

**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

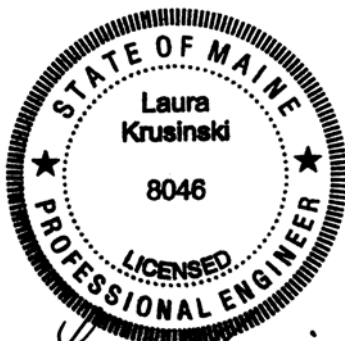
**RED BRIDGE
STATE ROUTE 2 OVER PENJEJAWOCK STREAM
BANGOR, MAINE**

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Penobscot County
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Fed No. STP-150(900)X
April 6, 2009

Soils Report No. 2009-10
Bridge No. 2711

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Red Bridge which carries Route 2 over Pennejawock Stream, in Bangor, Maine. The proposed replacement bridge will be a precast concrete three-sided box or precast concrete arch founded on strip spread footings constructed on bedrock. The superstructure curb-to-curb width will match the existing of 40 feet and will be centered on the existing alignment.

The following design recommendations are discussed in detail in this report:

Concrete Arch Stem Walls and Wingwalls – Precast arch or box stem walls on spread footings shall be designed to resist all lateral earth loads, vehicular loads, superstructure loads, and all reactions transferred to the footings through the arch walls. They shall be designed for all relevant strength and service limit states in accordance with AASHTO LRFD Bridge Design Specifications 4th Edition, 2007, (herein referred to as LRFD).

The design of precast stem walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure. A sliding resistance factor, ϕ_r , of 0.90 shall be applied to the nominal sliding resistance of arch walls and wingwalls founded on spread footings on bedrock. For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths (3/8) of the footing dimensions, in either direction.

Earth loads shall be calculated using an at rest earth pressure coefficient, K_o , of 0.47. The Designer may assume Soil Type 4 [Bridge Design Guide (BDG) Section 3.6.1] for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pounds per cubic foot (pcf). Additional lateral earth pressure on arch stem walls or wingwalls due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG.

Factored Bearing Resistance – The factored bearing pressure at the strength limit state for spread footings on bedrock should not exceed the factored bearing resistance of 26 kips per square foot (ksf). Based on presumptive bearing resistance values, a factored bearing resistance of 20 ksf may be used when analyzing the service limit state and for preliminary footing sizing, as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the footing concrete, which may be taken as $0.3 f'c$.

No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

GEOTECHNICAL DESIGN SUMMARY – CONTINUED

Return Wingwalls - Precast Concrete Modular Gravity (PCMG) walls founded on bedrock may be used as return wingwalls. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The bearing resistance for the PCMG wall founded on a leveling slab founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 26 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 20 ksf may be used to control settlement when analyzing the service limit state, and for preliminary base unit sizing.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, ϕ_{τ} , of 0.90 shall be applied to the nominal sliding resistance of the portion of precast concrete wall segments founded on leveling pads cast on bedrock, the wall unit stems bearing on leveling fill, and the area of soil within the precast concrete units in contact with leveling fill soil placed on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 ($0.80 \cdot \tan 30^\circ$) at the bedrock subgrade to precast concrete interfaces, and a maximum frictional coefficient of 0.58 ($\tan 30^\circ$) at the bedrock subgrade with leveling fill to soil-infill interfaces.

For the lowest PCMG unit, the eccentricity of factored loads at the strength limit state shall not exceed three-eighths ($3/8^{\text{ths}}$) of the footing dimensions, in either direction.

Scour and Riprap - For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the Maine Bridge Design Guide (BDG).

Settlement - The grades of bridge approaches and side slopes will be not raised, therefore post-construction settlement due to compression of the foundation soils is anticipated to be less than 0.5 inch and will have minimal effect on the finished structure. Any settlement of the buried structure will be due to elastic settlement of the bedrock, which is assumed to occur during construction and be less than 0.5 inch.

Frost Protection - Foundations placed on bedrock are not subject to heave by frost, therefore, there are no frost embedment requirements for project footings cast directly on sound bedrock. In the situation that any foundations are placed on compacted granular borrow, the footings should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations – In accordance with LRFD 3.10.1, seismic analysis is not required for buried structures.

Construction Considerations – Cofferdams and temporary earth support systems will be required for the construction of precast buried structures, strip footings and wingwalls. Preparation of the bedrock subgrade for spread footings to support arch walls or wingwalls may require excavation of bedrock to create level benches or a completely level surface. All

loose and disturbed bedrock and soil debris should be removed from bearing surfaces and the surfaces washed with high-pressure water and air before concrete is placed for footings.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Red Bridge which carries State Route 2 over Penjajawock Stream, in Bangor, Maine. This report summarizes the soil information obtained at the site during the subsurface investigations, and presents foundation recommendations and geotechnical design parameters for bridge replacement.

Red Bridge was built in 1936 and is a 14-foot single span, concrete slab superstructure, supported on full-height, mass concrete gravity abutments. The abutments contain portions of the pre-1936 unreinforced concrete and dry laid, field stone abutments. The substructures were rehabilitated in 1986 and grout bag repairs were executed in 1996.

Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports indicate the substructures are in serious condition. An upstream gabion retaining wall has failed due to undermining, and the stream is shifting northward and undercutting the embankment. Both abutments have open horizontal cracks of $\frac{3}{4}$ to 2.5-inch and there is some shifting of the substructures. The south abutment is tipping back above the horizontal crack. The north abutment also has an open vertical crack.

MaineDOT Bridge Maintenance inspection reports (2007) assign the deck and superstructure a condition rating of 4 – poor, and the substructure a condition rating of 3 – serious. Bridge Maintenance inspection reports (2007) assign the structure a Bridge Sufficiency Rating of 46.7. The May 2008 MaineDOT Scope Review Team report considered the serious condition of the substructures and recommended bridge replacement.

Preliminary foundation alternatives were provided by the geotechnical team member in a Preliminary Design Report (PDR) Meeting on October 15, 2008. Subsequent preliminary engineering assessments by the MaineDOT Bridge Program resulted in the recommendation of a bridge replacement project with foundations consisting of spread footings founded directly on bedrock or on seal concrete founded on bedrock.

2.0 GEOLOGIC SETTING

Red Bridge on State Route 2 in Bangor, Maine crosses the Penjajawock Stream which flows into the Penobscot River as shown on Sheet 1 - Location Map, presented at the end of this report. The proposed precast bridge structure will be relocated slightly to the north to accommodate a new stream alignment.

The Maine Geologic Survey (MGS) Surficial Geology of Bangor Quadrangle, Maine, Open-file No. 77-24 (1977) indicates that Red Bridge in Bangor is at a contact of glacial till and glacial marine deposits. Glacial till is a heterogeneous mixture of sand, silt, clay and stones, and includes two varieties: basal till and ablation till. Basal till is fine grained and very compact. Ablation till is loose, sandy and stoney. The till unit generally overlies

bedrock, and was deposited directly by glacial ice. Till deposits typically conform to the bedrock surface, and were deposited directly by the glacial ice. Glacial marine deposits, also known as the Presumpscot Formation, is commonly a clayey silt, but sand is also abundant at the surface in some areas.

The Bedrock Geologic Map of Maine, MGS, (1985), cite the bedrock at the Red Bridge site as the Vassalboro Formation and consists of calcareous sandstone, interbedded sandstone and impure limestone.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three (3) test borings. All borings were terminated with bedrock cores. Phase I test borings BB-BPS-101 and BB-BPS-102 were drilled behind the locations of the existing north and south abutments, respectively. One additional boring, BB-BPS-201 was drilled behind the north abutment to explore subsurface conditions for a possible stream realignment. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. The Phase I borings were drilled on June 11 and 12, 2008 using the Maine Department of Transportation (MaineDOT) drill rig. The phase two boring was drilled on January 30, 2009, by Northern Test Boring, Inc. of Gorham, Maine.

The borings were 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 drill rig and Northern Test Boring drill rig are equipped with automatic hammers. The hammers have been calibrated. All N-values discussed in this report are corrected values computed by applying average energy transfer factors to the raw field N-values. The hammer efficiency factors and both the raw field N-value and the corrected N-value are shown on the boring logs.

The bedrock was cored in the three borings using an NQ-2 core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The Geotechnical Team Member or a MaineDOT Certified Subsurface Investigator logged the subsurface conditions encountered. The borings were located in the field using taped distant measurements to site features after completion of the drilling program.

Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs, found at the end of this report.

4.0 LABORATORY TESTING

Laboratory testing of samples obtained in the borings consisted of five (5) standard grain size analyses, two (2) grain size analyses with hydrometer, seven (7) natural water content tests, and one (1) Atterberg Limits test. The results of soil laboratory tests are included as Appendix B - Laboratory Data, at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of granular fill, weathered glacial marine deposits, alluvium, and glacial till, all underlain by metamorphic bedrock. An interpretive subsurface profile depicting the generalized soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile, found at the end of this report. A brief summary description of the strata encountered is as follows:

5.1 Fill

A layer of fill was encountered in the all three of the test borings drilled in the vicinity of the proposed precast structure. The encountered fill layer is approximately 10 to 20 feet thick. The fill unit generally consisted of brown, dry to moist, SAND, little silt, and silty SAND, some to little gravel and sandy GRAVEL, trace silt. A subunit of black, moist asphalt fragments, some sand and gravel was encountered in the fill unit in BB-BPS-102. An approximately 0.80 foot thick layer of concrete was encountered in the fill soil in BB-BPS-201.

Corrected SPT N-values in the granular fill layer ranged from 15 to 56 blows per foot (bpf) indicating that the fill unit is medium dense to very dense in consistency.

Grain size analyses were conducted on three (3) samples from the fill soil unit. Grain size analyses resulted in the soil being classified as A-1-a, A-1-b and A-2-4 under the AASHTO Soil Classification System and as GW-GM and SM under the Unified Soil Classification System. Natural water contents of tested samples ranged from approximately 3 to 12%.

5.2 Weathered Glacial Marine Silt

An approximately 5.8-foot thick layer of weathered glacial marine silt was encountered below the granular fill in BB-BPS-101. The deposit consisted of brown, mottled, damp to moist, SILT, some sand, little clay, trace fine gravel and grey, wet, SILT, some clay, little sand, trace gravel with blocky structure. Corrected SPT N-values in silt layer ranged from 6 to 9 blows per foot (bpf) indicating that the silt is medium stiff to stiff in consistency. Two pocket penetrometer tests indicated an estimated unconfined compressive strength of 1500 psf, which correlates to a soil with medium stiff consistency.

