GEOTECHNICAL DESIGN REPORT

18-0005

September 4, 2019

Explorations and Geotechnical Engineering Services

Staples Bridge #1238 Replacement
Card Mill Road over Great Works River
North Berwick, Maine
WIN 22336

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

S.W.COLE ENGINEERING, INC.

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

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TABLE OF CONTENTS

LIST OF ATTACHMENTS

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Subject: Geotechnical Design Report
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WIN 022336.00

Dear Laura:

In accordance with our Proposals dated February 15, 2018 and April 9, 2019, and project specific Assignment Letters #18 and #22 dated February 22, 2018 and April 19, 2019, we have made the requested subsurface explorations for the subject project. The purpose of our services was to obtain subsurface information in order to provide geotechnical design considerations and recommendations for foundations and earthwork associated with the proposed bridge replacement.

The services provided by S. W. Cole Engineering, Inc. (S.W.COLE) were conducted in accordance with our Multi-PIN Agreement with the Maine Department of Transportation (MaineDOT), No. 20150720000000000085, dated July 20, 2015. The contents of this report are subject to the limitations in Attachment A.

1.0 INTRODUCTION

1.1 Site Conditions

The existing Staples Bridge No. 1238 carries Card Mill Road over the Great Works River in North Berwick, Maine. The site location is shown on the "Site Location Map" attached in Appendix B. Based on the Highway Bridge Inspection Report dated August 17, 2015, we understand the existing single-span structure is 26 feet long (end-to-end), 18 feet wide (curb-to-curb) with a 19 foot clear span and zero skew. The existing bridge consists of steel girders with timber deck and paved wearing surface supported on stacked stone abutments. We understand the existing crossing was constructed in 1928 with a superstructure replacement in 1987. We understand the 1987 superstructure replacement included construction of concrete load distribution slabs at the abutments. Based on site observations, the southeast corner of the east abutment has been reconstructed with concrete. The Bridge Inspection Report indicates scouring of the river channel in the southeast corner.

The Preliminary Design Report (PDR) indicates the preferred replacement option consists of an off-alignment bridge replacement. We understand the replacement structure will consist of a 48-foot single-span, 16-foot wide (curb-to-curb) bridge constructed ±40 feet downstream from the existing bridge. We understand construction will include about 150 linear feet of new roadway for the south approach and alignment with 2:1(H:V), or flatter, approach side slopes and 1.75:1 riprap slopes in front of the new abutments. The proposed replacement structure and alignment are shown on the "Boring Location Plan/Interpretive Subsurface Profile" attached in Appendix B.

1.2 Proposed Construction

Based on discussions with MaineDOT and the project team, we understand the proposed bridge replacement will be advertised as a "detail-build" project. At this time, we understand acceptable bridge substructure alternatives include:

- Conventional, cast-in-place concrete spread footings founded on bedrock; and
- Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) founded on bedrock.

The proposed bridge replacement substructure shall be designed for all applicable load combinations for all relevant service, strength, and extreme limit states in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load Resistance and Factor Design (LRFD) Bridge Design Specifications, 8th Edition, 2017; relevant Maine Department of Transportation (MaineDOT) Bridge Design Guide (BDG) sections; and project-specific special provisions.

2.0 EXPLORATIONS AND TESTING

2.1 Explorations

2.1.1 Preliminary Phase Explorations

Two borings (BB-NBGWR-101 and BB-NBGWR-102) and 13 probes (BP-NBGWR-101 through BP-NBGWR-113) were made for the existing alignment on May 29 to 31, 2018 by S. W. Cole Explorations, LLC. Borings BB-NBGWR-101 and BB-NBGWR-102 were drilled approximately 12 feet and 18 feet behind the faces of the existing west and east abutments, respectively. Seven test probes were made behind the existing west abutment and six test probes were made behind the existing east abutment. Test probes were spaced approximately 6-inches to 2 feet apart to investigate the geometry of the abutment back face. The exploration locations were selected and established in the field by S.W.COLE using taped measurements from existing site features. The "as-drilled" exploration locations are shown on the "Boring Location Plan/Interpretive Subsurface Profile" attached in Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix C.

2.1.2 Design Phase Explorations

Two bridge borings (BB-NBGWR-201 and BB-NBGWR-202), two bridge probes (BP-NBGWR-201 and BP-NBGWR-202), and five roadway borings (HB-NBGWR-201 through HB-NBGWR-205) were made for the proposed off-alignment replacement on May 20 and 21, 2019 by S. W. Cole Explorations, LLC. These exploration locations were selected and established in the field by S.W.COLE. The ground surface elevations of the test borings were provided by MaineDOT. The "as-drilled" exploration locations are shown on the "Boring Location Plan/Interpretive Subsurface Profile" attached in Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix C.

2.2 Testing

The explorations were drilled using a combination of solid- and hollow-stem augers, cased-wash-boring and NQ rock core drilling techniques. The soils in the test borings were sampled at 2- to 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) methods using a rope and cathead with safety hammer to drive the split-spoon. Upon encountering refusal on bedrock, borings BB-NBGWR-201 and BB-NBGWR-202 were advanced about 10 feet into bedrock using NQ2 rock coring. The uncorrected SPT blow counts, uncorrected and corrected SPT N-values and rock core intervals are shown on the logs.

Soils and rock core samples recovered from the test borings were visually classified in our laboratory and transported to the MaineDOT Laboratory in Bangor, Maine, for testing to assist soil classification and identification. Laboratory testing was performed on disturbed SPT samples obtained during the explorations. Laboratory testing was performed by the MaineDOT Materials Testing and Exploration Central Laboratory in Bangor, Maine, in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) testing procedures. Laboratory testing included 2 natural water content tests and 2 grain size analyses without hydrometer. A summary and results of the laboratory testing are provided in Appendix D.

3.0 SUBSURFACE CONDITIONS

3.1 Surficial and Bedrock Geology

The Maine Geological Survey (MGS) Surficial Geology of the North Berwick Quadrangle, Maine (Open-File No. 99-92)[1], indicates the surficial soils at the site consist of Presumpscot Formation (silt and silty clay) with marine regressive sand/ice contact deposits (coarse gravel and sand) and glacial till mapped in the vicinity. The subsurface conditions encountered were generally consistent with the mapped surficial geology within the site vicinity; however, the explorations also encountered a surface deposit of fill soils from previous site development.

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[1] Smith, G.W., 1999, Surficial Geology of the North Berwick Quadrangle, Maine: Maine Geological Survey, Open-File Map 99-92.

The MGS Bedrock Geology of the Kittery Quadrangle, Maine (Geologic Map 16-6)[2], indicates the bedrock at the site consists of medium-bedded, medium brownish gray, feldspathic quartz-biotite granofels, greenish calc-silicate granofels, and subordinate quartz-biotite schist of the Berwick Formation. The bedrock conditions encountered were generally consistent with the mapped bedrock geology within the site vicinity.

3.2 Subsurface Conditions

The test borings along the proposed alignment encountered a soils profile generally consisting of a surface layer of forest duff and topsoil or fill overlying marine sands overlying refusal surfaces (bedrock). The test borings along the existing alignment encountered a soils profile generally consisting of a surface layer of pavement and fill overlying marine sands overlying refusal surfaces. The principal strata encountered in the explorations are summarized below. Refer to the attached logs for more detailed descriptions of the subsurface findings at the exploration locations.

Topsoil: Test borings BB-NBGWR-201, BB-NBGWR-202 and HB-NBGWR-202 through HB-NBGWR-204 encountered a 0.3- to 0.8-foot thick surface layer of forest duff and topsoil. The topsoil generally consisted of sand, some silt, trace to little gravel with organics.

Pavement: An approximate 4 inch layer of pavement was observed in BB-NBGWR-101 and BB-NBGWR-102.

Fill: From the ground surface in HB-NBGWR-201 and HB-NBGWR-205 and below pavement in BB-NBGWR-101 and BB-NBGWR-102, a 0.5- to 5-foot thick layer of fill was observed. The fill generally consisted of sand with varying amounts of gravel and silt. Below the granular fill in BB-NBGWR-101, a 1.4-foot thick layer of sand, some silt, little gravel and trace clay and organics was observed.

The fill was generally loose to medium dense with SPT N_{60} values ranging from 7 to 28 blows per foot (bpf).

Marine Sand: Below the fill, the borings generally encountered marine sand mantling relatively shallow refusal at depths of 9.2 to 12.3 feet below ground surface (bgs) corresponding to Elevation (El.) 135.3 to 128.1 feet. In general, the marine sand deposit consisted of sand with varying amounts of gravel and silt with cobbles and boulders.

The marine sand was generally medium dense to very dense with SPT N_{60} values ranging from 18 bpf to 50 blows for 3 inches (sampler refusal).

Bedrock: Bedrock was encountered and sampled in borings BB-NBGWR-101, BB-NBGWR-102, BB-NBGWR-201 and BB-NBGWR-202. The top of bedrock varied from about

[2] Hussey, A.M., II, Bothner, W.A., and Thompson, P.J., 2016, Bedrock Geology of the Kittery 1:100,000 Quadrangle, Maine and New Hampshire: Maine Geological Survey, Geologic Map 16-6.

9.2 to 12.3 feet bgs (El. 135.3 to 128.1 feet). The bedrock consisted of grey, hard, fresh to slightly weathered, quartz-biotite granofels.

The following table summarizes the approximate depths to bedrock, corresponding top of bedrock elevations and Rock Quality Designation (RQD) where encountered.

Boring Number (Substructure)	Approximate Depth to Bedrock (feet)	Approximate Bedrock Elevation (feet)	RQD (RMQ)
BB-NBGWR-101	10.3	135.3	33 to 100% (Poor to Excellent)
BB-NBGWR-102	12.3	132.6	40 to 41% (Poor)
BB-NBGWR-201 (Abutment No 2)	9.2	128.1	7 to 16% (Very Poor)
BB-NBGWR-202 (Abutment No 1)	11.7	131.7	66 to 76% (Fair to Good)

Rock quality designation (RQD) values for the bedrock generally ranged from 7 to 100 percent corresponding to a Rock Mass Quality (RMQ) of very poor to excellent. Detailed descriptions of the rock core and RQD values for each core run are shown on the exploration logs in Appendix C.

