MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CUMBERLAND MILLS WEST BRIDGE
AND
CUMBERLAND MILLS EAST BRIDGE
OVER PRESUMPSCOT RIVER
WESTBROOK, MAINE

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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 the Cumberland Mills West and East Bridges over the Presumpscot River in Westbrook, Maine. In the area between the two bridges, there is a manmade island which extends downstream from the roadway. This island separates the river into east and west channels. The proposed alignment for the proposed bridges will be similar to the existing. The proposed bridge superstructures will be widened from the existing approximately 36 feet 8 inches to a new width of approximately 41 feet 4 inches. The following design recommendations are discussed in detail in the attached report:

General - Spread Footings on Bedrock- The proposed abutments and wingwalls will be founded on spread footings directly on bedrock or on seals constructed on bedrock. The abutment borings indicate that bedrock with Rock Quality Designations (RQD) corresponding to rock that is very highly fractured to slightly fractured, will be encountered at the bedrock surface, therefore, the bedrock surface shall be cleared of all loose bedrock and loose, decomposed bedrock.

The approximate bottom of footing or bottom of seal elevations are estimated to be:

Bridge	Substructure	Estimated Bottom of Footing or Seal Elevation
West	Abutment No. 1	36 to 38 feet
West	Abutment No. 2	33 to 35 feet
East	Abutment No. 3	30 to 32 feet
East	Abutment No. 4	33 to 35 feet

Abutment and Wingwall Design - Abutments and wingwalls shall be proportioned for all applicable load combinations and shall be designed for all relevant strength, extreme and service limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), failure by sliding and reinforced concrete structural failure.

Extreme limit state design checks for abutments shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load combinations relating to certain hydraulic events and ice (if warranted by ice history or stream constriction by the abutments). Where the footings or seals will be constructed directly on bedrock cleaned of all weathered, loose or potentially erodible rock, strength and extreme limit states do not need to consider foundation resistance after the design or check floods for scour. 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.

For footings or concrete seals on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions, in either direction.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock. Sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

Anchorage of footings to seals or of seals to bedrock may be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Cantilever-type abutments should be designed for active earth pressure over the abutment height. Earth loads for wingwalls shall also be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}).

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

Bearing Resistance –A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. The bearing resistance for abutment and wingwall footings founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf for footing widths of 10 feet or less and 17 ksf for footings widths of 12 feet or greater.

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

Cast-in-Place Retaining Wall - A cast-in-place cantilever retaining wall founded on bedrock will be constructed on the south side of the existing center island to allow for the roadway widening. The wall will be constructed in front of the existing wall with the area between the two walls backfilled. The existing wall will be removed to a specified elevation with the remained left in place and buried. A 4H:1V slope will be constructed behind the guardrail

down to the top of the new retaining wall and will allow the placement of a utility pole in this area. The wall should be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface. The wall will be backfilled with granular borrow for underwater backfill.

Bearing resistance for the wall founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. A factored bearing resistance of 20 ksf may be used when analyzing the service limit state assuming a resistance factor of 1.0.

Scour and Riprap - The cantilever-type abutments, wingwalls and retaining wall will be supported on spread footings founded directly on bedrock or on seals constructed on bedrock. Bedrock surfaces that are cleaned of all weathered, loose and potentially erodible rock are not considered to be scour susceptible. Grain size analyses were performed on granular soil samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses.

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states for any foundation bearing on native or till soils. For scour protection, any footings for wingwalls or retaining walls which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of abutment footings and wingwalls.

Settlement - The existing approach embankments at both bridge approaches will be widened to accommodate the widened roadway. Placement of the necessary fill will result in negligible densification of the underlying soils and minimal settlement of the embankments. Any settlement will occur during and immediately after construction of the widened embankments. Post-construction settlement will be minimal.

Any settlement of bridge abutments will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

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

Seismic Design Considerations - Seismic analysis is not required for single-span bridges regardless of seismic zone. The Cumberland Mills West and East Bridges are not on the National Highway System. The bridges are not classified as major structures. These criteria eliminate the MaineDOT requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD.

Construction Considerations - Construction activities will include construction of cofferdams and earth support systems to support the approach fills and control stream flow during construction of seals and footings for abutments, wingwalls and retaining walls. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for major structures.

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

The contractor should maintain the abutment, wingwall and retaining wall excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete. The final bedrock surface shall be approved by the Resident prior to placement of the footing concrete.

1.0 INTRODUCTION

The purpose this Geotechnical Design Report is to present geotechnical recommendations for replacement of the Cumberland Mills West and East Bridges which carry Cumberland Avenue over the Presumpscot River in Westbrook, Maine. This report presents the soils information obtained at the sites during the subsurface investigation, foundation recommendations and geotechnical design parameters for bridge replacement.

The two bridges span the Presumpscot River with one abutment each on a common, manmade island which separates the river into two channels. The West Bridge was built in 1954 and is an approximately 85 foot long, single span structure founded on concrete capped granite block abutments. The abutments are likely founded on a combination of native soils and bedrock. The East Bridge was also built in 1954 and is an approximately 103 foot long, single span structure founded on concrete capped granite block abutments. The abutments are likely founded on a combination of native soils and bedrock. The existing bridges both have a bridge deck width of approximately 35 feet.

Year 2009 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports assign the West Bridge substructures a condition rating of 5 – fair with a Bridge Sufficiency Rating of 54.2 and the East Bridge substructures a condition rating of 4 – poor with a Bridge Sufficiency Rating of 30.8. The West Bridge is considered to be in fair condition except the deck which is poor condition and the East Bridge is considered to be in overall poor condition.

The proposed bridge superstructures will be single span structures having similar span lengths to the existing bridges. The new bridge structures will maintain the existing alignment with some widening to the downstream side. The bridges will be supported by full height cast-in-place abutments founded spread footings on bedrock or concrete seals, if needed. The presence of shallow bedrock precludes the use of H-pile supported integral abutments. A cast-in-place cantilever retaining wall founded on bedrock will be constructed on the south side of the existing center island to allow for the roadway widening.

2.0 GEOLOGIC SETTING

The Cumberland Mills West and East Bridges in Westbrook, Maine cross the Presumpscot River as shown on Sheet 1 - Location Map presented at the end of this report. The Presumpscot River flows into Casco Bay at Martin's Point in Portland.

According to the Surficial Geologic Map, Portland West Quadrangle, Maine published by the Maine Geological Survey (1997) the surficial soils in the vicinity of the site consist of Presumpscot Formation deposits. The predominant matrix of the deposit is silt, clay, and minor sand deposits. These soils were generally deposited on the sea floor during the late-glacial marine submergence.

According to the Bedrock Geologic Map of Maine (1985) the bedrock in the vicinity of the site is identified as calcareous sandstone of the Vassalboro Formation. The Norumbega fault zone lies just to the south of the site.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling four (4) test borings, two (2) at each bridge location. Test borings BB-WCMW-101 and BB-WCMW-102 were drilled behind the location of the existing West Bridge abutments. Test borings BB-WCME-101 and BB-WCME-102 were drilled behind the location of the existing East Bridge abutments. These boring locations are shown in Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The borings were drilled between June 22 and 24, 2004 using the MaineDOT drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and graphically on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The bedrock was cored in all of the borings using an NQ 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, identified field and laboratory testing requirements and logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

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

4.0 LABORATORY TESTING

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

Laboratory testing consisted of four (4) grain size analyses and four (4) natural moisture content determinations. The tests were performed in the MaineDOT Materials and Testing Laboratory in Bangor, Maine. The results of this laboratory testing are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information is also shown on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is show 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 at each bridge is as follows:

5.1 Cumberland Mills West Bridge

Test borings BB-WCMW-101 and BB-WCMW-102 were drilled behind the locations of the existing West Bridge abutments. Subsurface conditions encountered at West Bridge generally consisted of fill soils over lying a thin layer of native sandy silt all of which is underlain by bedrock.

Fill Soils. A layer of fill was encountered behind the existing abutments in both of the borings. The layer was approximately 15 feet thick in boring BB-WCMW-101 and approximately 14.2 feet thick in boring BB-WCMW-102. Obstructions were encountered in the fill soils which make up the center island. The fill soils generally consisted of brown and dark brown, dry to wet, fine to coarse sand, little silt, trace brick, slag, organics and rock fragments. SPT N-values in the fill layer ranged from 11 to 35 blows per foot (bpf) indicating that the layer is medium dense to dense in consistency.

