19	24			Brown, moist, medium dense, fine to coarse SAND, trace silt, some gravel, (Fill).	G#266635 A-1-b, SW-SM WC=4.1%
5	2D	24/13	5.00 - 7.00	4/2/4/4	6	8	15		Similar to above, except loose.	
							21			
							38			
							23			
							17	51.50	9.00	
10	3D	24/12	10.00 - 12.00	24/56/14/13	70	88			Brown, very wet, very dense, fine to coarse SAND, some gravel, little silt, occasional cobbles.	G#266636 A-1-b, SM WC=11.6%
	R1	36/36	11.50 - 14.50				RC	49.00	11.50 — R1:BOULDER, roller coned with large roller coned through boulder, then dropped in NW Casing.	
								46.00	14.50	
15	4D	24/8	15.00 - 17.00	6/11/6/12	17	21	21		Grey brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, with broken rock fragments.	
							33			
							96			
							114			
							78			
20								40.50	20.00	
									Bottom of Exploration at 20.00 feet below ground surface. Broke casing, left 5 ft NW Casing in bore hole. Moved to BB-SBSR-104A.	
25										

Remarks:

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-SBSR-104

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-104A
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/13/13-5/14/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+78.7, 9.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.23	4½" Pavement — 0.38	
									See BB-SBSR-104 for description of upper 0 to 18.0 feet of soil.	
5										
								51.60	9.00	
10										
15							32			
							31			
							11			
							10			
							10			
20	1D	12/10	20.00 - 21.00	8/40	---			40.10	Black, wet, dense, fine to coarse SAND, some gravel, little silt, weathered ROCK in spoon tip. — 20.50	G#266637 A-1-b, SW-SM WC=14.7%
	R1	60/60	21.00 - 26.00	RQD = 52%			NQ-2	39.60	Weathered ROCK. — 21.00	
									Top of Intact Bedrock at Elev. 39.6 ft. R1:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous joints dipping at 20, 30, 40 and 60 degrees, joints are open, (Berwick Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec)	
25										

Remarks:

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-SBSR-104A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-104A
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	60.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/13/13-5/14/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+78.7, 9.0 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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										21.0-22.0 ft (5:30)	
										22.0-23.0 ft (5:40)	
										23.0-24.0 ft (5:45)	
	R2	60/58	26.00 - 31.00	RQD = 72%						24.0-25.0 ft (3:10)	
										25.0-26.0 ft (3:35) 100% Recovery	
										R2:Bedrock: Similar to R1, (Berwick Formation).	
										Rock Mass Quality = Fair.	
										R2:Core Times (min:sec)	
										26.0-27.0 ft (5:45)	
										27.0-28.0 ft (6:50)	
30										28.0-29.0 ft (4:10)	
										29.0-30.0 ft (3:10)	
								29.60		30.0-31.0 ft (3:35) 97% Recovery	
										31.00	
										Bottom of Exploration at 31.00 feet below ground surface.	
35											
40											
45											
50											

Remarks:

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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-201
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	46.6	**Auger ID/OD:**	N/A
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/9/14; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+99.2, 9.0 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	R1	60/60	0.70 - 5.70	RQD = 86%			NQ-2	46.10		Roller Coned ahead to 0.7 ft bgs.	
										0.50	
										Top of Bedrock at Elev. 46.1 ft. R1:Bedrock:Black with white rectangular crystals, hard, porphyritic, BASALT, (intrusive dike). Rock Mass Quality = Good. R1:Core Times (min:sec) 0.7-1.7 ft (5:10) 1.7-2.7 ft (5:00) 2.7-3.7 ft (3:30) 3.7-4.7 ft (3:20) 4.7-5.7 ft (3:15) 100% Recovery	
5								40.90		5.70	
										Bottom of Exploration at 5.70 feet below ground surface.	
10											
15											
20											
25											

Remarks:

15.0 ft from Bridge Deck to Ground.

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: Somesville Bridge #3412 carries Pine and Market Street over the Saco River
Location: Saco-Biddeford, Maine