Refusal Surface: The probes encountered refusal surfaces interpreted as the abutment granite blocks or bedrock, based on drilling action. Refusal surfaces were encountered in the probes at the following depths:

Exploration No.	Location	Station / Offset	Refusal Depth (feet)	Refusal Elevation (feet)
BP-NBGWR-101	Existing South Abutment	Sta. 15+66.2, 43.1 ft Lt.	12.7	132.3
BP-NBGWR-102	Existing South Abutment	Sta. 15+68.2, 43.1 ft Lt.	7.5	137.5
BP-NBGWR-103	Existing South Abutment	Sta. 15+70.1, 42.9 ft Lt.	8.5	136.5
BP-NBGWR-104	Existing South Abutment	Sta. 15+70.6, 42.2 ft Lt.	3.5	141.5
BP-NBGWR-105	Existing South Abutment	Sta. 15+71.2, 42.9 ft Lt.	3.1	141.9
BP-NBGWR-106	Existing South Abutment	Sta. 15+72.1, 42.7 ft Lt.	3.8	141.2
BP-NBGWR-107	Existing South Abutment	Sta. 15+72.9, 42.7 ft Lt.	0.8	144.2
BP-NBGWR-108	Existing North Abutment	Sta. 16+1.9, 39.4 ft Lt.	4.2	140.8
BP-NBGWR-109	Existing North Abutment	Sta. 16+0.7, 38.3 ft Lt.	6.5	138.5
BP-NBGWR-110	Existing North Abutment	Sta. 16+2.7, 39.4 ft Lt.	7.3	137.7
BP-NBGWR-111	Existing North Abutment	Sta. 16+4.6, 39.0 ft Lt.	7.5	137.5
BP-NBGWR-112	Existing North Abutment	Sta. 16+6.6, 38.5 ft Lt.	8.5	136.5
BP-NBGWR-113	Existing North Abutment	Sta. 16+8.4, 37.9 ft Lt.	8.5	136.5
BP-NBGWR-201	Abutment No. 2	Sta. 16+6.0, 6.0 ft Rt.	10.3	127.0
BP-NBGWR-202	Abutment No. 1	Sta. 15+41.0, 13.5 ft Rt.	3.3	146.1

Probe BP-NBGWR-202 encountered what was interpreted to be a cobble or boulder at a depth of 3.3 feet bgs.

The exploration locations are shown on Boring Location Plan/Interpretive Subsurface Profile attached in Appendix B.

3.3 Groundwater Conditions

The soils encountered at the test borings were damp to wet from the ground surface. The measured water levels within the borings ranged from about 1 to 9 feet bgs. It should be noted that water was introduced during drilling; therefore, water levels indicated may not represent stabilized groundwater conditions. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate seasonally, particularly in response to periods of snowmelt and precipitation, changes in site use and the water level of the Great Works River.

4.0 GEOTECHNICAL EVALUATION

S.W.COLE conducted geotechnical engineering evaluations in accordance with 2017 AASHTO LRFD Bridge Design Specifications, 8^{th} Edition (AASHTO LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through June 2018 (MaineDOT BDG) and offers the following:

4.1 Foundation Alternatives

During the PDR stage, on-alignment rehabilitation and replacement alternatives as well as an off-alignment bridge replacement alternative were evaluated. Based on the need for a temporary detour bridge for the on-alignment alternatives, the off-alignment alternative was selected as the preferred alternative. It is our understanding that the existing bridge will remain in-service during construction of the replacement structure.

The final replacement structure will be designed and constructed utilizing the detail-build method and the type of structure will be designed, detailed, and constructed in accordance with Special Provision 531 “Bridge Structure – Detail Build” which will be developed and provided in the final Contract Documents. Based on the subsurface conditions present at the site the following foundation support alternatives are considered feasible:

- Conventional, cantilever-type cast-in-place concrete abutments and wingwalls on spread footings founded on bedrock; and
- Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS) founded on bedrock.

4.2 Settlement and Global Stability

4.2.1 Settlement

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

Any settlement of abutments or wingwalls founded on bedrock will be due to elastic compression of the bedrock mass, and is anticipated to be less than ½ inch.

4.2.2 Global Stability

We performed global stability evaluations for the new approach embankment construction at Sta. 16+20 using SLOPE/W software. We evaluated the global stability considering the resistance factors outlined in AASHTO LRFD, Section 11.6.2.3 and guidance in Section C11.6.2.3 as follows:

Global Stability for Static Conditions

FS ≥ 1.5 (ϕ =0.75) for slopes or walls supporting a structural element

FS ≥ 1.3 (ϕ =0.65) for slopes or walls not supporting a structural element

Global Stability for Seismic Conditions

FS ≥ 1.0 (ϕ =1)

In accordance with LRFD Article 11.6.5.2.2, the seismic condition includes a seismic load by incorporating a horizontal seismic coefficient, k_h of 0.06 g, equal to one-half of the calculated acceleration coefficient (A_s) of 0.119 g. Results of our global stability model runs are summarized in the following table and included in Appendix E.

Model	Safety Factor	
	Static	Seismic
Sta. 16+25 – 7 foot new embankment with 2H:1V side slopes	1.43	1.24

Results of our global stability model runs indicates the global stability for 2H:1V side slopes are above the referenced safety factors per LRFD Section 11.6.2.3. Results are included as Appendix E.

4.3 Conventional, Cantilever-Type Structure

4.3.1 Abutment and Wingwall Design

The proposed bridge replacement structure (abutments and wingwalls) shall be evaluated for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and designed for all applicable load combinations for all relevant service, strength, and extreme limit states in accordance with 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition (AASHTO LRFD), relevant Maine Department of Transportation (MaineDOT) Bridge Design Guide (BDG) sections, MaineDOT Standard Specifications, and project-specific special provisions.

4.3.1.1 Strength Limit State Design

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

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

4.3.1.2 Service Limit State Design

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

4.3.1.3 Extreme Limit State Design

Extreme limit state design checks for abutments and wingwalls shall include bearing resistance, eccentricity (overturning), failure by sliding and structural failure with respect to extreme event load conditions relating to seismic forces, hydraulic events and ice. Resistance factors, 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. LRFD Figures C11.5.6-1 and C11.5.6-2 illustrate the typical load factors to produce the extreme factored effect for bearing resistance and sliding and eccentricity.

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

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

4.3.2 Bearing Resistance and Eccentricity

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

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

Limit State	Bearing Resistance Factor φ_b	Factored Bearing Resistance (ksf)	LRFD Reference
Service	1.0	20.0	Article 10.5.5.1
Strength	0.45	18.5	Table 10.5.5.2.2-1
Extreme	0.8	32.8	Article C11.5.8

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

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

4.3.3 Sliding Resistance

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

Limit State	Sliding Resistance Factor φ_τ	Reference
Strength	0.8	LRFD Article C10.5.5.2.2
Service	1.0	LRFD Article 10.5.5.1
Extreme	1.0	LRFD Article 10.5.5.3.3

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

Bedrock subgrade preparation should occur in the dry. For bedrock subgrade prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.7 at the bedrock-concrete seal interface. For bedrock subgrades prepared in-the-wet, sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.6 at the bedrock-concrete seal interface.

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

4.4 Geosynthetic Reinforced Soil Integrated Bridge System (GRS-IBS)

GRS-IBS abutments and wingwalls shall be designed in accordance with *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide* (FHWA-HRT-11-026; January, 2011) and *Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report* (FHWA-HRT-11-027; January, 2011) referred to herein as FHWA 2011. The design and construction recommendations of GRS-IBS structures (abutment and wingwalls) presented in this report are based on the criteria outlined in these FHWA publications.

GRS-IBS structures are a specific group of AASHTO-defined Mechanically-Stabilized Earth (MSE) walls that utilize pre-approved, large, wet cast blocks (in accordance with Special Provision 672), closely spaced geosynthetic reinforcement in the soil mass and an integrated approach. GRS-IBS structures do not include MSE walls with steel strap soil reinforcement and MSE walls with precast panels.

A GRS-IBS supported structure shall consist of the following components:

- GRS abutments and wing walls founded on bedrock or concrete seal on bedrock; and
- Integrated bridge seat and approach.

4.4.1 Abutment and Wingwall Design

The design of GRS abutments and wingwalls shall be evaluated for direct sliding, bearing capacity, and global stability failure modes. Because a GRS mass is relatively ductile and free of tensile strength, overturning about the toe, in a strict sense, is not a possible response due to earth pressures at the back of the mass or loading on its top (FHWA 2011). Additionally, internal stability shall be analyzed for vertical capacity, deformations and reinforcement strength.

The proposed GRS structure (abutment and wingwalls) will be a U-shaped structure with an abutment width of about 27 feet and a wingwall length of about 15 feet. AASHTO LRFD Article 11.10.10.2, states the guardrail should be placed a minimum distance of 3 feet from the face of mechanically stabilized earth walls. Final design layout of the GRS abutments and wingwalls should ensure that the face of the abutment is wide enough to allow for placement of a guardrail and permit a guardrail lay down length of 4 feet.

4.4.1.1 Facing Elements

The most commonly used facing element for GRS abutments and walls is the split face concrete masonry unit (CMU) with nominal dimensions of 8 inches by 8 inches by 16 inches. There are currently no CMU blocks available in the State of Maine that meet the freeze-thaw requirements of Standard Specification 672. The PCMB units shall meet the requirements of the Project Plans and Special Provision 672. Facing block elements as approved by MaineDOT shall consist of Redi-Rock® wet-cast blocks (18-inches high, 28-inches deep).

GRS abutments and wingwall facing elements shall be founded directly on bedrock or a concrete seal on bedrock. The upper 2 feet of the facing block elements are susceptible to movement. To prevent displacement, hollow cores blocks in the top 2 feet of the abutment facing blocks shall be filled with concrete fill and pinned together with No. 4 epoxy coated rebar embedded with a 2-inch minimum cover. After the top block void is filled with concrete and rebar is inserted, a thin layer of concrete is placed in top of the block to form the coping cap. The concrete shall be ASTM Class A concrete with 4,000 psi compressive strength. If the facing blocks are solid, No. 4 epoxy coated rebar shall be drilled and grouted into the blocks to tie the upper 2 feet of the facing blocks together.

4.4.1.2 Geosynthetic Reinforcement

Geosynthetic reinforcement shall consist of biaxial, woven polypropylene geotextile with an ultimate tensile strength of 4,800 pounds per foot (lb/ft) and tensile strength at 2 percent strain of 1,370 lb/ft in each direction of load-bearing. Limiting the required reinforcement strength to less than the reinforcement strength at 2 percent strain will ensure long-term performance and serviceability. Any geosynthetic meeting the requirements of this section can be used in the abutment but a geotextile fabric must be used for the integrated approach to encapsulate the material. The permittivity and apparent opening size of a geosynthetic need to be considered to ensure adequate long-term drainage particularly when the abutment may be submerged at any point.

The geosynthetic shall be placed in closely spaced layers less than or equal to 12 inches. Where 18-inch high blocks are used, the abutment GRS design should assume 9-inch maximum geosynthetic reinforcing spacing. In accordance with FHWA 2011, the GRS shall have a minimum base width of 6 feet. The minimum reinforcing length at the lowest level shall have a base-to-height (B/H) ratio, including the facing block of 0.3 or greater. Once the base length of the reinforcing is chosen, the length of the reinforcing should follow the cut slope up to a B/H ratio of 0.7. From there the reinforcing length can get progressively longer in reinforcement zones. The progressively longer lengths of reinforcing serve to improve the quality of construction and overall stability of the GRS abutment. The details of the reinforcement zones (i.e., numbers of layers and lengths) are determined during final design.

A bearing reinforcement zone is required under the bridge seat to support the increased loads due to the bridge. The bearing bed reinforcement spacing should be half the primary spacing (minimum). If the required strength in the bearing bed reinforcement zone at the 9-inch spacing does not exceed the allowable strength or the strength at 2 percent strain, intermediate layers of geotextile may not be necessary. The minimum length of the bearing bed reinforcement should be twice the setback plus the width of the bridge seat. The depth of the bearing reinforcement zone is determined based on internal stability design for required reinforcement strength. At a minimum there should be five bearing bed reinforcement layers.

