

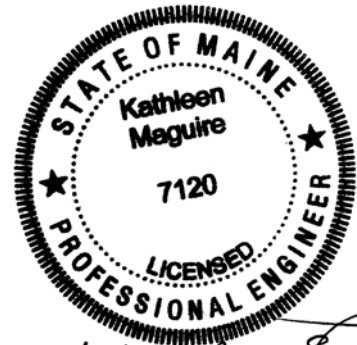
**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**LITTLE BRIDGE
OVER MILL BROOK
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 Little Bridge over Mill Brook in Westbrook, Maine. The Maine Department of Transportation (MaineDOT) Bridge Program has selected the Little Bridge site as a location to install a composite tubular arch bridge structure. The proposed 36 foot, single span replacement structure will be founded on driven H-piles. The following design recommendations are discussed in detail in the attached report:

H-piles - The use of H-pile supported arch stem walls/pile caps is a viable foundation system for use at the site. Piles should be fitted with driving points to protect the tips and improve penetration. Piles may be plumb, battered or a combination of both. The H-piles shall be design for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral, and flexural resistance. An L-Pile[®] analysis is recommended to evaluate the combined axial compression and flexure with factored axial loads, moments and pile head displacements applied. As the H-piles will be modeled as fully fixed at the pile head, the resistance of the piles should be evaluated for structural compliance with the interaction equation.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each arch stem wall/pile cap. The first pile driven at each arch stem wall/pile cap should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor, ϕ_{dyn} , of 0.65. The factored pile load should be shown on the plans.

Arch Stem Wall/Pile Cap – Arch stem wall/pile cap shall be designed for all relevant strength, service and extreme limit states and load combinations. The design of pile supported arch stem wall/pile caps at the strength limit state shall consider pile stability and structural resistance. Arch stem wall/pile cap design at the service limit state shall include settlement, horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination. Extreme limit state design checks for arch stem wall/pile cap supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads.

Calculation of passive earth pressure for resisting lateral forces/thrust from the arch should assume a K_p of 3.25, anticipating small footing movements and a resistance factor (ϕ_{ep}) of 0.5. Should the arch stem wall/pile cap rotation (γ/H) exceed 0.005, a Coulomb passive earth pressure coefficient (K_p) of 6.73 is recommended. A load factor for passive earth pressure is not specified in LRFD. For designing the pile cap reinforcing steel to resist passive earth pressure, use a maximum load factor, $\gamma_{EH} = 1.50$.

All arch stem wall/pile cap designs shall include a drainage system behind the arch stem wall/pile cap to intercept any groundwater.

Prefabricated Concrete Modular Block Gravity Wall – Precast Concrete Modular Gravity (PCMG) walls will be constructed on all four corners of the bridge to retain the roadway section and minimize impacts. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635 and plan notes.

Bearing Resistance – Bearing resistance for PCMG walls founded on a leveling slab on native silt shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 7 ksf for bases from 8.5 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

Scour and Riprap – The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. The riprap shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

Settlement - The grade of the existing bridge approaches will be maintained in the replacement of the structure. Post-construction settlements are anticipated to be negligible. Any settlement of the arch stem wall/pile cap will be due to the elastic compression of the piling and will be negligible.

Frost Protection - The arch stem wall/pile caps shall be embedded a minimum of 4.0 feet for frost protection. Any foundation placed on granular subgrade soils including the PCMG wall base shall 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. However, superstructure connections and minimum support length requirements shall be designed in accordance with LRFD requirements.

Construction Considerations - Construction of the arch stem wall/pile cap will require soil excavation and partial or full removal of the existing abutments. Construction activities may require cofferdams and earth support systems. Using the excavated native soils as structural backfill should not be permitted. The existing subbase and subgrade fill soils in the bridge approaches should not be used to re-base the new bridge approaches.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Little Bridge over Mill Brook in Westbrook, Maine. A subsurface investigation at the site has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing Little Bridge carries East Bridge Street over Mill Brook and was constructed in 1955. The bridge consists of twin, 11-foot diameter steel culverts with a total span of 26 feet. In 1979, the roadway grade was raised and the twin culverts were extended approximately 20 feet in both directions. The 2007 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the culverts have a condition rating of 4 indicating considerable damage while the channel protection condition is rated a 6 indicating bank slumping. The Bridge Sufficiency Rating is 71.9. The bridge has a scour critical rating of 8 meaning that the bridge foundations have been determined to be stable for the assessed or calculated scour condition. Inspection records note that the culverts have rust holes through the downstream end of both pipes and sporadic rust holes throughout the culverts.

The MaineDOT Bridge Program has selected the Little Bridge site as a location to install a rigidified, concrete-filled, composite, tubular arch bridge structure developed by the University of Maine's Advance Engineering Wood Composites (AEWC) Center in Orono, Maine. The carbon fiber tubes are inflated and infused with resin. After hardening, the tubes are transported to the bridge site and lowered into place and filled with concrete. The proposed arch structure will have a span length of approximately 36 feet and will be founded on an arch stem wall/pile cap on driven H-piles. The proposed bridge alignment will closely match the existing alignment. The roadway grade will closely match the existing grade for the length of the project.

2.0 GEOLOGIC SETTING

Little Bridge in Westbrook carries East Bridge Street over Mill Brook approximately 0.1 miles southwesterly of Route 302 as shown on Sheet 1 - Location Map found at the end of this report. Mill Brook flows in an easterly direction into the Presumpscot River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. Sand is dominant in some areas, but may be underlain by finer-grained sediments. The unit contains small areas of till not completely covered by marine sediments. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams and commonly has a branching network of steep-walled stream gullies. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine (1985), published by the Maine Geological Survey, the bedrock at the site is identified as calcareous sandstone or interbedded sandstone and impure limestone. This bedrock is identified as the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Test boring BB-WMB-101 was drilled at the south side of the existing structure. Test borings BB-WMB-102 and BB-WMB-102A were drilled at the north side of the existing structure. Boring BB-WMB-102A was terminated at a depth of approximately 15.5 feet below ground surface (bgs) prior to reaching bedrock due to the presence of boulders at the boring location.

The exploration locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The borings were drilled between December 17 and 22, 2009 by the MaineDOT drill crew. 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 end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Undisturbed tube samples were obtained and in-situ vane shear tests were made in the soft soils where possible. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in February of 2009 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value are shown on the boring logs. The bedrock was cored in two (2) of the borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of nine (9) standard grain size analyses, ten (10) grain size analyses with hydrometer, two (2) Atterberg Limits tests, one (1) 1-D consolidation test, and one (1) standard tube opening. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the test borings generally consisted of fill sands, underlain by sand, underlain by clayey silt, silt and clay, underlain by sand, underlain by glacial till, underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

5.1 Fill Sand

A layer of fill sand was encountered beneath the pavement in all of the borings. The thickness of the layer was approximately 9.0 to 11.5 feet. The soil generally consisted of:

- brown, damp, fine to coarse sand with trace silt,
- brown, damp, gravelly, fine to coarse sand with trace silt,
- dark brown, moist, fine to coarse sand with some gravel and some silt,
- light brown, moist, silty, fine to coarse sand with trace gravel and
- olive brown, moist, fine to coarse sand with some silt, little clay and little gravel.

Corrected SPT N-values in the fill sand ranged from 13 to 34 blows per foot (bpf) indicating that the soil is medium dense to dense in consistency. Water contents from three (3) samples obtained within the fill sand layer range from approximately 7% to 13%. Two (2) grain size analyses and one (1) grain size analysis with hydrometer conducted on samples of the fill sand indicate that the soil is classified as an A-1-b or A-4 by the AASHTO Classification System and a SM or SC-SM by the Unified Soil Classification System.

5.2 Upper Sand

A sand layer was encountered beneath the fill sand in all of the borings. The thickness of the upper sand layer ranged from approximately 9.0 to 14.5 feet. The upper sand generally consisted of:

- brown, damp, fine to coarse sand, with trace silt,
- light brown, damp, fine to coarse sand with little silt, little gravel and trace clay,
- brown damp, fine to coarse sand, little to some gravel and little to trace gravel, and
- olive brown, wet fine to coarse sand with little silt, little gravel and trace clay.

Cobbles and boulders were encountered within the upper sand layer in boring BB-WMB-101 at a depth of approximately 21.0 feet bgs and in boring BB-WMB-102 at a depth of approximately 15.5 feet bgs.

Corrected SPT N-values in the upper sand layer ranged from 13 to >50 bpf indicating that the upper sand is medium dense to very dense in consistency. Water contents from four (4) samples obtained within the upper sand layer range from approximately 5% to 12%. Two (2) grain size analyses and two (2) grain size analyses with hydrometer conducted on samples from the upper

sand layer indicate that the soil is classified as an A-1-b, A-2-4, or A-4 by the AASHTO Classification System and a SW-SM, SM, or SC-SM by the Unified Soil Classification System.

5.3 Clayey Silt, Silt and Clay

A layer of clayey silt, silt and clay was encountered below the upper sand layer. The thickness of the layer ranged from approximately 7.0 to 11.0 feet. This layer generally consisted of:

- grey, wet, clayey silt, with little to some sand and trace gravel
- grey, wet, silt, with some clay, with some sand, and trace gravel and
- grey, wet, clay, with some silt and little sand.

Corrected SPT N-value in the layer ranged from 3 to 7 bpf indicating that the soil is very soft to medium stiff in consistency. Four (4) water contents from samples obtained within the layer ranged from approximately 26% to 43%. Four (4) grain size analyses with hydrometer conducted on samples from this layer indicate that the soil is classified as an A-4 or A-6 by the AASHTO Classification System and a CL-ML or CL by the Unified Soil Classification System.

Table 5-1 below summarizes the results of Atterberg Limits tests from two (2) samples from the layer:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-WMB-101 1U	Silt	29.2	22	17	5	2.44
BB-WMB-102A 2D	Clay	33.7	28	18	10	1.57

Table 5-1 - Summary of Atterberg Limits Testing Results

Interpretation of these results indicates that layer is generally on the verge of becoming a viscous liquid if disturbed. For both of the samples the natural water content exceeds the liquid limit. This indicates that the layer has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as “quick”.

One-dimensional (1-D) consolidation testing was conducted on one (1) tube samples taken from the layer. The results of this test are included in Appendix B - Laboratory Data.

