STATE OF MAINE
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Letter of Transmittal

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Subject: Geotechnical Design Report, Little Falls Brook Bridge, Bristol, Maine

Document Type: 24

Date: 2/6/2015

Soils Report No.: 2015-02

Bridge #: 6488

Route: N/A

WIN: 017531.00

Town: Bristol

Attached is one (1) copy of Soils Report 2015-02, "GEOTECHNICAL DESIGN REPORT For the Construction of: LITTLE FALLS BROOK BRIDGE, HARRINGTON ROAD OVER LITTLE FALLS BROOK, BRISTOL, MAINE" dated: February 5, 2015.

This report is available in TEDOCS as Document # 1437890.

att: 1 of 2015-02

MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the construction of:

LITTLE FALLS BROOK BRIDGE
HARRINGTON ROAD OVER LITTLE FALLS BROOK
BRISTOL, MAINE

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Soils Report No. 2015-02
Bridge No. 6488

February 5, 2015

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the construction of Little Falls Brook Bridge which carries Harrington Road over Little Falls Brook in Bristol, Maine. The Maine Department of Transportation (MaineDOT) originally investigated a "detail-build" project delivery method for this project. To address the need to design, fabricate, and deliver a bridge structure in time for the planned Fall 2015 road closure, the MaineDOT revisited the "detail-build" project delivery method. Subsequently, MaineDOT has elected to design and specify a steel structural plate pipe arch on new, cast-in-place, concrete footings using the traditional "bid-build" project delivery method. The following summarized design recommendations for the design of the pipe arch and its foundations are discussed in detail in Section 8.0 of this report.

Spread Footings on Bedrock – Bedrock was encountered at depths ranging from approximately 10.1 to 12.2 feet below ground surface (bgs) at the proposed arch foundations. Based on the boring and power auger probes conducted at the anticipated locations for footings supporting a 18-foot span buried structure, the approximate bedrock surface is estimated to range from Elevation 14.0 to 16.1 feet. It is essential to note that the top of intact rock cannot be known for the entire foundation area prior to excavation. Intact rock may be encountered above and below the approximate bedrock surface elevations provided in this report.

The thickness of the arch footing may be designed to vary in thickness to accommodate variations in the bedrock surface with the top of the footing set at a minimum of the $Q_{1.1}$ design flow elevation.

Bearing Resistance of Spread Footings on Bedrock – The bearing resistance for cast-in-place spread footings constructed on bedrock shall be investigated at the service limit state using a factored bearing resistance of 20 kips per square foot (ksf) to control settlement and for preliminary footing sizing. The factored bearing resistance at the strength limit state for spread footings on bedrock shall not exceed 23 ksf. However, the service limit state bearing resistance may govern the design.

In no instance shall the applied bearing stress exceed the nominal structural resistance of the structural 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.

Arch Footing Design – Arch spread footings shall be designed for all relevant strength and service limit state load combinations in accordance with AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD), and MaineDOT Bridge Design Guide (BDG), 2003.

For the scour protection of arch spread footings, footings shall be constructed directly on bedrock surfaces cleaned of soil and all weathered, loose, and potentially erodible rock.

For spread footings on bedrock, the location of the resultant of the reaction forces shall be within the middle nine-tenths (9/10) of the footing.

Structural Plate Pipe Arch Design – The pipe arch shall be designed for all relevant strength and service limit state load combinations in accordance with AASHTO LRFD, and MaineDOT BDG. The loading specified for the structure shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25.

Earth Pressures and Surcharge Forces – Calculation of earth pressures on arch footings should assume an at-rest earth pressure coefficient, K_0, of 0.47, assuming the footings are prevented from movement. A resistance factor for at-rest or passive earth pressures mobilized to resist lateral sliding forces shall be, φ_{ep}, 0.50. For designing the arch footing reinforcing steel for at-rest earth pressure, a maximum load factor, γ_{EH}, of 1.50 is recommended.

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

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per the MaineDOT BDG.

Settlement – No significant vertical or horizontal alignment changes are planned for the buried structure. The structural plate pipe arch and its foundations will be constructed on bedrock. Therefore, any settlement of the foundations will be due to elastic compression of the bedrock and will be negligible.

Frost Protection – For foundations on bedrock heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary. Foundations placed on soil should be founded a minimum of 5.1 feet below finished exterior grade for frost protection.

Scour and Riprap – The buried structure will be founded on spread footings founded on bedrock. For scour protection of the arch footings, construct the footings directly on bedrock surfaces cleaned of all weathered, loose, highly fractured, and potentially erodible rock. Slope protection shall be extended far enough from the structure to protect the structural portion of the soil envelope surrounding the structure. The slopes shall be armored with a 3-foot thick layer of riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material.

Seismic Design Considerations – Seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore, seismic analysis is not required.

Construction Considerations – Construction of the pipe arch and its foundations will require soil and rock excavation and will require removal of the existing stone culvert.

Cofferdams and temporary earth support systems may be required to permit construction of arch footings in the dry.

The subgrade for the arch spread footings shall consist of sound bedrock. The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock subgrade surface shall be cleaned of all overburden soils and loose, dislodged, bedrock fragments by mechanical means. The final bearing surface of bedrock shall be washed with high pressure water and air prior to concrete being placed for the arch footings.

The slope of the bedrock surface subgrade for foundations shall not be steeper than 4H:1V or it shall be benched in level steps or excavated to be completely level. Anchors or dowels may also be engineered and employed to improving sliding resistance.

Excavation of bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with the MaineDOT Standard Specification 105.2.7. It is also recommended that the contractor conduct pre-and post-blast surveys, as well as blast vibration monitoring in accordance with industry standards at the time of the blast.

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

It is anticipated that there may be seepage of water from fractures and joints exposed in the bedrock surface. Surface water should be diverted from the foundation excavation throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by using a sump pump located in the corner of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that all foundations are constructed in the dry.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the construction of Little Falls Brook Bridge which carries Harrington Road over Little Falls Brook in Bristol, Maine. A subsurface investigation has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge construction. This report presents the subsurface information obtained at the site during the subsurface investigation, foundation design recommendations, and geotechnical design parameters for the bridge construction.

The existing structure is a dry-laid, granite block culvert, presumed on bedrock. No record was found for the date of original construction. The culvert is undersized, has significant voids between the stones and was originally scoped as a strut replacement by the MaineDOT Highway Program. An extended scoping and environmental review resulted in the determination that a span of approximately 14 feet would be required to provide full bankfull width (BFW). Due to the significantly larger structure required, this project was transferred to the MaineDOT Bridge Program for design.