Boring No.: BB-SBSR-202
WIN: 18233.00

Driller:	MaineDOT	**Elevation (ft.)**	61.1	**Auger ID/OD:**	5" Soild Stem
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/9/14; 08:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	16+76.3, 10.9 ft Rt.	**Casing ID/OD:**	NW & HW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	60.77	4" Pavement — 0.33	
5	1D	24/17	5.00 - 7.00	12/6/7/9	13	19			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
							a40	52.30	8.80 — Cobbles or Granite Block, augered thru, set in HW Casing. [a]40 blows for 0.5 ft.	
10							25	50.60	10.50 — Brown, wet, medium dense, fine to coarse SAND, some silt, some gravel.	
	2D	21.6/13	11.00 - 12.80	5/6/7/30(3.6")	13	19	66			
	R1	15.6/15.6	12.80 - 14.10				57	48.30	12.80 — R1: Granite Block, cored thru with large Roller Coned. Set in NW Casing.	
							60			
	3D	6/5	14.50 - 15.00	60	---			47.00	14.10 — Grey, wet, dense, fine to coarse SAND, some gravel, trace silt. Cobble from 15.0-15.6 ft bgs.	
15							38			
							72			
							70			
	R2	31.2/31.2	18.70 - 21.30	RQD = 0%			b100 NQ-2	42.40	[b]100 blows for 0.7 ft. 18.70 — Top of Bedrock at Elev. 42.4 ft. R2:Bedrock: Dark grey, fine grained, very hard, cemented, SANDSTONE, with calcite and quartz veins, numerous open joints, (Berwick Formation). Rock Mass Quality = Very poor.	
20								39.80	R2:Core Times (min:sec) 18.7-19.7 ft (4:02) 19.7-20.7 ft (5:30) 20.7-21.3 ft (6:00) 100% Recovery — 21.30	
25									**Bottom of Exploration at 21.30 feet below ground surface.**	

Remarks:

Granite block moved while coring, tore up core barrel, could not get back down.

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.

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Saco-Biddeford Work Number: 18233.00

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-SBSR-101, 1D	13+20.4	6.8 Rt.	5.0-7.0	266626	1	4.4			SW-SM	A-1-b	0
BB-SBSR-101A, 1D/A	13+18.4	6.8 Rt.	10.5-12.0	266627	1	14.2			SC-SM	A-1-b	II
BB-SBSR-101A, 2D	13+18.4	6.8 Rt.	15.0-17.0	266628	1	22.2			SC-SM	A-2-4	II
BB-SBSR-101A, 5D	13+18.4	6.8 Rt.	31.0-33.0	266629	1	12.1			SC-SM	A-2-4	III
BB-SBSR-101A, 6D	13+18.4	6.8 Rt.	36.0-38.0	266630	1	9.0			SC-SM	A-1-b	II
BB-SBSR-102, 2D	13+23.9	8.0 Lt.	5.0-7.0	266631	2	4.7			SW-SM	A-1-b	0
BB-SBSR-102, 3D/A	13+23.9	8.0 Lt.	11.0-12.0	266632	2	13.5			SC-SM	A-1-b	III
BB-SBSR-102, 4D	13+23.9	8.0 Lt.	15.0-17.0	266633	2	12.4			SC-SM	A-1-b	II
BB-SBSR-102, 5D	13+23.9	8.0 Lt.	25.0-25.7	266634	2	26.1			SM	A-2-4	II
BB-SBSR-104, 1D	16+75.7	9.0 Lt.	1.0-3.0	266635	3	4.1			SW-SM	A-1-b	0
BB-SBSR-104, 3D	16+75.7	9.0 Lt.	10.0-12.0	266636	3	11.6			SM	A-1-b	II
BB-SBSR-104A, 1D	16+78.7	9.0 Lt.	20.0-21.0	266637	3	14.7			SW-SM	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.