5.4 Lower Sand

A lower sand layer was encountered beneath the clayey silt, silt and clay layer. The thickness of the lower sand layer ranged from approximately 20.5 to 24.5 feet. The lower sand generally consisted of grey, wet to saturated, fine to coarse sand, trace to some gravel, little silt, and trace clay. Corrected SPT N-values in the lower sand layer ranged from 4 to 55 bpf indicating that the soil is very loose to very dense in consistency. Water contents from seven (7) samples

obtained within the lower sand layer range from approximately 9% to 19%. Five (5) grain size analyses and three (3) grain size analyses with hydrometer conducted on samples from the lower sand layer indicate that the soil is classified as an A-2-4 or A-1-b by the AASHTO Classification System and a SC-SM or SM by the Unified Soil Classification System.

5.4 Glacial Till

A thin layer of glacial till was encountered beneath the lower sand in boring BB-WMB-101. The thickness of the glacial till was approximately 1.8 feet. The glacial till generally consisted of grey, wet, gravelly, fine to coarse sand, with some silt. One (1) attempted SPT sample within the glacial till resulted in a spoon refusal. One (1) water content from a sample obtained within the glacial till layer was approximately 9%. One (1) grain size analyses conducted on a sample from the glacial till layer indicate that the soil is classified as an A-2-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

5.5 Bedrock

Bedrock was encountered and cored in borings BB-WMB-101 and BB-WMB-102A. The Table 5-2 summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB- WMB -101/ Abutment No. 1	55.3 feet	-19.8 feet	67%
BB- WMB -102 and BB-WMB-102A/ Abutment No. 2	53.5 feet	-19.1 feet	58%

Table 5-2 - Summary of Bedrock Depths, Elevations and RQD

Weathered bedrock was encountered at the bedrock surface in boring BB-WMB-101. The weathered bedrock had a thickness of approximately 0.3 feet. The bedrock is identified as grey, fine grained, slightly metamorphosed, unweathered sandy mudstone with quartz bands. Bedding dips at 10 to 30 degrees. The bedrock is part of the Cushing Formation. The rock quality designation (RQD) of the bedrock was determined to range from 58 to 67 percent indicating a rock mass quality of fair.

5.6 Groundwater

Groundwater was observed at a depths ranging from approximately 20.0 to 21.0 feet below the existing ground surface. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The MaineDOT Bridge Program has selected Little Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure developed by the University of Maine's AEWCA Advanced Structures & Composites Center in Orono, Maine. AEWCA's tubular arches are made of Fiber Reinforced Plastic (FRP) composite materials. The carbon fiber tubes are inflated and infused with resin. After hardening, the tubes are transported to the bridge site and lowered into place and filled with concrete. The tubular arches are covered with a corrugated, FRP composite deck material and backfill is placed over the tubular structure.

The following foundation alternatives may be considered for the bridge replacement:

- Spread footings,
- Driven H-piles,
- Driven pipe piles, or
- Drilled shafts

Due to the depth of overburden at the site the use of driven H-pile or pipe pile supported arches is recommended. For the purposes of this report it is assumed that driven H-piles will be used to support the structure. If, during final design, it is determined that the use of pipe piles is necessary the pipe pile resistances will be developed and provided to the designer. Prefabricated Concrete Modular Gravity (PCMG) Walls will be required to support the bridge approaches.

The design of the FRP tubular arches and associated headwalls is the responsibility of the AEWCA and will be supplied to the designer and Contractor prior to construction of the structure.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for cast-in-place concrete or precast concrete stem walls or pile caps supported on driven steel H-piles to support the tubular arches which will make up the replacement structure.

7.1 Driven H-Piles

The use of H-pile supported arch stem walls/pile caps is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial and lateral loads. Piles should be 50 ksi, Grade A572 steel H-piles. Piles should be fitted with driving points to protect the tips and improve penetration. Piles may be plumb, battered or a combination of both.

Pile lengths at the proposed arch stem wall/pile caps may be estimated based on Table 7-1 below:

Location	Estimated Arch Stem Wall/Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length
Abutment No.1 BB-WMB-101	10 feet	55.3 feet	-19.8 feet	30 feet
Abutment No.2 BB-WMB-102 and BB-WMB-102A	10 feet	53.5 feet	-19.1 feet	29 feet

Table 7-1 – Estimated Pile Lengths for Plumb H-Piles

These pile lengths do not take into account the pile length embedded in the pile cap, the additional five (5) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. The design and check floods for scour are defined in AASHTO LRFD Bridge Design Specifications 5th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5.

Since the H-piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2.

7.1.1 Strength Limit State

The nominal structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to recalculate the nominal structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e ". Preliminary estimates of the factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c , of 0.50 (severe driving conditions) and an unbraced length (ℓ) of 48 inches and an effective length factor (K) of 1.0.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, ϕ_{stat} , of 0.45.

The drivability of the five (5) proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is $\phi_{\text{dyn}} = 0.65$.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 7-2 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* $\phi_c=0.50$	Geotechnical Resistance $\phi_{\text{stat}}=0.45$	Drivability Resistance $\phi_{\text{dyn}}=0.65$	Governing Resistance
HP 12x53	380	251	319	251
HP 12x74	534	350	386	350
HP 14x73	528	312	384	312
HP 14x89	644	379	447	379
HP 14x117	849	497	494	494

* based on preliminary assumption of $t=48''$ and $K=1.0$

Table 7-2 – Factored Axial Resistances for H-Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial geotechnical resistance is less than the factored axial structural resistance for four of the piles and the factored axial drivability resistance is less than the factored axial structural resistance for the 14 x 117 pile. Therefore, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored resistance shown in the last column of Table 7-2 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor $\phi_c=0.7$ and the flexural resistance factor $\phi_f=1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.12.2.2.1-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2.

7.1.2 Service and Extreme Limit States

For the service and extreme limit states resistance factors, ϕ , of 1.0 are recommended for structural and geotechnical pile resistances. It is the responsibility of the structural engineer to recalculate P_n based on refined elastic critical buckling resistance (P_e) evaluations.

The calculated factored axial structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 7-3 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* $\phi=1.0$	Geotechnical Resistance $\phi=1.0$	Drivability Resistance $\phi=1.0$	Governing Resistance
HP 12x53	759	558	491	491
HP 12x74	1069	779	594	594
HP 14x73	1055	694	591	591
HP 14x89	1287	843	688	688
HP 14x117	1698	1104	760	760

*based on preliminary assumption of $t=48''$ and $K=1.0$

**Table 7-3 - Factored Axial Resistances for H-Piles at the
Service and Extreme Limit States**

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the service and extreme limit states should not exceed the factored drivability resistance shown in the last column of Table 7-3 above.

7.1.3 Lateral Pile Resistance

Lateral loads may be reacted by plumb or battered piles. The designer should perform a series of lateral pile resistance analyses to evaluate pile top deflections and bending stresses under strength limit state design lateral loads using L-Pile[®] software or FB-Pier[®] software. Similar software for analyzing pile response under lateral loads where the nonlinear soil behavior is modeled using soil-resistance (p-y) curves may be used. These analyses should take into consideration pile batter, if any. Lacking a performance criteria at this time for allowable lateral displacements at the pile head, the designer should consider performing lateral pile analyses to determine maximum factored lateral loads permissible based on the allowable displacement criteria. Furthermore, the designer should evaluate the associated pile stresses under factored lateral loads.

Recommended geotechnical parameters for generation of p-y curves in lateral pile analyses are provided in Tables 7-4 and 7-5 below. In general, the model developed should emulate the soil

at the site by using the soil layers (referenced in Tables 7-4 and 7-5 by elevations) and appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. It is recommended that the analyses be conducted assuming a fixed pile-head boundary condition.

Soil Layer	Elevation of Soil Layer at Abutment No. 1 (feet)	Elevation of Soil Layer at Abutment No. 2 (feet)	Water Table Condition	Effective Unit Weight lbs/in ³ (lbs/ft ³)
Sand Fill	35.5 to 24.0	34.4 to 25.4	Above	0.0723 (125)
Upper Native Sand	24.0 to 9.5	25.4 to 16.4	Above	0.0694 (120)
Clayey Silt	9.5 to 2.5	16.4 to 5.4	Below	0.0307 (53)
Lower Native Sand (loose to medium dense)	2.5 to -7.5	5.4 to -4.6	Below	0.0336 (58)
Lower Native Sand (dense to very dense)	-7.5 to -19.8	-4.6 to -19.1	Below	0.0336 (58)

Table 7-4 - Soil Parameters for Generation of Soil-Resistance (p-y) Curves

Soil Layer	k _s (lb/in ³)	Cohesion (lb/in ²)	E ₅₀ for clays	Friction Angle
Sand Fill	90	-	-	32°
Upper Native Sand	60	-	-	32°
Clayey Silt	30	375	0.020	-
Lower Native Sand (loose to medium dense)	20	-	-	30°
Lower Native Sand (dense to very dense)	125	-	-	36°

Table 7-5 - Soil Parameters for Generation of Soil-Resistance (p-y) Curves

7.1.4 Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each arch stem wall/pile cap. The first pile driven at each arch stem wall/pile cap should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance

when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Arch Stem Wall/Pile Cap

Arch stem walls/pile caps shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported arch stem wall/pile caps at the strength limit state shall consider pile stability and structural resistance.

A resistance factor of $\phi = 1.0$ shall be used to assess arch stem wall/pile cap design at the service limit state including: settlement, horizontal movement, overall stability and scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65. Extreme limit state design checks for arch stem wall/pile cap supported on H-piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ , for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

Calculation of passive earth pressure for resisting lateral forces/thrust from the arch should assume a K_p of 3.25, anticipating small footing movements and a resistance factor (ϕ_{ep}) of 0.5 per LRFD Table 10.5.5.2.2-1. Should the arch stem wall/pile cap rotation (y/H) exceed 0.005, a Coulomb passive earth pressure coefficient (K_p) of 6.73 is recommended. A load factor for passive earth pressure is not specified in LRFD. For designing the pile cap reinforcing steel to resist passive earth pressure, use a maximum load factor, $\gamma_{EH} = 1.50$.

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

All arch stem wall/pile cap design shall include a drainage system behind the arch stem wall/pile cap to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the arch stem wall/pile cap and wingwalls with weep holes will provide adequate drainage.

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

7.3 Precast Concrete Modular Block Retaining Wall

Precast Concrete Modular Gravity (PCMG) walls will be constructed on all four corners of the bridge to retain the roadway section and minimize impacts. These walls shall be designed by a

Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635 which is included in Appendix D found at the end of this report.