Due to the shallow bedrock at the site, replacement alternatives are limited to open bottom structures with cast-in-place concrete footings founded on bedrock. The Preliminary Design Report (PDR) initially identified aluminum and steel structural plate pipe arches and precast concrete rigid frames as feasible alternatives, and recommended a “detail-build” method of project delivery. A 28-day road closure with an approximate 14 mile detour in fall of 2015 was planned. Subsequently, to address the need to fabricate and deliver a structure in time for the fall road closure, the MaineDOT Bridge Program has elected to design and specify a steel structural plate arch on new, cast-in-place footings founded directly on bedrock. To satisfy environmental requirements and provide necessary hydraulic capacity, the replacement structure will have a span of 18 feet to provide a clear width of 16 feet. The new pipe arch will be located on the same horizontal alignment as the existing strut, at a 15 degree skew.

2.0 GEOLOGIC SETTING

The existing stone culvert carries Harrington Road over Little Falls Brook approximately 0.49 miles west of State Route 130, as shown on Sheet 1 – Location Map.

The Maine Geologic Survey (MGS) Surficial Geology of the Bristol Quadrangle (1972) indicates the surficial soils in the vicinity of the bridge project consist of glacial-marine deposits of the Presumpscot Formation. These deposits generally consist of clay and silt that washed out of the Late Wisconsinan glacier and accumulated on the ocean floor when the relative sea level was higher than at present.

The Bedrock Geologic Map of Maine, MGS, (1985), indicates the bedrock in the vicinity of the bridge site as an unnamed sedimentary rock. The Bedrock Geology of the Bath Quadrangle, Maine, Map No. 02-152, MGS, (2002), cites the bedrock at the bridge site as being gneiss with relict zones of schist, of the Cross River Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling one (1) test boring and four (4) power auger probes outside of the four corners of the existing structure. Test boring BB-BLFB-101 was located at the southeast corner and drilled in the eastbound travel lane of Harrington Road, approximately 12 feet outside of the existing stone culvert. Power augers PA-BLFB-101, PA-BLFB-102, PA-BLFB-103, and PA-BLFB-104 were drilled approximately 22 to 55 feet outside of the existing stone culvert. The boring and power auger probe locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The boring and power auger probes were drilled on March 11, 2014 by the MaineDOT Drill Crew. Details and sampling methods used, field data obtained, and soil and bedrock conditions encountered are presented in the boring log and Power Auger Probe Summary Sheet provided in Appendix A – Boring Log and Power Auger Probes and on Sheet 3 – Boring Log.

The boring was drilled using cased wash boring and solid stem auger techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT dill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633-05 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in July 2013 and was found to deliver approximately 43 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.867 to the raw field N-values. This hammer efficiency factor (0.867) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring log.

The boring was advanced to bedrock and terminated with a bedrock core. The bedrock was cored using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The power auger probes were drilled through the roadway surface using solid stem auger techniques and were taken to refusal depths ranging from approximately 4.1 to 10.7 feet bgs. No soil samples were taken in the probes. The Power Auger Probe Summary Sheet can be found on Sheet 3 – Boring Log and in Appendix A – Boring Log and Power Auger Probes.

A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring and power auger probe locations, drilling methods, designated type and depth of sampling techniques, reviewed the draft boring log and identified field and laboratory testing requirements. The boring and power auger probes were located in the field using taped measurements at the completion of the drilling programs.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test boring to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of one (1) standard grain size analysis with natural water content, one (1) grain size analysis with hydrometer and natural moisture content, and one (1) Atterberg Limits test. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring log provided in Appendix A – Boring Log and Power Auger Probes and on Sheet 3 – Boring Log.

5.0 SUBSURFACE CONDITIONS ENCOUNTERED IN BORINGS

Subsurface conditions encountered in the test boring consisted of granular fill and silts with cobbles underlain by bedrock. The boring log is provided in Appendix A – Boring Log and Power Auger Probes and on Sheet 3 – Boring Log. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Fill Soils

A layer of granular fill was encountered in the boring. The thickness of the fill layer encountered was approximately 8 feet. The layer consisted of brown, moist, fine to coarse sand, little gravel, little silt, and occasional cobbles. One (1) corrected SPT N-value in the fill was 14 blows per foot (bpf) indicating the soil is medium dense in consistency. One (1) grain size analysis resulted in the soil being classified as A-1-b under the AASHTO soil classification system and SM under the Unified Soil Classification System (USCS). The measured water content of the sample tested was approximately 4 percent.

5.2 Native Silt

A layer of native silt was encountered in the boring. The encountered thickness was approximately 4.2 feet. The layer encountered consisted of olive, wet, silt, some sand, trace clay, trace gravel with occasional cobbles. One (1) corrected SPT N-value in the deposit was 12 bpf indicating that the soil is stiff in consistency. One (1) grain size analysis resulted in the soil being classified as A-4 under the AASHTO soil classification system and CL-ML under the USCS. The measured water content of the sample tested was approximately 24.4

percent. One (1) Atterberg Limits test resulted in the silt sample being classified as non-plastic.

5.3 Bedrock

Bedrock was encountered and cored at a depth of approximately 12.2 feet bgs in the boring.

The bedrock at the site is identified as dark grey, fine to medium grained, hard, banded schist with zones of quartz. The bedrock cored is part of the Cross River Formation. The RQD of the bedrock was calculated to be approximately 73 percent correlating to a Rock Mass Quality of fair.

5.4 Groundwater

Groundwater was not observed in the boring. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, and construction activities.

6.0 SUBSURFACE CONDITIONS ENCOUNTERED IN PROBES

A series of four (4) probes were drilled using solid stem auger techniques through the roadway surface outside the existing stone culvert alignment in order to estimate the bedrock surface elevation at the project site. One (1) power auger probe (PA-BLFB-101) refused at 4.1 feet bgs on what is assumed to be a cobble. Three (3) power auger probes (PA-BLFB-102, PA-BLFB-103 and PA-BLFB-104) refused at depths ranging from approximately 10.1 to 10.7 feet bgs interpreted to be the bedrock surface.

Table 1, below, summarizes refusal depths encountered in the probes.

Power Auger Probe (PA)	Station	Offset (feet)	Approx. Depth to Bedrock or Power Auger Refusal (feet)	Approx. Elevation of Bedrock Surface[1] (feet)
PA-BLFB-103	9+85.9	6.7 Lt.	10.1	14.6
PA-BLFB-101	9+88.2	7.6 Rt.	4.1	N.A.[2]
PA-BLFB-102	9+79.6	7.9 Rt.	10.6	14.1
PA-BLFB-104	10+27.9	9.6 Lt.	10.7	16.7

[1] The power auger refusal surface is interpreted to be the bedrock surface
[2] PA-BLFB-101 encountered a refusal surface at a depth of 4.1 feet bgs on what is believed to be a cobble.