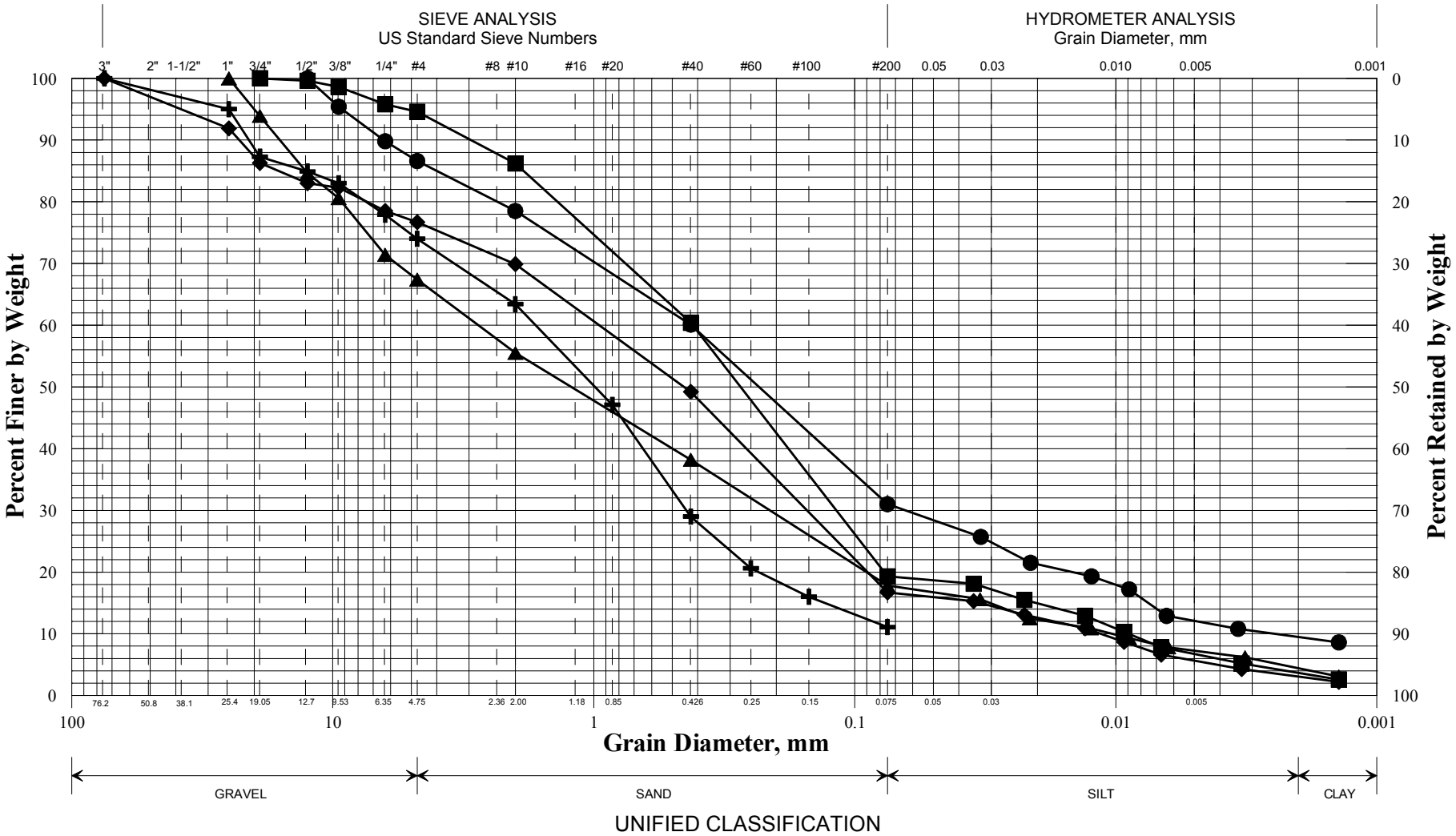
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



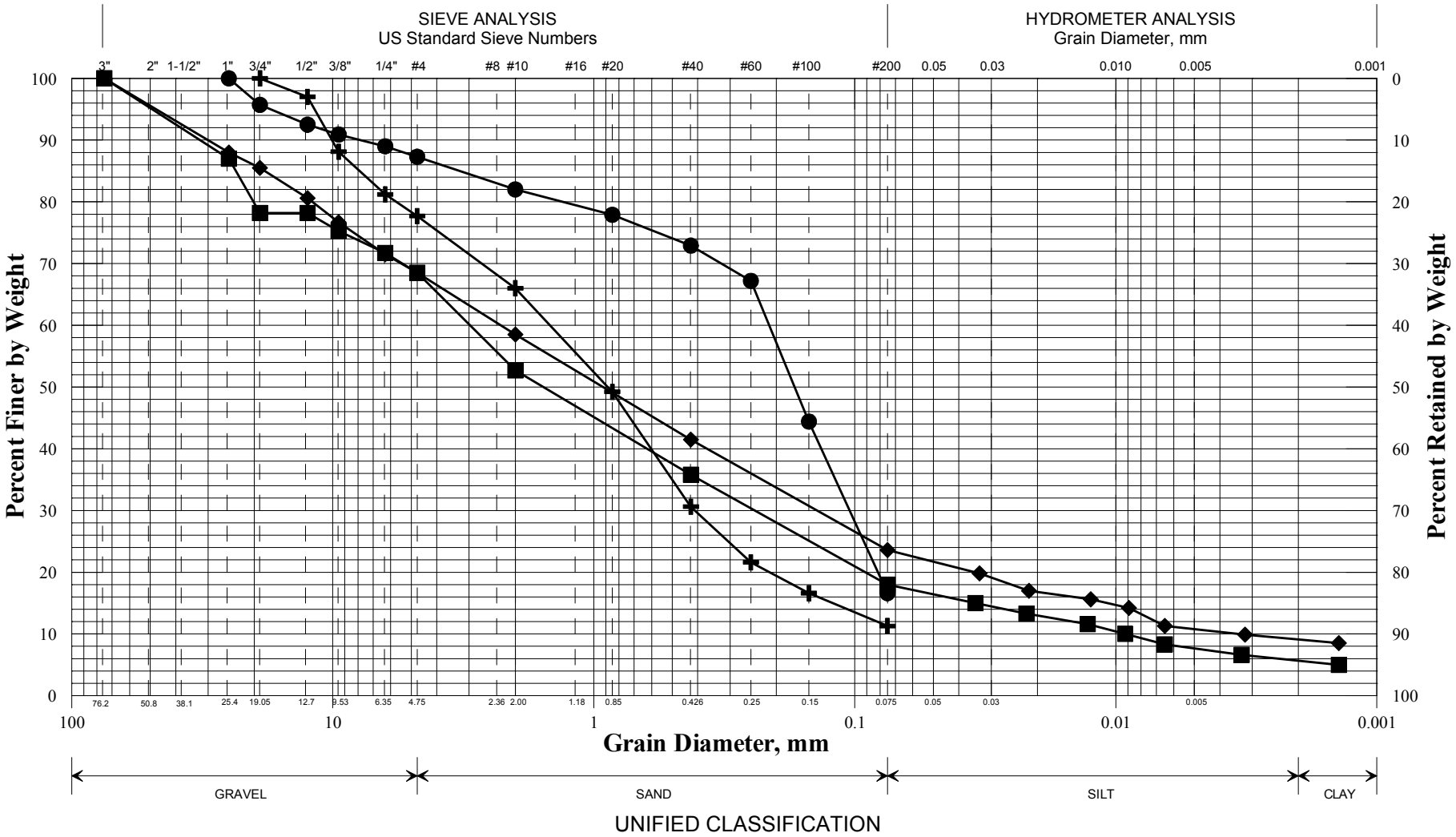
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBSR-101/1D	13+20.4	6.8 RT	5.0-7.0	SAND, some gravel, little silt.	4.4			
◆	BB-SBSR-101A/1DA	13+18.4	6.8 RT	10.5-12.0	SAND, some gravel, little silt, trace clay.	14.2			
■	BB-SBSR-101A/2D	13+18.4	6.8 RT	15.0-17.0	SAND, little silt, trace gravel, trace clay.	22.2			
●	BB-SBSR-101A/5D	13+18.4	6.8 RT	31.0-33.0	SAND, some silt, little gravel, trace clay.	12.1			
▲	BB-SBSR-101A/6D	13+18.4	6.8 RT	36.0-38.0	SAND, some gravel, little silt, trace clay.	9.0			
✖									