The PCMG wall designs shall consider a live load surcharge estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic = 0 feet	Distance from wall pressure surface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥ 20	2.0	2.0

Table 7-6 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Bearing resistance for PCMG walls founded on a leveling slab on native silt shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 7 ksf for bases from 8.5 to 12 feet wide. The bearing resistance factor, ϕ_b , for spread footings on soil is 0.45. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for PCMG bottom unit of the PCMG wall shall be checked for the extreme limit state with a resistance factor of 1.0. The PCMG units shall be designed so that the nominal bearing resistance after the design scour event provides adequate resistance to support the unfactored strength limit state loads with a resistance factor of 1.0. The overall stability of the wall system should be investigated at the Service I Load Combination with a resistance factor ϕ , of 0.65.

The designer shall apply a sliding resistance factor ϕ_τ of 0.90 to the nominal sliding resistance of precast concrete wall segments founded on sand. For footings on soil the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth ($1/4^{\text{th}}$) of the footing dimensions in either direction (LRFD Article 10.6.3.3). Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of $\tan 30^\circ$ at the foundation soil to soil infill interface and a maximum frictional coefficient of $0.8 \times (\tan 30^\circ)$ at the foundation soil to concrete module interface. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

7.4 Scour and Riprap

Grain size analyses were performed on soil samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analysis. 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, $D_{50} = 0.64$ mm
- Average diameter of particle at 95 percent passing, $D_{95} = 24.5$ mm
- Soil Classification AASHTO Soil Type A-1-b, A-2-4 and A-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 and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch stem wall/pile caps and wingwalls. 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 Standard Specifications 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.

7.5 Settlement

The grade of the existing bridge approaches will be maintained in the replacement of the structure. Post-construction settlements are anticipated to be negligible. Any settlement of the arch stem wall/pile cap will be due to the elastic compression of the piling and will be negligible.

7.6 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1195 F-degree days. In a granular soil with a water content of approximately 10%, this correlates to a frost depth of approximately 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade

for frost protection. See Appendix C- Calculations at the end of this report for supporting documentation.

7.7 Seismic Design Considerations

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

7.8 Construction Considerations

Construction of the arch stem wall/pile cap will require soil excavation and partial or full removal of the existing abutments. Construction activities may require cofferdams and earth support systems. The removal of the existing abutments may require the replacement of excavated soils with compacted granular fill prior to pile driving.

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

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

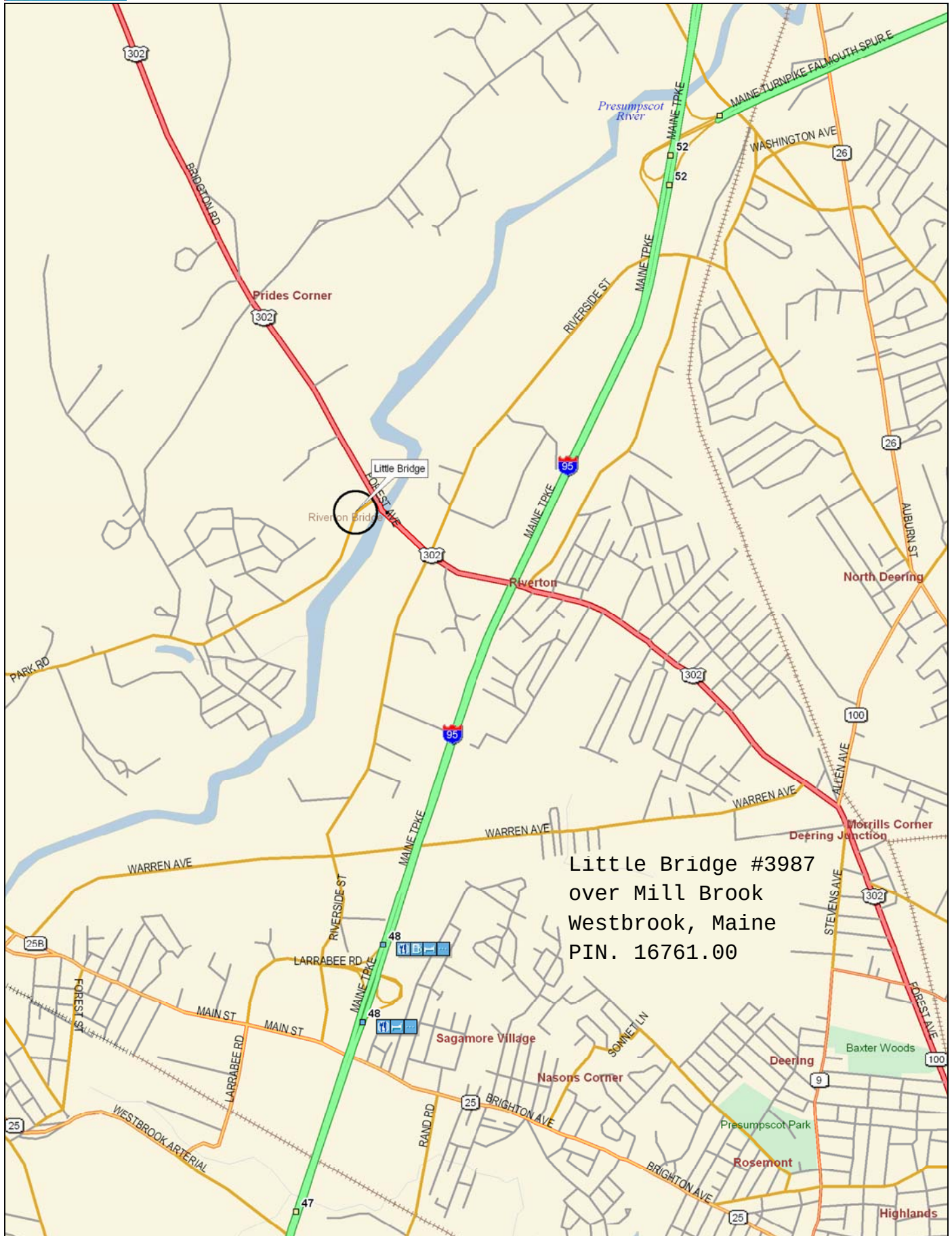
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements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

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Little Bridge in Westbrook 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 plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



[illegible][illegible]

Maine Department of Transportation				Project: Little Bridge #3887 over Mill Brook		Boring No.: BB-WMB-102	
Soil/Rock Exploration Log				Location: Westbrook, Maine		P.I.N.: 16761.00	
US CUSTOMARY UNITS							
Driller: MainDOT		Elevation (ft.): 34.4		Auger 10/001: 5' Dia.			
Operator: Giguere/Giglio		Datum: NAVD88		Sampler: Standard Split Spoon			
Logged By: B. Wither		CME 450		Hammer wt./fall: 140#/30"			
Date Started/Finished: 12/17/09/12/20/14/00		Drilling Method: Split Stem Auger		Core Barrels: np/2"			
Boring Location: B-WB-4, 12.7 ft.		Casing 10/001: N/A		Water Level: None Observed			
Hammer Efficiency Factor: 0.84		Hammer Type: Automatic ☒ Hydraulic ☐		Rope & Cathod ☐			
Definitions: S = Split Spoon Sample SA = Unconsolidated Split Spoon Sample attempt NS = Not a Split Spoon Sample TH = Unconsolidated Thin Wall Tube Sample attempt TS = Insitu Vane Test Result NP = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SA = Split Stem Auger NS = Not a Split Stem Auger EC = Electric Core WH = Weight of 100lb. Hammer NP = Not a Pusher Penetration Test WH = Weight of rod or casing WH = Weight of one section S₁₀₀ = Insitu Field Vane Shear Strength Test S₁₀ = Pusher Torque Shear Strength Test S₁ = Uncoring Compressive Strength Test S₁₀₀ = Rock Shear Strength Hammer Efficiency Factor = Annual Calibration Value S₁₀₀ = SPT Empirically corrected for hammer efficiency & Split Stem Analysis NP = Hammer Efficiency Factor/RODM=uncorrected S₁₀₀ = Consolidation Test S₁₀₀ = Lab Vane Shear Strength Test EC = water content, percent L₁ = Liquid Limit P₁ = Plasticity Index NP = Plasticity Index							
Sample Information				Visual Description and Remarks			
Depth (ft.)	Sample No.	Pen./Rec. In.	Sample Depth (ft.)	Pen./Rec. In.	Sample Depth (ft.)	Pen./Rec. In.	Sample Depth (ft.)
0	10	24/12	1.00 - 3.00	11/14/8/13	22	31	33.85
5	20	24/18	5.00 - 7.00	5/7/11/24	24	34	29.40
10	30	24/13	10.00 - 12.00	6/5/4/5	9	13	25.40
15	40	6/6	15.00 - 15.50	50 16"	---	---	18.90
20							
24							
Pavement 0-55 Brown, damp, dense, gravelly, fine to coarse SAND, trace silt, (FILL). 5-10 Olive-brown, moist, dense, fine to coarse SAND, little silt, little clay, little gravel, (FILL). 10-25 Light brown, damp, medium dense, fine to coarse SAND, little silt, little clay, little gravel, trace clay. 25-50 Olive-brown, wet, dense, fine to coarse SAND, little silt, little clay, little gravel. Bottom of Exploration at 15.50 feet below ground surface. Large Boulder REFUSAL, moved to BB-WMB-102A. Laboratory Testing Results: AASHTO and Unified Class GW24661 A-4, SC-SM MC10.05 GW24662 A-4, SC-SM MC10.05 GW24663 A-4, SC-SM MC10.05							
Stationation lines represent approximate boundaries between soil types. Transitions may be gradual. * Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made. Page 1 of 1 Boring No.: BB-WMB-102							