Grain size analyses were conducted on two (2) samples from the glaciomarine silt deposit. Grain size analyses resulted in the soil being classified as A-4 under the AASHTO Soil Classification System and as CL and CL-ML under the Unified Soil Classification System. An Atterberg Limits test on one sample indicated the soil is nonplastic. Natural water contents of tested samples ranged from approximately 22 to 26%.

5.3 Fine Sand

An approximately 7.8-foot thick, discontinuous layer of fine, sandy alluvium was encountered below the granular fill in BB-BPS-201. The deposit consisted of grey-brown, damp to wet, fine SAND, to fine SAND, little silt, trace gravel. Corrected SPT N-values in fine sand layer were 18 and 19 blows per foot (bpf) indicating that the sand is medium dense in consistency.

One grain size analysis was conducted on a sample from the alluvial deposit. The grain size analysis resulted in the soil being classified as A-2-4 under the AASHTO Soil Classification System and SM under the Unified Soil Classification System. The natural water content of the tested sample was approximately 20%.

5.4 Glacial Till

A layer of glacial till was encountered in test borings BB-BPS-101 and BB-BPS-201, which are located behind the existing north abutment. The encountered till layer is approximately 5.6 to 8.6 feet thick. The till soils encountered in BB-BPS-101 consisted of brown to grey, wet, SAND, some to little gravel, some to little silt, with stained and oxidized lenses. The till soils encountered in BB-BPS-201 consisted of cobbles and gravel.

Corrected SPT N-values in glacial till layers ranged from 6 to 12 bpf in BB-BPS-101 indicating that the till unit is loose to medium dense in consistency. The till subunit in BB-BPS-201 was cored with a NQ-2 core barrel.

5.5 Bedrock

Bedrock at the site was encountered and cored at a depth of approximately 24.4 feet below ground surface (bgs) and approximate Elevation 5.1 feet in boring BB-BPS-101. In the two borings drilled along the northern wall of the proposed buried box or arch, bedrock was encountered and cored at depth of approximately 20 feet bgs and approximate Elevation 10.5 feet in boring BB-BPS-102 and at a depth of approximately 29.40 feet and approximate Elevation 2.0 feet in BB-BPS-201.

The bedrock at the site is identified as grey, fine grained, calcareous metasedimentary GREENSCHIST, hard, slightly weathered to fresh, joint set along bedding, dipping at irregular to steep angles, very closely spaced, tight to open with silt infilling. The rock quality designation (RQD) of the bedrock was determined to range from 24 to 95 percent, correlating to a rock mass quality of very poor to excellent.

The table below summarizes top of bedrock elevations at the proposed buried structure location.

Proposed Substructure	Boring	Station	Depth to Bedrock (feet)	Elevation of Bedrock Surface (feet)
Precast Box - South Arch Wall	BB-BPS-102	6+15.6	20.0	10.5
Precast Box – North Arch Wall	BB-BPS-201	6+66.1	29.4	2.0
Precast Box - North Arch Wall	BB-BPS-101	6+54	24.4	5.1

Table 1. Top of Bedrock Elevations

6.0 FOUNDATION ALTERNATIVES AND RECOMMENDATIONS

A Preliminary Design Report (PDR) Meeting for Red Bridge was held on October 15, 2008, during which foundation alternatives were provided to the team by the geotechnical team member. The following foundations were considered for the replacement bridge substructures and evaluated for practicality and effectiveness:

- Full height reinforced concrete abutments founded on new spread footings supported on bedrock or on seal concrete founded on bedrock.
- Three-sided precast concrete box or precast concrete arch with the box/arch stem walls founded on strip spread footings constructed on bedrock.

A precast three-sided box or precast arch structure, founded on strip spread footings founded directly on bedrock or on seal concrete on bedrock, was selected by the team to replace Red Bridge. This section presents only geotechnical design recommendations for a precast, buried structure.

6.1 General - Spread Footings on Bedrock

Supporting stem walls for a precast arch or 3-sided box stem on spread footings founded on bedrock is a practical and effective foundation alternative. The borings encountered bedrock approximately 20 to 30 feet below the bridge approaches at the locations of the three borings. It is therefore considered feasible that cofferdams, seals (if required) and spread footings could be practically and economically constructed to bear on bedrock.

The borings indicate that suitable bedrock with RQD's ranging from 24 to 95 percent will be encountered at the bedrock surface, however, the bedrock surface shall be cleared of all loose rock, disturbed bedrock and soil. Based on borings conducted at the site and top of bedrock

elevations encountered, the bottom of footing or seal elevations are estimated to be approximately 20 feet below the south wall of the proposed buried precast structure and approximately 24 to 30 feet below the roadway surface along the north wall of the proposed buried structure.

6.2 Arch/Box Stem Wall and Footing Design

Precast stem walls and spread footings shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 12.5 and shall be designed for all relevant strength and service limit states. In addition to the typical loads, spread footing should consider all reactions transferred to the footings through the arch/box walls. The design of precast stem walls founded on spread footings at the strength limit state shall consider nominal bearing resistance, eccentricity (overturning), lateral sliding and structural failure.

In accordance with LRFD Article 12.5.5, the resistance factor values for the geotechnical design of foundations for buried structures shall be as specified in LRFD Section 10 – Foundations.

The design of buried structures shall also consider foundation resistance after scour due to the design flood. In accordance with LRFD Article 10.5.5.3.2, the foundation resistance after scour shall be adequate to support the unfactored Strength Limit State loads with a resistance factor of 1.0.

A sliding resistance factor, ϕ_{τ} , of 0.90 shall be applied to the nominal sliding resistance of precast stem wall spread footings on bedrock. Sliding computations for resistance to lateral reactions from arch thrust shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

For footings on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{\text{th}}$) of the footing dimensions, in either direction.

A resistance factor of 1.0 shall be used to assess spread footing design at the service limit state, including: settlement, excessive horizontal movement and scour at the design flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65

Arch and box stem walls shall be designed as restrained, rigid frames. Earth pressures shall be calculated using an at rest earth pressure coefficient, K_o , of 0.47. The designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the BDG. The live load surcharge on arch stem walls may be

estimated as a uniform earth pressure due to an equivalent height of soil (h_{eq}) taken from the table below:

Precast Arch or 3-Sided Box Stem Wall Height (feet)	h_{eq} (feet)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Table 2. Equivalent height of soil for live load surcharge

Precast arch or 3-sided box designs shall include a drainage system behind the stem walls to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG.

Backfill within 10 feet of the precast stem walls shall conform to MaineDOT Specification 709.19, Granular Borrow for Underwater Backfill. This gradation specifies 10 percent or less of material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure. The contractor shall compact backfill in the controlled backfill zone in accordance with the manufacturer's recommendations but no less than 92% of the AASHTO T-180 maximum dry density.

6.3 Factored Bearing Resistance

Spread footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads are specified in LRFD Article 11.5.5 and 12.5 and shall include reactions due to thrust in the arch or box walls. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2.

The bearing resistance for footings for precast arches and retaining walls founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 26 ksf. This assumes a bearing resistance factor, ϕ_b , for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. The calculated factored bearing resistance is based on fractured bedrock with an average RQD of at least 60%. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. See Appendix C – Calculations, for supporting documentation.

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

6.4 PCMG Retaining Walls

Precast Concrete Modular Gravity (PCMG) walls may be incorporated as return wingwalls. The walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The PCMG should be founded on bedrock. The PCMG wall shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to 2.0 feet of soil.

The bearing resistance for the PCMG wall founded on a leveling slab founded on bedrock shall be investigated at the strength limit stated using factored loads and a factored bearing resistance of 26 ksf. The stress distribution may be assumed to be a linear distribution over the effective footing base as shown in LRFD Figure 11.6.3.2-2. Based on presumptive bearing resistance values, a factored bearing resistance of 20 ksf may be used to control settlement when analyzing service limit state load combinations and for preliminary footing sizing. See Appendix C – Calculations, for supporting documentation.

The bearing resistance for the bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, the PCMG wall units should be designed so that the nominal bearing resistance, in conjunction with the depth of scour determined for the check flood for scour, provide adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. In general, spread footings at stream crossings should be founded a minimum of 2 feet below the calculated scour depth.

Failure by sliding shall be investigated by the wall subcontractor. A sliding resistance factor, ϕ_{τ} , of 0.90 shall be applied to the nominal sliding resistance of the portion of precast concrete wall segments founded on leveling pads cast on bedrock, the wall unit stems bearing on leveling fill soil, and the soil within the precast concrete units in contact with leveling fill soil placed on bedrock. Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.46 ($0.80 \cdot \tan 30^{\circ}$) at the bedrock subgrade to precast concrete interfaces, and a maximum frictional coefficient of 0.58 ($\tan 30^{\circ}$) at the bedrock subgrade with leveling fill to soil-infill interfaces. Recommended values of sliding frictional coefficients are based on LRFD Articles 10.6.3.4, 11.11.4.2 and Table 10.5.5.2.2-1.

For lowest PCMG unit on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eighths ($3/8^{\text{th}}$) of the footing dimensions, in either direction.

6.5 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limits states. These changes in foundation conditions shall be investigated at arch/box wall footings and wingwalls.

In general, for scour protection, any footings which are constructed on soil deposits should be embedded at least 2 feet below the design scour depth and armored with 3 feet of riprap for scour protection. Refer to BDG Section 2.3.11 for information regarding scour design.

For scour protection, bridge approach slopes and slopes at wingwalls should be armored with 3 feet of riprap as per Section 2.3.11.3 of the BDG. Stone riprap shall conform to Item number 703.26 of the Standard Specification and be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation or terminated at the surface of bedrock-exposed streambeds. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to Item number 703.19 of the Standard Specification. Riprap may be placed at the toes of wingwalls and retaining walls, as required.

6.6 Settlement

The grades of the bridge approaches and side slopes will not be raised in the construction of the proposed bridge, therefore post-construction settlement due to compression of the foundation soils will be negligible. Settlement of the bridge abutments due to elastic settlement of the bedrock is anticipated to occur during construction of the abutments, and is generally assumed to be less than 0.5 inches.

6.7 Frost Protection

Spread footings for precast arch or three-sided box stem walls and return wingwalls will be founded on bedrock. PCMG retaining walls should be constructed directly on bedrock. Therefore, heave due to frost is not a design issue, and no requirements for minimum depth of embedment are necessary.