WIN
018233.00
Town
Saco,Biddeford
Reported by/Date
WHITE, TERRY A 7/10/2013

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

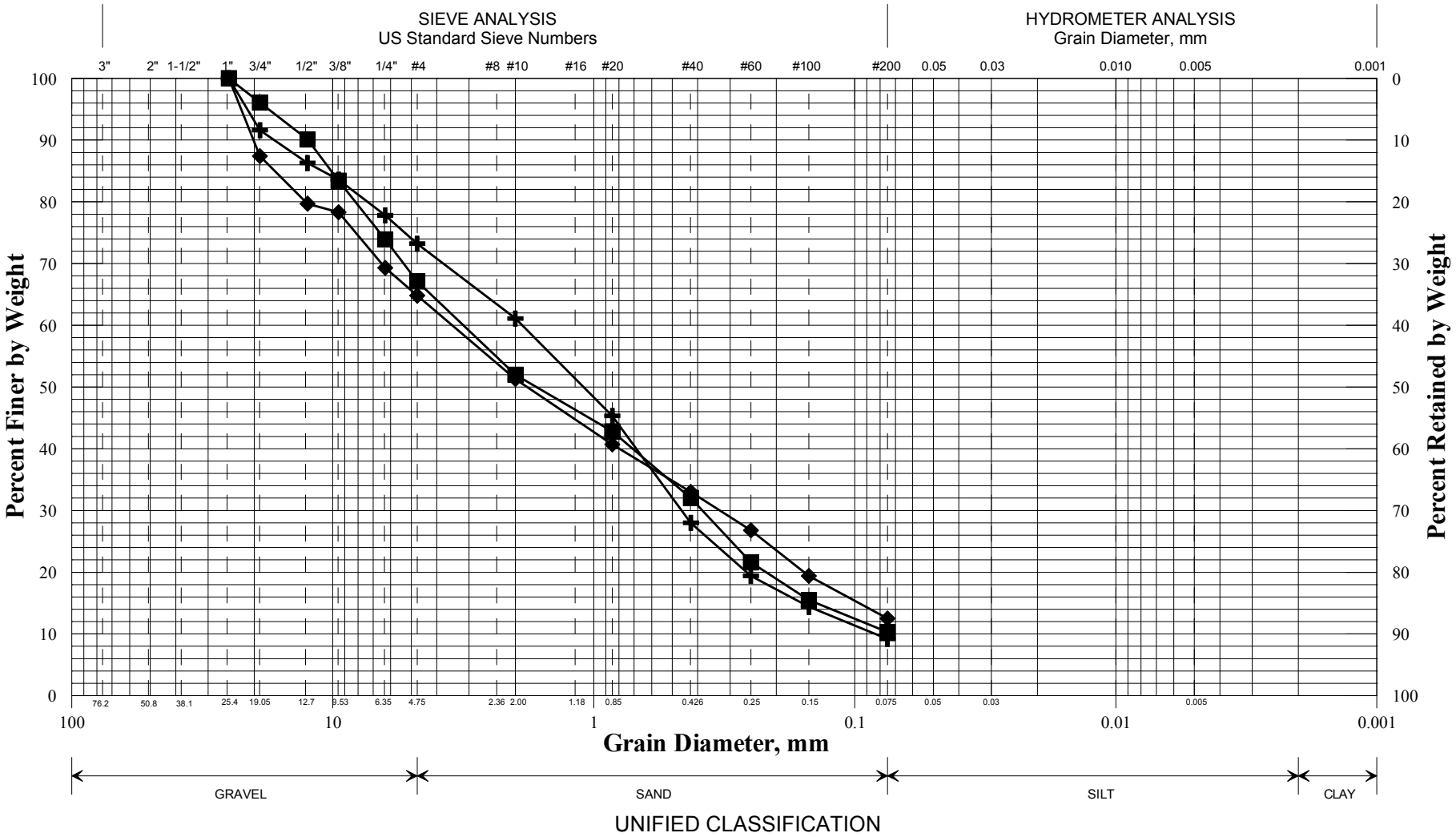


UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBSR-102/2D	13+23.9	8.0 LT	5.0-7.0	SAND, some gravel, little silt.	4.7			
◆	BB-SBSR-102/3DA	13+23.9	8.0 LT	10.0-12.0	SAND, some gravel, little silt, trace clay.	13.5			
■	BB-SBSR-102/4D	13+23.9	8.0 LT	15.0-17.0	SAND, some gravel, little silt, trace clay.	12.4			
●	BB-SBSR-102/5D	13+23.9	8.0 LT	25.0-25.7	SAND, little silt, little gravel.	26.1			
▲									
✖									

WIN
018233.00
Town
Saco,Biddeford
Reported by/Date
WHITE, TERRY A 7/10/2013

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBSR-104/1D	16+75.7	9.0 LT	1.0-3.0	SAND, some gravel, trace silt.	4.1			
◆	BB-SBSR-104/3D	16+75.7	9.0 LT	10.0-12.0	SAND, some gravel, little silt.	11.6			
■	BB-SBSR-104A/1D	16+78.7	9.0 LT	20.0-21.0	SAND, some gravel, trace silt.	14.7			
●									
▲									
✖									

WIN
018233.00
Town
Saco,Biddeford
Reported by/Date
WHITE, TERRY A 7/8/2013

Appendix C

Calculations

Earth Pressure:

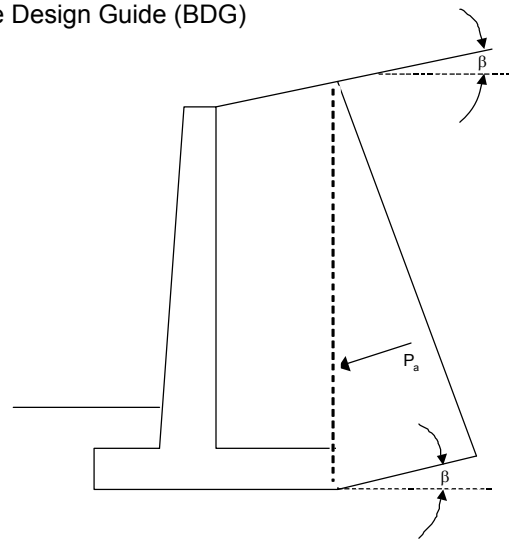
Active Earth Pressure - Rankine Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-7

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

unit weight: $\gamma_{type4} := 125 \cdot pcf$

Internal Friction Angle: $\phi_{type4} := 32 \cdot deg$

Cohesion: $c_{sand} := 0 \cdot psf$



Generally use Rankine for long heeled cantilever walls where the failure surface is un interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot deg$ assume horizontal backfill surface

$$K_{a\_rankine\_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}$$

$K_{a\_rankine\_slope} = 0.31$

Pa is oriented at an angle of β to the vertical plane.

At-Rest Earth Pressure
from Principles of Foundation Engineering, BM Das, 4th Edition
Eq. 6.3

$K_o := 1 - \sin(\phi_{type4})$ $K_o = 0.47$

Bearing Resistance for Abutments, Wingwalls, Pier and Cast-in-Place Retaining Wall on Bedrock:

SERVICE LIMIT STATE

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 6th Edition 2012
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

For Broken Rock of any kind:

Type of Bearing Material: Weathered or broken rock of any kind

Consistency In Place: medium hard rock

Bearing Resistance: Ordinary Range (ksf) 16 to 24

Recommended Value of Use: 20 ksf

Resistance factor at the **service limit state** = 1.0 (LRFD Article 10.5.5.1)

$q_{factored\_bc} := 20 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only at the service limit state.