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					TERMS DESCRIBING DENSITY/CONSISTENCY																														
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES																															
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	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 <table><tr><th>Descriptive Term</th><th>Portion of Total</th></tr><tr><td>trace</td><td>0% - 10%</td></tr><tr><td>little</td><td>11% - 20%</td></tr><tr><td>some</td><td>21% - 35%</td></tr><tr><td>adjective (e.g. sandy, clayey)</td><td>36% - 50%</td></tr></table> <table><tr><th>Density of Cohesionless Soils</th><th>Standard Penetration Resistance N-Value (blows per foot)</th></tr><tr><td>Very loose</td><td>0 - 4</td></tr><tr><td>Loose</td><td>5 - 10</td></tr><tr><td>Medium Dense</td><td>11 - 30</td></tr><tr><td>Dense</td><td>31 - 50</td></tr><tr><td>Very Dense</td><td>> 50</td></tr></table>				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					
		Descriptive Term	Portion of Total																																
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Medium Dense	11 - 30																																		
Dense	31 - 50																																		
Very Dense	> 50																																		
(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines																																	
GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.																																	
SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines																																
	(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																																
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.	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. <table><tr><th>Consistency of Cohesive soils</th><th>SPT N-Value blows per foot</th><th>Approximate Undrained Shear Strength (psf)</th><th>Field Guidelines</th></tr><tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily Penetrates</td></tr><tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr><tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr><tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr><tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr><tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr></table>				Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	Hard	>30	over 4000	Indented by thumbnail with difficulty
		Consistency of Cohesive soils	SPT N-Value blows per foot					Approximate Undrained Shear Strength (psf)	Field Guidelines																										
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	Stiff	9 - 15	1000 - 2000					Indented by thumb with great effort																											
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail																																
Hard	>30	over 4000	Indented by thumbnail with difficulty																																
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					Rock Quality Designation (RQD): RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table><tr><th>Rock Mass Quality</th><th>RQD</th></tr><tr><td>Very Poor</td><td><25%</td></tr><tr><td>Poor</td><td>26% - 50%</td></tr><tr><td>Fair</td><td>51% - 75%</td></tr><tr><td>Good</td><td>76% - 90%</td></tr><tr><td>Excellent</td><td>91% - 100%</td></tr></table> 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				Rock Mass Quality	RQD	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%															
Rock Mass Quality	RQD																																		
Very Poor	<25%																																		
Poor	26% - 50%																																		
Fair	51% - 75%																																		
Good	76% - 90%																																		
Excellent	91% - 100%																																		
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information					Sample Container Labeling Requirements: <table><tr><td>PIN</td><td>Blow Counts</td></tr><tr><td>Bridge Name / Town</td><td>Sample Recovery</td></tr><tr><td>Boring Number</td><td>Date</td></tr><tr><td>Sample Number</td><td>Personnel Initials</td></tr><tr><td>Sample Depth</td><td></td></tr></table>				PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth																		
PIN	Blow Counts																																		
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Sample Number	Personnel Initials																																		
Sample Depth																																			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Little Bridge #3987 over Mill Brook Location: Westbrook, Maine		Boring No.: BB-WMB-101 PIN: 16761.00																																																																																																																																																																																																																								
Driller: MaineDOT			Elevation (ft.) 35.5		Auger ID/OD: 5" Solid Stem																																																																																																																																																																																																																									
Operator: Giguere/Giles			Datum: NAVD88		Sampler: Standard Split Spoon																																																																																																																																																																																																																									
Logged By: B. Wilder			Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																									
Date Start/Finish: 12/17,22/09			Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"																																																																																																																																																																																																																									
Boring Location: 7+43.3, 9.2 Lt.			Casing ID/OD: HW & NW		Water Level*: 20.0' bgs.																																																																																																																																																																																																																									
Hammer Efficiency Factor: 0.84			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																											
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Little Bridge #3987 over Mill Brook Location: Westbrook, Maine				Boring No.: BB-WMB-101 PIN: 16761.00			
Driller: MaineDOT				Elevation (ft.): 35.5				Auger ID/OD: 5" Solid Stem			
Operator: Giguere/Giles				Datum: NAVD88				Sampler: Standard Split Spoon			
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 12/17,22/09				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"			
Boring Location: 7+43.3, 9.2 Lt.				Casing ID/OD: HW & NW				Water Level*: 20.0' bgs.			
Hammer Efficiency Factor: 0.84				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR/C = weight of rods or casing WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N_{uncorrected} = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N_{uncorrected} S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
25	6D	24/20	25.00 - 27.00	3/2/2/2	4	6		9.50		Brown, wet, loose, silty, medium to coarse SAND, wood. Grey, wet, medium stiff, Clayey SILT, little sand, trace gravel, plastic.	G#241455 A-4, CL-ML WC=42.5%
30	1U	24/18	30.00 - 32.00	WOR/CHP				2.50		cHydraulic Push Grey, wet, very soft, SILT, some clay, some sand (layer in bottom of tube), trace gravel, plastic. Failed 55x110 mm vane attempt.	G,C#241456 A-4, CL-ML WC=29.2% LL=22 PL=17 PI=5
35	MD	24/0	35.00 - 37.00	2/1/2/1	3	4	CHP			Grey, wet, loose, fine SAND (in wash). Changed to NW Casing at 35.0' bgs. Grey, saturated, loose, fine to coarse SAND, little gravel, little silt, trace clay.	G#241457 A-2-4, SC-SM WC=11.3%
							4				
40	7D	24/10	39.00 - 41.00	4/2/1/3	3	4	4			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, trace clay. Grey, wet, dense, fine to coarse SAND, little gravel, little silt.	G#241458 A-1-b, SC-SM WC=9.3%
							4				
							102				
45							41				G#241459 A-2-4, SM
							49				
							47				
50							66				
	9D	24/12	49.00 - 51.00	13/21/12/6	33	46	39				
Remarks: 500# down pressure on Core Barrel.											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3	
* 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-WMB-101	

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS					Project: Little Bridge #3987 over Mill Brook Location: Westbrook, Maine			Boring No.: BB-WMB-102 PIN: 16761.00																																																																																																																																																																																																											
Driller: MaineDOT			Elevation (ft.) 34.4			Auger ID/OD: 5" Dia.																																																																																																																																																																																																													
Operator: Giguere/Giles			Datum: NAVD88			Sampler: Standard Split Spoon																																																																																																																																																																																																													
Logged By: B. Wilder			Rig Type: CME 45C			Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																													
Date Start/Finish: 12/17/09; 12:30-14:00			Drilling Method: Solid Stem Auger			Core Barrel: NQ-2"																																																																																																																																																																																																													
Boring Location: 8+09.4, 12.7 Rt.			Casing ID/OD: N/A			Water Level*: None Observed																																																																																																																																																																																																													
Hammer Efficiency Factor: 0.84			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																
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	4D	6/6	15.00 - 15.50	50 (6")	---			18.90		Olive-brown, wet, dense, fine to coarse SAND, little silt, little clay, little gravel.		G#241463 A-4, SC-SM WC=11.8%																																																																																																																																																																																																							
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[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Little Bridge #3987 over Mill Brook Location: Westbrook, Maine				Boring No.: BB-WMB-102A PIN: 16761.00									
Driller: MaineDOT				Elevation (ft.): 34.4				Auger ID/OD: 5" Solid Stem									
Operator: Giguere/Giles				Datum: NAVD88				Sampler: Standard Split Spoon									
Logged By: B. Wilder				Rig Type: CME 45C				Hammer Wt./Fall: 140#/30"									
Date Start/Finish: 12/17,22/09				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"									
Boring Location: 8+03.6, 12.8 Rt.				Casing ID/OD: HW				Water Level*: 21.0' bgs.									
Hammer Efficiency Factor: 0.84				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐													
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR/C = weight of rods or casing WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N_{uncorrected} = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N_{uncorrected} S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																	
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.						
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)									
25	2D	24/20	25.50 - 27.50	2/1/1/1	2	3	53	5.40		Grey, wet, soft, CLAY, some silt, little sand.	G#241465 A-6, CL WC=33.7% LL=28 PL=18 PI=10						
							39										
							31										
							24										
30							24						Grey, saturated, loose, fine to coarse SAND, little silt, trace clay, trace gravel. Failed 55x110 mm vane attempt.	G#241466 A-2-4, SC-SM WC=18.7%			
	3D	24/18	30.00 - 32.00	2/2/2/2	4	6	23										
	MV		30.00 - 30.37	Would not push			24										
							20										
35							35									Grey, wet, medium dense, fine to coarse SAND, little gravel, little silt.	G#241467 A-1-b, SM WC=10.1%
							26										
	4D	24/12	35.00 - 37.00	7/6/5/5	11	15	29										
							22										
40							23		Cobble from 40.0-40.6' bgs, Roller Coned ahead to 41.0' bgs.	G#241468 A-2-4, SM WC=12.8%							
							42										
							56										
							102										
45	5D	24/10	41.00 - 43.00	14/16/7/7	23	32	55					Grey, wet, dense, fine to coarse SAND, little gravel, little silt.					
							56										
							56										
							71										
50							95										
							126										
							128										
							197										
							220										
Remarks: 500# down pressure on Core Barrel.																	
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3							
* 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-WMB-102A							

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS						Project: Little Bridge #3987 over Mill Brook							Boring No.: BB-WMB-102A																																																																																																																																																																																																																																																																										
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<table><thead><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/AASHTO and Unified Class</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr></thead><tbody><tr><td rowspan="4">50</td><td>6D</td><td>24/11</td><td>50.00 - 52.00</td><td>37/17/10/17</td><td>27</td><td>38</td><td>119</td><td rowspan="12"> -19.10 -24.10 </td><td rowspan="12"> </td><td rowspan="12"> Grey, wet, dense, fine to coarse SAND, little silt, trace gravel. a150 blows for 0.5'. Top of Bedrock at Elev. -19.1'. Bedrock: Grey, fine grained, slightly metamorphosed, unweathered, sandy, MUDSTONE, with quartz bands. Bedding dips at 20 to 30 degrees. (Cushing Formation). Rock Mass Quality = Fair R1:Core Times (min:sec) 53.5-54.5' (1:58) 54.5-55.5' (2:35) 55.5-56.5' (1:37) 56.5-57.5' (2:05) 57.5-58.5' (2:28) 87% Recovery Bottom of Exploration at 58.50 feet below ground surface. </td><td rowspan="12"> G#241469 A-2-4, SM WC=10.4% </td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>126</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>122</td></tr><tr><td>R1</td><td>60/52</td><td>53.50 - 58.50</td><td>RQD = 58%</td><td></td><td></td><td>a150</td><td></td></tr><tr><td rowspan="4">55</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">60</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">65</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">70</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td rowspan="4">75</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr></tbody></table>																			Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	50	6D	24/11	50.00 - 52.00	37/17/10/17	27	38	119	-19.10 -24.10		Grey, wet, dense, fine to coarse SAND, little silt, trace gravel. a150 blows for 0.5'. Top of Bedrock at Elev. -19.1'. Bedrock: Grey, fine grained, slightly metamorphosed, unweathered, sandy, MUDSTONE, with quartz bands. Bedding dips at 20 to 30 degrees. (Cushing Formation). Rock Mass Quality = Fair R1:Core Times (min:sec) 53.5-54.5' (1:58) 54.5-55.5' (2:35) 55.5-56.5' (1:37) 56.5-57.5' (2:05) 57.5-58.5' (2:28) 87% Recovery Bottom of Exploration at 58.50 feet below ground surface.	G#241469 A-2-4, SM WC=10.4%								126								122	R1	60/52	53.50 - 58.50	RQD = 58%			a150		55																																									60																																									65																																									70																																									75																																								
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Appendix B