In the situation that any foundations are placed on compacted granular borrow, the footings should be designed with an appropriate embedment for frost protection. According to the BDG, Bangor, Maine has a design freezing index of approximately 1726 F-degree days. An assumed water content of 20% was used for moist, coarse grained soils above the water table. These components correlate to a frost depth of 6.0 feet. Therefore, any foundations placed on soil should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

6.8 Seismic Design Considerations

In accordance with LRFD Article 3.10.1, seismic analysis is not required for buried structures.

6.9 Construction Considerations

Construction of strip footings for a precast arch or three-sided box stem walls, and footings for return wingwalls will require soil excavations and may require cofferdam construction, and earth support systems.

The arch/box stem wall footing subgrade should consist of 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 surface shall be cleared of all loose and decomposed bedrock and soil. The bearing surface shall then be washed with high-pressure water and air prior to concrete being placed for the footing.

The bedrock surface shall be stepped to create level benches or excavated to be level overall. Elsewhere, the bedrock surface slope shall be less than 4 horizontal to 1 vertical (4H:1V) or it shall be benched in level steps or excavated to be completely level. 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.

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

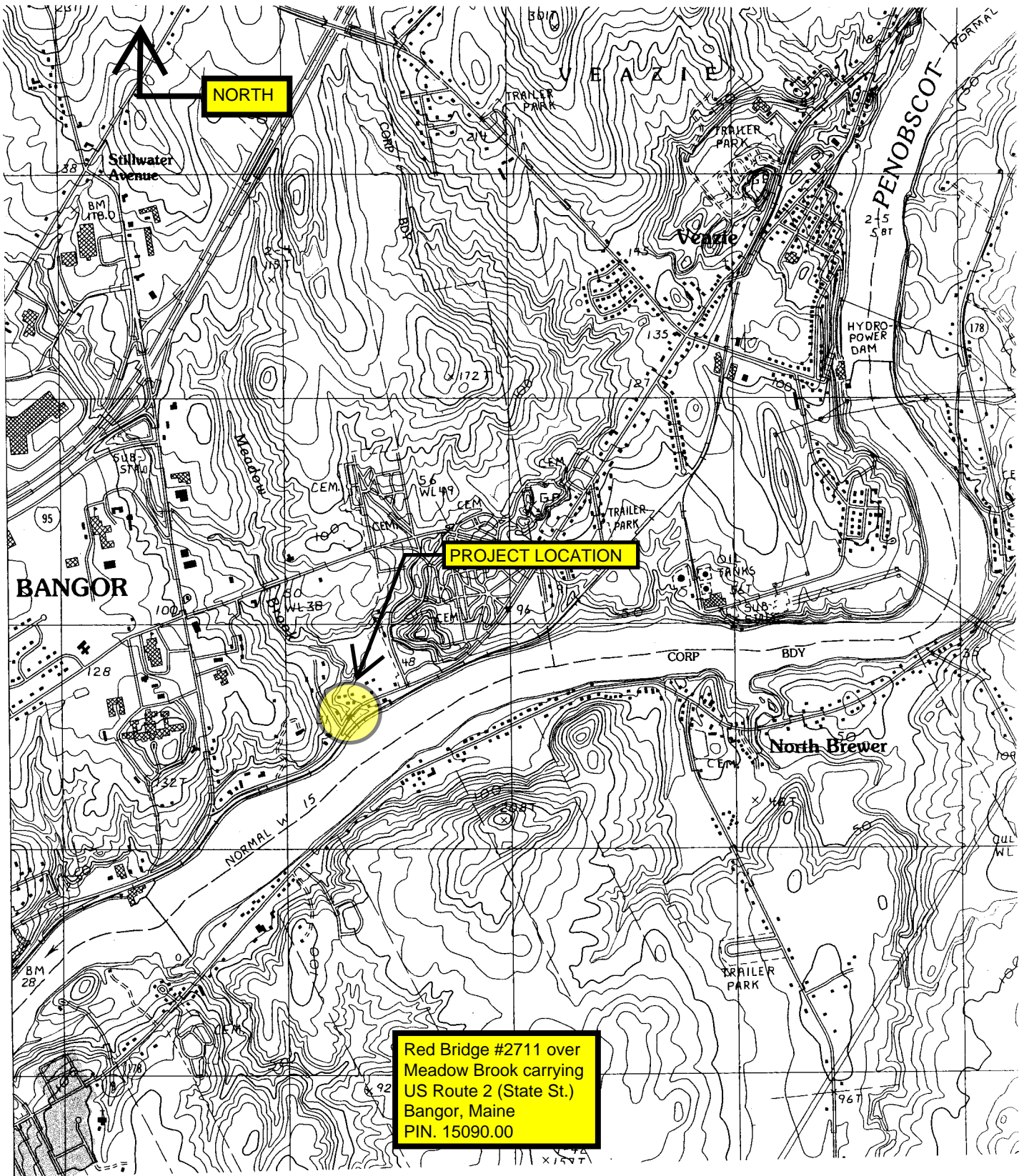
It is anticipated that there will be seepage of water from fractures and joints exposed in the bedrock surface. Water should be controlled by pumping from sumps. The contractor should maintain the excavation so that all foundations are constructed in the dry.

7.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Red Bridge in Bangor, 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.

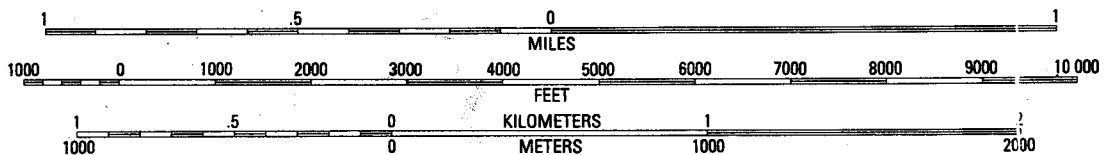
Sheets



Red Bridge #2711 over
Meadow Brook carrying
US Route 2 (State St.)
Bangor, Maine
PIN. 15090.00