Native Sandy Silt. A layer of sandy silt was encountered behind the western abutment in boring BB-WCMW-101. The sandy silt was not encountered in the eastern boring BB-WCMW-102. The layer was approximately 6.7 feet thick in boring BB-WCMW-101. This layer generally consisted of grey, wet, sandy silt, with trace gravel and organics. SPT N-values in the layer ranged from 2 to 16 bpf indicating that the layer is soft to very stiff in consistency. A grain size analysis was conducted on one sample from the silt layer. The moisture content of the sample was determined to be approximately 29%. The grain size analysis resulted in the soil being classified as an A-4 under the AASHTO Soil Classification System and as an ML under the Unified Soil Classification System. Laboratory test results can be found in Appendix B - Laboratory Data. This testing information is also shown on the boring logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

A layer of wood was encountered in boring BB-WCMW-101 at a depth of approximately 22.0 feet below ground surface (bgs). The layer observed in the spilt spoon was approximately 3 inches thick.

Bedrock. The bedrock was cored in both of the borings at the West Bridge. Bedrock was encountered at a depth of 21.7 feet bgs (El. 31.8 feet) in boring BB-WCMW-101 and at a depth of 14.2 feet bgs (El. 40.3 feet) in boring BB-WCMW-102. The bedrock was observed to be grey and white calcareous sandstone. The RQD of the bedrock at the West Bridge ranged from 28% to 60% indicating that the rock is of poor to fair quality. The top of the bedrock surface appears to slope downward from east to west.

Table 1 summarizes approximate depths to bedrock, corresponding top of bedrock elevations and RQD at the boring locations:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB-WCMW-101	21.7 feet	31.8 feet	60%
BB-WCMW-102	14.2 feet	40.3 feet	28%

Table 1 – Summary of Bedrock Depths, Elevations and RQD for West Bridge

Groundwater. Groundwater measurements were not taken during drilling activities. The depth to groundwater is inferred to be at a depth of approximately 10 feet bgs based on the moisture content of the soil samples. Groundwater levels will vary with seasonal precipitation and construction activity.

5.2 Cumberland Mills East Bridge

Test borings BB-WCME-101 and BB-WCME-102 were drilled behind the location of the existing East Bridge abutments. Subsurface conditions encountered at East Bridge generally consisted of fill soils over lying a thin layer of native sandy silt all of which is underlain by bedrock.

Fill Soils. A layer of fill was encountered behind the existing abutments in both of the borings. The layer was approximately 18.6 feet thick in boring BB-WCME-101 and approximately 10.8 feet thick in boring BB-WCME-102. The fill soils generally consisted of grey, brown and dark brown, damp to wet, fine to coarse sand and silty sand, with trace gravel, brick, slag, organics and rock fragments. SPT N-values in the fill layer ranged from 6 to 21 bpf indicating that the layer is loose to medium dense in consistency. Three grain size analyses were conducted on samples from the fill layer. The moisture content of the samples ranged from approximately 16% to 21%. The grain size analysis resulted in the soil being classified as an A-4 or A-2-4 under the AASHTO Soil Classification System and as an SM under the Unified Soil Classification System. Laboratory test results can be found in Appendix B - Laboratory Data. This testing information is also shown on the boring logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

Native Sandy Silt. A layer of sandy silt was encountered behind the western bridge abutment in the area of the manmade island in boring BB-WCME-101. The sandy silt was not encountered in the eastern boring BB-WCME-102 behind the eastern bridge abutment. The layer was approximately 3.9 feet thick in boring BB-WCME-101. This layer generally consisted of grey, wet, sandy silt with broken rock fragments. One SPT N-value in the layer was recorded as greater than 50 bpf indicating that the layer is hard in consistency.

Bedrock. The bedrock was cored in both of the borings at the East Bridge. Bedrock was encountered at a depth of 22.5 feet bgs (El. 32.0 feet) in boring BB-WCME-101 and at a depth of 10.8 feet bgs (El. 42.7 feet) in boring BB-WCME-102. The bedrock was observed to be grey and white calcareous sandstone. The RQD of the bedrock at the East Bridge ranged

from 25% to 64% indicating that the rock is of very poor to fair quality. The top of the bedrock surface appears to slope downward from east to west.

Table 2 summarizes approximate depths to bedrock, corresponding top of bedrock elevations and RQD at the boring locations:

Boring Number	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
BB-WCME-101	22.5 feet	32.0 feet	25%
BB-WCME-102	10.8 feet	42.7 feet	64%

Table 2 – Summary of Bedrock Depths, Elevations and RQD for East Bridge

Groundwater. Groundwater measurements were not taken during drilling activities. The depth to groundwater is inferred to be at a depth of approximately 10 feet bgs based on the moisture content of the soil samples. Groundwater levels will vary with seasonal precipitation and construction activity.

6.0 FOUNDATION ALTERNATIVES

Assessment of subsurface conditions at the site indicate the most effective foundation type for this site to be cantilever-type abutments and wingwalls on spread footings founded directly on bedrock or on seals constructed on bedrock. The use of a pile supported integral structure was ruled out by the design team during the Preliminary Design Report (PDR) phase.

A cast-in-place cantilever retaining wall founded on bedrock will be constructed on the south side of the existing center island to allow for the roadway widening.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following subsections will discuss the foundation considerations and recommendations for full height cast-in-place cantilever abutments founded on bedrock and a cast-in-place cantilever retaining wall founded on bedrock.

7.1 General - Spread Footings on Bedrock

Bedrock was encountered at depths ranging from approximately 11 to 23 feet below the existing roadway surface across the site. It is therefore considered feasible that spread footings or seals, if required, could be practically and economically constructed to bear on bedrock within moderately shallow excavations requiring cofferdams and temporary soil support systems.

The borings indicate that suitable bedrock with an average RQD of approximately 44 percent will be encountered at the bedrock surface; however, the bedrock surface shall be cleared of all loose bedrock and highly fractured bedrock. Based on borings conducted at the site and

top of bedrock elevations encountered in those borings, the estimated bottom of footing or bottom of seal elevations are estimated to be:

Bridge	Substructure	Estimated Bottom of Footing or Seal Elevation
West	Abutment No. 1	36 to 38 feet
West	Abutment No. 2	33 to 35 feet
East	Abutment No. 3	30 to 32 feet
East	Abutment No. 4	33 to 35 feet

Table 3 – Estimated Bottom of Footing or Seal Elevations for Abutments

7.2 Abutment and Wingwall Design

Abutments and wingwalls shall be proportioned for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and shall be designed for all relevant strength, extreme and service limit states. The design of project abutments and wingwalls founded on spread footings at the strength limit state shall consider bearing resistance, eccentricity (overturning), failure by sliding and reinforced concrete structural failure.

For scour protection of abutment and wingwall footings, construct the footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock. Strength and extreme limit state designs do not need to consider foundation resistance after the design or check floods for scour.

Extreme limit state design checks for abutments shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to extreme event load conditions relating to certain hydraulic events and ice (if warranted by ice history or stream constriction by the abutments). Resistance factors, φ, for the extreme limit state shall be taken as 1.0.

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 over all stability of foundations are typically investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. Shear failure along adversely oriented joint surfaces in the rock mass below the foundations is not anticipated, therefore; a global stability evaluation may be waved.

For footings or concrete seals on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed three-eights (3/8) of the footing dimensions in either direction. The eccentricity corresponds to the resultant of reaction forces falling within the middle three-fourths (3/4) of the footing.

For sliding analyses, a sliding resistance factor, φ_τ, of 0.90 shall be applied to the nominal sliding resistance of abutments and wingwalls founded on spread footings on bedrock.

Sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.70 at the bedrock-concrete interface.

Anchorage of footings to seals or of seals to bedrock may be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Cantilever-type abutments should be designed for active earth pressure over the abutment height. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, of 0.31 is recommended. Earth loads for wingwalls shall also be calculated using an active earth pressure coefficient, K_a, of 0.31, calculated using Rankine Theory.

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 MaineDOT BDG for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5. The live load surcharge on walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from the Table 2 below:

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

Table 4 - Equivalent Height of Soil for Estimating Live Load Surcharge

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

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

Slopes above the wingwalls should be constructed with riprap and not exceed 1.75H:1V.

7.3 Bearing Resistance

Substructure 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. The stress distribution may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2.

A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination. The bearing resistance for abutment and wingwall footings founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf for footing widths of 10 feet or less and 17 ksf for footings widths of 12 feet or greater. This assumes a bearing resistance factor, $\varphi_{b,}$ for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods.

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

7.4 Cast-in-Place Retaining Wall

A cast-in-place cantilever retaining wall founded on bedrock will be constructed on the south side of the existing center island to allow for the roadway widening. The wall will be constructed in front of the existing wall with the area between the two walls backfilled. The existing wall will be removed to a specified elevation with the remained left in place and buried. A 4H:1V slope will be constructed behind the guardrail down to the top of the new retaining wall and will allow the placement of a utility pole in this area.