The integration zone is part of the integrated approach of GRS-IBS behind the bridge superstructure to limit the development of tension cracks and to blend the approach way on to the roadway. The integration zone reinforcement layers should be blended to create a smooth transition. The number of reinforcement layers in the integration zone depends on the height of the superstructure but each wrapped layer should be no more than 12 inches in height. The top layer of the integration zone should extend beyond the cut slope to prevent moisture infiltration.

4.4.1.3 Reinforced Backfill Material

The reinforced backfill is a major structural component for the GRS abutment. Abutment reinforced backfill shall consist of clean, crushed angular (not rounded), hard, durable particles or fragments of stone or gravel. These materials shall be free from organic matter or deleterious material such as shale or other soft particles that have poor durability. Reinforced backfill typically consists of open-graded backfill due to the relative ease of construction and favorable drainage characteristics. Since the lower sections of the abutments will be submerged, open-graded backfill should be used because it is free draining. Open-graded backfill material shall meeting the following gradation. The friction angle of the open-graded backfill shall be no less than 38 degrees.

Open Graded Backfill MaineDOT Standard Specification 518.02 Designation SP-2-89	
Sieve Size	**Percent Finer by Weight**
½ inch	100
⅜ inch	90-100
No. 4	20 to 55
No. 8	5 to 30
No. 16	0 to 10
No. 50	0 to 5
No. 200	0 to 1.5

The backfill shall be compacted to a minimum of 95 percent of maximum dry density according to AASHTO T-99. Lifts of 6 to 9 inches, depending on facing block size, should be compacted using vibratory roller compaction equipment. Since the facing elements are not rigidly connected to the reinforcing, hand-operated compaction equipment is required within 3 feet of the back of the wall. The top 5 feet of the abutment shall be compacted to 100 percent of the maximum dry density according to AASHTO T-99.

The lateral stress distribution due to the weight of the GRS fill is found using the Rankine active earth pressure coefficient, K_{ar}, of 0.24. This K_{ar} is used for calculating the required reinforcement strength. For the internal stability analysis of the GRS mass, the ultimate load carrying capacity of the GRS mass is computed using the Rankine passive earth pressure coefficient, K_{pr}, of 4.2.

4.4.1.4 Integrated Approach Backfill

The GRS located directly behind the beam is necessary to provide a smooth integrated transition from the approach way to the bridge deck. FHWA 2011 recommends that the fill material used for this transition be well-graded gravel.

However, in an effort to specify one type of aggregate for all GRS components (the reinforced fill zone and the integration zone) it is recommended that the fill material in the integration zone consist of the open-graded gravel. The integrated approach backfill shall be compacted to a minimum of 95 percent of maximum dry density according to AASHTO T-99.

4.4.2 Bearing Resistance

For GRS abutments and wingwall footings founded on competent, sound bedrock we recommend the following factored bearing resistance at the strength limit state of 26.7 kips. A factored bearing resistance at the service limit state of 20 kips shall be used for the GRS-IBS abutments founded on bedrock to control settlement when analyzing the service limit state as allowed in AASHTO LRFD Table C10.6.2.6.1-1. These factored bearing resistances must be greater than the applied factored vertical bearing pressure determined by the structural designer for the applicable limit state.

The following resistances factors shall be used in design of the GRS-IBS structure.

Application	Resistance Factor	LRFD Reference
Bearing Resistance, ϕ_{bc}	0.65	Table 11.5.7-1
Direct Sliding, ϕ_r	1.0	Table 11.5.7-1
Global Stability, ϕ	0.65	Article 11.6.2.3
Vertical Capacity, ϕ_{cap}	0.45	FHWA 2011
Reinforcement Strength, ϕ_{reinf}	0.4	FHWA 2011

4.5 Earth Pressure and Surcharge

4.5.1 Earth Pressure

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

Structure Type	Rankine Active Earth Pressure Coefficient, k_a	At-rest Earth Pressure Coefficient, k_o
Conventional, cantilever-type cast-in-place concrete	0.28	0.47
Geosynthetic Reinforced Soil Integrated Bridge System	0.24	N/A

The resultant Rankine earth pressure is orientated perpendicular to the wall back-face.

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

- Internal Friction Angle, ϕ = 32 degrees
- Total Unit Weight, γ = 125 pcf

In accordance with FHWA 2011, the designer may assume the following soil properties for the open-graded backfill for the GRS structure:

- Internal Friction Angle, ϕ = 38 degrees
- Total Unit Weight, γ = 125 pcf

The friction angle of the open-graded backfill shall be confirmed with laboratory direct shear testing in accordance with AASHTO T 236 prior to use.

4.5.2 Surcharge Pressure

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 if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads, is permitted per LRFD Article 3.11.6.5.

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

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

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

4.6 Frost

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

However, foundations placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. Based on the MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, the design freezing index for the North Berwick, Maine area is approximately 1,200 freezing degree-days. Based on Section 5.2.1 of the MaineDOT BDG and assuming a water content of 10% for new granular fills, the maximum seasonal frost penetration is estimated to be approximately 6.1 feet. Considering this, we recommend foundations constructed within or on granular fill be founded with least 6.1 feet of soil cover to provide frost protection.

4.7 Seismic Design Considerations

Seismic site class was evaluated in accordance with AASHTO Section 3.10.3.1 Method B (average N-value method). An N-value of 100 bpf was assumed for the profile below the refusal surface. Based on the subsurface information, the average N-value fell between 50 and 100 bpf corresponding to an AASHTO Site Class C as defined in AASHTO Table 3.10.3.1-1.

The USGS online Seismic Design Maps Tool was used to obtain the seismic design parameters for the site. Based on the assigned site class (AASHTO Site Class C) and site coordinates, the software provides the recommended AASHTO Response Spectrum for a 7% probability of exceedance in 75 years (1,000-year return period). The results for the project site are summarized below:

Recommended Seismic Design Parameters[3]	
Site Class	C
PGA	0.099 g
S_s	0.191 g
S_1	0.045 g
F_{pga}	1.2
F_a	1.2
F_v	1.7
A_s	0.119 g
S_{DS}	0.229 g
S_{D1}	0.077 g
Seismic Zone (based on S_{D1})	Zone 1

NOTE: Site Coordinates: N43.316808, W70.744094

4.8 Scour and Riprap

4.8.1 Conventional, Cantilever-Type Structure

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

Bridge and channel soil slopes above the soil-bedrock interface shall be armored with at least 3 feet of riprap. Riprap shall conform to MaineDOT Standard Specification 703.26 “Plain and Hand Laid Riprap” and should be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a 1 foot layer of MaineDOT Standard Specification 703.19 “Granular

[3] U.S. Geological Survey, Seismic Design Map, , accessed July 6, 2018
http://earthquake.usgs.gov/designmaps/us/application.php

Borrow Material for Underwater Backfill" and a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

4.8.2 GRS-IBS Structure

A grain size analysis was performed on a soil sample taken from the approximate streambed elevation to generate a grain size curve for determining parameters to be used in scour analyses. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 6.7 mm
- Average diameter of particle at 95 percent passing, D_{95} = 38.1 mm
- Soil Classification AASHTO Soil Type A-1-a.

The grain size curve is included on Sheet 1 attached in Appendix D.

The design of scour countermeasures for GRS abutments is outlined in FHWA 2011 (Section 5.3 Hydraulic Design Considerations), including a typical cross section for the detailing of a riprap apron adapted from HEC-23. At a minimum, for scour protection and protection of the GRS-IBS structure, the bridge approach slopes and slopes at abutments should be armored with 3 feet of plain riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap shall conform to MaineDOT Standard Specification 703.26 "Plain and Hand Laid Riprap" and should be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a 1 foot layer of MaineDOT Standard Specification 703.19 "Granular Borrow Material for Underwater Backfill" and a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

4.9 Construction Considerations

Construction activities for new abutments and any retaining walls will require earth and loose, weathered bedrock excavation. The construction of cofferdams will be needed to control the Great Works River during placement of seal or spread footing concrete. Earth support systems may be needed to support the approach soils. New approach embankment construction will be needed.

4.9.1 Bedrock Removal and Subgrade Preparation

If included in design, the abutment and wingwall foundation subgrades should consist of sound bedrock or mud slab over sound bedrock. The borings and probes at the proposed abutment locations encountered bedrock at depths of approximately 9 to 12 feet below the existing ground surface. The bedrock cored at the site indicates that the bedrock is moderately to highly fractured. The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations for the abutments are made.

The bedrock surface shall be cleared of all loose fractured bedrock, decomposed bedrock and soil to expose sound, intact bedrock. The final bearing surface shall be solid. If the bedrock surface is observed to slope steeper than 4H:1V at the subgrade elevation in any direction, the bedrock shall be benched to create level steps or excavated to be completely level. Excavation of highly sloped and loose fractured bedrock material shall be made using all conventional excavation methods (digging bucket, ripper tooth, hoe-ramming) possible in attempt to create level steps or be completely level. Based on the proximity to the existing bridge structure, we recommend bedrock excavation by blasting be avoided. Anchors or dowels may also be designed and employed to improve sliding resistance where the prepared bedrock surface is steeper than 4H:1V in any direction. The bottom of footing or concrete seal elevation may vary based on the presence of fractured bedrock and the variability of the bedrock surface.

The final bearing surface shall then be washed with high pressure water and air prior to concrete being placed for the footing. The final bedrock subgrade surfaces shall be approved by the Resident or Project Geotechnical Engineer prior to placement of concrete seal or footing concrete.

4.9.2 Dewatering

The Contractor should control groundwater and surface water infiltration into excavations throughout construction. The contractor should use temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment or other means to divert surface water and groundwater, if significant seepage is encountered, during construction.

Excavations for abutments and wingwalls, if included in design, will extend below the level of the Great Works River and groundwater will seep from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

4.9.3 Approach Embankments

Approach embankment areas should be cleared and grubbed to remove existing surficial topsoil, soils with organics, vegetation and stumps. The borings along the proposed approach alignment encountered topsoil and soils with organics to depths of approximately 0.2 to 2.5 feet below the existing ground surface. The final subgrade surfaces shall be approved by the Resident or Project Geotechnical Engineer prior to placement of approach embankment fills.

Approach embankment slopes outside of abutment and wingwall backfill envelope should be designed as earth fill slopes no steeper than 2H:1V. Slopes steeper than 2H:1V typically require reinforcement or rock fill surfacing. We recommend new embankment fill be thoroughly and systematically compacted to the full limit of the slope. Where new fill slope extensions are constructed over existing slopes, we recommend benching the existing slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, to prevent creation of a preferential slip plane under the new embankment fill.

4.9.4 Reuse of Excavated Soil and Bedrock

The native silty soils or existing fill soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that prior to placement and compaction it may be necessary to spread out and dry portions of these soils that are wet of optimum moisture content.

4.9.5 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices guidelines to minimize erosion of the fine-grained soils at the site.

4.9.6 Weather Considerations

The silty native soils are susceptible to disturbance and rutting as a result of exposure to water or construction traffic. We recommend the contractor protect the subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that the contractor remove and replace the disturbed materials and replace with compacted Gravel Borrow.