Table 1. Summary of Power Auger Probe Refusal Elevations

The probe information is presented in the Power Auger Summary Sheet provided in Appendix A – Boring Log and Power Auger Probes.

7.0 FOUNDATION ALTERNATIVES

Due to the shallow bedrock at the site, the November 2014 PDR considered several open bottom structures of varying spans. All of the considered structures would be founded on a new, cast-in-place, concrete footing on bedrock:

- a steel structural plate arch,
- an aluminum structural plate arch,
- a 3-sided precast concrete frame or arch.

Assessment of subsurface conditions indicates that, for all aforementioned open bottom structures, cast-in-place spread footings constructed directly on bedrock are the most effective foundation alternatives due to the shallow bedrock. Subsequently, to address the need to fabricate and deliver a structure in time for the fall construction window, the MaineDOT Bridge Program has elected to design and specify a 18-foot span, steel structural plate arch with a clear span of 16-feet, on cast-in-place spread footings on bedrock.

8.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections provide foundation considerations and recommendations for a steel structural plate arch on cast-in-place spread footings on bedrock. The design recommendations in this Section are provided in accordance with AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014 (LRFD) and MaineDOT Bridge Design Guide, (BDG), 2003.

8.1 Spread Footings on Bedrock

Bedrock was encountered at depths ranging from approximately 10.1 to 12.2 feet bgs at the proposed arch foundations. It is considered feasible that spread footings can be practically and economically constructed to bear on bedrock at this location, possibly without temporary soil support systems. The boring indicates that bedrock with an RQD of approximately 72 percent will be encountered at the bedrock surface at the boring location, however, approximately 6 inches of fractured bedrock was encountered at the boring location. The Contractor should anticipate the need to clear the bearing area of all weathered, loose, highly fractured and potentially erodible bedrock encountered during construction.

Based on the boring and power auger probes conducted at the anticipated locations for footings supporting a 18-foot span buried structure, the approximate bedrock surface is estimated to range from approximate Elevation 14.0 to 16.1 feet. It is essential to note that the top of intact rock cannot be known for the entire foundation area prior to excavation. Intact rock may be encountered above and below the approximate bedrock surface elevations provided in this report. The thickness of the arch footings may be designed to vary in thickness to accommodate variations in the bedrock surface with the top of the footing set to a minimum elevation of the $Q_{1.1}$ design flow at 16.5 feet.

A subfooting leveling course of Fill Class concrete, containing MaineDOT Standard Specification 703.01 – Fine Aggregate for Concrete, conforming to MaineDOT Standard Specification 502.05 – Structural Concrete with wire mesh reinforcing, may be constructed, as required, for the full length of the arch footing to level highly irregular bedrock surfaces. The bearing surface of the leveling course should be roughened to improve sliding resistance. If bedrock underlying the proposed arch footing is loose or unsound, the unsound bedrock shall be excavated and the subfooting leveling course constructed in place of the unsound material and considered incidental to the contract.

8.2 Bearing Resistance of Spread Footings on Bedrock

Cast-in-place spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads shall be as specified in LRFD Article 11.5.5.

The bearing vertical stress shall be calculated assuming a triangular or trapezoidal pressure distribution over an effective base as shown in LRFD Figure 11.6.3.2-2 for foundations on rock.

The bearing resistance of cast-in-place spread footings constructed on bedrock shall be investigated at the service limit state using factored loads and a factored bearing resistance of 20 ksf. Resistance factors for the service limit state are taken as 1.0. A service limit state factored bearing resistance of 20 ksf is intended to control settlement and for preliminary footing sizing as allowed in LRFD C10.6.2.1.

Once the dimension of the cast-in-place spread footing is determined, the designer shall confirm that the factored bearing resistance at the strength limit state is greater than the applied factored vertical bearing pressure. The factored bearing resistance at the strength limit state has been calculated to be 23 ksf. This factored bearing resistance assumes a resistance factor, φ_b, for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. However, the service limit state bearing resistance may govern the design. See Appendix C – Calculations for supporting calculations.

In no instance shall the bearing stress exceed the nominal structural resistance of the structural 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.

8.3 Arch Footing Design

Spread footings for arches shall be designed for all relevant strength and service limit state load combinations specified in AASHTO LRFD Articles 12.5.2 and 12.5.3 and Table 3.4.1-1. Reinforced concrete footings shall be designed in accordance with LRFD Article 10.6 to resist all earth loads, live loads, dead loads, and lateral reactions transferred through the pipe arch. The design of spread footings at the strength limit state shall consider:

- bearing resistance,
- eccentricity,
- lateral sliding,
- reinforced-concrete structural failure.

For the scour protection of steel pipe arch spread footings, project plans shall require construction of footings directly on bedrock surfaces cleaned of all soil and weathered, loose, or potentially erodible rock. With these precautions, strength limit state designs do not need to consider rock scour due to abrasion, or plucking due to the design flood for scour.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.80 shall be applied to the nominal sliding resistance of cast-in-place arch spread footings constructed on bedrock, assuming the rock subgrade with be prepared in-the-wet. If the rock subgrade is prepared in-the-dry and cleaned with high pressure water and air prior to placing seal concrete, a sliding resistance factor, φ_τ , of 0.90 may be assumed.

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

Anchorage of the footings to bedrock may be required to resist horizontal footing reactions and improve stability. Dowels should be reinforcing bars engineered by the designer and be embedded into the footings and bedrock by depths determined by the designer. If bedrock is observed to slope steeper than 4H:1V at the footing subgrade elevation, the bedrock should be benched to create level steps.

For spread footings cast directly 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. This eccentricity corresponds to the resultant of reaction forces falling within the middle nine-tenths (9/10) of the footing.

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

8.4 Structural Plate Pipe Arch Design

The steel pipe arch and its foundations shall be designed by the appropriate methods specified in LRFD Articles 12.7 to resist the load combinations specified in Articles 12.5.2 and 12.5.3. Resistance factors for steel pipe arches shall be as specified in Table 12.5.5-1. Additional design features for the steel pipe arches are detailed in MaineDOT BDG Section 8 – Buried Structures and in LRFD Article 12.6.

The steel plate pipe arch shall also be designed in accordance with all relevant MaineDOT Standard Specifications and MaineDOT Special Provisions. Such specifications may include, but are not limited to, Standard Specification 509 – Structural Plate Pipes, Pipe Arches, Arches, and Metal Box Culverts. The loading specified for the structure shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25.