**VEAZIE QUADRANGLE
MAINE-PENOBSCOT CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)**

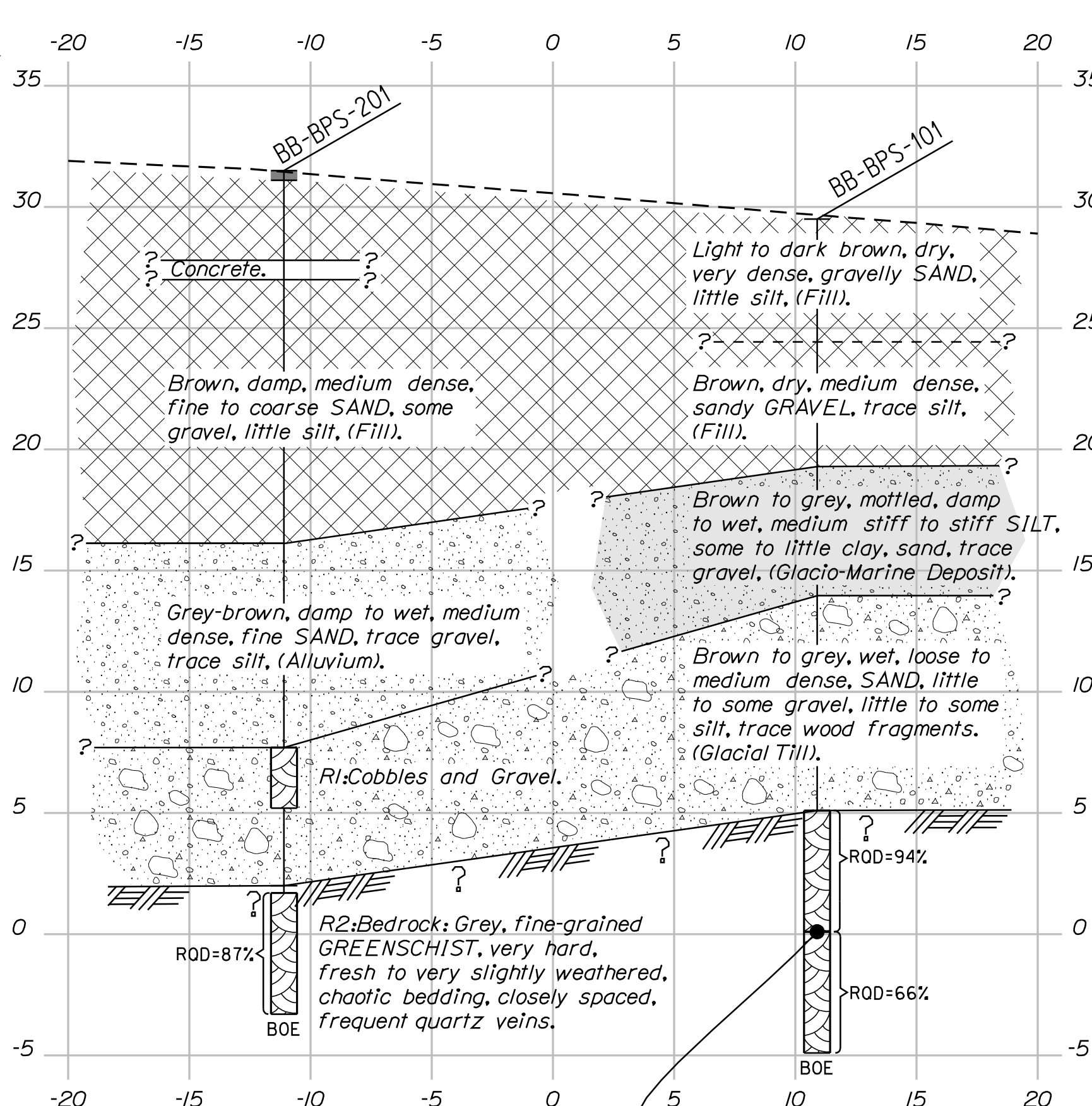
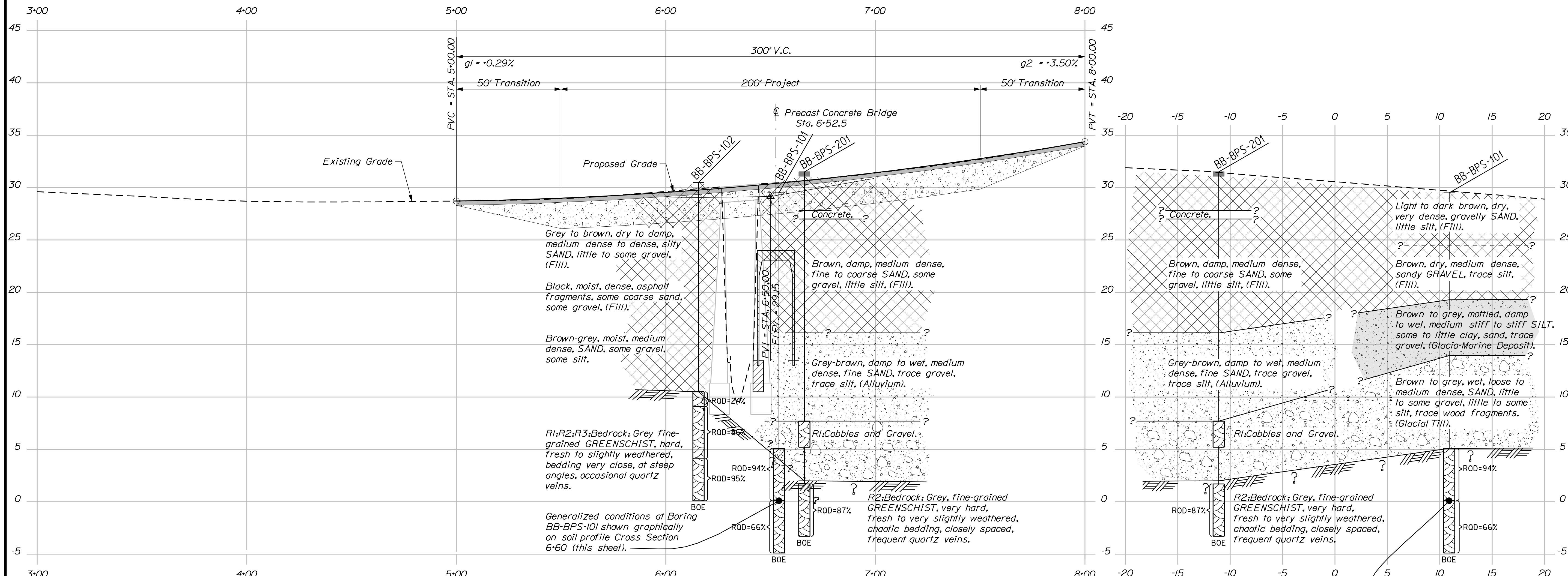
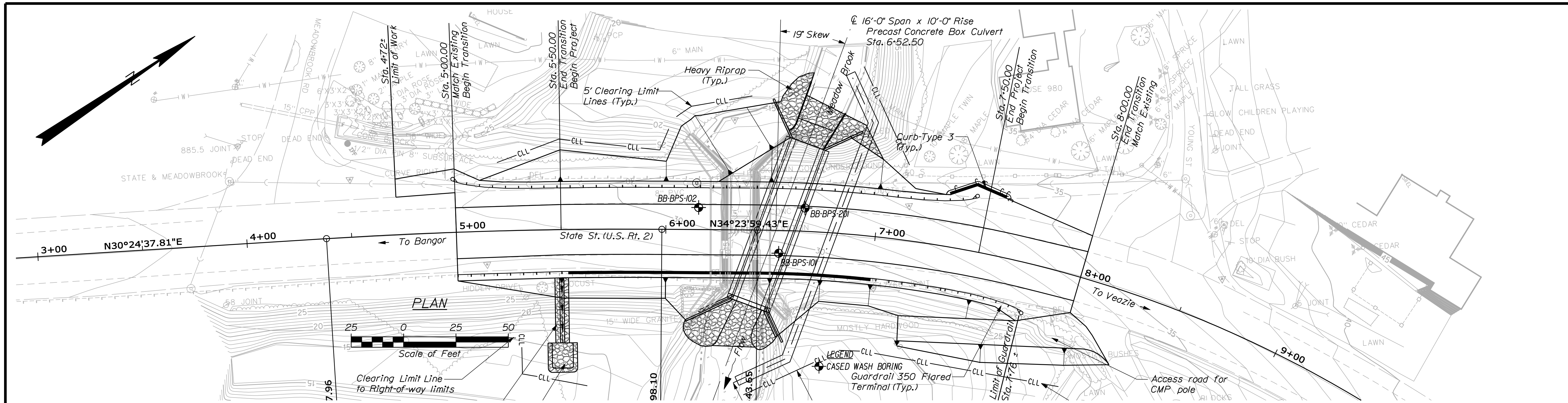
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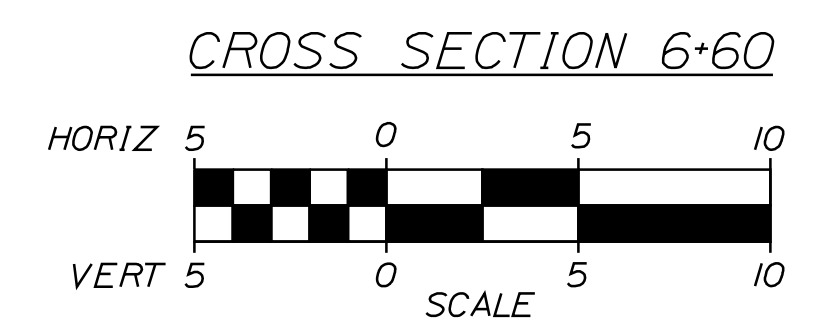
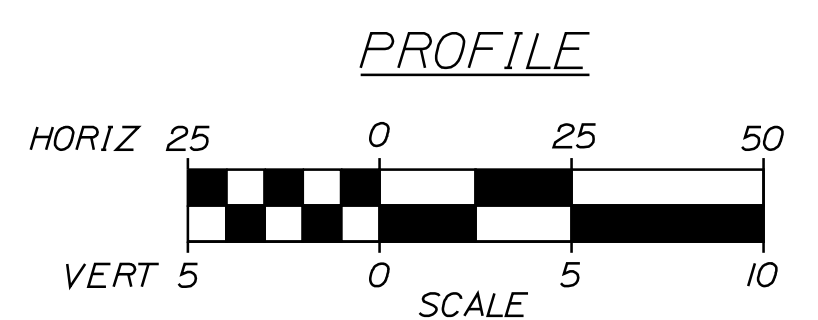
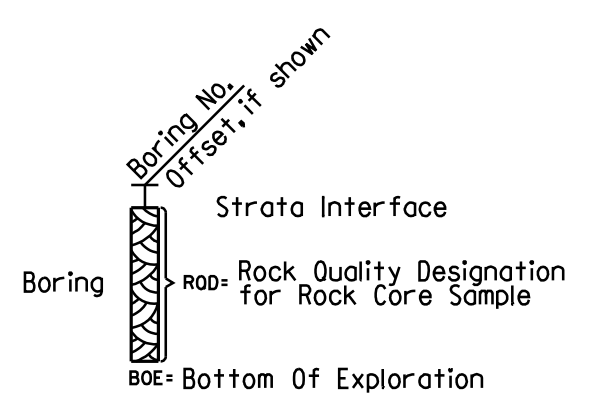
Date: 4/16/2009

Username: terry.white

Filename: ... \GEOTECH\MSTA\006_BLP&BSP\ldgn Division: GEOTECH



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	
15090.00		PIN 15090.00	
BRIDGE NO. 2711		BRIDGE PLANS	
PROJ. MANAGER	BY	DATE	SIGNATURE
DESIGN DETAILED	L. KRUSINSKI	JUNE 2008	
CHECKED/REVIEWED	T. WHITE		
DESIGN DETAILED			
DESIGN DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			
RED BRIDGE		PENOBSCOT COUNTY	
PENJEWOCK STREAM		BANGOR	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE			
SHEET NUMBER			
2			
OF 3			

Maine Department of Transportation		Project: Red Bridge #211 over Meadow Brook on Route 2 (State Street)		Boring No.: BB-BPS-102	
Soil/Block Exploration Log		Location: Bangor, Maine		PIN: 15090.00	
US CUSTOMARY UNITS		US CUSTOMARY UNITS		US CUSTOMARY UNITS	
Driller: MaineDOT	Elevation (ft.): 30.5	Auger ID/OD: 5" Solid Stem	Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Operator: E. Giguere/C. Giles	Rig Type: CME 45C	Hammer Wt./Fall: 140W/30"	Logged By: L. Krusinski	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"
Date Start/Finish: 6/11/08-6/12/08	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"	Boring Location: 6454.1, 10.9 Rt.	Water Level: None Observed	
Boring Location: 6454.1, 10.6 Lt.	Hammer Type: Automatic 55	Hydraulic	Rope & Cathead		
Hammer Efficiency Factor: 0.77	<small> A = Rock Core Sample S = Split Spoon Sample T = Pocket Torque Shear Strength Test W = Water Content, percent B = Unsuccessful Split Spoon Sample attempt SA = Solid Stem Auger U = Uncollected Undersize Strength Test WC = water content, percent C = Thin Wall Tube Sample SS = Hollow Stem Auger V = Vertical Field Shear Strength Test X = Liquid Limit Y = Plasticity Index Z = Consolidation Test D = Unsuccessful Thin Wall Tube Sample attempt SW = Weight of 100lb. hammer RC = Roller Cone Unconsolidated = Raw Field SPT Results PL = Plastic Limit C = Grain Size Analysis E = In Situ Cone Shear Test PE = Pocket Penetrometer W = Weight of rod or casing NG = SPT uncorrected corrected for hammer efficiency G = Grain Size Analysis No = Unsuccessful In Situ Cone Shear Test attempt WP = Weight of one person X = Consolidation Test </small>				
Sample Information		Visual Description and Remarks		Laboratory Testing Results/AASHTO and Unified Class	
Depth (ft.)	Sample No.	Pen./Rec. (in)	Sample Depth (ft.)	Blows 1/6 in. (ft.)	Blows 1/6 in. (ft.)
0	10	24/16	1.00 - 3.00	14/18/6/9	24
Gray to brown, dry to damp, dense, silty fine to coarse SAND, some rounded to subangular coarse gravel, little fine gravel. (F111).					
5	20	24/8	5.00 - 7.00	3/9/4/5	13
Brown, damp, medium dense, silty, fine to medium SAND, little fine rounded gravel, trace coarse sand, similar to 10. (F111).					
10	30	24/8	10.00 - 12.00	18/17/6/6	25
0184 blows for 7". Black, moist, sandy, asphalt pavement fragments, some coarse sand and fine gravel. (F111).					
15	40	24/10	15.00 - 17.00	7/13/10/10	23
Brown-grey, moist, medium dense, fine to medium SAND, some rounded fine to coarse gravel, little silt.					
20	R1	16-8/9-6	20.00 - 21.40	ROD = 24%	
	R2	60/51-6	21.40 - 25.40	ROD = 86%	
Top of Bedrock at Elev. 10.5'. R1: Bedrock Grey, fine grained, calcareous metasedimentary (GREENSCHIST); hard, slightly weathered, slightly fractured, core blocked off open silt seam at 0.8 ft. R2: Core Times (minsec) 20-29.4' (1235) 51% Recovery Core Blocker					
25	R3	48/48	26.40 - 30.40	ROD = 95%	
R2: Bedrock Grey, fine grained, calcareous metasedimentary (GREENSCHIST); hard, slightly weathered, slightly fractured, core blocked off open silt seam at 0.8 ft. R3: Core Times (minsec) 21-22.4' (1240) 22-24-23.4' (1416) 23-4-24.4' (1420) 24-4-25.4' (1420) 24-4-25.4' (1420) 24-4-25.4' (1420) 25-4-26.4' (1420) 26-4-27.4' (1420) 27-4-28.4' (1410) 28-4-29.4' (1330) 29-4-30.4' (1400) 100% Recovery					
30					
Bottom of Exploration at 30.40 feet below ground surface.					