The cast-in-place cantilever retaining wall will be founded on bedrock. The wall should be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface. The wall will be backfilled with granular borrow for underwater backfill.

Bearing resistance for the wall founded on bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 14 ksf. The bearing resistance factor, $\phi_{b,}$ for spread footings on rock is 0.45. A factored bearing resistance of 20 ksf may be used when analyzing the service limit state assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

For scour protection of the retaining wall footings, construct the footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock. Strength and extreme limit state designs do not need to consider foundation resistance after the design or check floods for scour.

Extreme limit state design checks for retaining walls shall include bearing resistance, eccentricity, failure by sliding and structural failure with respect to certain hydraulic events

and ice (if warranted by ice history or stream constriction). Resistance factors, φ, for the extreme limit state shall be taken as 1.0.

Any irregularities in the existing bedrock surface or irregularities created during the excavation process will be backfilled with un-reinforced Class S fill concrete to the bearing elevation.

7.5 Scour and Riprap

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

Grain size analyses were performed on granular soil samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.13 mm
- Average diameter of particle at 95 percent passing, D_{95} = 5.5 mm
- Soil Classification AASHTO Soil Type A-4 or A-2-4

The grain size curves are included in Appendix B- Laboratory Data found at the end of this report.

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

Bridge approach slopes, slopes at wingwalls and slopes at the toes of any footings on granular soils shall be armored with 3 feet of riprap. Riprap shall conform to Special Provisions 610 and 703. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class “1” Erosion Control Geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

7.6 Settlement

The existing approach embankments at both bridge approaches will be widened to accommodate the widened roadway. Placement of the necessary fill will result in negligible densification of the underlying soils and minimal settlement of the embankments. Any settlement will occur during and immediately after construction of the widened embankments. Post-construction settlement will be minimal.

Any settlement of bridge abutments will be due to elastic compression of the bedrock mass, and is estimated to be less than 0.5 inch.

7.7 Frost Protection

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

In the event that any foundation is placed on granular subgrade soils, it should be designed with an appropriate embedment for frost protection. According to the MaineDOT frost depth maps for the State of Maine (MaineDOT Bridge Design Guide [BDG] Figure 5-1); the site has a design-freezing index of approximately 1200 F-degree days. Using the laboratory determined water content for the site soils of approximately 20%, this correlates to a frost depth of 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on granular soils and not those founded on bedrock. See Appendix C - Calculations at the end of this report for supporting documentation.

7.8 Seismic Design Considerations

In conformance with LRFD Table 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, the Cumberland Mills West and East Bridges are not on the National Highway System (NHS). The bridges are not classified as major structures since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak ground acceleration coefficient (PGA) = 0.089g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.283g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.108g

- Site Class D (stiff soil with a average SPT blow count between 15 and 50 blows per foot for the upper 100 ft of the soil profile)
- Seismic Zone 1, based on a $S_{D1} < 0.15g$

See Appendix C – Calculations at the end of this report for supporting documentation.

7.9 Construction Considerations

Construction activities will include construction of cofferdams and earth support systems to support the approach fills and control stream flow during construction of seals and footings for abutments, wingwalls and the retaining wall. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for major structures.

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil. The final bearing surface shall be solid. The bedrock surface slope shall be less than 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 contractor should maintain the abutment, wingwall and retaining wall excavations so that the foundations can be constructed in the dry. The cleanliness and condition of the bedrock surface should be confirmed by the Resident prior to placing concrete.

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

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

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.

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

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

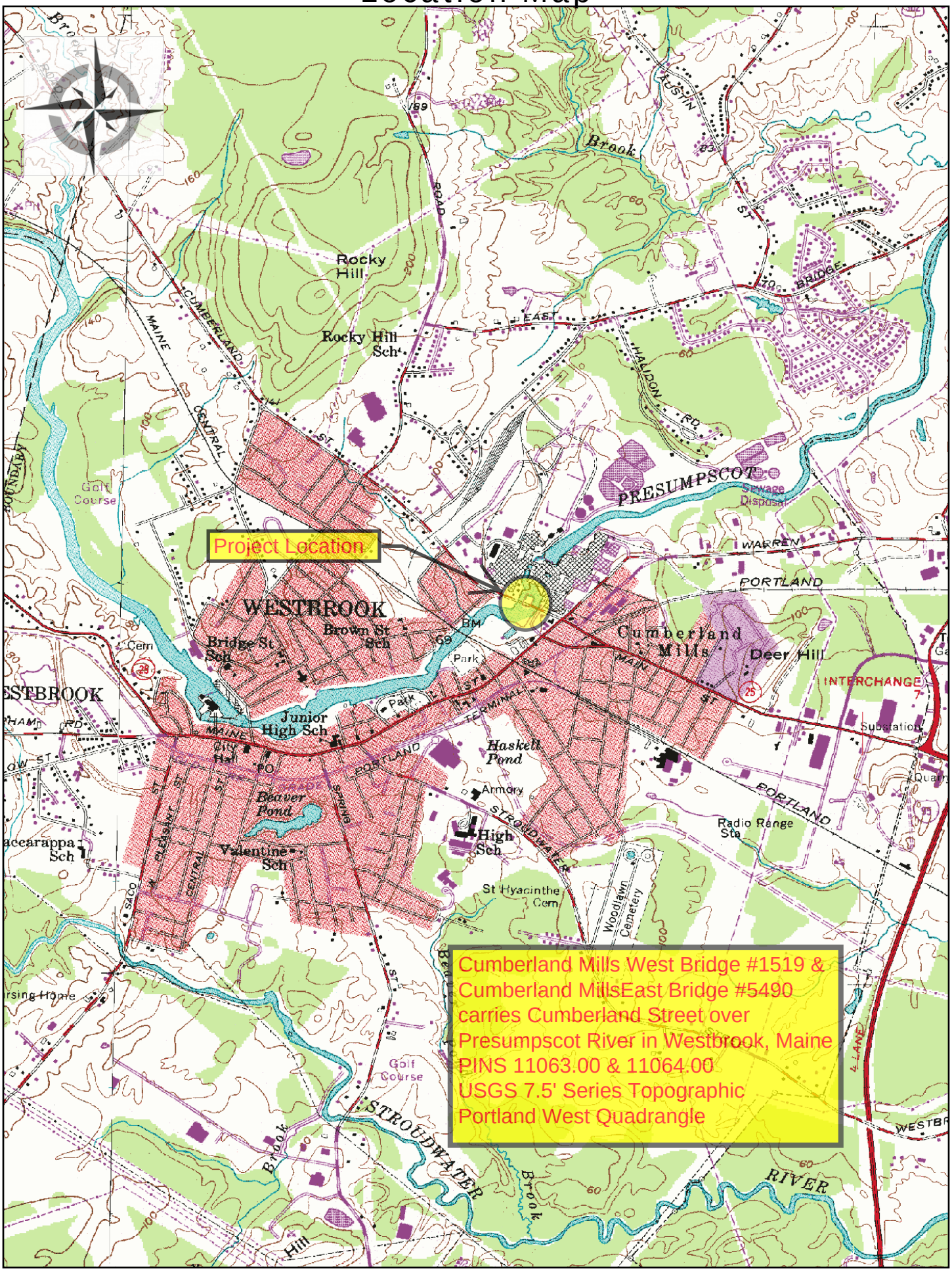
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Cumberland Mills West and East Bridges in Westbrook, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is 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.