8.5 Earth Pressures and Surcharge Forces

Calculation of earth pressures acting on arches and their footings should assume an at-rest earth pressure coefficient, K_o, of 0.47, assuming the arch footings are to be prevented from movement. Calculation of earth pressures mobilized to resisting outward thrust forces from the arches shall also assume an at-rest earth pressure coefficient, K_o, of 0.47. A resistance factor for at-rest earth pressures mobilized to resist lateral forces is not specified in LRFD, therefore use the resistance factor for passive pressure, φ_{ep}, of 0.50 per LRFD Table 10.5.5.2.2-1. For designing footing reinforcing steel for at-rest earth pressures to resist outward thrust forces, a maximum load factor, γ_{EH}, of 1.50 is recommended. For vertical earth pressure, the maximum load factor from LFRD Table 3.4.1-2 shall apply.

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

Additional lateral earth pressure due to construction surcharge or live load surcharge may be required for the arch footings. The live load surcharge on arch footings and 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 may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 3 below:

Arch Wall Height (feet)	Traffic Normal to Wall h_{eq} (feet)	Traffic Parallel to Wall h_{eq} (feet)
5	4.0	5.0
10	3.0	3.5
≥20	2.0	2.0

Table 2. Equivalent Height of Soil for Estimating Live Load Surcharge

8.6 Settlement

No significant vertical or horizontal alignment changes are currently planned for the strut replacement. The structural plate pipe arch and its foundations will be constructed on bedrock. Therefore, we expect that any settlement will be due to elastic compression of the bedrock which will be negligible.

8.7 Frost Protection

For foundations on bedrock heave due to frost is not a design issue and no requirements for minimum depth of embedment are necessary. Foundations placed on placed on native soils should be designed with an appropriate embedment for frost protection. According to BDG Figure 5-1, Maine Design Freezing Index Map, Bristol has a design freezing index of approximately 1250 F-degree days. An assumed water content of 20% was used for coarse-grained soils at the potential elevation of a footing. These components correlate to a frost depth of 5.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Bristol was assigned a design freezing index (DFI) from the Modberg DFI database for Belfast which lies on a DFI contour similar to that of Bristol. An assumed water content of 20% was used for coarse-grained soils. These components correlate to a frost depth of approximately 5.27 feet.

We recommend that foundations constructed on soil be designed with an embedment of 5.1 feet for frost protection. See Appendix C – Calculations for supporting calculations.

8.8 Scour and Riprap

The buried structure will be founded on spread footings founded on bedrock. For scour protection of the arch footings, construct the footings directly on bedrock surfaces cleaned of soil and all weathered, loose, highly fractured and potentially erodible rock.

We recommend that sideslopes and structure backfill envelop be armored with a minimum 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 – Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile (MaineDOT Standard Specification 722.03 – Erosion Control Geotextile) and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation unless the streambed consists of bedrock. The riprap slopes shall be constructed no steeper than a maximum 1.75H:1V extending from the edge of the roadway down to the existing ground surface.

8.9 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore, seismic analysis is not required.

8.10 Construction Considerations

Construction of the pipe arch and its foundations will require soil and rock excavation and removal of the existing stone culvert. Cofferdams and temporary earth support systems may be required to permit construction of arch footings in-the-dry.

Construction activities should not be permitted to disturb the bedrock mass or create any open fissures. Irregularities in the existing bedrock surface or irregularities created during the excavation process should be backfilled with Fill Class structural concrete with fine aggregate (MaineDOT Standard Specifications 502.05 and 703.01). The bottom of the footing elevation may vary based on the presence of fractured bedrock.

The subgrade for the arch spread footings shall consist of sound bedrock. The nature, slope, and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavation is made. The bedrock subgrade surface shall be cleaned of all overburden soils and loose or dislodged bedrock fragments by mechanical means. Mechanical means include expansive agents, hydraulic hoe ram, hydraulic splitters or wedging and prying. The final bearing surface of bedrock shall be washed with high pressure water and air prior to concrete being placed for the arch footings.

The slope of the bedrock subgrade for foundations shall be no steeper than 4H:1V or it shall be benched in level steps or excavated to be completely level. Anchoring, doweling, or other means of improving sliding resistance, may also be employed where the prepared bedrock surface is steeper than 4H:1V in any direction at arch footings.

Excavation of bedrock material may be done using conventional excavation methods, but may require drilling and blasting techniques. Blasting should be conducted in accordance with MaineDOT Standard Specification 105.2.7. It is also recommended that the contractor conduct pre-blast and post-blast surveys, as well as blast vibration monitoring, in accordance with industry standards at the time of the blast.

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

The soil envelope and backfill for the structural plate pipe arch shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 3 inches. The granular borrow backfill should be placed in loose measured lifts of 6 or 8 inches thick, as described in MaineDOT Standard Specification 509.05, and compacted to the manufacturer's specifications. To minimize post-construction settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO

T-180 maximum dry density. The soil envelope and backfill width is recommended to be a minimum of 3 feet wide at the base of the footing and extended upwards to the elevation of the subgrade (MaineDOT BDG Section 8.4.2).