Maine Department of Transportation		Project: Red Bridge #211 over Meadow Brook on Route 2 (State Street)		Boring No.: BB-BPS-101	
Soil/Block Exploration Log		Location: Bangor, Maine		PIN: 15090.00	
US CUSTOMARY UNITS		US CUSTOMARY UNITS		US CUSTOMARY UNITS	
Driller: MaineDOT	Elevation (ft.): 29.5	Auger ID/OD: 5" Solid Stem	Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Operator: E. Giguere/C. Giles	Rig Type: CME 45C	Hammer Wt./Fall: 140W/30"	Logged By: L. Krusinski/C. Beebe	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"
Date Start/Finish: 6/11/08-6/12/08	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"	Boring Location: 6454.1, 10.9 Rt.	Water Level: None Observed	
Boring Location: 6454.1, 10.9 Rt.	Hammer Type: Automatic 55	Hydraulic	Rope & Cathead		
Hammer Efficiency Factor: 0.77	<small> A = Rock Core Sample S = Split Spoon Sample T = Pocket Torque Shear Strength Test W = Water Content, percent B = Unsuccessful Split Spoon Sample attempt SA = Solid Stem Auger U = Uncollected Undersize Strength Test WC = water content, percent C = Thin Wall Tube Sample SS = Hollow Stem Auger V = Vertical Field Shear Strength Test X = Liquid Limit Y = Plasticity Index Z = Consolidation Test D = Unsuccessful Thin Wall Tube Sample attempt SW = Weight of 100lb. hammer RC = Roller Cone Unconsolidated = Raw Field SPT Results PL = Plastic Limit C = Grain Size Analysis E = In Situ Cone Shear Test PE = Pocket Penetrometer W = Weight of rod or casing NG = SPT uncorrected corrected for hammer efficiency G = Grain Size Analysis No = Unsuccessful In Situ Cone Shear Test attempt WP = Weight of one person X = Consolidation Test </small>				
Sample Information		Visual Description and Remarks		Laboratory Testing Results/AASHTO and Unified Class	
Depth (ft.)	Sample No.	Pen./Rec. (in)	Sample Depth (ft.)	Blows 1/6 in. (ft.)	Blows 1/6 in. (ft.)
0	10	24/17	1.00 - 3.00	35/25/19/19	44
Light to dark brown, dry, very dense, fine to coarse angular to subrounded gravelly fine SAND, some medium to coarse sand, little silt.					
5	20	24/12	5.00 - 7.00	5/6/7/11	13
Brown, dry, medium dense, sandy fine to coarse GRAVEL, subrounded to subangular, trace silt. (F111).					
10	30	24/14	10.00 - 12.00	7/5/2/2	7
Brown, moist, damp to moist, stiff, SILT, some sand, little clay, trace fine gravel. (Weathered Presumpscot Formation)					
15	40/AB	24/18	15.00 - 17.00	ND/4/2/3/5	5
40/AB: 15.0-15.8' bgs. Grey, wet, medium stiff, SILT, some clay, little sand, trace gravel, blocky. (Presumpscot Formation). Sp = 1.5 ksf					
	50	24/8	20.00 - 22.00	5/4/5/5	5
Brown, wet, medium dense, SAND, some fine gravel, little silt, trace wood fragments.					
25	R1	60/60	24.40 - 28.40	ROD = 94%	
083 blows for 0.4'. Roller Cone ahead to 24.4' bgs., wood fragments on roller cone. Top of Bedrock at Elev. 5.1'. R1: Bedrock Green-grey, fine grained, calcareous metasedimentary (GREENSCHIST); hard, fresh, bedding/foliation very close, irregular to steep angles, second joint set perpendicular to foliation, one open silt seam in fractured portion 2 ft into core run, waxy portions, frequent quartz veins. R2: Core Times (minsec) 24-4-25.4' (1235) 25-4-26.4' (1250) 26-4-27.4' (1250) 27-4-28.4' (1250) 28-4-29.4' (1310) 29-4-30.4' (1310) 100% Recovery R3: Bedrock Grey, fine grained, calcareous metasedimentary (GREENSCHIST); hard, fresh to slightly weathered, foliation close, irregular dip, but predominantly moderate dip, highly fractured zone 3.4-5.0 ft into core run, surfaces with silt infilling. R4: Core Times (minsec) 29-4-30.4' (1310) 30-4-31.4' (1310) 31-4-32.4' (1310) 32-4-33.4' (1410) 33-4-34.4' (1235) 100% Recovery					
30	R2	60/60	28.40 - 34.40	ROD = 66%	
Bottom of Exploration at 34.40 feet below ground surface.					

Maine Department of Transportation		Project: Red Bridge #211 over Meadow Brook on Route 2 (State Street)		Boring No.: BB-BPS-201	
Soil/Block Exploration Log		Location: Bangor, Maine		PIN: 15090.00	
US CUSTOMARY UNITS		US CUSTOMARY UNITS		US CUSTOMARY UNITS	
Driller: Northern Test Borings, Inc.	Elevation (ft.): 31.4	Auger ID/OD: 5" Solid Stem	Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon
Operator: Mike/Nick	Rig Type: Diedrick D-50 Truck	Hammer Wt./Fall: 140W/30"	Logged By: B. Wilder	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"
Date Start/Finish: 1/30/09-2/10-14/00	Drilling Method: Cased Wash Boring	Core Barrel: ND-2"	Boring Location: 6466.1, 11.1 Lt.	Water Level: None Observed	
Boring Location: 6466.1, 11.1 Lt.	Hammer Type: Automatic 55	Hydraulic	Rope & Cathead		
Hammer Efficiency Factor: 0.623	<small> A = Rock Core Sample S = Split Spoon Sample T = Pocket Torque Shear Strength Test W = Water Content, percent B = Unsuccessful Split Spoon Sample attempt SA = Solid Stem Auger U = Uncollected Undersize Strength Test WC = water content, percent C = Thin Wall Tube Sample SS = Hollow Stem Auger V = Vertical Field Shear Strength Test X = Liquid Limit Y = Plasticity Index Z = Consolidation Test D = Unsuccessful Thin Wall Tube Sample attempt SW = Weight of 100lb. hammer RC = Roller Cone Unconsolidated = Raw Field SPT Results PL = Plastic Limit C = Grain Size Analysis E = In Situ Cone Shear Test PE = Pocket Penetrometer W = Weight of rod or casing NG = SPT uncorrected corrected for hammer efficiency G = Grain Size Analysis No = Unsuccessful In Situ Cone Shear Test attempt WP = Weight of one person X = Consolidation Test </small>				
Sample Information		Visual Description and Remarks		Laboratory Testing Results/AASHTO and Unified Class	
Depth (ft.)	Sample No.	Pen./Rec. (in)	Sample Depth (ft.)	Blows 1/6 in. (ft.)	Blows 1/6 in. (ft.)
0	10	14.4/12	2.50 - 3.70	23/24/50/24.4"	---
PAVEMENT: Frost Depth 2'2" bgs. ---					
5	20	24/18	5.00 - 7.00	9/14/14/9	28
Brown, damp, medium dense, fine to coarse SAND, some fine to coarse gravel, little silt. (F111).					
10	30	24/16	10.00 - 12.00	6/8/6/5	14
Similar to above.					
15	40	24/20	15.00 - 17.00	12/8/9/10	17
Brown grey, damp, medium dense, fine SAND. (Alluvium).					
20	R1	30/4	23.80 - 26.30	ROD = N/A%	
Grey-brown, wet, medium dense, fine SAND, little silt, trace gravel. (Alluvium). Roller Cone ahead to 23.0' bgs. R1: Cobbles (Schist) and Gravel. R1 Core Times (minsec) 23.8-24.8' (1200) 24.8-25.8' (1248) 25.8-26.3' (1145) Cobbles and Gravel.					
25	R2	60/60	29.80 - 34.80	ROD = 87%	
0150 blows for 0.4'. Top of Bedrock at Elev. 2.0'. Roller Cone ahead from 29.4-29.8' bgs. R2: Bedrock Grey, fine-grained, metasedimentary, very closely spaced, frequent quartz veins, fucosolar formation. R2: Core Times (minsec) 29.8-30.8' (1420) 30.8-31.8' (1433) 31.8-32.8' (1446) 32.8-33.8' (1527) 33.8-34.8' (1530) 100% Recovery					
30					
Bottom of Exploration at 34.80 feet below ground surface.					