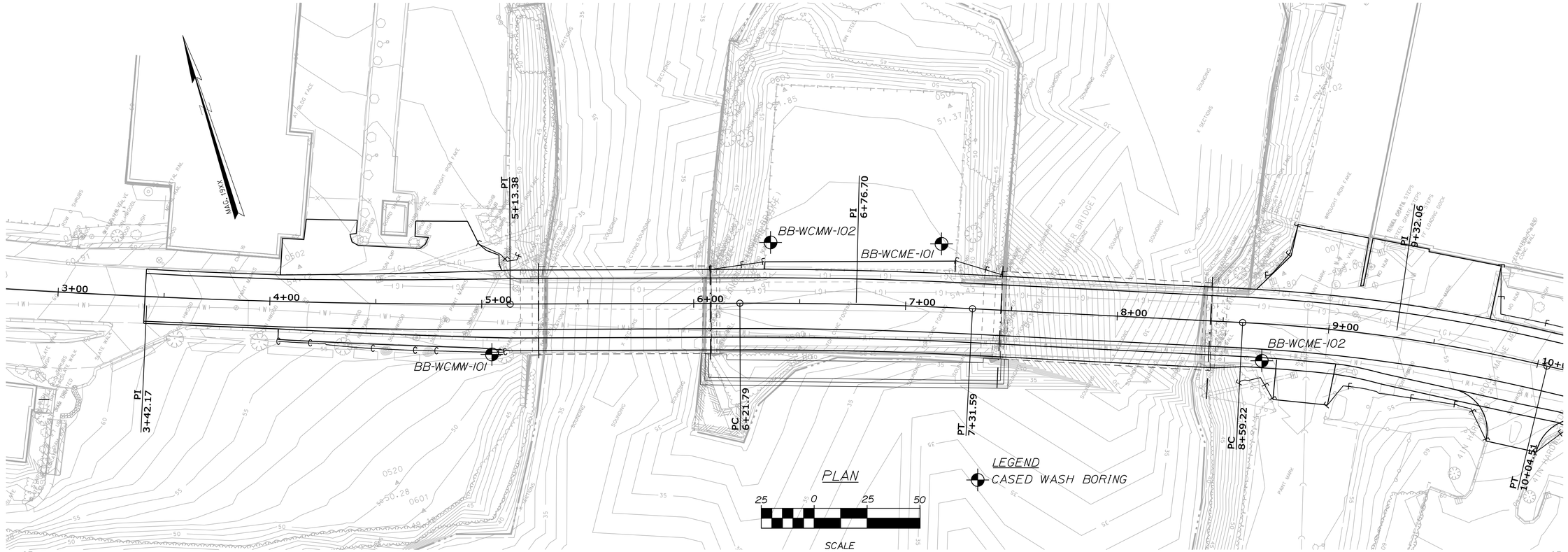
Sheets

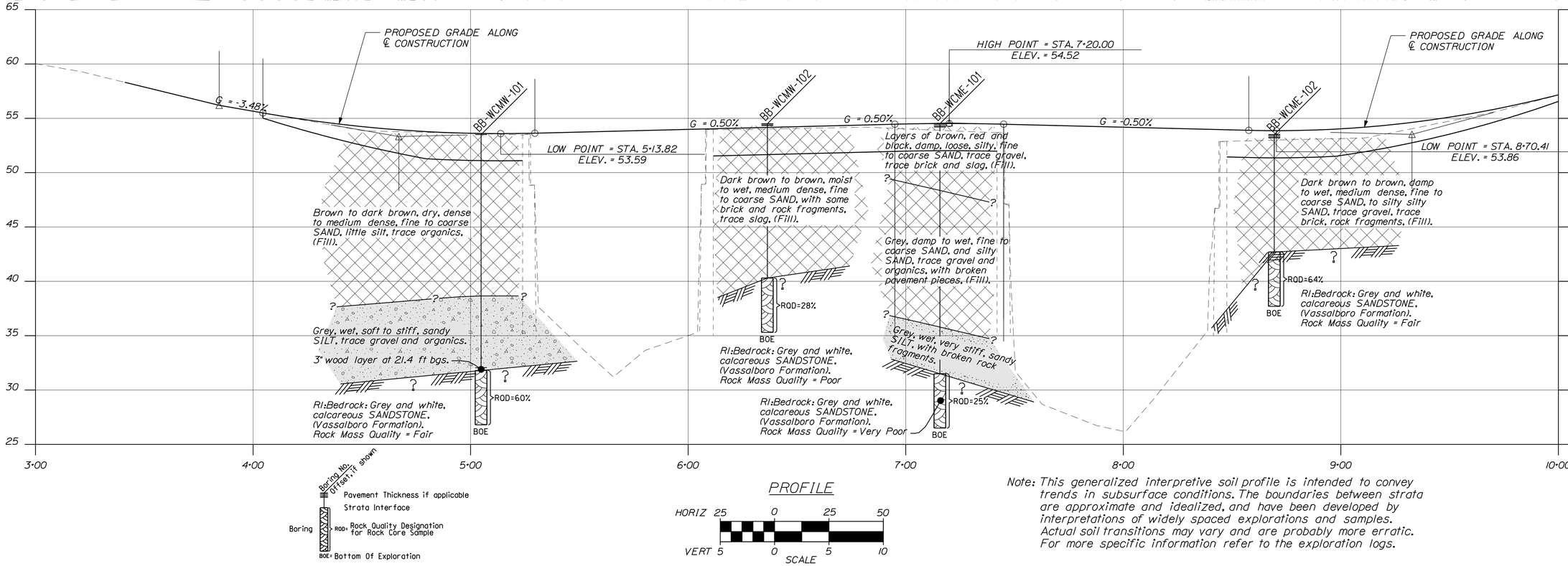
Location Map



Map Scale 1:24000

Filename: ...\geotech\mslo\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:1/27/2011





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

PIN
BRIDGE NO. 1519 & 5410 **11063.00 - 11064.00** BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	J Wentworth	
DESIGN-DETAILED	K. MAGUIRE	JAN 2011
CHECKED-REVIEWED	T. WHITE	-----
DESIGN2-DETAILED2	-----	-----
DESIGN3-DETAILED3	-----	-----
REVISIONS 1	-----	-----
REVISIONS 2	-----	-----
REVISIONS 3	-----	-----
REVISIONS 4	-----	-----
FIELD CHANGES	-----	-----

SIGNATURE

P.E. NUMBER

DATE

CUMBERLAND MILLS WEST & EAST BRIDGE
PRESUMPSCOT RIVER
WESTBROOK CUMBERLAND COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\mslo\007_BORING LOGS1.dgn Username: terry.white Date:1/27/2011 Division: GEOTECH

Maine Department of Transportation — Boring No.: BB-WCMW-101

Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Cumberland Mills West Bridge — Location: Westbrook, Maine — PIN: 11063.00

Driller:	MaineDOT	Elevation (ft.):	53.5	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoons
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/23/04-6/23/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	5+04.8, 23.7 Rt.	Casing ID/OD:	3.5/3.0"	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger; S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_u = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); MOH = weight of 140lb. hammer; WOR = weight of rods; WOC = weight of casing; WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#/6 in.) Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						554			Brown, dry, fine to coarse SAND, little silt.	
5	10	24/2	5.00 - 7.00	8/18/17/9	35	40			Dark brown, dry, dense, fine to coarse SAND, little silt, trace organics.	
10	MD	24/0	10.00 - 12.00	7/7/9/9	16	16			No recovery.	
15	20	24/20	15.00 - 17.00	4/1/1/1	2	15	38.50		Grey, wet, soft, sandy SILT, trace gravel and organics. — 15.00	GM76151 A-4i, MI WC=29.1%
20	30	24/8	20.00 - 22.00	4/4/12/50|.2|	16	22			Grey, wet, very stiff, sandy SILT, wood layer (3") in bottom of spoon, rock in nose of spoon.	
	R1	60/54	21.70 - 26.70	ROD = 60%		924 ND	31.80		924 blows for 0.5'. — 21.70 Top of Bedrock at Elev. 31.8 ft. Bedrock: Grey and white, calcareous SANDSTONE. (Vassalboro Formation). Rock Mass Quality = Fair. R1: Bedrock Core Times (min/sec) 21.7-22.7' (5|42) 22.7-23.7' (5|50) 23.7-24.7' (6|23) 24.7-25.7' (5|03) 25.7-26.7' (5|30) 90% Recovery	
25							26.80		Bottom of Exploration at 26.70 feet below ground surface. — 26.70	
30										
35										
40										
45										

Remarks:

Stratification lines represent approximate boundaries between soil typest transitions may be gradual. — Page 1 of 1

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made. — Boring No.: BB-WCMW-101

Maine Department of Transportation — Boring No.: BB-WCMW-102

Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Cumberland Mills West Bridge — Location: Westbrook, Maine — PIN: 11063.00

Driller:	MaineDOT	Elevation (ft.):	54.5	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoons
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/23/04-6/23/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	6+36.4, 28.6 Lt.	Casing ID/OD:	3.5/3.0"	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger; S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_u = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); MOH = weight of 140lb. hammer; WOR = weight of rods; WOC = weight of casing; WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#/6 in.) Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						554	54.25		3" Pavement — 0.17	
5	10	24/10	2.00 - 4.00	6/10/23/21	33				Dark brown, moist, dense, fine to coarse SAND with some brick, and rock fragments. (fill).	
	20	24/14	5.00 - 7.00	13/11/7/4	18	22			Dark brown, moist, medium dense, fine to coarse SAND with some brick, trace slag. (fill).	
10						36			Obstruction at 6.5' bgs.	
						192			Weashed ahead to 10.0' bgs.	
	30	24/6	10.00 - 12.00	2/3/8/10	11	8			Brown, wet, medium dense, fine to coarse SAND with brick. (fill).	
15						12				
						13				
	R1	60/60	14.20 - 19.20	ROD = 28%		9				
20						623 ND	40.30		923 blows for 0.2'. — 44.20 Top of Bedrock at Elev. 40.3 ft. Bedrock: Grey and white, calcareous SANDSTONE. (Vassalboro Formation). Rock Mass Quality = Poor. R1: Bedrock Core Times (min/sec) 14.2-15.2' (4|30) 15.2-16.2' (4|27) 16.2-17.2' (4|30) 17.2-18.2' (5|50) 18.2-19.2' (5|15) 100% Recovery	
25							35.30		Bottom of Exploration at 19.20 feet below ground surface. — 19.20	
30										
35										
40										
45										