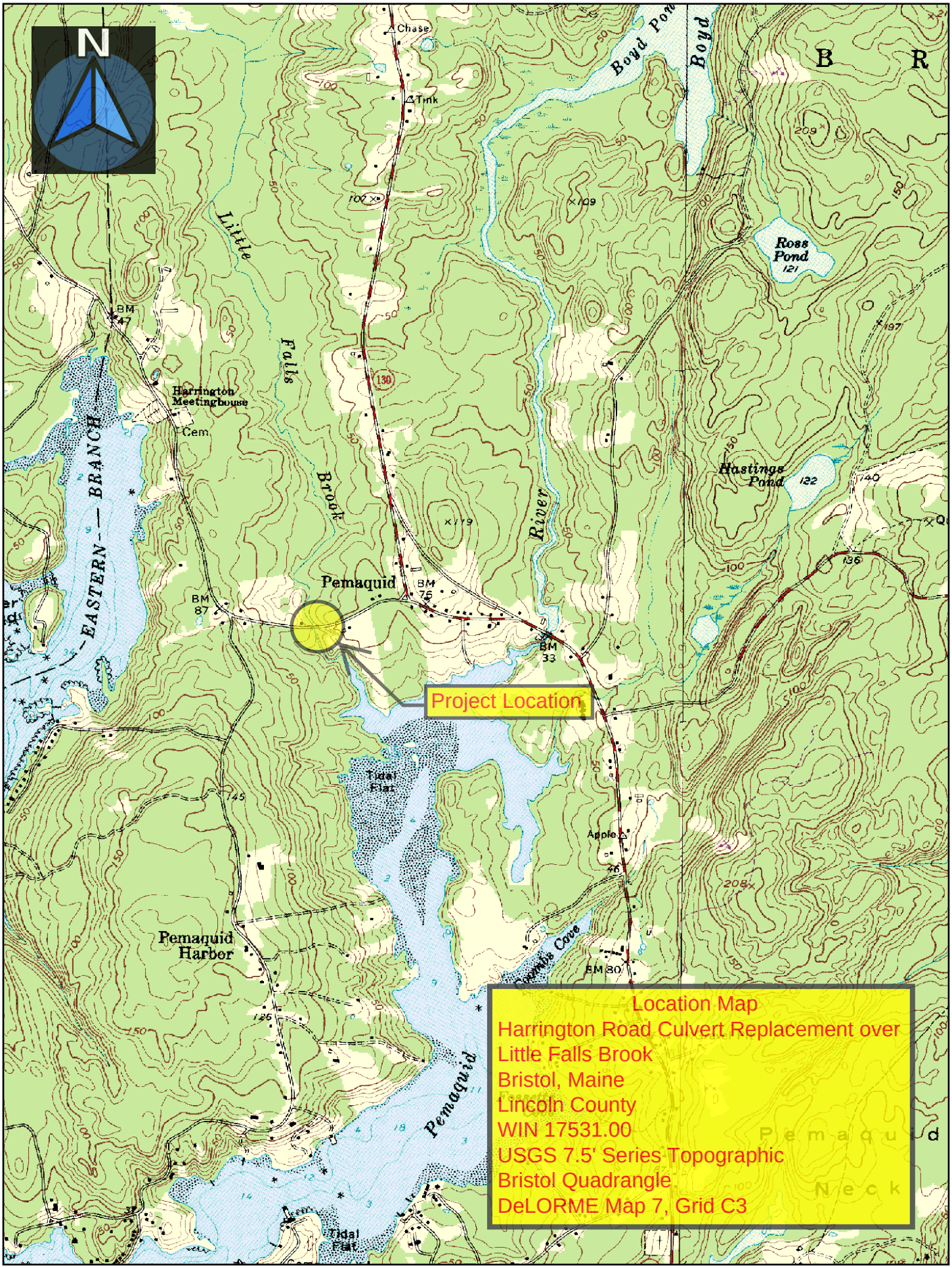
It is anticipated that there may be seepage of water from fractures and joints exposed in the bedrock surface. Surface water should be diverted from the foundation excavation throughout the period of construction. Water encountered at the base of the foundation excavation should be removed by using a sump pump located in the corner of the excavation outside of the foundation footprint. The contractor should maintain the excavation so that all foundations are constructed in the dry.

9.0 Closure

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed construction of Little Falls Brook Bridge in Bristol, Maine, in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied. In the event that any changes in the nature, design, or location of the proposed project are planned; this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

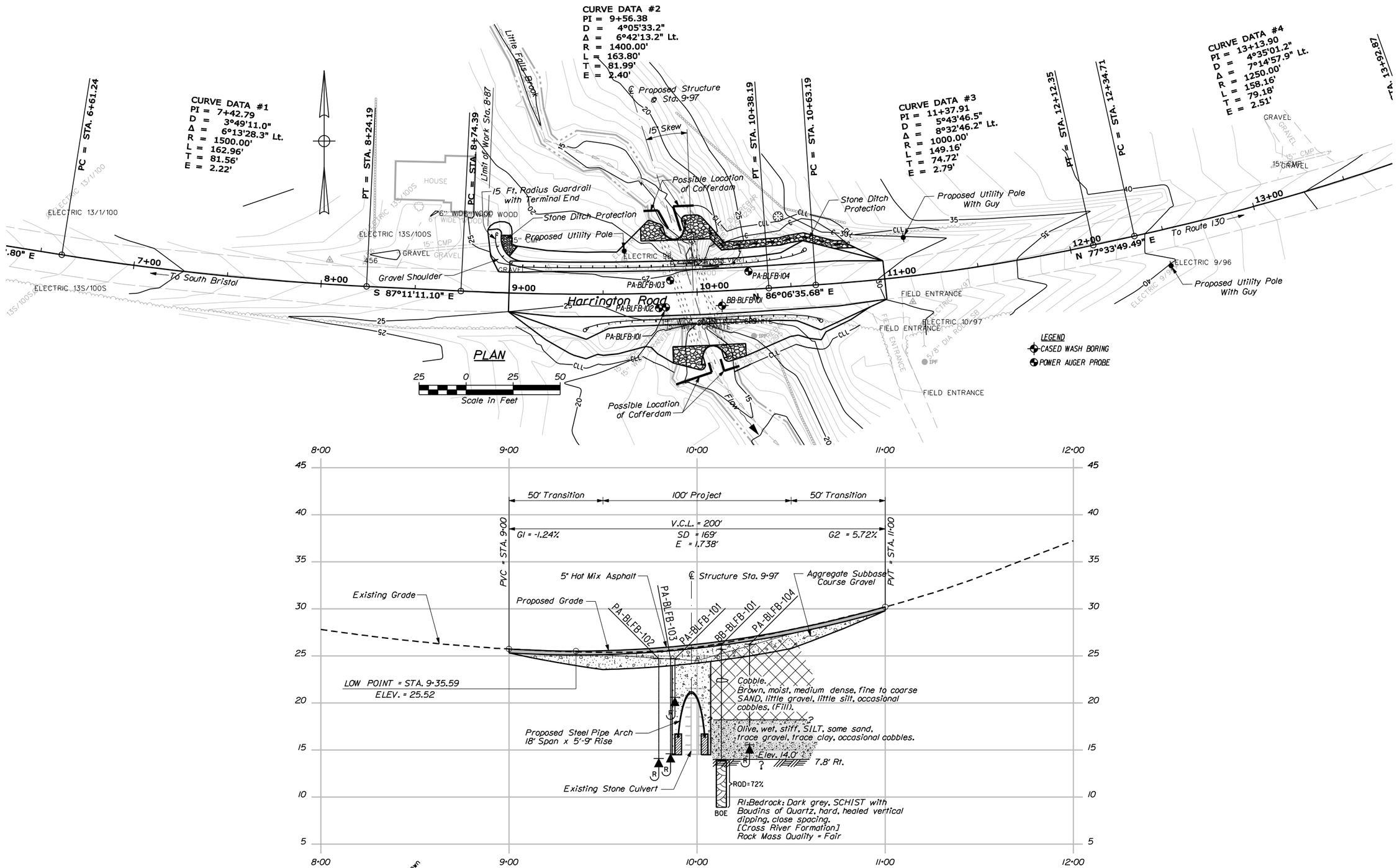
We also recommend that we be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design.