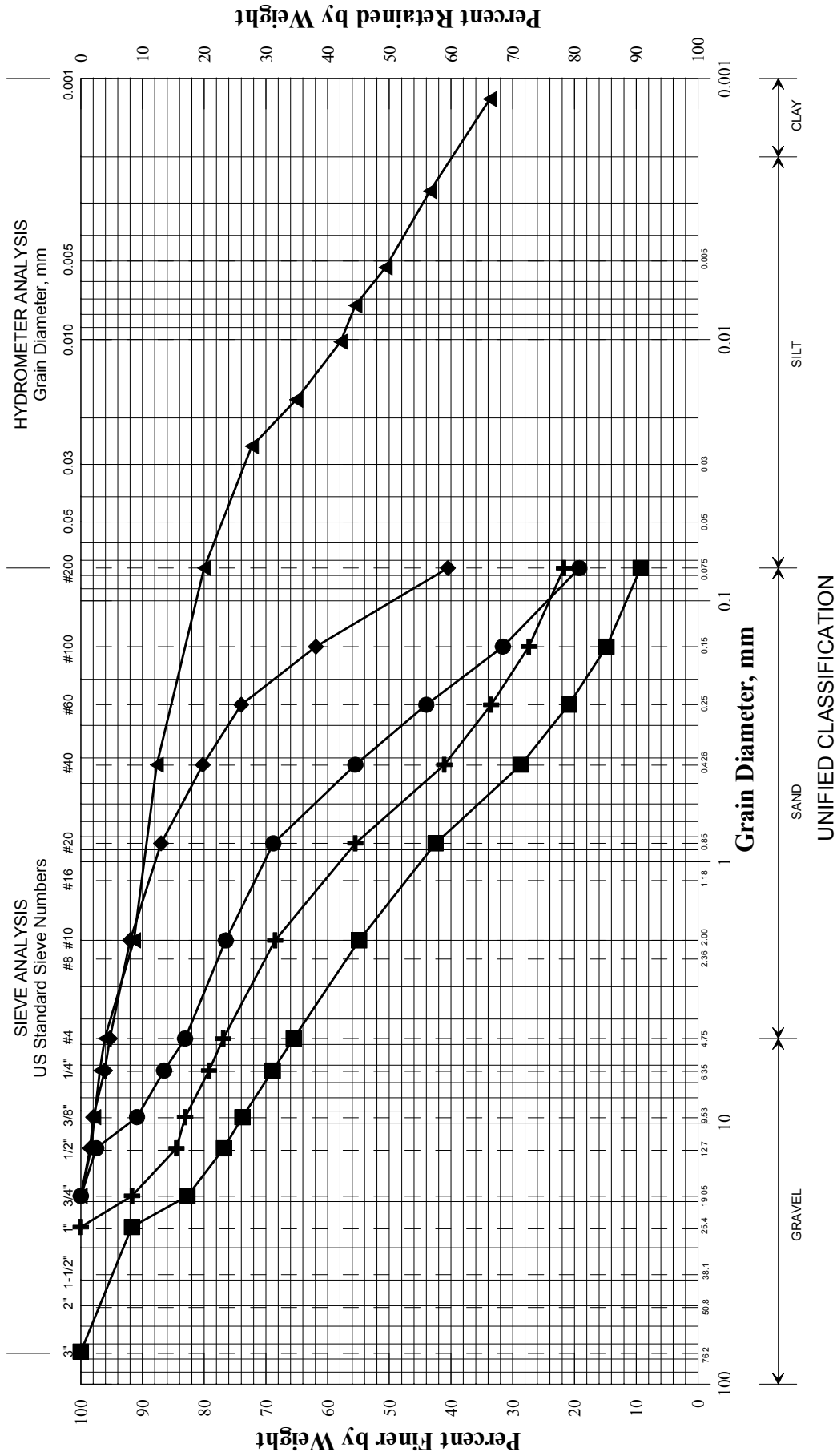
Laboratory Data

Town(s): Westbrook Project Number: 16761.00

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

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

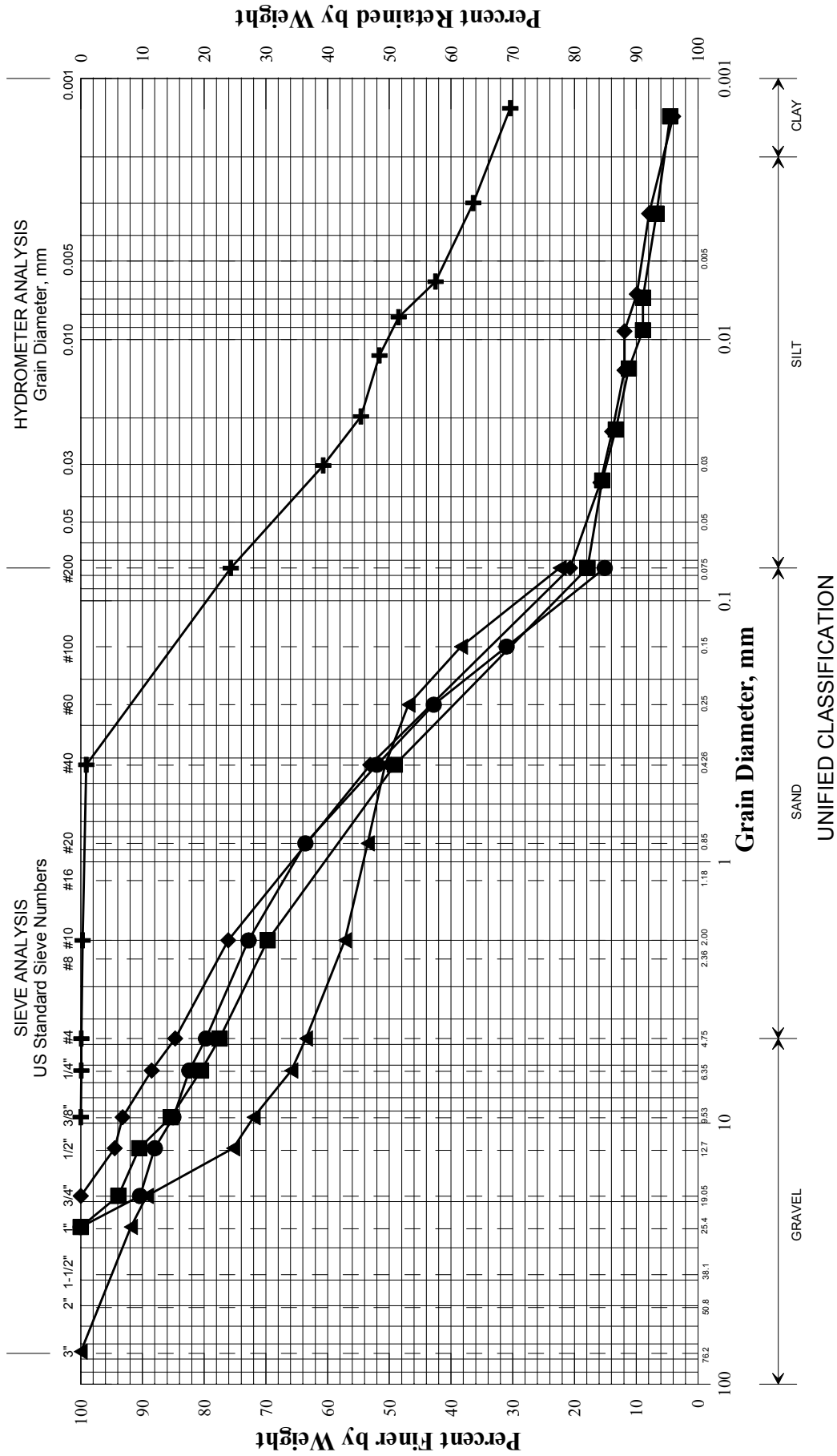
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-101/2D	7+43.3	9.2 LT	5.0-7.0	SAND, some gravel, some silt.	6.7			
◆	BB-WMB-101/3D	7+43.3	9.2 LT	10.0-12.0	Silty SAND, trace gravel.	13.1			
■	BB-WMB-101/4D	7+43.3	9.2 LT	15.0-17.0	SAND, some gravel, trace silt.	5.1			
●	BB-WMB-101/5D	7+43.3	9.2 LT	20.5-21.0	SAND, little silt, little gravel.	10.2			
▲	BB-WMB-101/6D	7+43.3	9.2 LT	25.0-27.0	Clayey SILT, little sand, trace gravel.	42.5			
×									