Remarks:

Stratification lines represent approximate boundaries between soil typest transitions may be gradual. — Page 1 of 1

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made. — Boring No.: BB-WCMW-102

Maine Department of Transportation — Boring No.: BB-WCME-101

Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Cumberland Mills East Bridge — Location: Westbrook, Maine — PIN: 11064.00

Driller:	MaineDOT	Elevation (ft.):	54.5	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/22/04-6/22/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	7+15.7, 29.9 Lt.	Casing ID/OD:	3.0/3.5"	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger; S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_u = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); MOH = weight of 140lb. hammer; WOR = weight of rods; WOC = weight of casing; WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#/6 in.) Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						554	54.20		Pavement — 0.30	
5	10	24/12	2.00 - 4.00	7/4/4/3	8				Layers of brown, red and black, damp, loose, silty fine to coarse SAND, trace gravel, trace brick and slag. (fill).	
	20	24/13	5.00 - 7.00	3/4/2/2	6	6			Grey, damp, loose, silty fine to coarse SAND, trace gravel and organics.	
10						6				
						9				
	30	24/6	10.00 - 12.00	4/3/3/2	6	8				
15						5			Grey, wet, loose, fine to coarse SAND, some silt, trace gravel and organics, with broken pavement pieces. (fill) .	GM76152 A-4, SM WC=16.0%
						3				
	40	24/8	15.00 - 17.00	3/5/4/4	9	6				
20						5				
						14			Grey, wet, loose, fine to coarse SAND, some silt, trace gravel, (fill).	GM76453 A-2-4, SM WC=20.6%
						12				
						15				
						34	35.90		— 18.60	
	50	10/6	20.00 - 20.83	35/55|.35|	---	89			Grey, wet, hard, sandy SILT with broken rock fragments. Weashed ahead 2.5'.	
25						23				
						924 ND	32.00		— 22.50 Roller coned ahead to 23.0' bgs. Top of Bedrock at Elev. 32.0 ft. Bedrock: Grey and white, calcareous SANDSTONE. (Vassalboro Formation). Rock Mass Quality = Very Poor. R1: Bedrock Core Times (min/sec) 23.0-24.0' (6|15) 24.0-25.0' (6|10) 25.0-26.0' (5|50) 26.0-27.0' (6|30) 27.0-28.0' (7|00) 100% Recovery	
	R1	60/60	23.00 - 28.00	ROD = 25%		ND				
30							26.50		Bottom of Exploration at 28.00 feet below ground surface. — 28.00	
35										
40										
45										

Remarks:

Stratification lines represent approximate boundaries between soil typest transitions may be gradual. — Page 1 of 1

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made. — Boring No.: BB-WCME-101

Maine Department of Transportation — Boring No.: BB-WCME-102

Soil/Rock Exploration Log — US CUSTOMARY UNITS

Project: Cumberland Mills East Bridge — Location: Westbrook, Maine — PIN: 11064.00

Driller:	MaineDOT	Elevation (ft.):	53.5	Auger ID/OD:	5" SSA
Operator:	C. Mann	Datum:	NGVD	Sampler:	Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	6/24/04-6/24/04	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ
Boring Location:	8+69.4, 17.5 Rt.	Casing ID/OD:	3.0/3.5"	Water Level*:	None Observed

Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; R = Rock Core Sample; V = Insitu Vane Shear Test; SSA = Solid Stem Auger; S_u = Insitu Field Vane Shear Strength (psf); T_v = Pocket Torvane Shear Strength (psf); q_u = Unconfined Compressive Strength (ksf); $S_{u(lab)}$ = Lab Vane Shear Strength (psf); MOH = weight of 140lb. hammer; WOR = weight of rods; WOC = weight of casing; WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#/6 in.) Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						554	53.20		4" Pavement — 0.30	
5	10	24/6	5.00 - 7.00	4/18/5/4	21	6			Dark brown, damp, medium dense, fine to coarse SAND, little silt, trace brick and rock fragments. (fill).	
						6			Change to brown silty fine to coarse SAND in nose of spoon.	
10						8				
						12				
						11				
	20 B1	10/6 60/60	10.00 - 10.83 +0.80 15.80	10/3110.33 ROD = 84%	---	928 ND	42.70		928 blows for 0.8'. Brown, wet, medium dense, fine to coarse silty SAND, trace gravel. Spoon bounce at 10.8' bgs. — 40.80 Top of Bedrock at Elev. 42.7 ft. Bedrock: Grey and white, calcareous SANDSTONE. (Vassalboro Formation). Rock Mass Quality = Fair. R1: Bedrock Core Times (min/sec) 10.8-11.8' (6|00) 11.8-12.8' (6|45) 12.8-13.8' (6|10) 13.8-14.8' (6|20) 14.8-15.8' (5|50) 100% Recovery	GM76154 A-4, SM WC=16.2%
15							37.70		— 15.80 Bottom of Exploration at 15.80 feet below ground surface.	
20										
25										
30										
35										
40										
45										

Remarks:

Stratification lines represent approximate boundaries between soil typest transitions may be gradual. — Page 1 of 1

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made. — Boring No.: BB-WCME-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PIN
BRIDGE NO. 1519 & 5463.00 - 11064.00 — BRIDGE PLANS

PROJ. MANAGER	J Wentworth	BY
DESIGN-DETAILED	K. MAGUIRE	T. WHITE
CHECKED-REVIEWED	------	------
DESIGN2-DETAILED2	------	------
DESIGN3-DETAILED3	------	------
REVISIONS 1	------	------
REVISIONS 2	------	------
REVISIONS 3	------	------
REVISIONS 4	------	------
FIELD CHANGES	------	------

DATE: JAN 2011

SIGNATURE

P.E. NUMBER

DATE

CUMBERLAND MILLS WEST & EAST BRIDGE
PRESUMPSCOT RIVER
WESTBROOK CUMBERLAND COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-sill 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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills West Bridge
Location: Westbrook, Maine

Boring No.: BB-WCMW-101
PIN: 11063.00

Driller:	MaineDOT	**Elevation (ft.)**	53.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoons
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/23/04-6/23/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+04.8, 23.7 Rt.	**Casing ID/OD:**	3.5/3.0"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA			Brown, dry, fine to coarse SAND, little silt.	
5	1D	24/2	5.00 - 7.00	8/18/17/9	35	40			Dark brown, dry, dense, fine to coarse SAND, little silt, trace organics.	
						17				
						17				
						21				
						23				
10	MD	24/0	10.00 - 12.00	7/7/9/9	16	16			No recovery.	
						16				
						27				
						19				
						24				
15	2D	24/20	15.00 - 17.00	4/1/1/1	2	15	38.50		15.00 Grey, wet, soft, sandy SILT, trace gravel and organics.	G#176151 A-4, ML WC=29.1%
						13				
						11				
						11				
						10				
20	3D	24/8	20.00 - 22.00	4/4/12/50(.2)	16	22			Grey, wet, very stiff, sandy SILT, wood layer (3") in bottom of spoon, rock in nose of spoon.	
	R1	60/54	21.70 - 26.70	RQD = 60%		[a]24 NQ	31.80		[a]24 blows for 0.5' 21.70 Top of Bedrock at Elev. 31.8 ft. Bedrock: Grey and white, calcareous SANDSTONE, (Vassalboro Formation). Rock Mass Quality = Fair. R1: Bedrock Core Times (min/sec) 21.7-22.7' (5:42) 22.7-23.7' (5:50)	
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.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills West Bridge
Location: Westbrook, Maine

Boring No.: BB-WCMW-101
PIN: 11063.00

Driller:	MaineDOT	**Elevation (ft.)**	53.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoons
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/23/04-6/23/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	5+04.8, 23.7 Rt.	**Casing ID/OD:**	3.5/3.0"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25									23.7-24.7' (6:23) 24.7-25.7' (5:03) 25.7-26.7' (5:30) 90% Recovery	
							26.80		26.70 **Bottom of Exploration at 26.70 feet below ground surface.**	
30										
35										
40										
45										
50										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills West Bridge
Location: Westbrook, Maine