Sheets



Map Scale 1:24000

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



Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

17531.00

WIN 17531.00

BRIDGE PLANS

BRIDGE NO. 6488

		BY	DATE
PROJ. MANAGER	S. SMITH		
DESIGN-DETAILED	N. HARTLEY		
CHECKED-REVIEWED			
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	JUL 2014
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

LITTLE FALLS BRIDGE
LITTLE FALLS BROOK
BRISTOL LINCOLN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\geotech\msta\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:2/5/2015

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:2/5/2015

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Falls Bridge #6488 carries Harrington Road over Little Falls
Location: Bristol, Maine

Boring No.: BB-BLFB-101
WIN: 17531.00

Driller:	MaineDOT	Elevation (ft.)	26.2	Auger ID/OD:	5" Solid Stem
Operator:	Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/11/2014; 08:00-11:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+12.9, 7.8 ft Rt.	Casing ID/OD:	NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.867		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows 1/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	25.70	6" PAVEMENT. 0.50	
									Frost, 2.0-3.0 ft bgs.	
									Cobble from 3.6-4.0 ft bgs.	
5	1D	24/11	5.00 - 7.00	4/5/5/11	10	14			Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, occasional cobbles. (Fill)	G#243178 A-1-b, SM WC=4.0%
								18.20	8.00	
10	2D	24/20	10.00 - 12.00	3/3/5/9	8	12	30		Olive, wet, stiff, SILT, some sand, trace gravel, trace clay, occasional cobbles.	G#243179 A-4, CL-ML WC=24.4% Non-Plastic
							87			
	R1	60/60	12.30 - 17.30	RQD = 72%			@100 NQ-2	14.00 / 13.90	@100 blows for 0.2 ft. 12.20	
15									Top of Bedrock at Elev. 14.0 ft. Roller Coned ahead to 12.3 ft bgs. 12.30	
									R1:Bedrock: Dark Grey, Schist with Boudins of Quartz, hard, healed vertical dipping, close spacing. Cross River Formation. Rock Mass Quality = Fair	
									R1:Core Times (min:sec)	
									12.3-13.3 ft (3:16)	
									13.3-14.3 ft (3:36)	
									14.3-15.3 ft (3:41)	
									15.3-16.3 ft (3:51)	
									16.3-17.3 ft (4:05) 100% Recovery	
								8.90	17.30	
20									Bottom of Exploration at 17.30 feet below ground surface.	
25										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-BLFB-101

State of Maine - Department of Transportation
Power Auger Probe Summary Sheet

Town(s): Bristol **Work Number: 17531.00**

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Water Depth (Ft.)	Comments / Date 3/11/2014
9+88.2	7.6 Rt.		4.1			PA-BLFB-101
9+79.6	7.9 Rt.		10.6			PA-BLFB-102
9+85.9	6.7 Lt.		10.1			PA-BLFB-103
10+27.9	9.6 Lt.		10.7			PA-BLFB-104
All borings were very dense from 4.0-7.0 ft bgs.						

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

17531.00

WIN 17531.00

BRIDGE NO. 6488

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	S. SMITH	
DESIGN-DETAILED	N. HARTLEY	
CHECKED-REVIEWED		
DESIGN2-DETAILED2	B.SLAVEN	T.WHITE	FEB 2015
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

LITTLE FALLS BRIDGE
LITTLE FALLS BROOK
BRISTOL LINCOLN COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Log and Power Auger Probes

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Falls Bridge #6488 carries Harrington Road over Little Falls Brook
Location: Bristol, Maine

Boring No.: BB-BLFB-101
PIN: 17531.00

Driller:	MaineDOT	**Elevation (ft.)**	26.2	**Auger ID/OD:**	5" Soild Stem
Operator:	Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/2014; 08:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+12.9, 7.8 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	25.70	6" PAVEMENT. 0.50	
									Frost, 2.0-3.0 ft bgs.	
									Cobble from 3.6-4.0 ft bgs.	
5	1D	24/11	5.00 - 7.00	4/5/5/11	10	14			Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, occasional cobbles. (Fill)	G#243178 A-1-b, SM WC=4.0%
								18.20	8.00	
10	2D	24/20	10.00 - 12.00	3/3/5/9	8	12	30		Olive, wet, stiff, SILT, some sand, trace gravel, trace clay, occasional cobbles.	G#243179 A-4, CL-ML WC=24.4% Non-Plastic
							87			
	R1	60/60	12.30 - 17.30	RQD = 72%			a100 NQ-2	14.00	a100 blows for 0.2 ft.	
								13.90	12.20	
									Top of Bedrock at Elev. 14.0 ft. Roller Coned ahead to 12.3 ft bgs. 12.30	
15									R1:Bedrock: Dark Grey, Schist with Boudins of Quartz, hard, healed vertical dipping, close spacing. Cross River Formation. Rock Mass Quality = Fair	
									R1:Core Times (min:sec)	
									12.3-13.3 ft (3:16)	
									13.3-14.3 ft (3:36)	
									14.3-15.3 ft (3:41)	
								8.90	15.3-16.3 ft (3:51)	
									16.3-17.3 ft (4:05) 100% Recovery	
									17.30	
20									**Bottom of Exploration at 17.30 feet below ground surface.**	
25										

Remarks:

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

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

State of Maine - Department of Transportation

Power Auger Probe Summary Sheet

Town(s): Bristol Work Number: 17531.00

Station (Feet)	Offset (Feet)	Weathered Rock (Feet)	Refusal (Feet)	No Refusal (Feet)	Water Depth (Ft.)	Comments / Date 3/11/2014
9+88.2	7.6 Rt.		4.1			PA-BLFB-101
9+79.6	7.9 Rt.		10.6			PA-BLFB-102
9+85.9	6.7 Lt.		10.1			PA-BLFB-103
10+27.9	9.6 Lt.		10.7			PA-BLFB-104
All borings were very dense from 4.0-7.0 ft bgs.						

MaineDOT Drill Crew
Logged By: B. Wilder
Drill Rig: CME 45C

5" Solid Stem Auger

Appendix B

Laboratory Test Results

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Bristol Work Number: 17531.00

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-BLFB-101, 1D	10+12.9	7.8 Rt.	5.0-7.0	243178	1	4.0			SM	A-1-b	II
BB-BLFB-101, 2D	10+12.9	7.8 Rt.	10.0-12.0	243179	1	24.4	-N	P-	CL-ML	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.