4.9.7 GRS-IBS

For scour protection, the GRS abutments should be moved away from the channel. Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the abutments, slope construction and riprap placement are of critical importance. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the Plans.

Careful attention should be given to the installation of the first row of blocks. Since all other courses of block are built off the first row, it is essential to ensure that the bottom row is level and even for construction.

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill.

In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

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.

5.0 CLOSURE

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

Sincerely,

S. W. Cole Engineering, Inc.

Michael St. Pierre, P.E.
2019.09.04 15:44:46 -04'00'

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

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

Robert E. Chaput, P.E.
Senior Geotechnical Engineer

MAS:tjm-rec

S.W.COLE ENGINEERING, INC.

APPENDIX A

Limitations

This report has been prepared for the exclusive use of the Maine Department of Transportation for specific application to the Staples Bridge #1238 Replacement carrying Card Mill Road over Great Works River (MaineDOT WIN 022336.00) in North Berwick, Maine. S. W. Cole Engineering, Inc. (S.W.COLE) has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

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

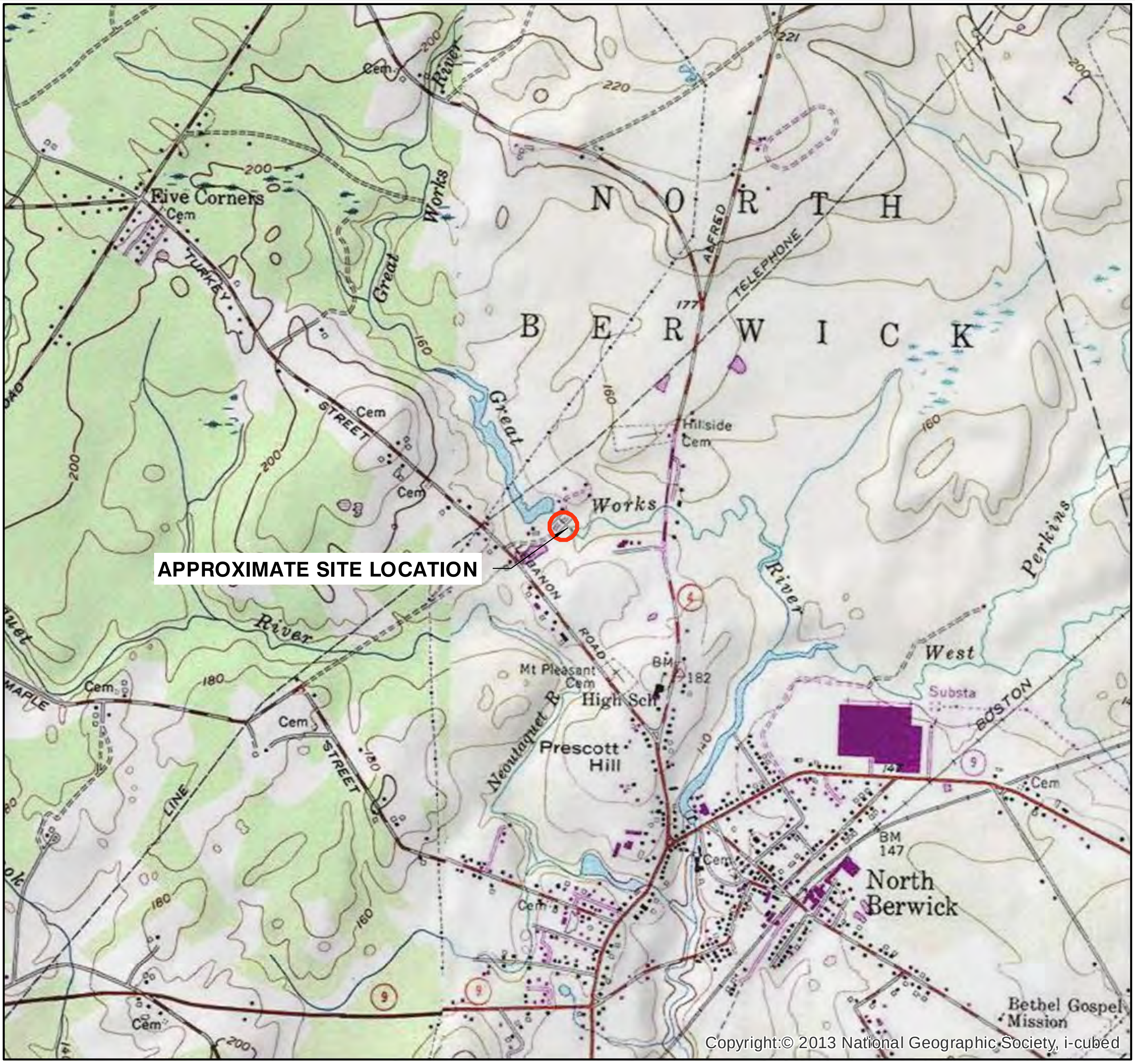
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations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

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

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

S.W.COLE ENGINEERING, INC.

APPENDIX B
Figures



S.W.COLE ENGINEERING, INC.

MAINE DEPARTMENT OF TRANSPORTATION

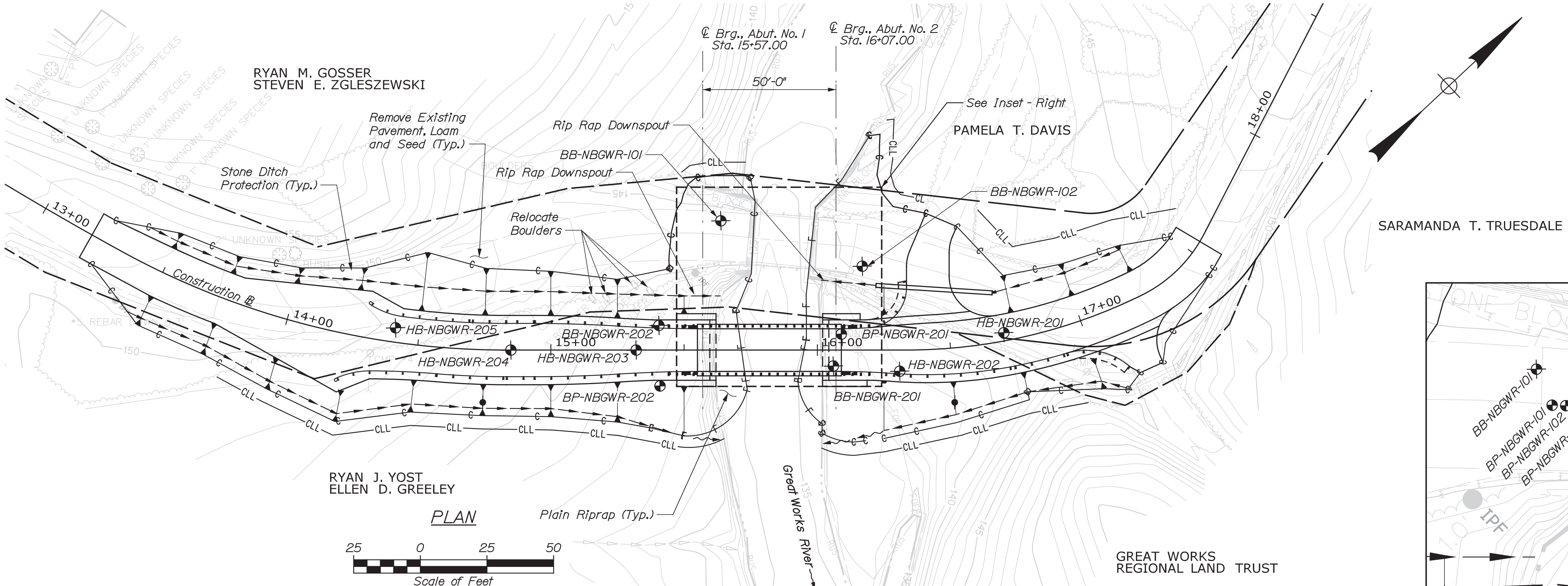
SITE LOCATION MAP

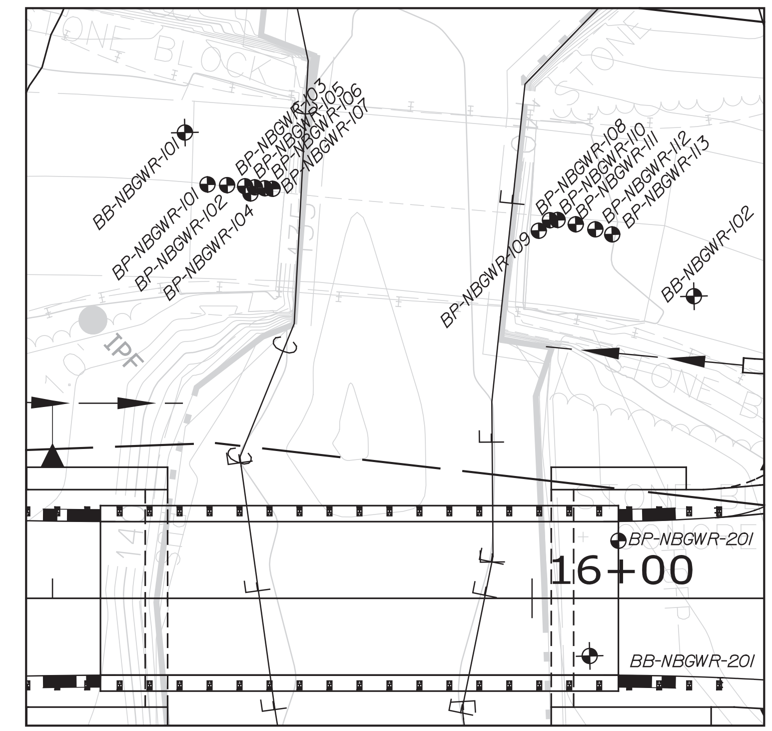
STAPLES BRIDGE #1238 REHABILITATION
CARD MILL ROAD OVER GREAT WORKS RIVER
NORTH BERWICK, MAINE
WIN 022336