Boring No.: BB-WCMW-102
PIN: 11063.00

Driller:	MaineDOT	**Elevation (ft.)**	54.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoons
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/23/04-6/23/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	6+36.4, 28.6 Lt.	**Casing ID/OD:**	3.5/3.0"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	54.33		2" Pavement	
									0.17	
	1D	24/10	2.00 - 4.00	6/10/23/21	33				Dark brown, moist, dense, fine to coarse SAND with some brick, and rock fragments, (Fill).	
5	2D	24/14	5.00 - 7.00	13/11/7/4	18	22			Dark brown, moist, medium dense, fine to coarse SAND with some brick, trace slag, (Fill).	
						36			Obstruction at 6.5' bgs.	
						192				
						[a]WA			[a]Washed ahead to 10.0' bgs.	
10						4				
	3D	24/6	10.00 - 12.00	2/3/8/10	11	8			Brown, wet, medium dense, fine to coarse SAND with brick, (Fill).	
						12				
						13				
						9				
	R1	60/60	14.20 - 19.20	RQD = 28%		[b]23 NQ	40.30		[b]23 blows for 0.2'.	
15									14.20	
									Top of Bedrock at Elev. 40.3 ft. Bedrock: Grey and white, calcareous SANDSTONE, (Vassalboro Formation). Rock Mass Qualilty = Poor. R1: Bedrock Core Times (min/sec) 14.2-15.2' (4:30) 15.2-16.2' (4:27) 16.2-17.2' (4:30) 17.2-18.2' (5:50) 18.2-19.2' (5:15) 100% Recovery	
							35.30		19.20	
20									**Bottom of Exploration at 19.20 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.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills East Bridge
Location: Westbrook, Maine

Boring No.: BB-WCME-101
PIN: 11064.00

Driller:	MaineDOT	**Elevation (ft.)**	54.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/22/04-6/22/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	7+15.7, 29.9 Lt.	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	54.20		Pavement 0.30	
	1D	24/12	2.00 - 4.00	7/4/4/3	8				Layers of brown, red and black, damp, loose, silty fine to coarse SAND, trace gravel, trace brick and slag, (Fill).	
5	2D	24/13	5.00 - 7.00	3/4/2/2	6	6			Grey, damp, loose, silty fine to coarse SAND, trace gravel and organics.	G#176152 A-4, SM WC=16.0%
						6				
						9				
						8				
						8				
10	3D	24/6	10.00 - 12.00	4/3/3/2	6	5			Grey, wet, loose, fine to coarse SAND, some silt, trace gravel and organics, with broken pavement pieces, (Fill) .	
						3				
						6				
						5				
						5				
15	4D	24/8	15.00 - 17.00	3/5/4/4	9	14			Grey, wet, loose, fine to coarse SAND, some silt, trace gravel, (fill).	G#176453 A-2-4, SM WC=20.6%
						12				
						15				
						34	35.90		18.60	
						89				
20	5D	10/6	20.00 - 20.83	35/55(.35)	---	33			Grey, wet, hard, sandy SILT with broken rock fragments. [a]Washed ahead 2.5'.	
						[a]WA				
						23				
						58	32.00		22.50	
	R1	60/60	23.00 - 28.00	RQD = 25%		[b]RC NQ			[b]Roller coned ahead to 23.0' bgs. Top of Bedrock at Elev. 32.0 ft. Bedrock: Grey and white, calcareous SANDSTONE, (Vassalboro Formation). Rock Mass Quality = Very Poor. R1: Bedrock Core Times (min/sec)	
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills East Bridge
Location: Westbrook, Maine

Boring No.: BB-WCME-101
PIN: 11064.00

Driller:	MaineDOT	**Elevation (ft.)**	54.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/22/04-6/22/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	7+15.7, 29.9 Lt.	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25									23.0-24.0' (6:15) 24.0-25.0' (6:10) 25.0-26.0' (5:50) 26.0-27.0' (6:30) 27.0-28.0' (7:00) 100% Recovery	
							26.50		28.00 **Bottom of Exploration at 28.00 feet below ground surface.**	
30										
35										
40										
45										
50										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cumberland Mills East Bridge
Location: Westbrook, Maine

Boring No.: BB-WCME-102
PIN: 11064.00

Driller:	MaineDOT	**Elevation (ft.)**	53.5	**Auger ID/OD:**	5" SSA
Operator:	C. Mann	**Datum:**	NGVD	**Sampler:**	Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	6/24/04-6/24/04	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ
Boring Location:	8+69.4, 17.5 Rt.	**Casing ID/OD:**	3.0/3.5"	**Water Level*:**	None Observed

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
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-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SSA	53.20		4" Pavement	
									0.30	
5	1D	24/6	5.00 - 7.00	4/16/5/4	21	6			Dark brown, damp, medium dense, fine to coarse SAND, little silt, trace brick and rock fragments, (Fill).	
						6			Change to brown silty fine to coarse SAND in nose of spoon.	
						8				
						12				
						11				
10	2D	10/6	10.00 - 10.83	10/31(0.3)	---	[a]28			[a]28 blows for 0.8'.	G#176154
	R1	60/60	10.80 - 15.80	RQD = 64%		NQ	42.70		Brown, wet, medium dense, fine to coarse silty SAND, trace gravel. Spoon bounce at 10.8' bgs.	A-4, SM
									10.80	WC=16.2%
									Top of Bedock at Elev. 42.7 ft. Bedrock: Grey and white, calcareous SANDSTONE, (Vassalboro Formation). Rock Mass Quality = Fair. R1: Bedrock Core Times (min/sec) 10.8-11.8' (6:00) 11.8-12.8' (6:45) 12.8-13.8' (6:10) 13.8-14.8' (6:20) 14.8-15.8' (5:50) 100% Recovery	
15							37.70		15.80	
									Bottom of Exploration at 15.80 feet below ground surface.	
20										
25										

Remarks:

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Westbrook Project Numbers:11063.00 11064.00

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
Cumberland Mills West Bridge 11063.00											
BB-WCMW-101, 2D	5+04.8	23.7 Rt.	15.0-17.0	176151	1	29.1			ML	A-4	IV
Cumberland Mills East Bridge 11064.00											
HB-WCME-101, 2D	7+15.7	29.9 Lt.	5.0-7.0	176152	2	16.0			SM	A-4	III
HB-WCME-101, 4D	7+15.7	29.9 Lt.	15.0-17.0	176153	2	20.6			SM	A-2-4	II
BB-WCME-102, 2D	8+69.4	17.5 Rt.	10.0-10.8	176154	2	16.2			SM	A-4	III

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.

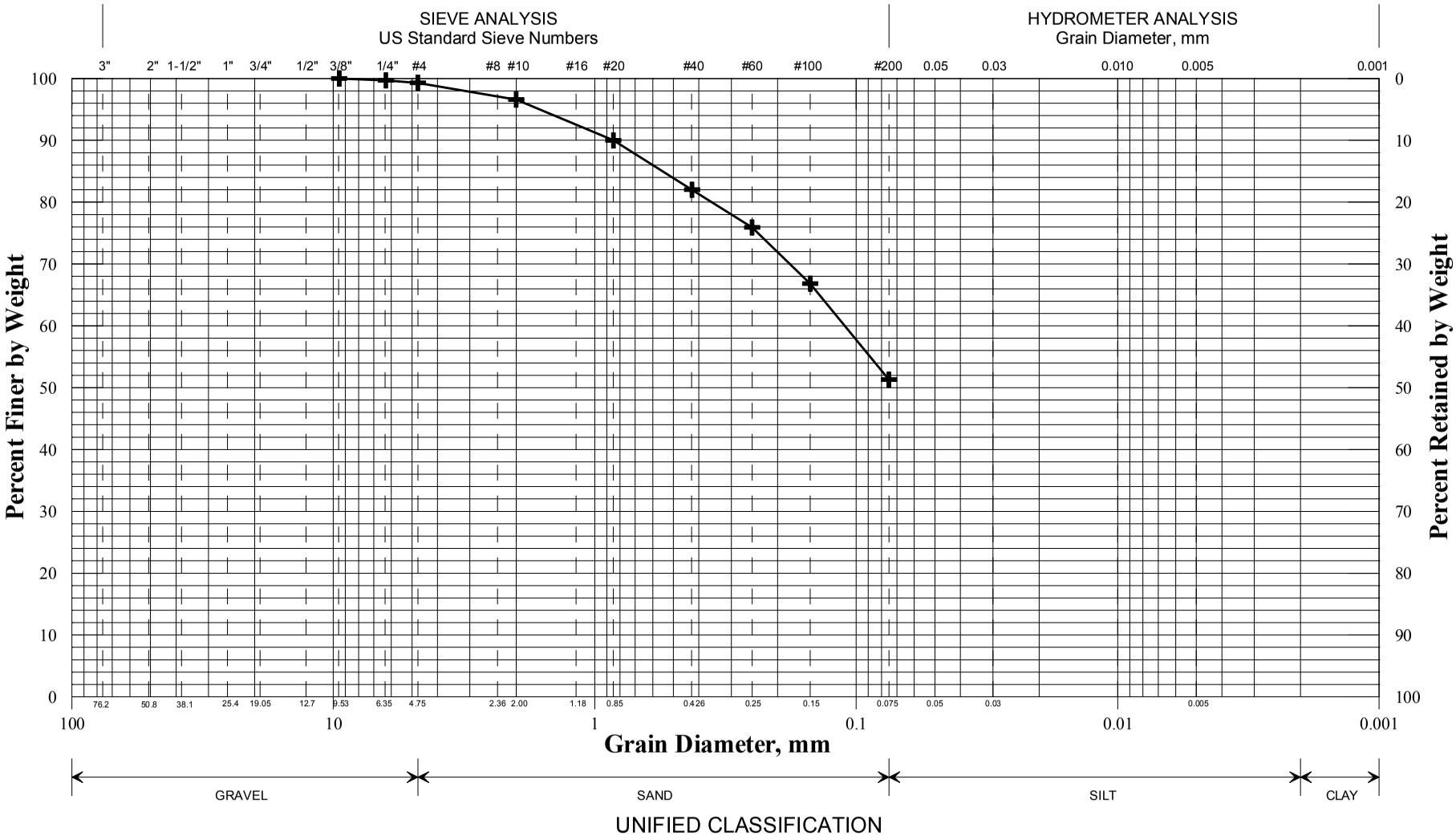
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