STRENGTH LIMIT STATE

Determine Bearing Resistance using RMR Method

From AASHTO LRFD Bridge Design Specifications 6th Edition 2012
Section 10.4.6.4 Rock Mass Strength

Bedrock at the site is Sandstone which was found to be "very poor to good" in quality.
RQD ranged from 0 to 86%. (Average 48% - poor)

Determine RMR from Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

From AASHTO - RMR is determined as the sum of the five relative ratings listed in Table 10.4.6.4-1

1. Strength of intact rock

From Standard Specifications for Highway Bridges 17th Edition - 2002
Table 4.4.8.1.2B uniaxial compressive strength for Sandstone = 1,400 to 3,600 ksf = 9,700 to 25,000 psi

Use: $q_u := 1500 \cdot ksf$ $q_u = 10417 \cdot psi$

From Table 10.4.6.4.-1:
For Uniaxial Compressive Strength = 1080 to 2160 ksf: Relative Rating = 7

2. Drill Core Quality

Bedrock RQD = Average 48% (poor) From Table 10.4.6.4.-1: RQD 25% to 50%: Relative Rating = 8

3. Spacing of joints

Assume Spacing of 2 inches to 1 foot From Table 10.4.6.4.-1: Relative Rating = 10

4. Condition of joints

Assume slightly rough surfaces <0.05 in, soft joint wall rock From Table 10.4.6.4.-1: Relative Rating = 12

5. Groundwater conditions

General Conditions = Water under moderate pressure From Table 10.4.6.4.-1: Relative Rating = 4

Raw RMR = 41

Adjustment to RMR for joint Orientations from Table 10.4.6.4-2

Assume Strike and Dip Orientations of Joints = Fair For Foundations: Rating = -7

Adjusted RMR = 34 $RMR := 34$

Determine Rock Mass Class from Adjusted RMR Rating

For Adjusted RMR =34 From LRFD Table 10.4.6.4.-3: Class No. = IV - Poor Rock

Determine Rock Type from LRFD Table 10.4.6.4.-4

Rock Type C - Sandstone

Determine Rock Property constants *m* and *s:*

Reference: The Hoek and Brown Failure Criterion - a 1988 Update,
15th Canadian Rock Mechanics Symposium

m/m_i= exp ((RMR-100)/14) Eq 18 - for disturbed rock masses

where m_i = m for intact rock $m_i := 15$ From LRFD Table 10.4.6.4-4

$$m_{Cpoor} := m_i \cdot \exp\left(\frac{RMR - 100}{14}\right) \qquad m_{Cpoor} = 0.134$$

s = exp ((RMR-100)/6) Eq 19 - for disturbed rock masses

$$s_{Cpoor} := \exp\left(\frac{RMR - 100}{6}\right) \qquad s_{Cpoor} = 0.00002$$

<u>Determine nominal and factored bearing resistance of Bedrock:</u>

Foundation Shape correction factor:

$C_{f1} := 1.0$ From Foundations on Rock, Wyllie, Table 5.4 pg 138

Uniaxial Compressive Strength - Sandstone

$$q_{uc} := \begin{pmatrix} 9700 \\ 10417 \\ 15000 \\ 25000 \end{pmatrix} \cdot psi$$

Upper and lower bounds from from Standard Specifications for Highway Bridges 17th Edition - 2002 Table 4.4.8.1.2B

Determine Nominal Bearing Resistance:

From Foundations on Rock, Wyllie, Table 5.4 pg 138

$$q_{nom} := C_{f1} \cdot \sqrt{s_{Cpoor}} \cdot q_{uc} \cdot \left[1 + \sqrt{m_{Cpoor} \cdot \left(s_{Cpoor}^{\frac{-1}{2}}\right) + 1}\right] \qquad q_{nom} = \begin{pmatrix} 39 \\ 42 \\ 60 \\ 100 \end{pmatrix} \cdot ksf$$

Determine Factored Bearing Resistance at the Strength Limit State:

From Table 10.5.5.2.2-1 Resistance factor for footing on rock $\phi_b := 0.45$

The factored resistance $q_R = \phi_b \times q_n$ equation 10.6.3.1.1-1 AASHTO LRFD

$$q_R := \phi_b \cdot q_{nom} \qquad q_R = \begin{pmatrix} 18 \\ 19 \\ 27 \\ 45 \end{pmatrix} \cdot ksf$$

Recommend 19 ksf for Strength Limit State

Determine Factored Bearing Resistance at the Extreme Limit State:

Use a bearing resistance factor of 0.8 for Extreme Limit State for cantilever semi-gravity walls per LRFD Article C11.5.8. Use for pier for consistency with the theory of preventing collapse for the Extreme Event.