Job No.	18-0005	Scale	1:24000
Date:	09/04/2019	Sheet	1

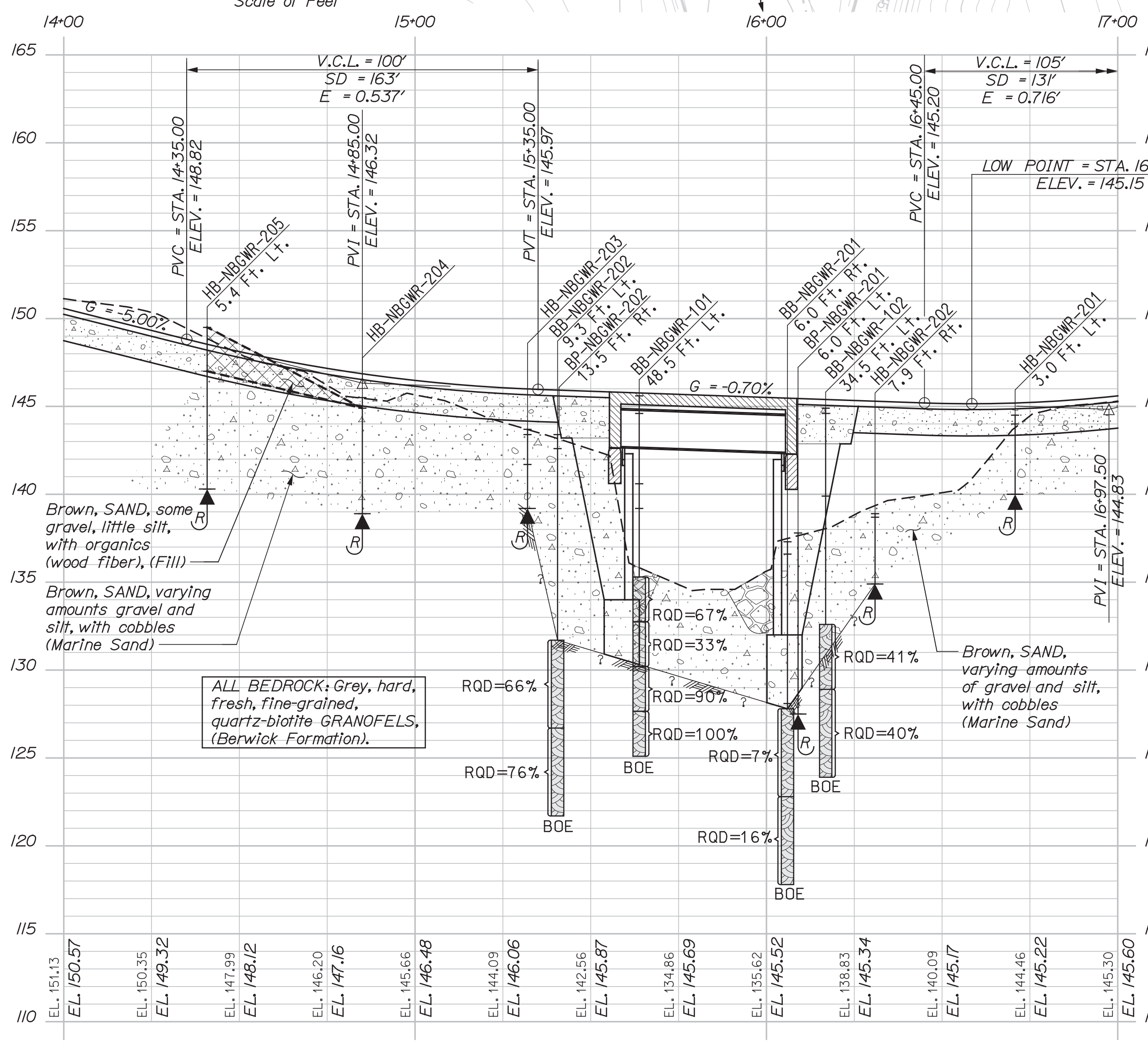
NOTE:
SITE LOCATION MAP PREPARED FROM ESRI ArcGIS ONLINE AND DATA PARTNERS INCLUDING USGS AND © 2007 NATIONAL GEOGRAPHIC SOCIETY.

R:\2018\18-0005\GIS\MXDs\18-0005 SLM.mxd, 8/7/2018 3:35:33 PM, 1:24,000,





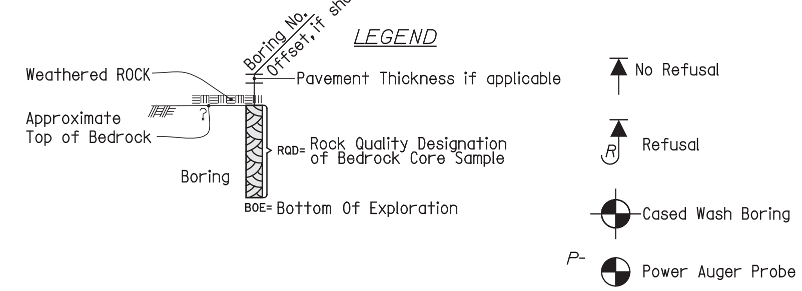
INSET
SCALE: 1" = 10'



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.

LEGEND



PROFILE

Horiz. 25 0 25 50
Vert. 5 0 5 10
Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2233(600)

WIN 022336.00

BRIDGE PLANS

BRIDGE NO. 1238

	BY	DATE
PROJ. MANAGER JOEL KITTREDGE		
DESIGN-DETAILED	E.BROWNELL	D.BURGESS 08-08-19
CHECKED-REVIEWED	E.BROWNELL	J.BURGESS 08-08-19
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

STAPLES BRIDGE
GREAT WORKS RIVER
NORTH BERWICK YORK COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 2

Filename: ...\00\BRIDGE\MSTA\005_BLP&ISP.dgn Division: BRIDGE Username: common Date:9/4/2019

S.W.COLE ENGINEERING, INC.

APPENDIX C

Boring Logs & Key to Soil and Rock Descriptions and Terms

UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

MODIFIED BURMISTER SYSTEM

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: BB-NBGWR-201
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	137.3	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/20/2019	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 16+06.0, 6.0 ft Rt	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	1.0 ft (during drill)
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	1/1/1/2	2	2	HSA	136.6		Forest Duff over dark brown, moist to wet, very loose, Sandy SILT, little gravel, with organics (rootlets), (Topsoil). —0.7— Brown, moist to wet, very loose, SAND, some silt, little gravel. Red-brown mottling from 0.8 to 1.2 feet bgs.	
5	2D	24/14	4.00 - 6.00	43/14/21/25	35	35				Brown, wet, dense, Sandy Gravel, trace silt, with cobbles, (Marine Sand).	G#337285 A-1-a, GW-GM WC=12.2%
										Frequent cobbles below 7 feet bgs	
10	R1	60/24	9.50 - 14.50	RQD = 7%			~~RC~~ NQ2	128.1		—9.2— Top of Bedrock at Elev. 128.1 ft. Advanced by roller cone from 9.2 to 9.5 feet bgs to seat casing. R1:Bedrock: Grey, fine-grained, quartz biotite GRANOFELS with quartzite veins, trace pyrite, hard, fresh, joints are moderate dipping (35-55 degrees), very close to close and open to healed with quartzite, (Berwick Formation). Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 9.5-10.5 ft (1:20) 10.5-11.5 ft (4:20) 11.5-12.5 ft (4:40) 12.5-13.5 ft (3:30) 13.5-14.5 ft (9:50) 40% Recovery.	
15	R2	60/49	14.50 - 19.50	RQD = 16%				117.8		R2:Bedrock: Similar to R1 except joints are low to moderate dipping (5 to 55 degrees). Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 14.5-15.5 ft (3:15) 15.5-16.5 ft (2:15) 16.5-17.5 ft (2:18) 17.5-18.5 ft (3:24) 18.5-19.5 ft (9:58) 82% Recovery. —19.5—	
20										**Bottom of Exploration at 19.5 feet below ground surface.**	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: BB-NBGWR-202
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	143.4	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/21/2019	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 15+40.6, 9.3 ft Lt	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	1.0 ft (during drill)
Hammer Efficiency Factor: 0.60		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	1/1/2/8	3	3	HSA	142.6		Forest duff over dark brown, moist to damp, very loose, Sandy SILT, little gravel, with organics (rootlets), (Topsoil). 0.8	
										Red-brown, damp, loose, SAND, little silt, trace gravel, (Marine Sand).	
								139.9		3.5	
5	2D	24/6	5.00 - 7.00	7/15/16/20	31	31				Brown with orange staining, wet, dense, Sandy GRAVEL, little silt, (Marine Sand).	
										Frequent cobbles below 7.5 feet bgs.	
10	3D	20/12	10.00 - 11.67	14/20/29/50-2"	49	49	RC			Similar to above except very dense.	
	R1	60/48	11.70 - 16.70	RQD = 66%			NQ2	131.7		11.7 Top of Bedrock at Elev. 131.7 ft. R1:Bedrock: Grey, fine-grained, quartz biotite GRANOFELS with quartzite veins hard, fresh, joints are moderate dipping to steep (35-85 degrees), very close to moderate close and tight to open, (Berwick Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 11.7-12.7 ft (4:15) 12.7-13.7 ft (3:55) 13.7-14.7 ft (4:30) 14.7-15.7 ft (4:20) 15.7-16.7 ft (5:15) 80% Recovery.	
15	R2	60/59	16.70 - 21.70	RQD = 76%						R2:Bedrock: Similar to R1. Rock Mass Quality = Good. R2:Core Times (min:sec) 16.7-17.7 ft (5.18) 17.7-18.7 ft (5:50) 18.7-19.7 ft (6:48) 19.7-20.7 ft (2:52) 20.7-21.7 ft (9:58) 98% Recovery.	
20								121.7		21.7 **Bottom of Exploration at 21.7 feet below ground surface.**	
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: HB-NBGWR-201
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	144.5	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/20/2019	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	Sta. 16+70.8, 3.1 ft Lt	**Casing ID/OD:**	N/A	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	10/9/6/4	15	15	HSA	144.0		Dark brown, damp, medium dense, SAND, some gravel, little silt, with organics (rootlets) and asphalt fragments, (Fill). 0.5	
										Brown, moist, medium dense, Gravelly SAND, little silt, (Marine Sand).	
								140.0		4.5	
5										**Bottom of Exploration at 4.5 feet below ground surface.** Auger refusal.	
10											
15											
20											
25											

Remarks:

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Boring No.: HB-NBGWR-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: HB-NBGWR-202
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	138.9	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/20/2019	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	Sta. 16+30.9, 7.9 ft Rt	**Casing ID/OD:**	N/A	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/0	0.00 - 2.00	1/1/1/1	2	2	HSA	138.7		Forest duff over topsoil. 0.2	
										From auger cutting. Brown, moist to wet, Sandy SILT, little gravel, (Marine Sand).	
										Cobble at 3 feet bgs.	
								134.9		4.0	
										Bottom of Exploration at 4.0 feet below ground surface. Auger refusal.	
5											
10											
15											
20											
25											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: HB-NBGWR-203
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	143.7	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/21/2019	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	Sta. 15+32.0, 0.0 ft	**Casing ID/OD:**	N/A	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.00 - 2.00	1/1/2/3	3	3	HSA	143.4		Forest Duff over dark brown, moist, loose, Sandy SILT, trace gravel, with organics (rootlets), (Topsoil). 0.3	
										Brown, damp, loose, Sandy SILT, little gravel. 2.0	
	2D	5/4	2.00 - 2.42	50-4"	- -			141.7		Grey with red mottling, moist, very dense, Silty SAND, some gravel, (Marine Sand). Cobble at 2.4 feet bgs. 4.5	
5								139.2		**Bottom of Exploration at 4.5 feet below ground surface.** Auger refusal.	
10											
15											
20											
25											

Remarks:

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Boring No.: HB-NBGWR-203

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: HB-NBGWR-204

WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	145.5	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/21/2019	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	Sta. 14+85.0, 0.0 ft	**Casing ID/OD:**	N/A	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.00 - 2.00	1/1/3/9	4	4	HSA	144.9		Forest Duff over dark brown, moist, loose, SILT, some sand, little gravel, with organics (rootlets), (Topsoil). 0.6	
	2D	24/18	2.00 - 4.00	14/17/14/11	31	31				Brown with orange staining, damp to moist, dense, Sandy SILT, some gravel, (Marine SAND).	G#337284 A-4, CL WC=13.0%
5	3D	21/6	5.00 - 6.75	9/12/45/50-1"	57	57				Brown, wet, very dense, Silty SAND, some gravel, (Marine Sand).	
								138.9		6.6 **Bottom of Exploration at 6.6 feet below ground surface.** Sampler Refusal.	
10											
15											
20											
25											