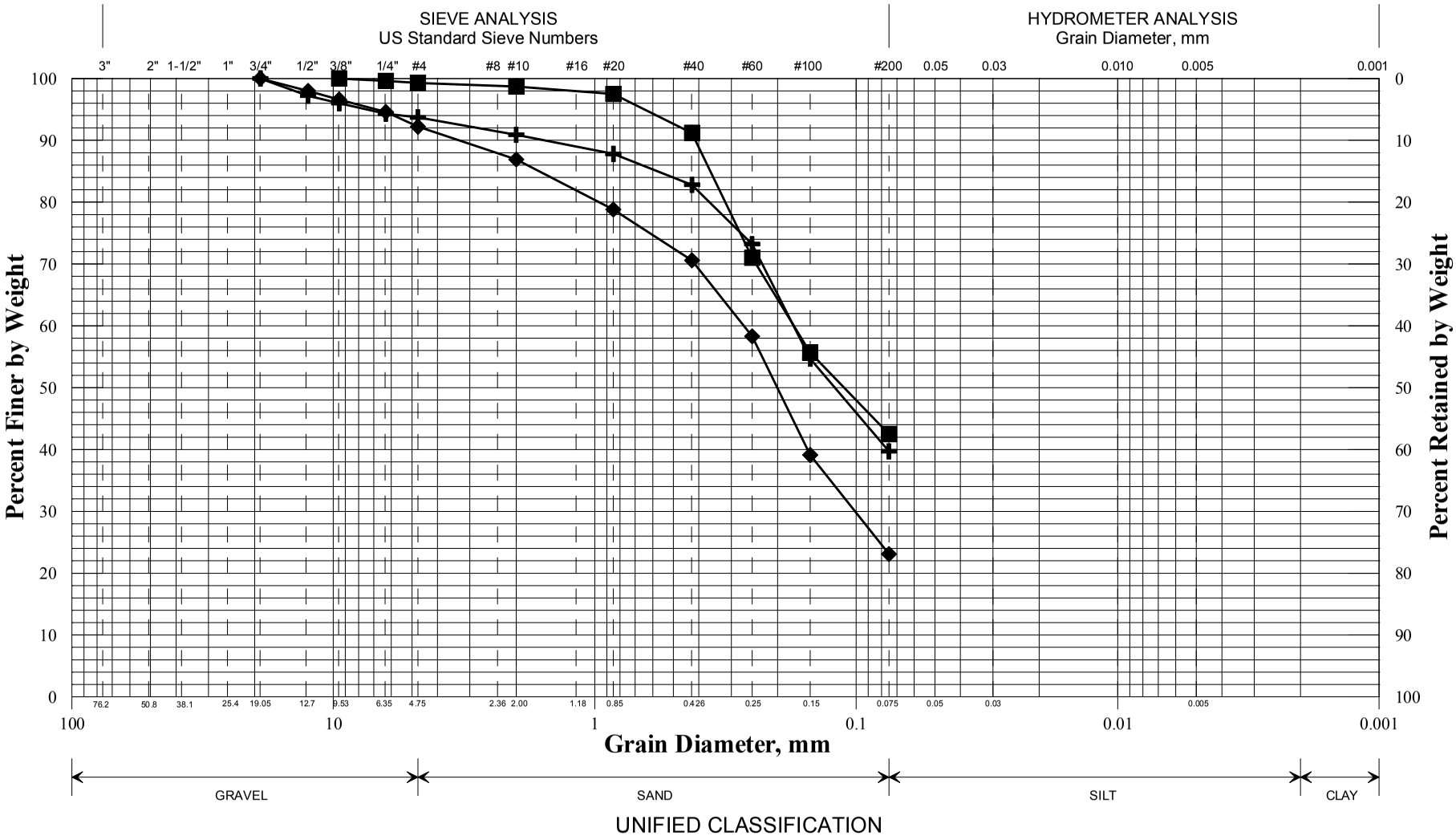
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-WCMW-101/2D	5+04.8	23.7 RT	15.0-17.0	Sandy SILT, trace gravel.	29.1			
◆									
■									
●									
▲									
✕									

PIN
011063.00
Town
Westbrook
Reported by/Date
WHITE, TERRY A 1/18/2011

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-WCME-101/2D	7+15.7	29.9 LT	5.0-7.0	Silty SAND, trace gravel.	16.0			
◆	BB-WCME-101/4D	7+15.7	29.9 LT	15.0-17.0	SAND, some silt, trace gravel.	20.6			
■	BB-WCME-102/2D	8+69.4	17.5 RT	10.0-10.8	Silty SAND, trace gravel.	16.2			
●									
▲									
✕									

PIN
011064.00
Town
Westbrook
Reported by/Date
WHITE, TERRY A 1/18/2011

Appendix C

Calculations

Earth Pressure:

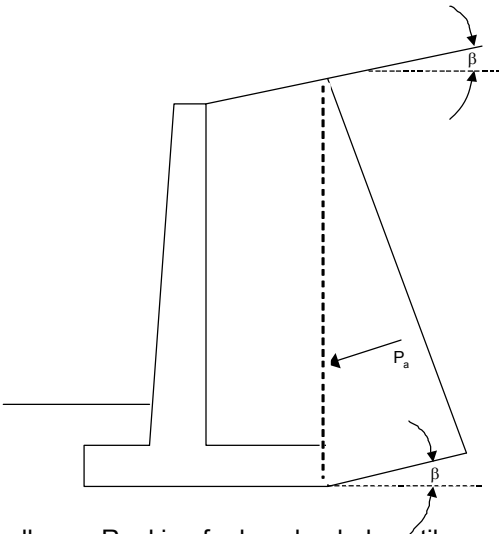
Active Earth Pressure - Rankine Theory
from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-7

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

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

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

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



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

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

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

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

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

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

Bearing Resistance:

SERVICE LIMIT STATE

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 5th Edition 2010
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Weathered or broken bedrock of any kind except argillaceous rock (shale)

Consistency In Place: medium hard rock

Bearing Resistance: Ordinary Range (ksf) 16 to 24

Recommended Value of Use: 20 ksf

Recommended Value: $q_{nom} := 20 \cdot ksf$

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

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

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

STRENGTH LIMIT STATE

Method 1 - Determine Bearing Resistance using RMR Method

From AASHTO LRFD Bridge Design Specifications
Section 10.4.6.4 Rock Mass Strength

Bedrock at the site is Sandstone which was found to be "poor to fair" in quality.
RQD ranged from 28 to 64%. (Average 44% - poor)

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

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

1. Strength of intact rock

From Standard Specifications for Highway Bridges 17th Edition - 2002
Table 4.4.8.1.2B uniaxial compressive strength for Sandstone = 1400 to 3600 ksf

Use: $q_u := 2000 \cdot ksf$

From Table 10.4.6.4.-1:
For Uniaxial Compressive Strength = 1080 to 2160 ksf: Relative Rating = 9

2. Drill Core Quality

Bedrock RQD = 28 to 64% (poor to fair)