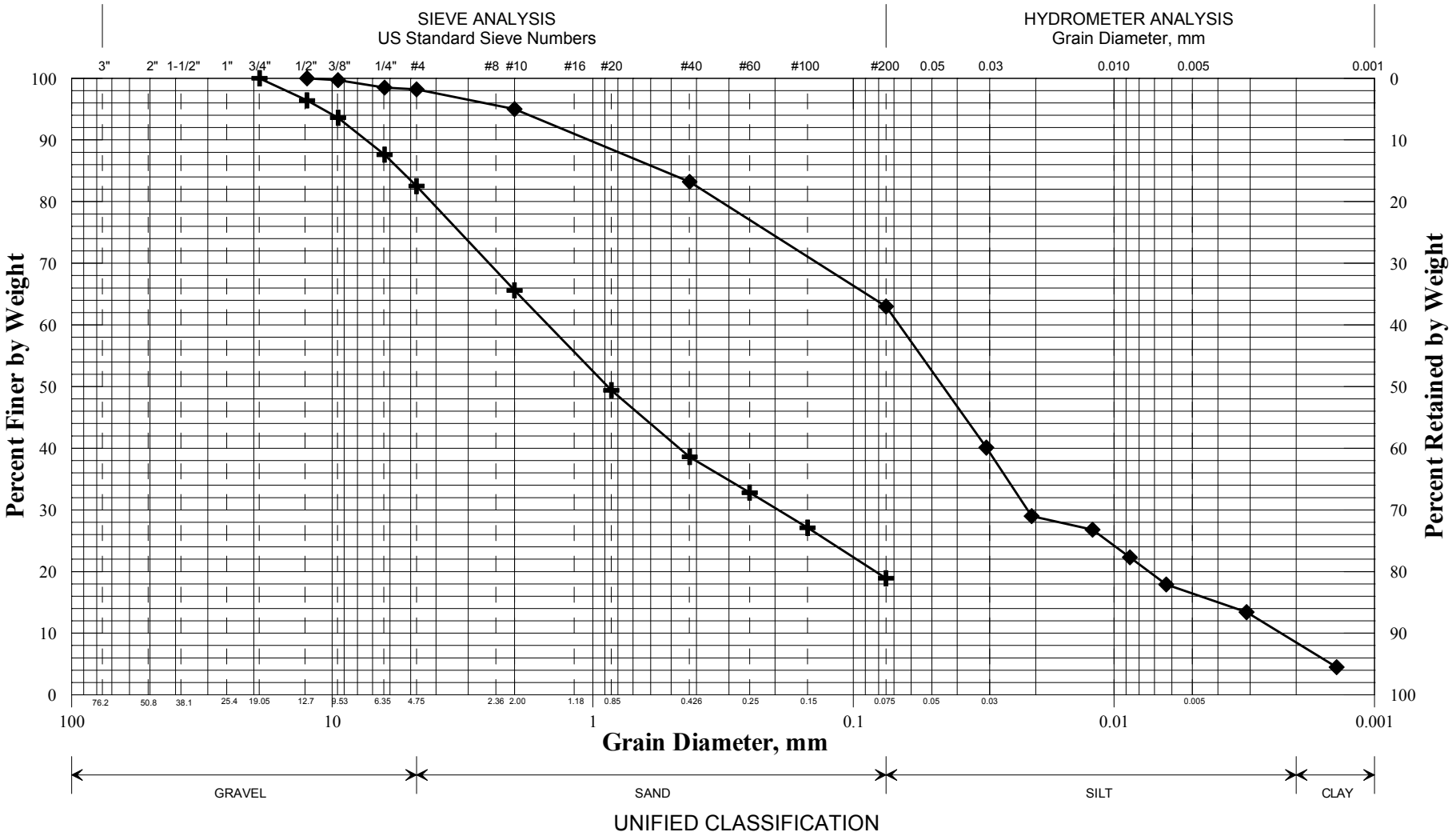
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-BLFB-101/1D	10+12.9	7.8 RT	5.0-7.0	SAND, little silt, little gravel.	4.0			
◆	BB-BLFB-101/2D	10+12.9	7.8 RT	10.0-12.0	SILT, some sand, trace clay, trace gravel.	24.4			NP
■									
●									
▲									
✖									

WIN
017531.00
Town
Bristol
Reported by/Date
WHITE, TERRY A 7/23/2014

SHEET 1

Appendix C

Calculations

Analysis

Calculation of nominal and factored bearing resistance on rock for Strength Limit State Analysis

Method

Use data from boring and calculate the nominal bearing resistance as follows:

1. Bedrock Properties from Boring
2. Estimation of Rock Mass Rating
3. Determine rock property constants s and m
4. Calculate nominal bearing resistance of bedrock, qn, using RMR/GSI method in Wylie "Foundations on Rock

References

1. AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014

2. AASHTO Standard Specifications for Highway Bridges, 17th Ed. 2002

3. Wyllie, Duncan C, "Foundations on Rock", Second Edition, 2009.

4. "The Hoek-Brown Failure Criterion - A 1988 Update", E. Hoek and E.T. Brown

A. Bedrock Properties from boring

Boring BB-BLFB-101

R1. Gray, fine grained, hard, Banded Schist with Quartz boudinage. The upper 36 inches include seven segments less than 3 inches. **RQD = 72%. Rock Mass Quality = Fair**

Compressive Strength

No UCT Tests conducted on rock samples from BB-BLFB-101

Estimated range of compressive strengths - Ref: Standard Specifications for Highway Bridges 17th Ed. 2002, Table 4.4.8.1.2B

Schist Co = 200-3,000 ksf or 1,400-21,000 psi
Quartz diorite Co = 200-2,100 ksf or 1,400-14,000 psi

Use 7000 psi or 1008 ksf

$q_{uc} := 7000 \cdot psi$

B. Determination of Rock Mass Rating (RMR) from LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Mass

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

1. Strength of intact rock

From Standard Specifications for Highway Bridges, 17th Edition, 2002
Table 4.4.8.1.2B uniaxial compressive strength - examine values for Schist and Quartz Diorite:

Schist Co = 200-3,000 ksf or 1,400-21,000 psi
Quartz Diorite Co = 2,00-2,100 ksf or 1,400-14,000 psi

Use 7000 psi or 1008 ksf

From LRFD Table 10.4.6.4-1 for Uniaxial compressive strength = 520-1080 ksf: Relative Rating = 4