Resistance factor - $\phi_{bc} := 0.8$

$$q_{rEE} := \phi_{bc} \cdot q_{nom} \qquad q_{rEE} = \begin{pmatrix} 31 \\ 33 \\ 48 \\ 80 \end{pmatrix} \cdot ksf$$

For Gravity and Semigravity Walls

Recommend 33 ksf for Extreme Limit State

Frost Protection:

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
Saco/Biddeford, Maine
DFI = 1100 degree-days

From the lab testing: soils are coarse grained with a water content = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1100 and wc =10%
Frost Penetration = 69.8 inches

$Frost\_depth := 69.8 \cdot in$ $\quad$ $Frost\_depth = 5.8 \cdot ft$

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Sanford

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

Air Design Freezing Index	=	1123 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	898 F-days
Mean Annual Temperature	=	46.8 deg F
Design Length of Freezing Season	=	116 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	60.0	10.0	125.0	28	34	2.0	1.6	1,800

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

****
Total Depth of Frost Penetration = 5.00 ft = 60.0 in.
****

$Frost\_depth_{modberg} := 60 \cdot in$

$Frost\_depth_{modberg} = 5 \cdot ft$ $\quad$ Use Frost Depth = 5.0 feet for design

Seismic:

Seismic Site Classification
Ref: LRFD Table C3.10.3.1-1
Method B: Average N for the top 100 feet of soil

BB-ALAR-101/101A					BB-SBSR-102					BB-ALAR-104/104A				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
6	18	sand	7	0.39	2	30	sand	3	0.10	2	24	sand	3	0.13
11	19	sand	5	0.26	6	15	sand	4	0.27	6	8	sand	5	0.63
16	5	sand	5	1.00	11	26	sand	5	0.19	11	50	sand	5	0.10
21	5	sand	5	1.00	16	5	sand	5	1.00	16	21	sand	8	0.38
26	9	sand	5	0.56	26	50	sand	13	0.26	21	100	bedrock	79	0.79
31	13	sand	5	0.38	30	100	bedrock	70	0.70					
37	91	sand	8	0.09										
40	100	bedrock	60	0.60										
SUM			**100**	**4.28**	**SUM**			**100**	**2.52**				**100**	**2.02**
			di/di/N	**23.36**				**di/di/N**	**39.70**				**di/di/N**	**49.4816**
												SUM	Nav	38
												15<Nav<50 bpf; Site Class D		

18233 Saco Biddeford Somesville Bridge

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years Saco
State - Maine
Zip Code - 04072
Zip Code Latitude = 43.508200
Zip Code Longitude = -070.436200
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.091	PGA - Site Class B
0.2	0.179	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04072
Zip Code Latitude = 43.508200
Zip Code Longitude = -070.436200
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.145	As - Site Class D
0.2	0.286	SDs - Site Class D
1.0	0.107	SD1 - Site Class D

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years

Biddeford

State - Maine
Zip Code - 04005
Zip Code Latitude = 43.484000
Zip Code Longitude = -070.459500
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.091	PGA - Site Class B
0.2	0.180	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04005
Zip Code Latitude = 43.484000
Zip Code Longitude = -070.459500
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.146	As - Site Class D
0.2	0.288	SDs - Site Class D
1.0	0.107	SD1 - Site Class D

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

Purpose - The ground motion parameters obtained in this analysis are for use with the design procedures described in AASHTO Guidelines for the Seismic Design of Highway Bridges (2007). The user may calculate seismic design parameters and response spectra (both for period and displacement), for Site Class A through E.

Description - This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. In most cases the user may perform an analysis for a site by specifying location by either latitude-longitude (recommended) or zip code. However, locations in Puerto and the Virgin Islands may only be specified by latitude-longitude.

Ground motion maps are included in PDF format. These maps may be opened using a map viewer that is part of the software package.

Data - The 2007 AASHTO maps are based on 5% in 50 year probabilistic data from the U.S. Geological Survey data sets for the following regions: 48 conterminous states (2002), Alaska (2006), Hawaii (1998), Puerto Rico and the Virgin Islands (2003). These were the most recent data available at the time of preparation of the AASHTO maps. The AASHTO maps are labelled with a probability of exceedance of 7% in 75 years which is approximately equal to the 5% in 50 year data.

Disclaimer - Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.

Appendix D

Special Provision

SPECIAL PROVISION
SECTION 511
COFFERDAMS

Section 511 is deleted in its entirety and replaced with the following:

511.01 Description This work shall consist of the complete design, construction, maintenance and removal of cofferdams and other related work, including dewatering and inspection, required to allow for the excavation of foundation units, to permit and protect the construction of bridge or other structural units and to protect adjacent Roadways, embankments or other structural units, in accordance with the Contract.

511.02 Materials As specified in the cofferdam Working Drawings.

511.03 Cofferdam Construction

A. Working Drawings. The Contractor shall submit Working Drawings, showing the materials to be used and the proposed method of construction of cofferdams to the Department. Construction shall not start on cofferdams until such Working Drawings have been submitted. Any review of or comment on, or any lack of review of or comment on, these Working Drawings by the Department shall not result in any liability upon the Department and it shall not relieve the Contractor of the responsibility for the satisfactory functioning of the cofferdam.