PIN	
016761.00	
Town	
Westbrook	
Reported by/Date	
WHITE, TERRY A	2/1/2010

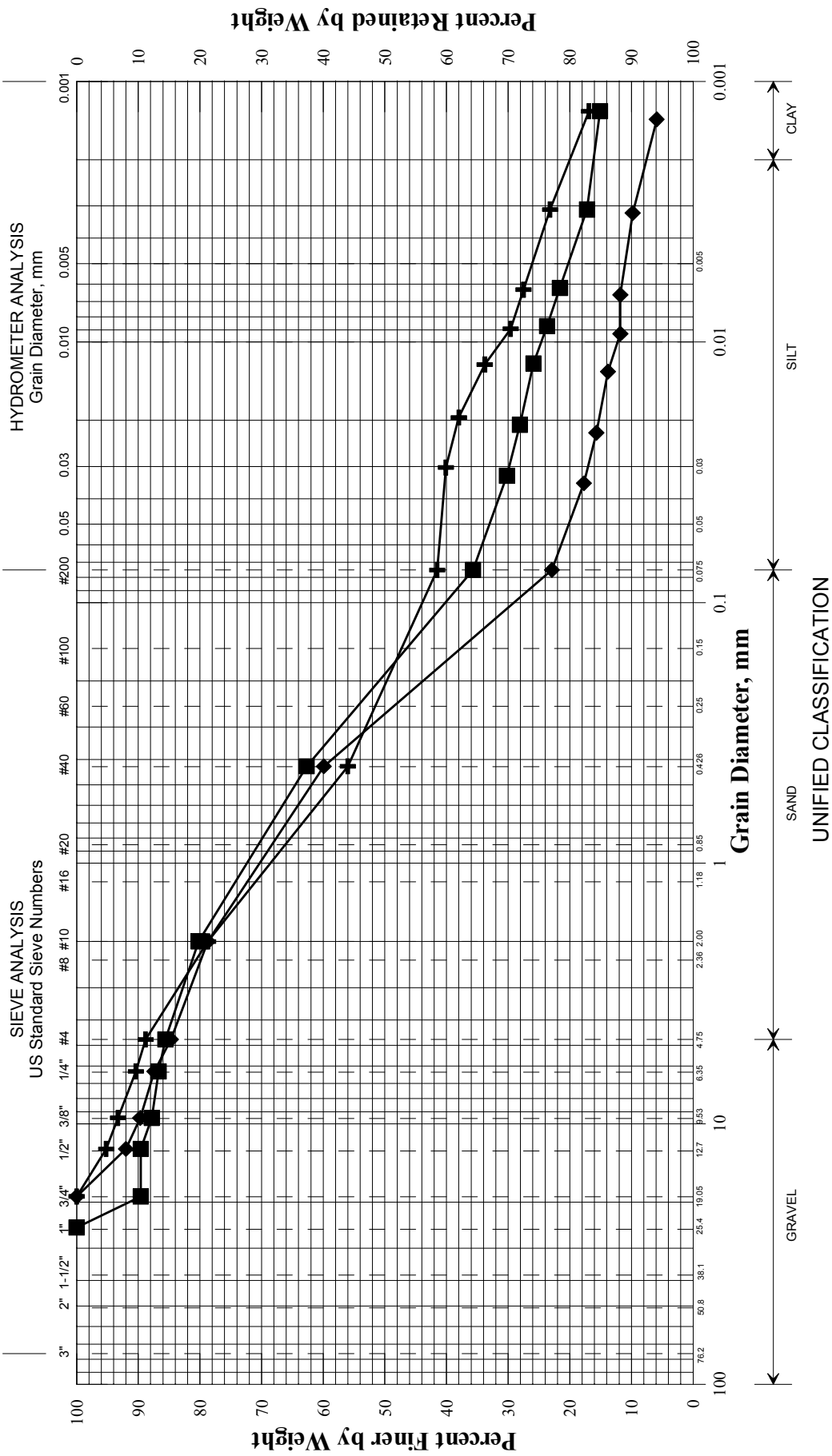
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-101/1U	7+43.3	9.2 LT	30.0-32.0	SILT, some clay, some sand, trace gravel.	29.2	22	17	5
◆	BB-WMB-101/7D	7+43.3	9.2 LT	39.0-41.0	SAND, little gravel, little silt, trace clay.	11.3			
■	BB-WMB-101/8D	7+43.3	9.2 LT	44.0-46.0	SAND, some gravel, little silt, trace clay.	9.3			
●	BB-WMB-101/9D	7+43.3	9.2 LT	49.0-51.0	SAND, little gravel, little silt.	11.1			
▲	BB-WMB-101/10D	7+43.3	9.2 LT	54.0-55.3	Gravelly SAND, some silt.	9.4			
×									

PIN	016761.00
Town	Westbrook
Reported by/Date	WHITE, TERRY A 2/1/2010

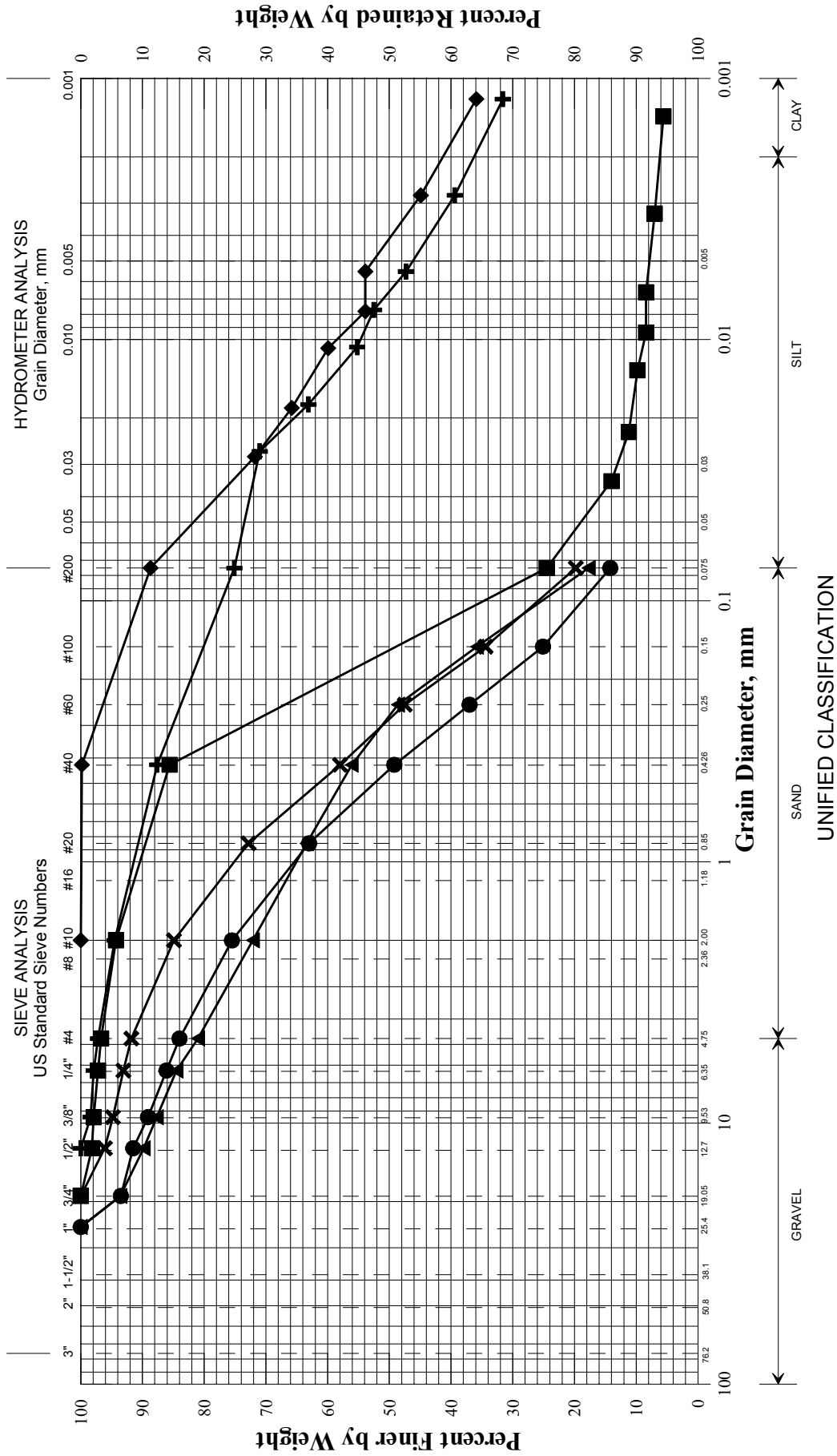
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-102/2D	8+09.4	12.7 RT	5.0-7.0	SAND, some silt, little clay, little gravel.	10.6			
◆	BB-WMB-102/3D	8+09.4	12.7 RT	10.0-12.0	SAND, little silt, little gravel, trace clay.	7.8			
■	BB-WMB-102/4D	8+09.4	12.7 RT	15.0-15.5	SAND, little silt, little clay, little gravel.	11.8			
●									
▲									
×									

PIN	
016761.00	
Town	
Westbrook	
Reported by/Date	
WHITE, TERRY A	2/1/2010

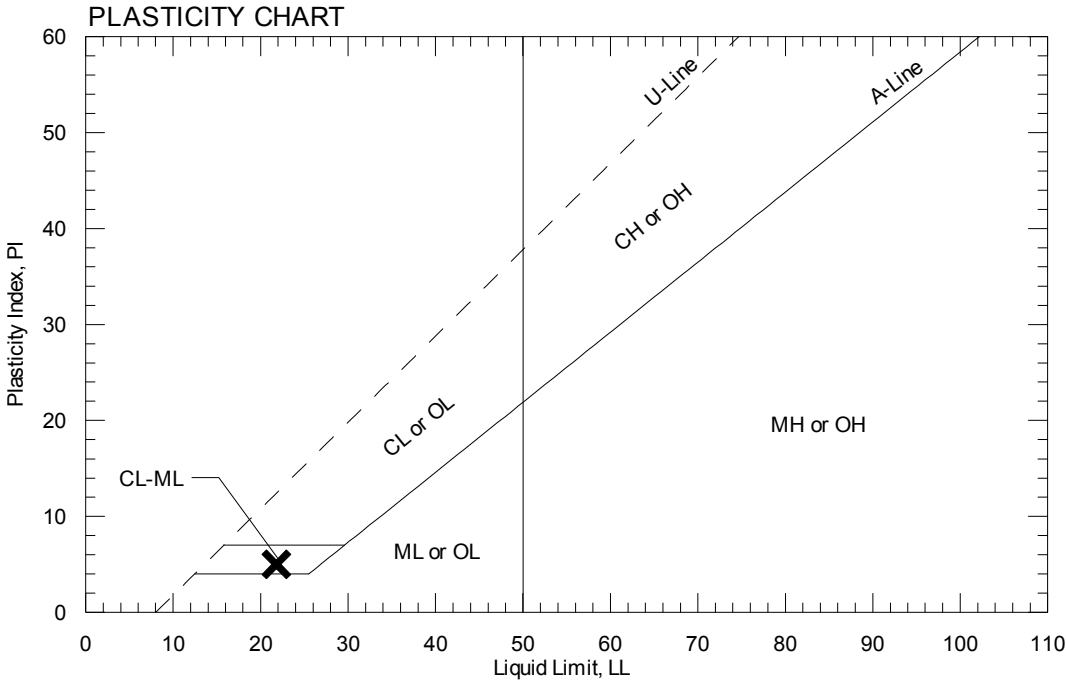
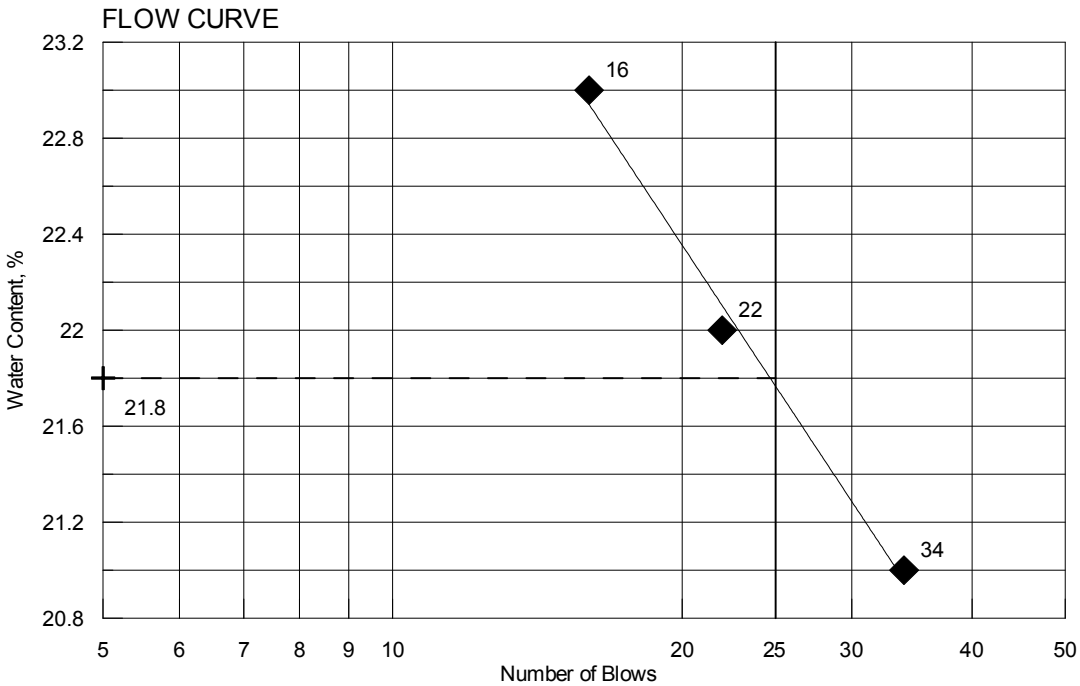
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