From Table 10.4.6.4.-1: RQD 25% to 50%: Relative Rating = 10

3. Spacing of joints

Assume Spacing of 2 in to 1 ft

From Table 10.4.6.4.-1: Relative Rating = 7

4. Condition of joints

Assume slightly rough surfaces <0.05 in, hard joint wall rock

From Table 10.4.6.4.-1: Relative Rating = 18

5 Groundwater conditions

General Conditions = Water under moderate pressure

From Table 10.4.6.4.-1: Relative Rating = 4

Raw RMR = 48

Adjustment to RMR for joint Orientations from Table 10.4.6.4-2

Assume Strike and Dip Orientations of Joints = Fair

For Foundations: Rating = -7

Adjusted RMR = 41

Determine Rock Mass Class from Total Rating

$RMR := 41$

From Table 10.4.6.4.-3: Class No. = III - Fair Rock

The shear strength of the rock evaluated using Hoek and Brown

$q_u := 2000 \cdot ksf$ average unconfined compressive strength of rock core

$\sigma_n := 0 \cdot ksf$ effective normal stress

From Table 10.4.6.4-4 determine *m* and *s*

Rock Quality = Fair Quality Rock Mass

Rock Type = C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage - *sandstone and quartzite*

By interpolation:

$m_r := 0.17$ $s_r := 0.00005$

Determine nominal and factored bearing resistance with Carter & Kulhawy 1988:

Reference: NCHRP 651 LRFD Design and Construction of Shallow Foundations for Highway Bridge Structures, 2010 Equation 82b pg. 40.

$$q_{nomCK} := \left[s_r^{0.5} + \left(m_r \left(s_r^{0.5} \right) + s_r \right)^{0.5} \right] \cdot q_u$$

$$q_{nomCK} := \left[0.00005^{0.5} + \left[0.17 \left(0.00005^{0.5} \right) + 0.00005 \right]^{0.5} \right] \cdot 2000 \cdot ksf$$

$$q_{nomCK} = 85 \cdot ksf$$

Determine the factored bearing resistance:

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

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

$q_R := \phi_b \cdot q_{nomCK}$ $q_R = 38 \cdot ksf$ High

Method 2 - Determine Bearing Resistance using Foundation Analysis and Design
by JE Bowles Fifth Edition Section 4-16 pg 277

Bedrock at the site is SANDSTONE which is poor to fair in quality.
RQD = 28 to 64%

Assume: $\phi := 45 \cdot deg$ internal friction angle rock

$c_r := 0 \cdot psi$ cohesion (rock)

Bearing Capacity factors by Stagg and Zienkiewicz 1968

$$N_c := 5 \cdot \left(\tan\left(45 \cdot deg + \frac{\phi}{2} \right)^4 \right) \qquad N_c = 170$$

$$N_q := \tan\left(45 \cdot deg + \frac{\phi}{2} \right)^6 \qquad N_q = 198$$

$$N_\gamma := N_q + 1 \qquad N_\gamma = 199$$

Terzaghi Shape factors from Table 4-1 pg 220

For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Assume $\gamma_r := 165 \cdot pcf$ for the rock

$D_f := 0 \cdot ft$ footing placed on bedrock surface - no embedment $q := \gamma_r \cdot D_f$ $q = 0 \cdot psf$

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$ Look at several footing widths

$$q_{nom} := c_r \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot \gamma_r \cdot B \cdot N_\gamma \cdot s_\gamma$$

$$q_{nom} = \begin{pmatrix} 99 \\ 131 \\ 164 \\ 197 \end{pmatrix} \cdot ksf$$

Reduce ultimate bearing based on average RQD = 44%

$q_{reduced} := q_{nom} \cdot (0.44)^2$

$$q_{reduced} = \begin{pmatrix} 19 \\ 25 \\ 32 \\ 38 \end{pmatrix} \cdot ksf$$

Assume this ultimate load is a nominal load. Apply 0.45 resistance factor to get factored resistance.

$q_{factored} := q_{reduced} \cdot 0.45$

$$q_{factored} = \begin{pmatrix} 9 \\ 11 \\ 14 \\ 17 \end{pmatrix} \cdot ksf \qquad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot ft$$

At the Strength Limit State:

Recommend a limiting factored bearing resistance of 14 ksf for footings 10 feet wide or less.
Recommend a limiting factored bearing resistance of 17 ksf for footings greater than 12 feet.

Method 3 - Determine Bearing Resistance using Rock Engineering Applications
by Franklin & Dusseault Section 3.6.4 pg 136

From Figure 3.11 Allowable bearing pressure from Rock Quality Designation

For RQD of 44%: $q_{all} := 6MPa$ $q_{all} = 125.313 \cdot ksf$ **High**

From Table 3.3 Allowable bearing pressure from Types and Condition of Rock

Broken Rock of any kind with moderately close spacing
of discontinuities (0.3 m or greater), except argillaceous shale

Presumed Allowable bearing Pressure = 1 Mpa

$q_{all2} := 1 \cdot MPa$ $q_{all2} = 21 \cdot ksf$

Frost Protection:

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

From the Design Freezing Index Map:
Westbrook, Maine
DFI = 1200 degree-days

From the lab testing: soils are coarse grained with a water content = ~20%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1200 and wc =20%
Frost Penetration = 60.4 inches

$Frost\_depth := 60.4in$ $Frost\_depth = 5 \cdot ft$

Use Frost Depth = 5.0 feet for design

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Portland

--- ModBerg Results ---

Project Location: Portland Wsfo Airport, Maine

Air Design Freezing Index	=	1195 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	956 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	68.1	20.0	125.0	34	46	3.8	1.9	3,600

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

****
Total Depth of Frost Penetration = 5.67 ft = 68.1 in.
****

$Frost\_depth_{modberg} := 68.1 \cdot in$

$Frost\_depth_{modberg} = 5.675 \cdot ft$

Seismic:

11063.00 and 11064.00 Westbrook Cumberland Mills West and East
Date and Time: 1/4/2011 3:21:54 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04092
Zip Code Latitude = 43.681700
Zip Code Longitude = -070.361800
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.089	PGA - Site Class B
0.2	0.177	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04092
Zip Code Latitude = 43.681700
Zip Code Longitude = -070.361800
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.143	As - Site Class D
0.2	0.283	SDs - Site Class D
1.0	0.108	SD1 - Site Class D

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

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

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

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

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

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

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 610
STONE FILL, RIPRAP, STONE BLANKET,
AND STONE DITCH PROTECTION

Add the following paragraph to Section 610.02:

Materials shall meet the requirements of the following Sections of Special Provision 703:

Stone Fill	703.25
Plain and Hand Laid Riprap	703.26
Stone Blanket	703.27
Heavy Riprap	703.28
Definitions	703.32

Add the following paragraph to Section 610.032.a.

Stone fill and stone blanket shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following paragraph to Section 610.032.b:

Riprap shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following to Section 610.032:

Section 610.032.d. The grading of riprap, stone fill, stone blanket and stone ditch protection shall be determined by the Resident by visual inspection of the load before it is dumped into place, or, if ordered by the Resident, by dumping individual loads on a flat surface and sorting and measuring the individual rocks contained in the load. A separate, reference pile of stone with the required gradation will be placed by the Contractor at a convenient location where the Resident can see and judge by eye the suitability of the rock being placed during the duration of the project. The Resident reserves the right to reject stone at the job site or stockpile, and in place. Stone rejected at the job site or in place shall be removed from the site at no additional cost to the Department.

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

703.25 Stone Fill Stones for stone fill shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for stone fill shall be angular and rough. Rounded, subrounded, or long thin stones will not be allowed. Stone for stone fill may be obtained from quarries or by screening oversized rock from earth borrow pits. The maximum allowable length to thickness ratio will be 3:1. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (500 lbs) shall have a maximum dimension of approximately 36 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension of 12 inches (200 lbs).

703.26 Plain and Hand Laid Riprap Stone for riprap shall consist of hard, sound durable rock that will not disintegrate by exposure to water or weather. Stone for riprap shall be angular and rough. Rounded, subrounded or long thin stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (200 lbs) shall have an average dimension of approximately 12 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension greater than 9 inches (50 lbs).

703.27 Stone Blanket Stones for stone blanket shall consist of sound durable rock that will not disintegrate by exposure to water or weather. Stone for stone blanket shall be angular and rough. Rounded or subrounded stones will not be allowed. Stones may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (300 lbs) shall have minimum dimension of 14 inches, and the maximum stone size (3000 lbs) shall have a maximum dimension of approximately 66 inches. Fifty percent of the stones by volume shall have average dimension greater than 24 inches (1000 lbs).

703.28 Heavy Riprap Stone for heavy riprap shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for heavy riprap shall be angular and rough. Rounded, subrounded, or thin, flat stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for heavy riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (500 lbs) shall have minimum dimension of 15 inches, and at least fifty percent of the stones by volume shall have an average dimension greater than 24 inches (1000 lbs).

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces
Subrounded: Particles have nearly plane sides but have well-rounded corners and edges
Rounded: Particles have smoothly curved sides and no edges