B. Construction. Construct cofferdams in conformance with the submitted Working Drawings. Cofferdams shall, in general, be carried below the elevation of the bottom of footings to adequate depths to ensure stability and adequate heights to seal off water. Cofferdams shall be braced to withstand pressure without buckling, secured in place to prevent tipping or movement and be as watertight as necessary for the safe and proper construction of the substructure Work inside them. With the exception of construction of a concrete foundation seal placed under water, the interior dimensions of cofferdams shall provide sufficient clearance for the construction and inspection of forms and to permit pumping outside of forms. The Contractor shall be responsible for the righting and resetting of cofferdams that have tilted or moved laterally, as required for construction.

During the placing and curing of seal concrete, maintain the water level inside the cofferdam at the same level as the water outside the cofferdam, to prevent flow through the concrete.

No timber or bracing shall be used in cofferdams in such a way as to remain in the substructure Work.

Cofferdams shall be constructed to protect fresh concrete against damage from the sudden rising of the water body, to prevent damage by erosion and to prevent damage to adjacent Roadways, embankments or other structural units.

Unless otherwise noted, cofferdams, including all sheeting and bracing involved, shall be removed after the completion of the substructure Work in a manner that prevents disturbance or injury to the finished Work.

Cofferdams shall be constructed, dewatered and removed in accordance with the requirements of Section 656 - Temporary Soil Erosion and Water Pollution Control and related Special Provisions.

C. Construction Inspection of Seal Cofferdams. Seal cofferdam excavations shall initially be inspected and approved by the Contractor.

For each seal cofferdam excavation, the Contractor shall submit a written procedure to the Resident for sediment/overburden removal and excavation inspection. For cofferdams where seal concrete is to be placed on bedrock, the inspection procedure shall describe the Contractor's final cleaning and inspection process for attaining cleanliness of each cofferdam excavation. For cofferdams where seal concrete is not excavated to bedrock, the procedure shall describe the Contractor's final cleaning and inspection process for attaining the bottom of seal elevation shown on the Plans.

The Contractor shall notify the Resident at least 48 hours prior to when each seal cofferdam excavation will be ready for final inspection by the Department. The Contractor shall allow adequate time for each occurrence of cofferdam excavation inspection by the Department. The Contractor shall provide and maintain access and equipment, such as steel probes, for the Resident and/or the Department's Dive Team to independently inspect each cofferdam excavation.

No seal concrete placement shall begin until the Department has approved the cofferdam excavation.

511.04 Pumping Pumping from the interior of any cofferdam shall be done in such a manner as to prevent any current of water that would carry away or segregate the concrete.

Pumping to dewater a sealed cofferdam shall not commence until the seal concrete has set sufficiently to withstand the hydrostatic pressure and meets the following minimum curing time, after the completion of the installation of the seal concrete:

1. When the temperature of the water body outside the cofferdam is greater than 40°F, a minimum of 5 days.
2. When the temperature of the water body outside the cofferdam is less than 40°F, a minimum of 7 days.

Procedures for the removal of all water and materials from cofferdams shall be described in the Soil Erosion and Water Pollution Control Plan as required in Section 656 Temporary Soil Erosion and Water Pollution Control and related Special Provisions.

511.05 Method of Measurement Cofferdams will be measured as one lump sum unit, as indicated on the Plans or called for in the Contract.

511.06 Basis of Payment The accepted quantity of cofferdam will be paid for at the Contract lump sum price for the respective cofferdam items, which price shall be full compensation for design, construction, maintenance, inspection and removal.

When required, the elevation of the bottom of the footing of any substructure unit may be lowered, without change in the price to be paid for cofferdams. However, if the average elevation of more than 25% of the area of the excavation is more than 3 feet below the elevation shown on the Plans, and if requested by the Contractor, then the additional costs incurred that are included in the cofferdam Pay Item will be paid for in accordance with Section 109.7 - Equitable Adjustments to Compensation. The Contractor shall immediately notify the Department when these additional costs commence. Failure of the Contractor to provide this notification will result in undocumented additional work that will be non-reimbursable. The Department will evaluate this additional work to determine an appropriate time extension, if warranted.

All costs for sedimentation control practices, including, but not limited to, constructing, maintaining, and removing sedimentation control structures, and pumping or transporting water and other materials for sedimentation control will not be paid for directly, but will be considered incidental to the cofferdam Pay Item(s).

All costs for related temporary soil erosion and water pollution controls, including inspection and maintenance, will not be paid for directly, but will be considered incidental to the cofferdam Pay Item(s).

All costs associated with preparation of Working Drawings, design calculations, written procedure for sediment/overburden removal and excavation inspection, and the inspection of the seal cofferdam excavation shall be considered incidental to the cofferdam Pay Item(s). There shall be no additional payment for repeated inspection by the Department of the same cofferdam excavation.

All costs for cofferdams and related temporary soil erosion and water pollution controls, including inspection and maintenance, will be considered incidental to related Pay Items, when a specific Pay Item for cofferdams is not included in the Contract.

Seal concrete will be evaluated under Section 502.

Payment will be made under:

Pay Item	Pay Unit
511.07 Cofferdam	Lump Sum