Remarks:

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Boring No.: HB-NBGWR-204

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: HB-NBGWR-205
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	149.5	**Auger ID/OD:**	2.25/4.5 inch Hollow Stem
Operator:	S. Shaw	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	A. Santiago	**Rig Type:**	Diedrich D-50	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	5/21/2019	**Drilling Method:**	Hollow Stem Auger	**Core Barrel:**	N/A
Boring Location:	Sta. 14+40.9, 5.4 ft Lt	**Casing ID/OD:**	N/A	**Water Level*:**	Not observed
Hammer Efficiency Factor:	0.60	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.00 - 2.00	4/4/3/8	7	7	HSA			Brown, damp, loose, SAND, some gravel, little silt, with organics (wood fiber), (Fill).	
										Similar to above except medium dense.	
	2D	24/8	2.00 - 4.00	8/12/16/12	28	28		147.0		2.5 — 2D(B) Brown, moist, medium dense, Sandy SILT, little gravel, (Marine Sand).	
5	3D	24/12	5.00 - 7.00	15/16/17/18	33	33				Similar to above except dense.	
								140.3		9.2 — **Bottom of Exploration at 9.2 feet below ground surface.** Auger Refusal.	
10											
15											
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: BB-NBGWR-101
WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	145.6 ft	**Auger ID/OD:**	HSA 2.25"/4.25"
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	E. Baron	**Rig Type:**	Mobile D53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	05-29-2018	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 15+63.8 ft, 48.5 ft Lt	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	±9.0 ft (after drilling)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐	Rope & Cathead ☒	

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/10	0.50 - 2.50	10/11/9/7	20	20	HSA	145.3		4" of Pavement (0.3)	
										Brown, damp, medium dense, SAND, some gravel, little silt, (Fill).	
	2D	24/12	2.50 - 4.50	7/15/3/2	18	18				Similar to above.	
5								140.6		(5.0)	
	3D	17/2	5.00 - 6.42	27/6/50-5"	- -					Dark brown mottled grey, moist, medium dense, SAND, some silt, little gravel, trace clay, trace organics, fine to medium sand.	
							OPEN	139.2		(6.4) Numerous cobbles and boulders from 6.4 to 9.3 ft bgs.	
	4D	3/3	9.50 - 9.75	50-3"	- -					Brown, wet, dense, SAND, some silt, some gravel, (Marine Sand).	
10	R1	24/21	10.50 - 12.50	RQD = 67%			NQ2	135.3		(10.3) Top of Bedrock at Elev. 135.3 ft. Advanced by rollercone from 10.3 to 10.5 feet bgs. R1:Bedrock: Grey, fine-grained, quartz-biotite GRANOFELS with quartzite veins, hard, fresh, joints are low angle to moderate dipping (5-55 degrees), very close to close and tight to open, (Berwick Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 10.5-11.5 ft (3:30) 11.5-12.5 ft (7:00) 87% Recovery.	
	R2	21/21	12.50 - 14.25	RQD = 33%						R2:Bedrock: Similar to R1 except joints are low angle to vertical. Rock Mass Quality = Poor. R2:Core Times (min:sec) 12.5-13.5 ft (3:45) 13.5-14.3 ft (4:30) 100% Recovery.	
15	R3	60/60	14.30 - 19.30	RQD = 90%						R3:Bedrock: Similar to R1 except joints are low angle and close to wide. Rock Mass Quality = Good. R3:Core Times (min:sec) 14.3-15.3 ft (3:45) 15.3-16.3 ft (4:15) 16.3-17.3 ft (3:30) 17.3-18.3 ft (3:30) 18.3-19.3 ft (3:15) 100% Recovery.	
20	R4	14/14	19.30 - 20.47	RQD = 100%				125.1		R4:Bedrock: Similar to R3 except no joints. Rock Mass Quality = Excellent. R4:Core Times (min:sec) 19.3-20.5 ft (3:30) 100% Recovery. (20.5)	
										Bottom of Exploration at 20.5 feet below ground surface.	
25											

Remarks:

bgs = below ground surface

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

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

Boring No.: BB-NBGWR-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Staples Bridge #1238 carries Card Mill Road over Great Works River
Location: North Berwick, Maine

Boring No.: BB-NBGWR-102

WIN: 022336

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	144.9 ft	**Auger ID/OD:**	5" Solid Stem Auger
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	E. Baron	**Rig Type:**	Mobile D53	**Hammer Wt./Fall:**	140 lbs/30"
Date Start/Finish:	05-30-2018	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta 16+16.9 ft; 34.5 ft Lt	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	±8 feet (during drilling)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/7	0.50 - 2.50	12/16/12/8	28	28	SSA	144.6		4" of Pavement (0.3)	
										Grey, damp, medium dense, Sandy GRAVEL, some silt, (Fill).	
	2D	24/6	2.50 - 4.50	10/12/9/16	21	21				Brown, damp, medium dense, Gravelly SAND, little silt, (Fill).	
5								139.9		(5.0)	
	3D	23/4	5.00 - 6.92	4/9/9/50-5"	18	18				Brown, moist, medium dense, SAND, some gravel, little silt, (Marine Sand).	
	4D	21/6	8.00 - 9.75	7/17/13/50-3"	30	30				Brown, wet, dense, Gravelly SAND, little silt, (Marine Sand).	
10	R1	72/59	10.00 - 16.00				NQ2			Advanced by rock core through cobbles from 10 to 11.3 feet bgs.	
				RQD = 41%				132.6		(12.3) Top of Bedrock at Elev. 132.6 ft. R1:Bedrock: Grey, fine-grained, quartz-biotite GRANOFELS with calcite-quartzite veins, hard, fresh, joints are low angle to stee (5-85 degrees), very close to close and tight to open with silt infilling in steep joint, (Berwick Formation). Rock Mass Quality = Poor. R1:Core Times (min:sec) 12.3-13.0 ft (2:45) 13.0-14.0 ft (3:00) 14.0-15.0 ft (2:45) 15.0-16.0 ft (2:30) 82% Recovery.	
15											
	R2	60/47	16.00 - 21.00	RQD = 40%						R2:Bedrock: Similar to R1 except joints are low angle. Rock Mass Quality = Poor. R2:Core Times (min:sec) 16.0-17.0 ft (2:45) 17.0-18.0 ft (2:45) 18.0-19.0 ft (3:00) 19.0-20.0 ft (2:45) 20.0-21.0 ft (2:30) 78% Recovery.	
20								123.9		(21.0)	
										Bottom of Exploration at 21.0 feet below ground surface.	
25											

Remarks:

bgs = below ground surface

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

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

Boring No.: BB-NBGWR-102

S.W.COLE ENGINEERING, INC.

APPENDIX D
Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): North Berwick Work Number: 22336.00

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-NBGWR-201, 2D	16+06	6.0 Rt.	4.0-6.0	337285	1	12.2			GW-GM	A-1-a	0
HB-NBGWR-204, 2D	14+85	CL	2.0-4.0	337284	1	13.0			CL	A-4	IV

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.

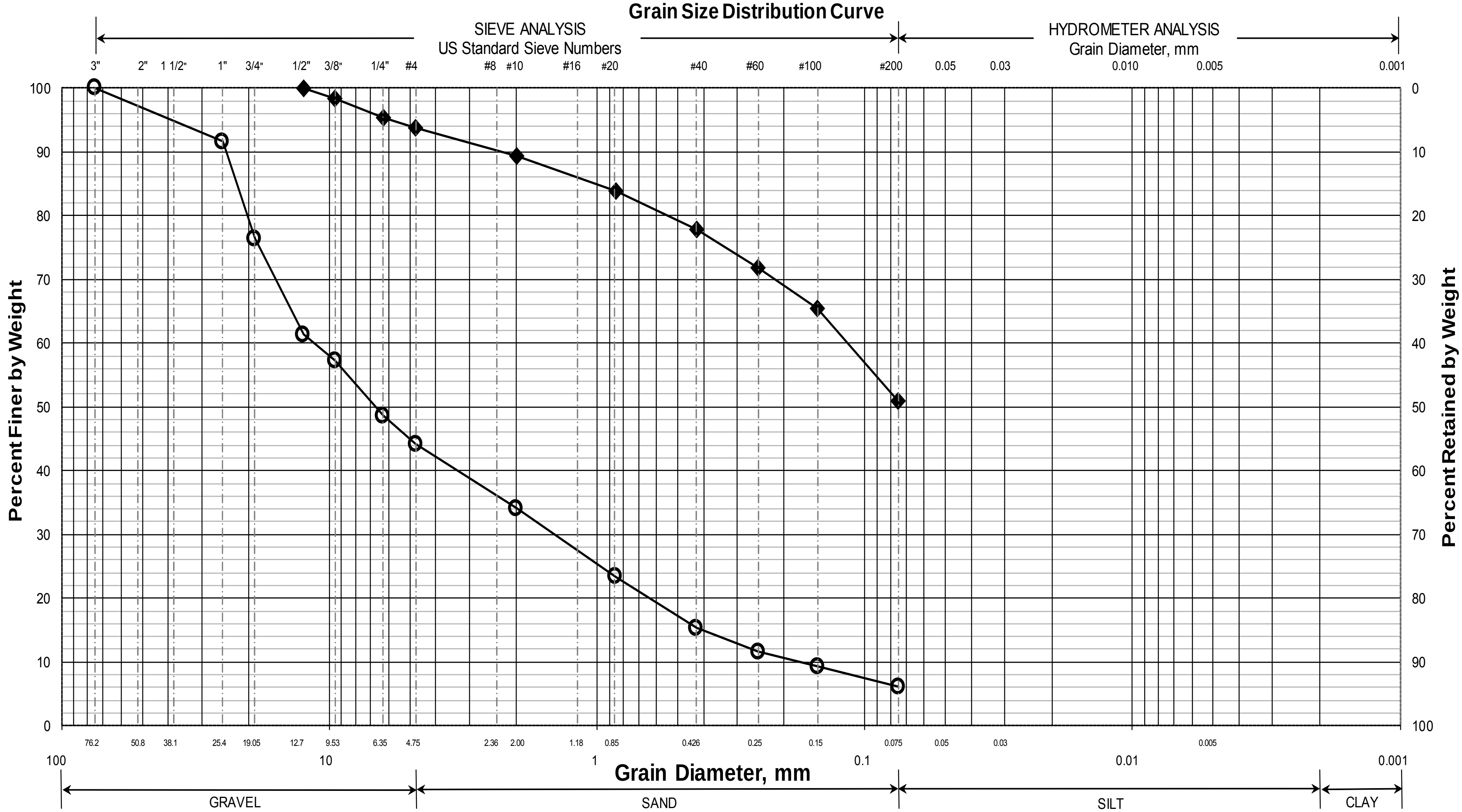
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