STATE OF MAINE DEPARTMENT OF TRANSPORTATION 15090.00 PIN 15090.00 BRIDGE NO. 2711 BRIDGE PLANS	PROJECT MANAGER: L. KRUSINSKI CHECKED/REVIEWED: T. WHITE DESIGNED/Detailed: L. KRUSINSKI DESIGNED/Checked: T. WHITE DATE: JUNE 2008	SIGNATURE: _____ P.E. NUMBER: _____ DATE: _____	
	BANGOR PENJEA WOCK STREAM PENOBSCOT COUNTY BORING LOGS	REVISIONS 1: _____ REVISIONS 2: _____ REVISIONS 3: _____ REVISIONS 4: _____ FIELD CHANGES: _____	SHEET NUMBER 3 OF 3

Appendix A

Boring Logs

Driller: MaineDOT	Elevation (ft.): 29.5	Auger ID/OD: 5" Solid Stem
Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: L. Krusinski/C. Beebe	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/11/08; 09:00-13:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+54, 10.9 Rt.	Casing ID/OD: HW	Water Level*: None Observed

Hammer Efficiency Factor: 0.77 Hammer Type: Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value
MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency
MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected
C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA				
1D	24/17	1.00 - 3.00	35/25/19/19	44	56					Light to dark brown, dry, very dense, fine to coarse angular to subrounded gravelly fine SAND, some medium to coarse sand, little silt.	
5	2D	24/12	5.00 - 7.00	5/6/7/11	13	17	25	24.50		Brown, dry, medium dense, sandy fine to coarse GRAVEL, subrounded to subangular, trace silt, (Fill).	G#210015 A-1-a, GW-GM WC=3.2%
10	3D	24/14	10.00 - 12.00	7/5/2/2	7	9	37	19.50		Brown, mottled, damp to moist, stiff, SILT, some sand, little clay, trace fine gravel. (Weathered Presumpscot Formation)	G#210018 A-4, CL-ML WC=22.0%
15	4D/AB	24/18	15.00 - 17.00	WOH/2/3/5	5	6	19	13.70		(4D/A) 15.0-15.8' bgs. Grey, wet, medium stiff, SILT, some clay, little sand, trace gravel, blocky. (Presumpscot Formation). q _p = 1.5 ksf q _p = 1.5 ksf	G#210016 A-4, CL WC=26.4% Non-Plastic
20	5D	24/8	20.00 - 22.00	5/4/5/5	9	12	33	15.80		(4D/B) 15.8-17.0' bgs. Brown to grey with stained/oxidized lenses, wet, loose, well graded SAND, some silt, little fine gravel. (Till).	
25	R1	60/60	24.40 - 29.40	RQD = 94%			483	5.10		Brown, wet, medium dense, SAND, some fine gravel, little silt, trace wood fragments.	G#210017 A-2-4, SM WC=15.9%
							483			a83 blows for 0.4'. Roller Coned ahead to 24.4' bgs., wood fragments on roller cone.	

Remarks:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Red Bridge #2711 over Meadow Brook on Route 2 (State Street) Location: Bangor, Maine	Boring No.: BB-BPS-101 PIN: 15090.00
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Driller: MaineDOT	Elevation (ft.): 29.5	Auger ID/OD: 5" Solid Stem
Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: L. Krusinski/C. Beebe	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/11/08; 09:00-13:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+54, 10.9 Rt.	Casing ID/OD: HW	Water Level*: None Observed
Hammer Efficiency Factor: 0.77	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf)
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (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										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
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.)				
25							RC NQ-2 CORE			24.40	Top of Bedrock at Elev. 5.1'. R1:Bedrock: Green-grey, fine grained, calcareous metasedimentary (GREENSCHIST), hard, fresh, bedding/foliation very close, irregular to steep angles, second joint set perpendicular to foliation, one open silt seam in fractured portion 2 ft into core run, vuggy portions, frequent quartz veins. R1:Core Times (min:sec) 24.4-25.4' (7:35) 25.4-26.4' (3:15) 26.4-27.4' (2:20) 27.4-28.4' (2:50) 28.4-29.4' (3:10) 100% Recovery R2:Bedrock: Grey, fine grained, calcareous metasedimentary (GREENSCHIST), hard, fresh to slightly weathered, foliation close, at irregular dip, but predominately moderate dip, highly fractured zone 3.4-5.0 ft into core run, surfaces with silt infilling, staining. R2:Core Times (min:sec) 29.4-30.4' (3:10) 30.4-31.4' (3:00) 31.4-32.4' (3:00) 32.4-33.4' (4:00) 33.4-34.4' (2:35) 100% Recovery	
30	R2	60/60	29.40 - 34.40	RQD = 66%				-4.90		34.40	Bottom of Exploration at 34.40 feet below ground surface.	
35												
40												
45												
50												

Remarks:

Driller: MaineDOT	Elevation (ft.): 30.5	Auger ID/OD: 5" Solid Stem
Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: L. Krusinski	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/11/08-6/12/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+15.6, 10.6 Lt.	Casing ID/OD: HW & NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.77 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value LL = Liquid Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PL = Plastic Limit
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA					
	1D	24/16	1.00 - 3.00	14/18/6/9	24	31					Grey to brown, dry to damp, dense, silty fine to coarse SAND, some rounded to subangular coarse gravel, little fine gravel, (Fill).	
5												
	2D	24/8	5.00 - 7.00	3/9/4/5	13	17	58				Brown, damp, medium dense, silty, fine to medium SAND, little fine rounded gravel, trace coarse sand, similar to 1D, (Fill).	
							33					
							32					
							80					
							167					
10												
	3D	24/8	10.00 - 12.00	18/17/8/6	25	32	184				184 blows for 7". Black, moist, dense, asphalt pavement fragments, some coarse sand and fine gravel. (Fill). Changed to NW Casing.	
							26					
							25					
							25					
							29					
15												
	4D	24/10	15.00 - 17.00	7/13/10/10	23	30	15				Brown-grey, moist, medium dense, fine to medium SAND, some rounded fine to coarse gravel, little silt.	G#210019 A-2-4, SM WC=12.4%
							41					
							45					
							119					
							211					
20												
	R1	16.8/9.6	20.00 - 21.40	RQD = 24%					10.50		Top of Bedrock at Elev. 10.5'. R1: Bedrock: Grey, fine grained, calcareous metasedimentary (GREENSCHIST), hard, slightly weathered, slightly fractured. Core blocked at open silt seam at 0.8 ft. R1: Core Times (min:sec) 20.0-21.0' (2:31) 21.0-21.4' (1:35) 57% Recovery Core Blocked	
	R2	60/51.6	21.40 - 26.40	RQD = 86%							R2: Bedrock: Grey, fine grained, calcareous metasedimentary (GREENSCHIST) hard, slightly weathered to fresh, bedding very close, dipping at steep angles, tight, occasional quartz and or pegmatite veins,	
25												

Remarks:

Driller: MaineDOT	Elevation (ft.): 30.5	Auger ID/OD: 5" Solid Stem
Operator: E. Giguere/C. Giles	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: L. Krusinski	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 6/11/08-6/12/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+15.6, 10.6 Lt.	Casing ID/OD: HW & NW	Water Level*: None Observed

Hammer Efficiency Factor: 0.77 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_u(lab) = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
25												
	R3	48/48	26.40 - 30.40	RQD = 95%							drill breaks along quartz veins, one silt seam at 1.3' into core run. R2: Core Times (min:sec) 21.4-22.4' (2:40) 22.4-23.4' (4:16) 23.4-24.4' (4:20) 24.4-25.4' (4:20) 25.4-26.4' (3:12) 86% Recovery R3: Bedrock: Grey-green, fine grained, metasedimentary (GREENSCHIST to GNEISS), hard, fresh to slightly weathered, very little bedding visible, at steep angles. R3: Core Times (min:sec) 26.4-27.4' (3:40) 27.4-28.4' (4:10) 28.4-29.4' (3:30) 29.4-30.4' (4:00) 100% Recovery	
30								0.10			Bottom of Exploration at 30.40 feet below ground surface.	
35												
40												
45												
50												

Remarks:

Driller: Northern Test Boring, Inc.	Elevation (ft.): 31.4	Auger ID/OD: 5" Solid Stem
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 1/30/09; 07:00-14:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+66.1, 11.1 Lt.	Casing ID/OD: HW	Water Level*: None Observed

Hammer Efficiency Factor: 0.623		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt	R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR/C = weight of rods or casing WO1P = Weight of one person	S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected	S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0									31.00		PAVEMENT. Frost Depth 2.2' bgs. —0.40	
	1D	14.4/12	2.50 - 3.70	23/24/50(2.4")	---				27.70		Brown, dry to damp, very dense, gravelly fine to coarse SAND, trace silt (Fill) —3.70	
									26.90		Concrete from 3.7-4.5' bgs. —4.50	
5	2D	24/18	5.00 - 7.00	9/14/14/9	28	29					Brown, damp, medium dense, fine to coarse SAND, some fine to coarse gravel, little silt, (Fill).	
											Similar to above.	
10	3D	24/16	10.00 - 12.00	6/8/6/5	14	15						G#212248 A-1-b, SM WC=5.8%
15	4D	24/20	15.00 - 17.00	12/8/9/10	17	18	59		15.40		Brown grey, damp, medium dense, fine SAND, (Alluvium). —16.00	
20	5D	24/18	19.50 - 21.50	9/9/9/12	18	19	99				Grey-brown, wet, medium dense, fine SAND, little silt, trace gravel, (Alluvium). Roller Coned ahead to 23.0' bgs.	G#212249 A-2-4, SM WC=20.3%
	R1	30/4	23.80 - 26.30	RQD = N/A%			150 NQ-2		7.60		R1: Cobbles (Schist) and Gravel.	
							148					R1: Core Times (min:sec)

Remarks:
Auto Hammer #185

Driller: Northern Test Boring, Inc.	Elevation (ft.): 31.4	Auger ID/OD: 5" Solid Stem
Operator: Mike/Nick	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: Diedrick D-50 Track	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 1/30/09; 07:00-14:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 6+66.1, 11.1 Lt.	Casing ID/OD: HW	Water Level*: None Observed

Hammer Efficiency Factor: 0.623 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
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 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25							89			23.8-24.8' (2:00) 24.8-25.8' (0:48) 25.8-26.3' (1:45) Cobbles and Gravel. a150 blows for 0.4'. Top of Bedrock at Elev. 2.0'. Roller Coned ahead from 29.4-29.8' bgs.		
						88						
						86						
						123						
30	R2	60/60	29.80 - 34.80	RQD = 87%		a150 NQ-2						
									R2: Bedrock: Grey, fine-grained, metasedimentary, very hard, fresh to slightly weathered, chaotic bedding, very closely spaced, frequent quartz veins. (Vassalboro Formation). R2: Core Times (min:sec) 29.8-30.8' (4:20) 30.8-31.8' (4:33) 31.8-32.8' (4:46) 32.8-33.8' (5:27) 33.8-34.8' (5:30) 100% Recovery Bottom of Exploration at 34.80 feet below ground surface.			
50												

Remarks:
Auto Hammer #185

* 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.