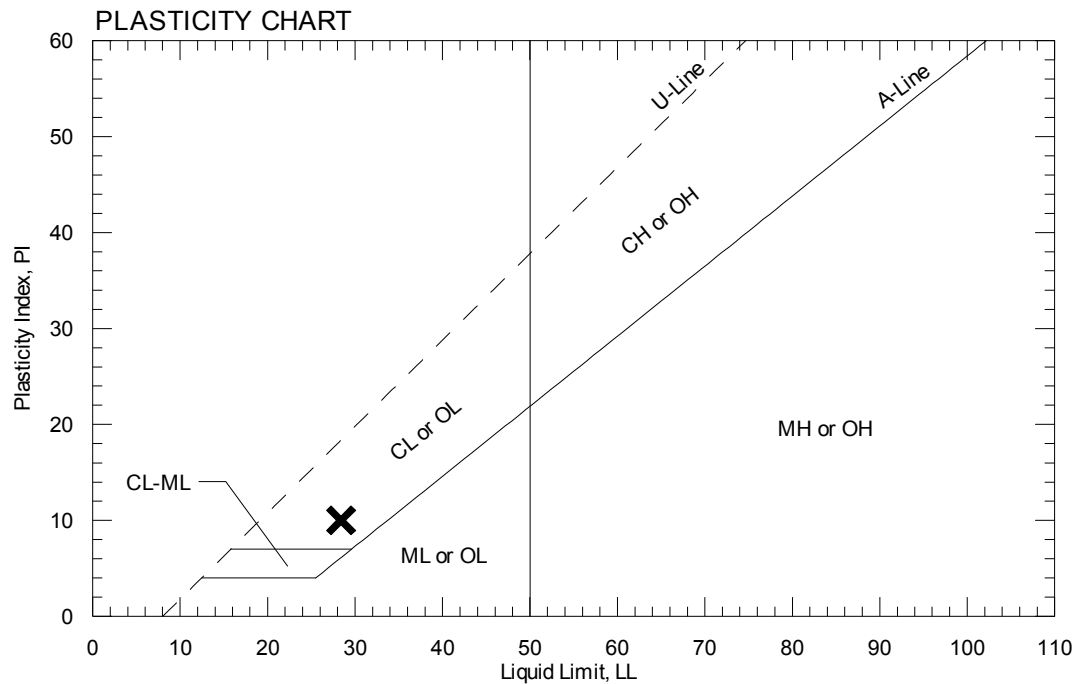
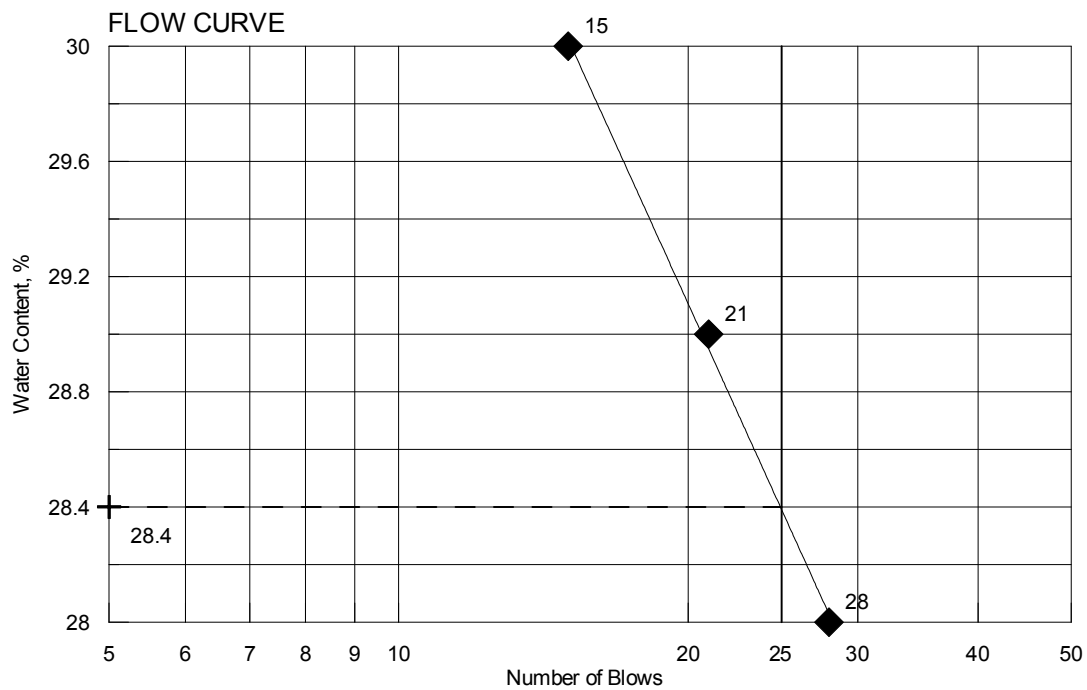
Boring/Sample No.		Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-102A/1D	8+03.6	12.8 RT	20.0-22.0	Clayey SILT, some sand, trace gravel.	26.2	28	18	10
◆	BB-WMB-102A/2D	8+03.6	12.8 RT	25.5-27.5	CLAY, some silt, little sand.	33.7			
■	BB-WMB-102A/3D	8+03.6	12.8 RT	30.0-32.0	SAND, little silt, trace clay, trace gravel.	18.7			
●	BB-WMB-102A/4D	8+03.6	12.8 RT	35.0-37.0	SAND, little gravel, little silt.	10.1			
▲	BB-WMB-102A/5D	8+03.6	12.8 RT	41.0-43.0	SAND, little gravel, little silt.	12.8			
×	BB-WMB-102A/6D	8+03.6	12.8 RT	50.0-52.0	SAND, little silt, trace gravel.	10.4			

Reported by/Date	
Westbrook	016761.00
Town	PIN
WHITE, TERRY A	2/1/2010

TOWN	Westbrook	Reference No.	241456
PIN	016761.00	Water Content, %	29.2
Sampled	12/21/2009	Plastic Limit	17
Boring No./Sample No.	BB-WMB-101/1U	Liquid Limit	22
Station	7+43.3	Plasticity Index	5
Depth	30.0-32.0	Tested By	BBURR