Maine Department of Transportation
Grain Size Distribution Curve



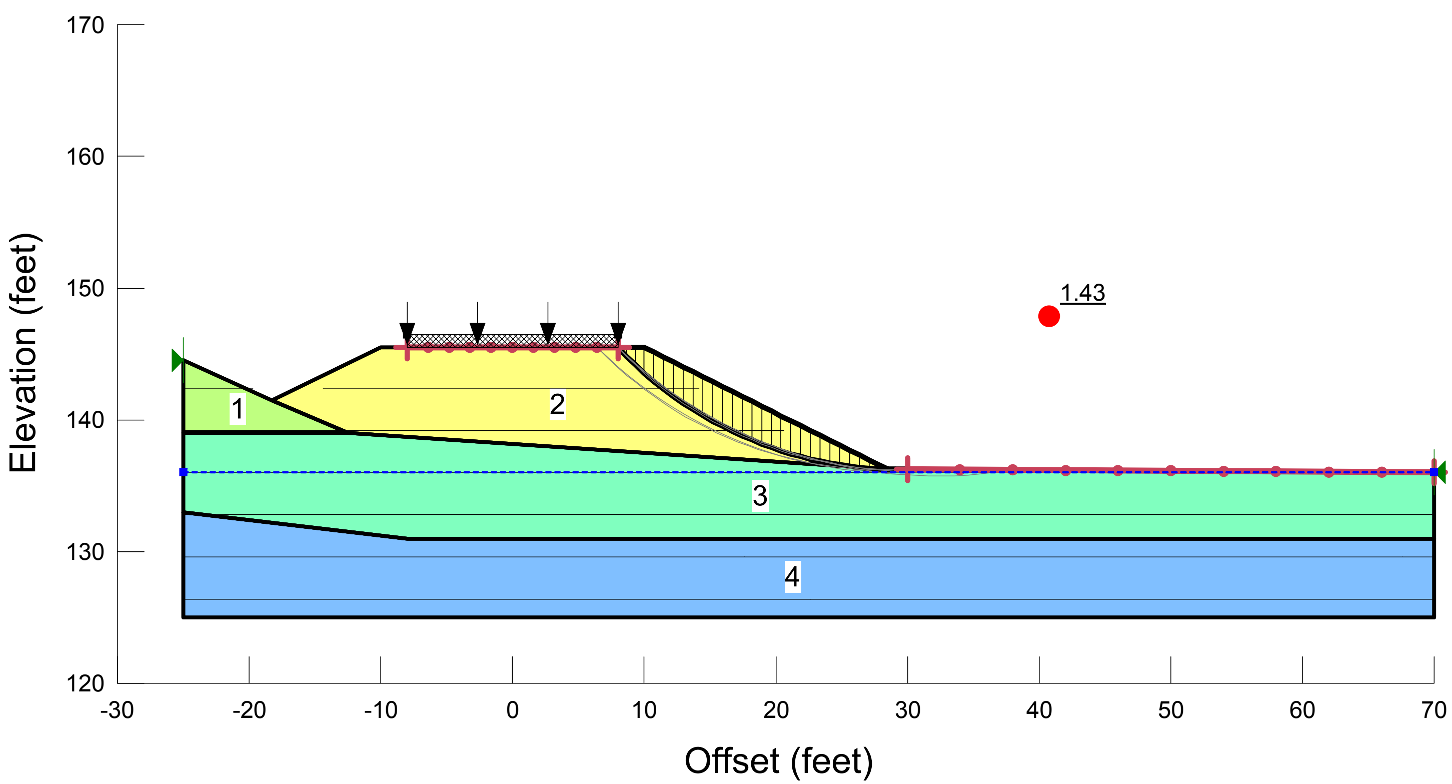
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-NBGWR-201/2D	16+06	6.0 RT	4.0-6.0	Sandy GRAVEL, trace silt.	12.2			
◆	HB-NBGWR-204/2D	14+85	CL	2.0-4.0	Sandy SILT, trace gravel.	13			
■									
●									
▲									
×									

WIN	
022336.00	
Town	
North Berwick	
Reported by/Date	
WHITE, TERRY A	7/15/2019

SHEET 1

APPENDIX E
Calculations

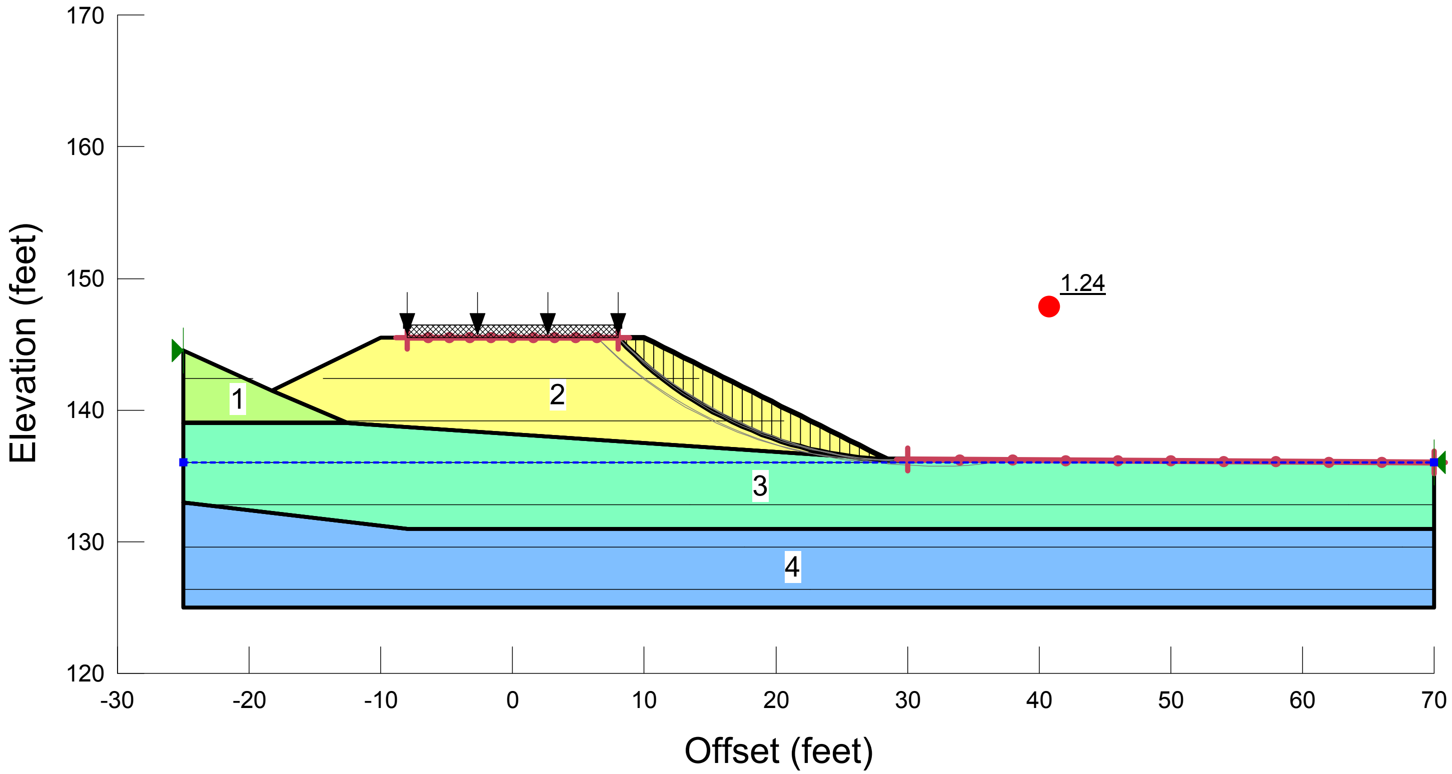


WIN 22336
Staples Bridge #1238
North Berwick, Maine

Station 16+25
North Approach

Static Conditions

Materials:
1. Existing Fill: Unit Wt = 120 pcf, Phi = 30 deg
2. New Fill: Unit Wt = 125 pcf, Phi = 32 deg
3. Marine Sand: Unit Wt = 130 pcf, Phi = 36 deg
4. Bedrock: Impenetrable



WIN 22336
Staples Bridge #1238
North Berwick, Maine

Station 16+25
North Approach

Seismic Conditions
kh = 0.5*amax = 0.06g

Materials:
1. Existing Fill: Unit Wt = 120 pcf, Phi = 30 deg
2. New Fill: Unit Wt = 125 pcf, Phi = 32 deg
3. Marine Sand: Unit Wt = 130 pcf, Phi = 36 deg
4. Bedrock: Impenetrable

Evaluation of Nominal and Factored Bearing Resistance on Rock

Service Limit State

From 2017 AASHTO LRFD Table 10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material:	weathered or broken bedrock of any kind
Consistency in Place:	medium hard rock
Bearing Resistance Range:	16 to 20 ksf
Recommended Bearing Resistance:	20 ksf

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

Resistance Factor Service Limit $\phi_r := 1.0$

Factored Bearing Resistance $q_{factored\_service} := \phi_r \cdot q_{nominal\_service}$

$q_{factored\_service} = 20\ ksf$

Recommend Nominal and Factored Bearing Resistance of 20 ksf (Service Limit State)

From 2017 LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values the service limit bearing resistances are limited to 1 inch of settlement

Strength and Extreme Limit States

Reference(s): Wyllie (2009) Foundations on Rock, 2nd Ed.
Hoek and Brown (1988) The Hoek-Brown Failure Criterion - A 1988 Update
AASHTO (2002) Standard Specifications for Highway Bridges, 17th Ed.
2012 AASHTO LRFD Bridge Design Specifications, 6th Ed.

Establish Bedrock Properties

No rock core compressive strengths performed. Evaluate based on observations of exposed bedrock at site and rock core obtained.

Estimate range or Compressive Strengths based on Table 4.4.8.1.2B from Standard Specifications for Highway Bridges.