2. Drill Core Quality

Bedrock RQD = 72% (fair) From LRFD Table 10.4.6.4-1, RQD 50% to 75%; Relative Rating = 13

3. Spacing of joints

Spacing of joints mostly follow the foliation.
From LRFD Table 10.4.6.4.-1 Spacing of joints 2 in. - 1 ft; Relative Rating = 10

4. Condition of joints

Assume slightly rough surfaces, separation <0.05 in., soft joint wall rock; Relative Rating 12

5. Groundwater conditions

General Conditions = Water under moderate pressure (Relative Rating 4) to Moist only (Relative Rating 7)

Relative Rating = 5

6. From LRFD Table 10.4.6.4.-2 Geomechanics Rating Adjustment for Joint Orientations

Strike and dip orientations of joints are Fair (-7) to Favorable (-2) use Relative Rating = -5

ADJUSTED RMR

$$RMR := 4 + 13 + 10 + 12 + 5 - 5$$

$$RMR = 39$$

Determine Rock Type for LRFD Table 10.4.6.4.-4

Rock Type - E = Coarse grained polymineralic igneous & metamorphic crystalline rocks
amphibolite, gabbro, gneiss, granite, diorite, quartz-diorite

Geomechanics Rock Mass Class Determined from Total Rating

From AASHTO LRFD Table 10.4.6.4-3, RMR = 39 is indicative of Poor Quality Rock Mass.

C. Rock Property Constants s and m (Ref. #1 and Ref. #4)

$RMR = 39$

Direct calculation of m and s is required, Reference 4 (Hoek and Brown, 1988), Equations 18 and 19 and Table 1. Assume isotropic behavior caused by the number and inconsistency of closely spaced discontinuity sets where none is significantly weaker than the other.

For a disturbed rock mass: m/mi = exp ((RMR-100)/14)

s = exp ((RMR-100)/6)

mi = m for intact rock

For rock type E, for intact rock, RMR=100, mi = 25 (Ref. # 3, Table 1) and s = 1

$m_i := 25$

$m := m_i \cdot \exp\left(\frac{RMR - 100}{14}\right)$ Equation 18, Ref. 3

$m = 0.32$

$s := \exp\left(\frac{RMR - 100}{6}\right)$ Equation 19, Ref. 3

$s = 3.843021 \times 10^{-5}$

D. Nominal and Factored Bearing Resistance of Bedrock

Correction Factor for Foundation Shape, from Wyllie Table 5.4 Pg. 138 (Ref. #2)

$C_{f1} := 1.0$ Conservative selection of Cfl = 1.0 for L/B>6

Nominal Bearing Resistance (Wyllie)

Reference #3: Wyllie "Foundations on Rock" Equation 5.4 Pg. 138

$$q_n := C_{f1} \cdot \sqrt{s} \cdot q_{uc} \cdot \left[1 + \sqrt{m \cdot \left(s^{\frac{-1}{2}}\right) + 1}\right]$$

$q_n = 52 \cdot ksf$

Factored Bearing Resistances

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$\phi_{bc} := 0.45$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 23 \cdot ksf$ Strength Limit State

Verify Nominal Bearing Resistance per Carter and Kulhawy (1988)

Reference : NCHRP, Report 651, LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, pg 40, Eq. 82b, and referred to in LRFD C.10.6.3.2.2. Same equation.

$$q_n := q_{uc} \cdot \left[\sqrt{s} + \sqrt{m \cdot \left(\sqrt{s}\right) + s}\right]$$

$q_n = 52 \cdot ksf$

Factored Bearing Resistance

Use a bearing resistance factor of 0.45 for Footings on Rock per LRFD Table 10.5.5.2.2-1

$\phi_{bc} := 0.45$

$q_r := q_n \cdot \phi_{bc}$

$q_r = 23 \cdot ksf$ Strength Limit State

Recommendation: Use 23 ksf for Factored Bearing Resistance at the Strength Limit State

Analysis

Calculation of nominal and factored bearing resistance on bedrock for Service Limit State Analysis

Method 1

Per AASHTO LRFD 10.6.2.4.4 - Settlement of Footings on Rock, "For footings bearing on fair to very good rock according to Geomechanics Classification system (i.e. RMR), as defined in Article 10.4.6.4, and designed in accordance with the provisions of this Section, elastic settlement may generally be assumed to be less than 0.5 inch."

Method 2

LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footings at the Service Limit State, based on *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, 7.2-142, "Presumptive Values of Allowable Bearing Pressures for Spread Foundations".

Abutment 1

Bearing Material:	Weathered or broken bedrock of any kind, except shale.
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16-24 ksf
AASHTO Recommended Value	20 ksf

$$q_{nominal} := 20 \cdot ksf$$

Resistance Factor for Service Limit State

$$\phi_r := 1.0$$

Per LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values for the factored bearing resistance for Service Limit State Analyses, settlement is typically limited to 1 inch

$$q_{factored} := \phi_r \cdot q_{nominal}$$

$$q_{factored} = 20 \cdot ksf$$ Service Limit State

Backfill engineering strength parameters

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

Unit weight $\gamma_1 := 125 \cdot pcf$

Internal friction angle $\phi_1 := 32 \cdot deg$

Cohesion $c_1 := 0 \cdot psf$

Restrained Arch Stem Walls and Footings

At-Rest Earth Pressure - Rankine Theory

Reference: Das, Principles of Foundation Engineering, 4th Edition, pg 336

$K_o := 1 - \sin(\phi_1)$

$K_o = 0.47$

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

From Design Freezing Index Map:**Bristol, Maine**
DFI = 1250 degree-days.
Case 1 - coarse grained granular fill soils W=20% (assumed).

For DFI = 1200 $d1 := 60.4$

For DFI = 1300 $d2 := 63.0$

$$d := in \cdot \left(\frac{d2 - d1}{10} \cdot 2 + d1 \right)$$

Depth of Frost Penetration $d = 61 \cdot in$ $d = 5.1 \cdot ft$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Belfast lies along the same Maine Design Freezing Index contour - use Belfast data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Belfast, Maine
Air Design Freezing Index = 1188 F-days
N-Factor = 0.80
Surface Design Freezing Index = 950 F-days
Mean Annual Temperature = 45.5 deg F
Design Length of Freezing Season = 118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	60.0	20.0	120.0	32	44	3.2	1.7	3,456
2-Coarse	3.3	20.0	120.0	32	44	3.2	1.7	3,456

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

Total Depth of Frost Penetration = 5.27 ft = 63.3 in.

Recommendation: 5.1 feet for design of foundations constructed on coarse grained soils