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																							
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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p>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.</p> <p style="text-align: center;">Modified Burmister System</p> <table border="0"> <tr> <td style="text-align: center;"><u>Descriptive Term</u></td> <td style="text-align: center;"><u>Portion of Total</u></td> </tr> <tr> <td>trace</td> <td>0% - 10%</td> </tr> <tr> <td>little</td> <td>11% - 20%</td> </tr> <tr> <td>some</td> <td>21% - 35%</td> </tr> <tr> <td>adjective (e.g. sandy, clayey)</td> <td>36% - 50%</td> </tr> </table> <table border="0"> <tr> <td style="text-align: center;"><u>Density of Cohesionless Soils</u></td> <td style="text-align: center;"><u>Standard Penetration Resistance</u> <u>N-Value (blows per foot)</u></td> </tr> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>> 50</td> </tr> </table>	<u>Descriptive Term</u>	<u>Portion of Total</u>	trace	0% - 10%	little	11% - 20%	some	21% - 35%	adjective (e.g. sandy, clayey)	36% - 50%	<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance</u> <u>N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50
		<u>Descriptive Term</u>	<u>Portion of Total</u>																								
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Medium Dense	11 - 30																										
Dense	31 - 50																										
Very Dense	> 50																										
(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.																									
<p>Desired Soil Observations: (in this order)</p> <p>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</p>				<p>Rock Quality Designation (RQD):</p> <p>RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core)</p> <p style="text-align: center;">Correlation of RQD to Rock Mass Quality</p> <table border="0"> <tr> <td style="text-align: center;"><u>Rock Mass Quality</u></td> <td style="text-align: center;"><u>RQD</u></td> </tr> <tr> <td>Very Poor</td> <td><25%</td> </tr> <tr> <td>Poor</td> <td>26% - 50%</td> </tr> <tr> <td>Fair</td> <td>51% - 75%</td> </tr> <tr> <td>Good</td> <td>76% - 90%</td> </tr> <tr> <td>Excellent</td> <td>91% - 100%</td> </tr> </table> <p>Desired Rock Observations: (in this order)</p> <p>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</p>		<u>Rock Mass Quality</u>	<u>RQD</u>	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%										
<u>Rock Mass Quality</u>	<u>RQD</u>																										
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<p>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</p>				<p>Sample Container Labeling Requirements:</p> <table border="0"> <tr> <td>PIN</td> <td>Blow Counts</td> </tr> <tr> <td>Bridge Name / Town</td> <td>Sample Recovery</td> </tr> <tr> <td>Boring Number</td> <td>Date</td> </tr> <tr> <td>Sample Number</td> <td>Personnel Initials</td> </tr> <tr> <td>Sample Depth</td> <td></td> </tr> </table>		PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth													
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Appendix B

Laboratory Data

State of Maine - Department of Transportation
Laboratory Testing Summary Sheet

Town(s): **Bangor**

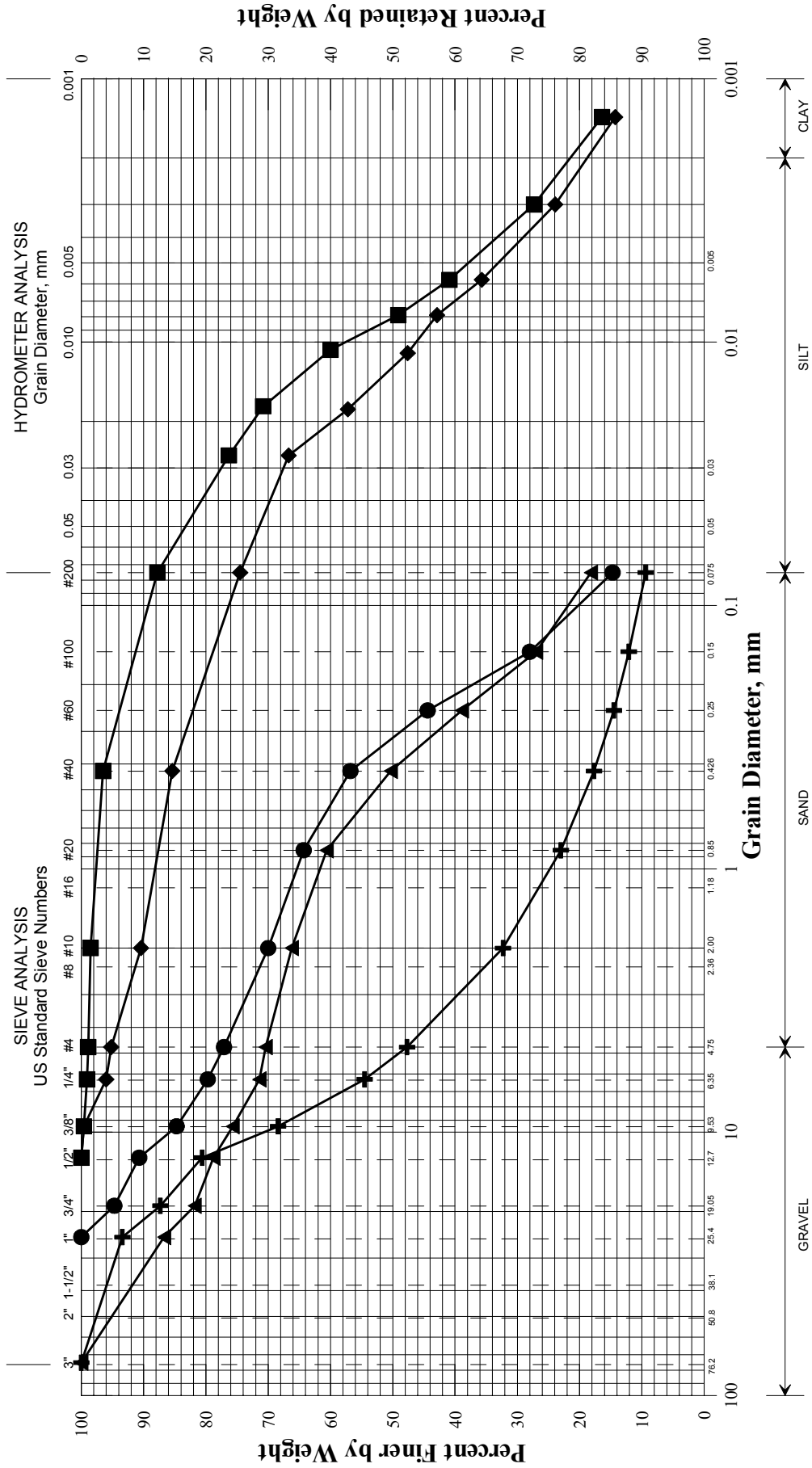
Project Number: **15090.00**

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-BPS-101, 2D	6+54	10.9 Rt.	5.0-7.0	210015	1	3.2			GW-GM	A-1-a	0
BB-BPS-101, 3D	6+54	10.9 Rt.	10.0-12.0	210018	1	22.0			CL-ML	A-4	IV
BB-BPS-101, 4D/A	6+54	10.9 Rt.	15.0-15.8	210016	1	26.4	-N	P-	CL	A-4	IV
BB-BPS-101, 5D	6+54	10.9 Rt.	20.0-22.0	210017	1	15.9			SM	A-2-4	II
BB-BPS-102, 4D	6+15.6	10.6 Lt.	15.0-17.0	210019	1	12.4			SM	A-2-4	II
BB-BPS-201, 3D	6+66.1	11.1 Lt.	10.0-12.0	212248	2	5.8			SM	A-1-b	II
BB-BPS-201, 5D	6+66.1	11.1 Lt.	19.5-21.5	212249	2	20.3			SM	A-2-4	II

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 MDOT 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
- PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