Classify as Rock Category D, fine-grained igneous crystalline rock - Diabase
Co = 450-12,000 ksf (3,100-83,000 psi)

Use $q_{uc} := 7000\ psi$ $q_{uc} = 1008\ ksf$

Deterine Rock Mass Rating (RMR) using 2012 AASHTO LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Masses

1. Strength of Intact Rock Material

For assumed Uniaxial Compressive Strength of 7,000 psi (1,008 ksf)

LRFD Table 10.4.6.4-1 for Uniaxial Compressive Strength = 520-1,080 ksf;
Relative Rating = 4 $RMR_1 := 4$

2. Drill Core Quality RQD
RQD ranged from 33-100% (Poor to Excellent); Weighted RQD = 60% (Fair)

LRFD Table 10.4.6.4-1 for RQD 50-75%;
Relative Rating = 13

$RMR_2 := 13$

3. Spacing of Joints
Very close to close joint spacing (2 to 12")

LRFD Table 10.4.6.4-1 for joint spacing of 2in to 1ft;
Relative Rating = 10

$RMR_3 := 10$

4. Condition of Joints
Joints with slightly rough surfaces, seperation <0.05" and soft joint wall rock

LRFD Table 10.4.6.4-1;
Relative Rating = 12

$RMR_4 := 12$

5. Groundwater Conditions
General conditions: Moist Only (Relative Rating = 7)
Water Under Moderate Pressure (Relative Rating = 4)

Use Relative Rating = 4

$RMR_5 := 4$

6. Strike and Dip Orientations
(from 2012 LRFD Table 10.4.6.4-2 Geomechanics Rating for Joint Orientations)

For Foundations, assume Fair rating

LRFD Table 10.4.6.4-2 for Strike and Dip Orientations;
Relative Rating = -7

$RMR_6 := 7$

$$RMR := RMR_1 + RMR_2 + RMR_3 + RMR_4 + RMR_5 - RMR_6$$

$$RMR = 36$$

From 2012 LRDF Table 10.4.6.4-3 Geomechanics Rock Mass Classes Determined from Total Ratings

RMR=36 is indicative of Poor Quality Rock Mass

From 2012 LRDF Table 10.4.6.4-4

Rock Type D - Fine grained polyminerallic igneous crystalline rocks

For Rock Type D and Intact Rock (RMR=100, mi=17)

Determine Rock Property Constants *s* and *m*

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

For Disturbed rock mass use Hoek and Brown (1988)
Eqn 18 m/mi = exp((RMR-100)/14)
Eqn 19 s = exp((RMR-100)/6)

where mi is the value of m for intact rock $m_i := 17$

$$m := m_i \cdot \exp\left(\frac{RMR-100}{14}\right) \qquad m = 0.176$$

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

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

$L_f := 28\ ft$ $B_f := 8.5\ ft$ $\frac{L_f}{B_f} = 3.294$

$C_{f1} := 1.05$

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

$$q_{nominal} = 41.1\ ksf$$

Factored Bearing Resistance - Strength I

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

$\varphi_b := 0.45$

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

$$q_{factored\_strength} = 18.5\ ksf$$

Strength Limit Factored Bearing Resistance

Factored Bearing Resistance - Extreme I

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

$\varphi_b := 0.8$

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

$$q_{factored\_extreme} = 32.8\ ksf$$

Strength Limit Factored Bearing Resistance

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

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

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

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

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

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

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

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

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

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

Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

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

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

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

Evaluation of Active Earth Pressure for CIP Substructure Design

Assumed Backfill Values

MaineDOT BDG Section 3.6.1 - Soil Type 4

$\gamma := 125\ pcf$ Unit Weight

$\phi := 38\ deg$ Friction Angle

$c := 0\ psf$ Cohesion

Wall Parameters

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

$\delta := \frac{2}{3} \cdot \phi$ $\delta = 25.33\ deg$ Interface Friction between Fill and Wall
LRFD Table 3.11.5.3-1, δ = 19 to 24deg

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

Rankine Active Earth Pressure Coefficient

$$k_{a\_r} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$k_{a\_r} = 0.24$ *for* $\beta = 0\ deg$

Calculated by / Date: MAS / August 2019
Checked by: TJM

Evaluation of Active Earth Pressure for GRS Structure Design

Assumed Backfill Values

FHWA-HRT-11-026, dated January 2011 (FHWA 2011)

$\gamma := 125\ pcf$ Unit Weight

$\phi := 38\ deg$ Friction Angle

$c := 0\ psf$ Cohesion

Wall Parameters

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

$\delta := \frac{2}{3} \cdot \phi$ $\delta = 25.33\ deg$ Interface Friction between Fill and Wall

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

Rankine Active Earth Pressure Coefficient

$$k_{a\_r} := \tan\left(45\ deg - \frac{\phi}{2}\right)^2 \qquad k_{a\_r} = 0.24$$

Rankine Passive Earth Pressure Coefficient

$$k_{p\_r} := \tan\left(45\ deg + \frac{\phi}{2}\right)^2 \qquad k_{p\_r} = 4.2$$

Calculated by / Date: MAS / August 2019
Checked by: TJM

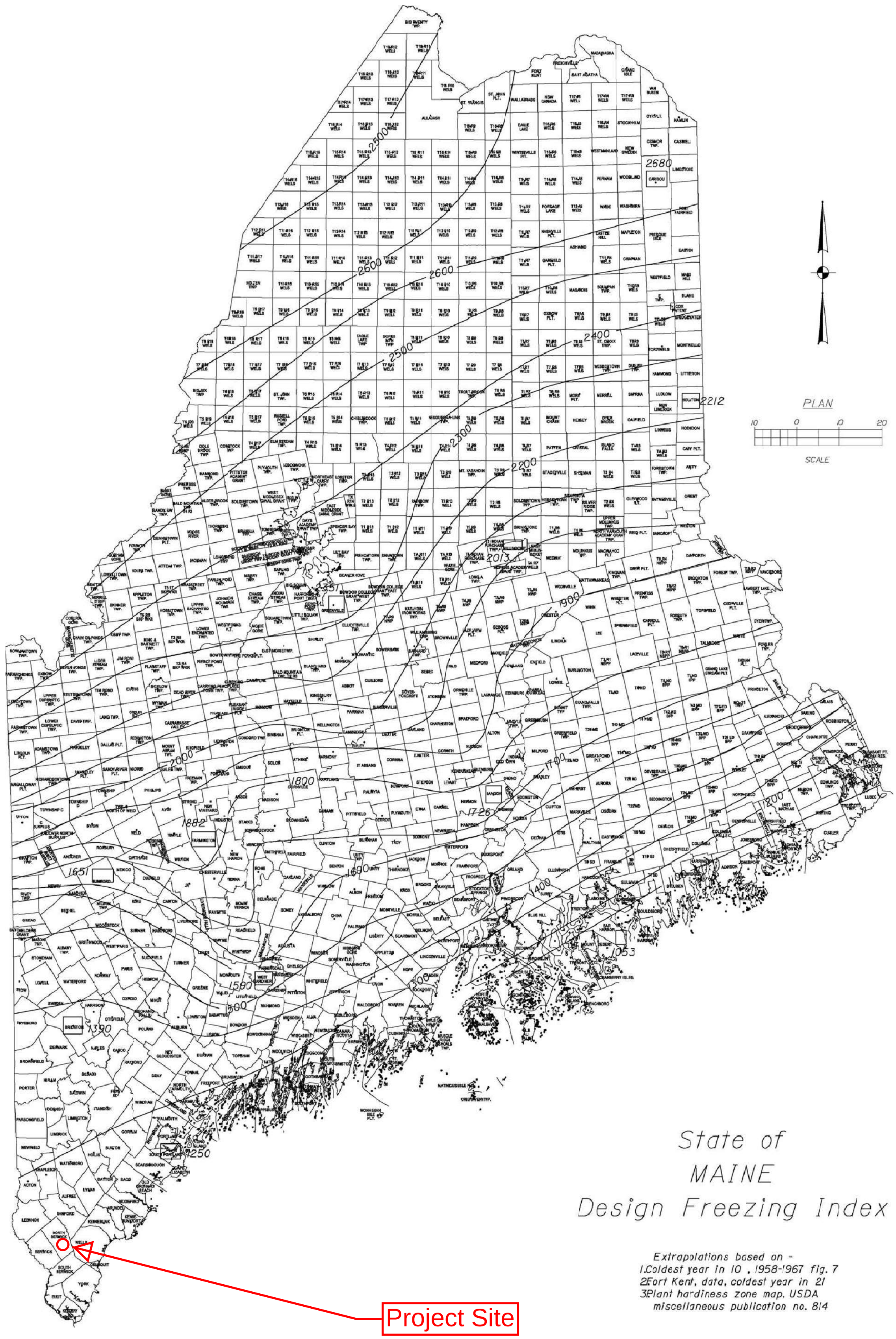


Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

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

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

Table 5-1 Depth of Frost Penetration

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

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

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

Data From Boring BB-NBGWR-101

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Fill	0	5	20	18							19.0	5	0.26
2	Native	5	10.3	100	100							100.0	5.3	0.05
3	Bedrock	10.3	100	100								100.0	89.7	0.90
												Σ =	**100**	**1.21**

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

N_bar = di/di/Ni = 82.43
Site Class C

Data From Boring BB-BSR-102

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Fill	0	5	28	21							24.5	5	0.20
2	Native	5	12.3	18	30							24.0	7.3	0.30
3	Bedrock	12.3	100	100								100.0	87.7	0.88
												Σ =	**100**	**1.39**

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

N_bar = di/di/Ni = 72.19
Site Class C

Data From Boring BB-NBGWR-201

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Native	0	9.2	2	35							18.5	9.2	0.50
2	Bedrock	9.2	100	100								100.0	90.8	0.91
												Σ =	**100**	**1.41**

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

N_bar = di/di/Ni = 71.16
Site Class C

Data From Boring BB-BSR-202

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Native	0	11.7	3	31	49						27.7	11.7	0.42
2	Bedrock	11.7	100	100								100.0	88.3	0.88
												Σ =	**100**	**1.31**

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

N_bar = di/di/Ni = 76.58
Site Class C

Calculated by / Date: MAS / June 2019
Checked by: TJB

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 43.316734
Longitude = -070.743965
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.099	PGA - Site Class B
0.2	0.191	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
Latitude = 43.316734
Longitude = -070.743965
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.119	As - Site Class C
0.2	0.229	SDs - Site Class C
1.0	0.077	SD1 - Site Class C

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Map Response Spectra for Site Class B
Latitude = 43.316734
Longitude = -070.743965
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.099	0.000	T = 0.0, Sa = PGA
0.048	0.191	0.004	T = To, Sa = Ss
0.200	0.191	0.075	T = 0.2, Sa = Ss
0.238	0.191	0.106	T = Ts, Sa = Ss
0.300	0.152	0.133	
0.400	0.114	0.178	
0.600	0.076	0.267	
0.800	0.057	0.355	
1.000	0.045	0.444	T = 1.0, Sa = S1
1.200	0.038	0.533	
1.400	0.032	0.622	
1.600	0.028	0.711	
1.800	0.025	0.800	
2.000	0.023	0.889	
2.200	0.021	0.978	
2.400	0.019	1.066	
2.600	0.017	1.155	
2.800	0.016	1.244	
3.000	0.015	1.333	
3.200	0.014	1.422	

3.400	0.013	1.511	
3.600	0.013	1.600	
3.800	0.012	1.688	
4.000	0.011	1.777	

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Design Response Spectra for Site Class C
Latitude = 43.316734
Longitude = -070.743965
As = FpgaPGA, SDs = FaSs, SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.119	0.000	T = 0.0, Sa = As
0.067	0.229	0.010	
0.200	0.229	0.090	T = 0.2, Sa = SDs
0.337	0.229	0.254	T = Ts, Sa = SDs
0.400	0.193	0.302	
0.600	0.129	0.453	
0.800	0.097	0.604	
1.000	0.077	0.755	T = 1.0, Sa = SD1
1.200	0.064	0.906	
1.400	0.055	1.057	
1.600	0.048	1.209	
1.800	0.043	1.360	
2.000	0.039	1.511	
2.200	0.035	1.662	
2.400	0.032	1.813	
2.600	0.030	1.964	
2.800	0.028	2.115	
3.000	0.026	2.266	
3.200	0.024	2.417	
3.400	0.023	2.568	
3.600	0.021	2.719	
3.800	0.020	2.870	
4.000	0.019	3.021	