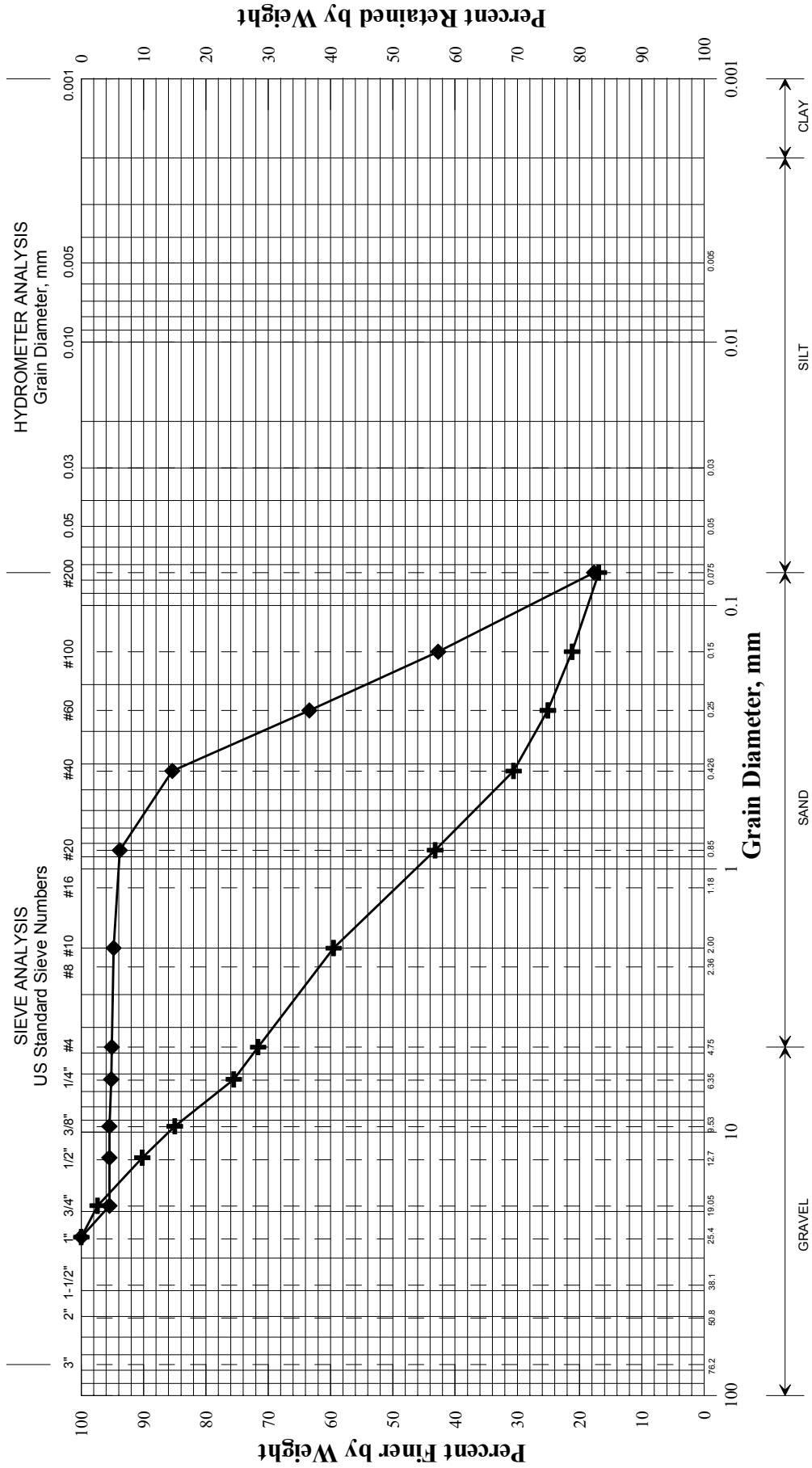


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	6+54	10.9 RT	5.0-7.0	Sandy GRAVEL, trace silt.	3.2			
◆	6+54	10.9 RT	10.0-12.0	SILT, some sand, little clay, trace gravel.	22.0			
■	6+54	10.9 RT	15.0-15.8	SILT, some clay, little sand, trace gravel.	26.4			NP
●	6+54	10.9 RT	20.0-22.0	SAND, some gravel, little silt.	15.9			
▲	6+15.6	10.6 LT	15.0-17.0	SAND, some gravel, little silt.	12.4			
×								

PIN	015090.00
Town	Bangor
Reported by/Date	WHITE, TERRY A 7/28/2008

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-BPS-2017/3D	6+66.1	11.1 LT	10.0-12.0	SAND, some gravel, little silt.	5.8			
BB-BPS-2017/5D	6+66.1	11.1 LT	19.5-21.5	SAND, little silt, trace gravel.	20.3			

PIN	015090.00
Town	Bangor
Reported by/Date	WHITE, TERRY A 2/6/2009

Appendix C

Calculations

Bearing Resistance - Abutment 1 and 2 Spread Footing Foundations

Method 1

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

Description of Bearing Material:

Metasedimentary GREENSCHIST, hard, fresh, except for occasional highly fractured zones. RQD in general ranges from 24 to 95 percent, (core run of 24% due to core blocking.)

Use averaged RQD of 60% for design.

Bearing Material:	Weathered or broken bedrock of any kind except argillite (shale).
Consistency in Place:	Medium hard rock
Allowable Bearing Pressure	Range: 16 - 24 ksf
<u>Recommended Value</u>	20 ksf

Use a factored bearing resistance of 20 ksf for service limit state analysis - and for preliminary sizing of the footing.

Method 2

Method: *AASHTO Standard Specifications - 17th Edition, 2002*

Section 4.4.8.1.1 - Competent Rock

Figure 4.4.8.1.1.A - for footings supported on competent rock.

Lowest RQD of rock is 25%

Allowable contact stress 30 tsf (60 ksf)

Method 3

AASHTO Standard Specifications - 17th Edition, 2002

Section 4.4.8.1.2. Footings on Broken or Jointed Rock

Table 4.4.8.1.2.A - for footings supported on jointed rock.

- | | |
|--|---|
| a. estimated RMR, Rock Mass Rating, | Fair to Excellent. RQD Range is 24-95%. Use average of 60 percent |
| b. Rock Category per 4.4.8.1.2B | E, Schist |
| c. Unconfined compressive strength, Co estimated (range of 1,400 - 21,000 psi) | 5,000 psi |

d. N_{ms} , per Table 4.4.8.1.2A

Table states to use $N_{ms}=.081$

e. Q_{ult}

$N_{ms} \times C_o$

Nominal Bearing Resistance

$$Q_{nom} := 0.081 \cdot 5000 \cdot \text{psi} \quad Q_{nom} = 58.32 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi := 0.45$$

$$Q_{factored} := Q_{nom} \cdot \phi$$

$$Q_{factored} = 26.244 \cdot \text{ksf}$$

Recommend a factored bearing resistance 26 ksf for the Strength Limit State Analysis.

Frost Protection

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

From Design Freezing Index Map: **Bangor**
DFI = 1726 degree-days

Case I - Soils at elevation of possible footings of are sand and gravel, assume WC=20%

Interpolate between frost depth of 72.4 inches at 1700 DFI and 74.5 inches at 1800 DFI

Result:

Depth of Frost Penetration =

$$d := \frac{74.5 - 72.4}{100} \cdot 26 \cdot \text{in} + 72.4 \cdot \text{in}$$

$$d = 6.079 \cdot \text{ft}$$

Recommend an embedment depth of 6 feet for foundations constructed on compacted fill soils

Active, Passive, At Rest Earth Pressures

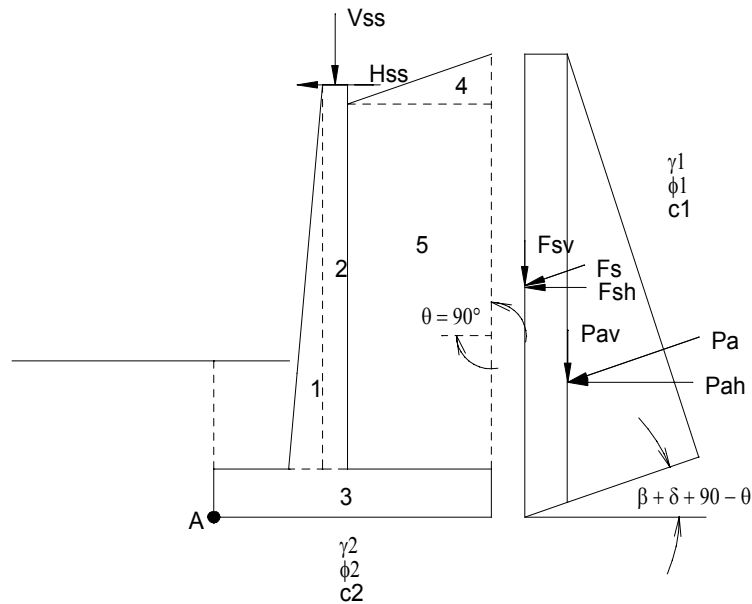
Backfill engineering strength parameters

Soil Type 4 Properties from Bridge Design Guide (BDG)

Unit weight $\gamma_1 := 125 \cdot \text{pcf}$

Internal friction angle $\phi_1 := 32 \cdot \text{deg}$

Cohesion $c_1 := 0 \cdot \text{psf}$



Active Earth Pressure - Rankine Theory

Either Rankine or Coulomb may be used for **long heeled** cantilever walls, where the failure surface is uninterrupted by the top of the wall stem. In general, use Rankine though. The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

- For cantilever walls with horizontal backslope

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi_1}{2}\right)^2 \quad K_a = 0.307$$

- For a sloped backfill

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$

$$K_{\text{aslope}} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \quad K_{\text{aslope}} = 0.307$$

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

Coulomb Theory

In general, for cases where the back face of the wall interferes with the development of a full sliding surface in the backfill, as assumed by Rankine Theory, use Coulomb.

- Coulomb theory applies for gravity, semigravity and prefab modular walls with steep back faces
- Coulomb theory also applies to concrete cantilever walls with short heels where the sliding surface is restricted by the top of wall - the wedge of soil does not move.
- Interface friction is considered in Coulomb.

Angle of back face of wall to the horizontal, θ :

$$\theta := 90 \cdot \text{deg}$$

Friction angle between fill and wall, δ :

Per LRFD Table 3.11.5.3-1, for "Clean sand, silty sand-gravel mixture, single-size hard rock fill against Formed or precast concrete" $\delta = 17$ to 22 degrees; select 20 degrees.

$$\delta := 20 \cdot \text{deg} \quad \text{for a gravity shaped wall where the interface friction is between soil and concrete}$$

to $\delta := 24 \cdot \text{deg}$ per BDG Table 3-3

Per LRFD Figure C3.11.5.3-1, for a cantilever wall where the sliding surface is a plane from the footing heel to the top of the wall, $\delta = 1/3$ to $2/3 \Phi$

$$\delta := \frac{2}{3} \cdot \phi_1$$

$$\delta = 21.333 \cdot \text{deg}$$

(If δ is taken as 0 and the slope of the backslope is horizontal, there is no difference in the active earth pressure coefficient when using either Rankine or Coulomb)

$$K_{\text{ac}} := \frac{\sin(\theta + \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta - \delta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}} \right)^2} \quad K_{\text{ac}} = 0.275$$

Orientation of Coulomb Pa

- In the case of gravity shaped walls and prefab walls, Pa is oriented δ degrees up from a perpendicular line to the backface.
- In the case of short heeled cantilever walls where the top of the wall interferes with the failure surface, Pa is oriented at an angle of $\phi/3$ to $2/3 \cdot \phi$ to the normal of a vertical line extending up from the heel of the wall

Passive Earth Pressure - Rankine Theory

Bowles does not recommend use of Rankine method for Kp when B>0.

β = Angle of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$

$$K_{pslope} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}$$

$$K_{pslope} = 3.255$$

Pp is oriented at an angle of β to the vertical plane

Passive Earth Pressure - Coulomb Theory

Interface friction is considered in Coulomb.

For a smooth vertical wall with horizontal backfill $\delta = \beta = 0$ and $\theta = 90$ degrees (refer: Bowles, 5th edition, pag 596)

θ = Angle of back face of wall to the horizontal

$\theta := 90 \cdot \text{deg}$

δ = friction angle between fill and wall taken as specified in LRFD Table 3.11.5.3-1 (degrees)

$$\delta := \frac{2}{3} \cdot \phi_1 \quad \delta = 0.372 \quad \delta := 0$$

$$K_{pc} := \frac{\sin(\theta - \phi_1)^2}{\sin(\theta)^2 \cdot \sin(\theta + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 + \beta)}{\sin(\theta + \delta) \cdot \sin(\theta + \beta)}}\right)^2} \quad K_{pc} = 3.255$$

At-rest Earth Pressure - Rankine Theory

Das, 2nd Edition, Principles of Foundation Engineering, pg 252, for normally consolidated granular soil

$$K_0 := 1 - \sin(\phi_1)$$

$$K_0 = 0.47$$