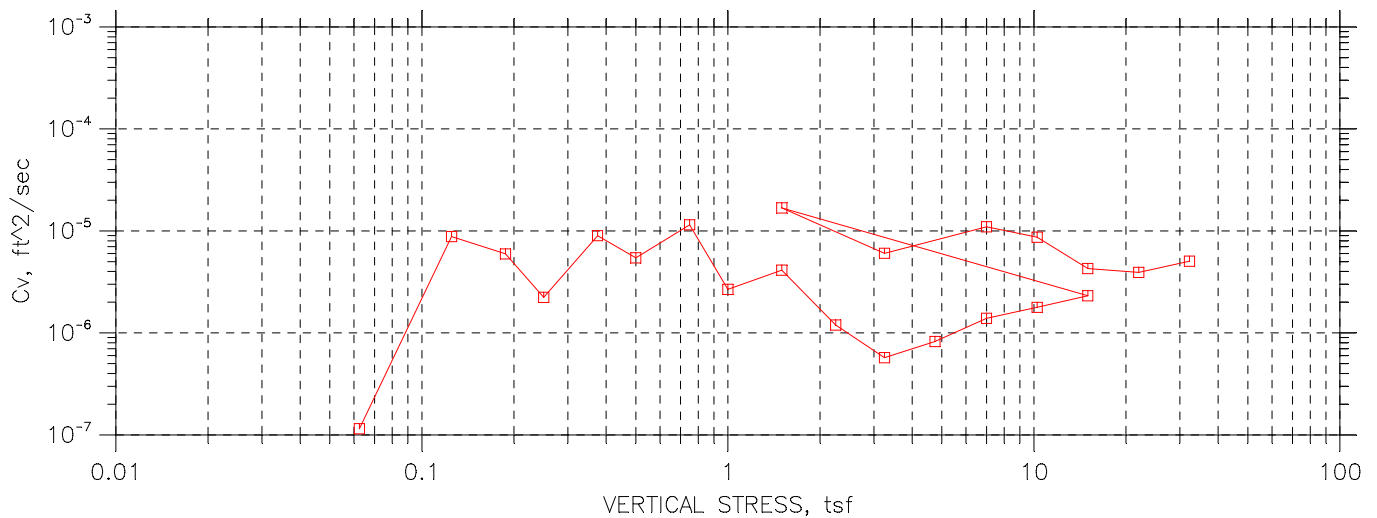
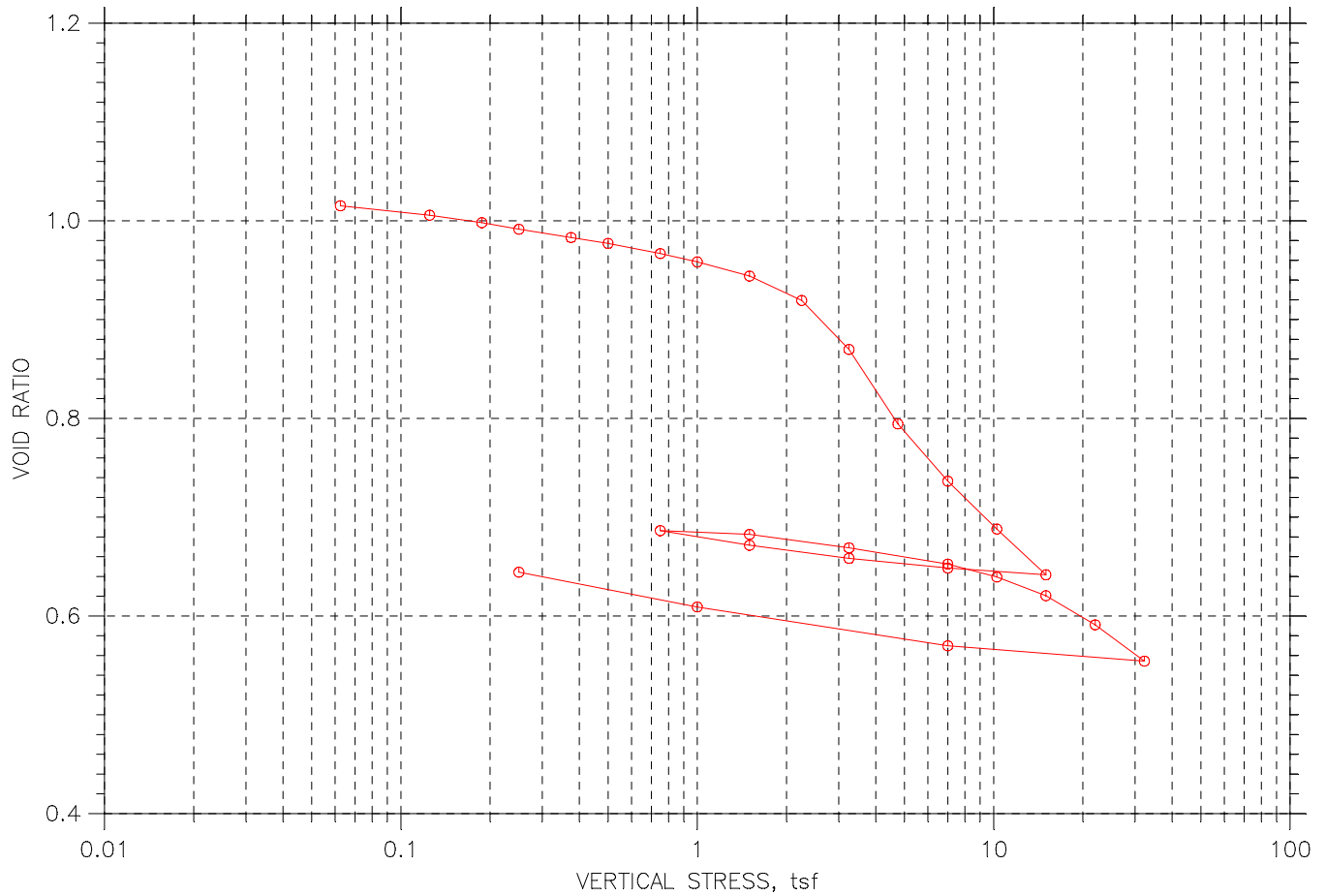


TOWN	Westbrook	Reference No.	241465
PIN	016761.00	Water Content, %	33.7
Sampled	12/17/2009	Plastic Limit	18
Boring No./Sample No.	BB-WMB-102A/2D	Liquid Limit	28
Station	8+03.6	Plasticity Index	10
Depth	25.5-27.5	Tested By	BBURR



CONSOLIDATION TEST DATA

SUMMARY REPORT



	Project: LITTLE BRIDGE	Location: WESTBROOK	Project No.: 016761.00
	Boring No.: BB-WMB-101	Tested By: Brian Fogg	Checked By: K Maguire
	Sample No.: 1U	Test Date: 1/8/10	Depth: 30-32 FT
	Test No.: 241456	Sample Type: Shelby Tube	Elevation: 5.5-3.5 FT
	Description: Grey, SILT, some clay, some sand, trace gravel.		
	Remarks:		

CONSOLIDATION TEST DATA

Project: LITTLE BRIDGE
 Boring No.: BB-WMB-101
 Sample No.: 1U
 Test No.: 241456

Location: WESTBROOK
 Tested By: Brian Fogg
 Test Date: 1/8/10
 Sample Type: Shelby Tube

Project No.: 016761.00
 Checked By: K Maguire
 Depth: 30-32 FT
 Elevation: 5.5-3.5 FT

Soil Description: Grey, SILT, some clay, some sand, trace gravel.
 Remarks:

Measured Specific Gravity: 2.74
 Initial Void Ratio: 1.02
 Final Void Ratio: 0.64

Liquid Limit: 22
 Plastic Limit: 17
 Plasticity Index: 5

Initial Height: 1.02 in
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	45	RING	RING	67
Wt. Container + Wet Soil, kg	0.22064	0.4084	0.39739	0.20206
Wt. Container + Dry Soil, kg	0.17945	0.3716	0.3716	0.1763
Wt. Container, kg	0.06404	0.26204	0.26204	0.06686
Wt. Dry Soil, kg	0.11541	0.10956	0.10956	0.10944
Water Content, %	35.69	33.59	23.54	23.54
Void Ratio	---	1.02	0.64	---
Degree of Saturation, %	---	90.27	100.09	---
Dry Unit Weight, pcf	---	84.703	104.02	---

CONSOLIDATION TEST DATA

Project: LITTLE BRIDGE
Boring No.: BB-WMB-101
Sample No.: 1U
Test No.: 241456

Location: WESTBROOK
Tested By: Brian Fogg
Test Date: 1/8/10
Sample Type: Shelby Tube

Project No.: 016761.00
Checked By: K Maguire
Depth: 30-32 FT
Elevation: 5.5-3.5 FT

Soil Description: Grey, SILT, some clay, some sand, trace gravel.
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	Log min	Coefficient of Consolidation		
							Sq.Rt. ft ² /sec	Log ft ² /sec	Ave. ft ² /sec
1	0.0625	0.002132	1.015	0.21	51.1	0.0	1.15e-007	0.00e+000	1.15e-007
2	0.125	0.006972	1.006	0.69	1.1	0.3	5.55e-006	2.12e-005	8.79e-006
3	0.188	0.01075	0.998	1.06	1.0	0.0	5.95e-006	0.00e+000	5.95e-006
4	0.25	0.01409	0.991	1.39	2.6	0.0	2.23e-006	0.00e+000	2.23e-006
5	0.375	0.01829	0.983	1.80	0.6	0.0	8.98e-006	0.00e+000	8.98e-006
6	0.5	0.02129	0.977	2.10	1.0	0.0	5.45e-006	0.00e+000	5.45e-006
7	0.75	0.02649	0.967	2.61	0.7	0.3	8.58e-006	1.73e-005	1.15e-005
8	1	0.03072	0.958	3.02	2.1	0.0	2.67e-006	0.00e+000	2.67e-006
9	1.5	0.03793	0.944	3.73	2.1	0.6	2.65e-006	9.37e-006	4.13e-006
10	2.25	0.05039	0.919	4.96	4.5	0.0	1.19e-006	0.00e+000	1.19e-006
11	3.25	0.07531	0.870	7.41	9.1	0.0	5.71e-007	0.00e+000	5.71e-007
12	4.75	0.1132	0.794	11.14	6.9	4.9	7.06e-007	9.90e-007	8.24e-007
13	7	0.1424	0.736	14.01	3.4	3.0	1.31e-006	1.48e-006	1.39e-006
14	10.3	0.1668	0.688	16.42	2.2	2.5	1.91e-006	1.67e-006	1.78e-006
15	15	0.1901	0.642	18.71	1.8	1.7	2.27e-006	2.36e-006	2.31e-006
16	7	0.1866	0.649	18.37	0.0	0.0	2.48e-004	0.00e+000	2.48e-004
17	3.25	0.1816	0.658	17.88	0.2	0.1	1.98e-005	3.12e-005	2.42e-005
18	1.5	0.175	0.672	17.23	1.1	0.0	3.69e-006	0.00e+000	3.69e-006
19	0.75	0.1675	0.686	16.49	2.4	2.7	1.71e-006	1.53e-006	1.62e-006
20	1.5	0.1695	0.683	16.68	0.2	0.0	1.68e-005	0.00e+000	1.68e-005
21	3.25	0.1763	0.669	17.35	0.7	0.6	5.64e-006	6.49e-006	6.04e-006
22	7	0.1846	0.652	18.17	0.5	0.2	8.29e-006	1.62e-005	1.10e-005
23	10.3	0.1912	0.639	18.82	0.7	0.2	5.68e-006	1.85e-005	8.69e-006
24	15	0.2007	0.621	19.75	1.3	0.5	2.97e-006	7.62e-006	4.27e-006
25	22	0.2155	0.591	21.21	1.1	0.8	3.37e-006	4.70e-006	3.92e-006
26	32.3	0.234	0.554	23.04	0.9	0.5	4.07e-006	6.62e-006	5.04e-006
27	7	0.2262	0.570	22.27	0.0	0.0	1.83e-004	0.00e+000	1.83e-004
28	1	0.2064	0.609	20.32	0.9	0.0	3.88e-006	0.00e+000	3.88e-006
29	0.25	0.1887	0.644	18.57	7.3	8.3	5.23e-007	4.63e-007	4.91e-007

Appendix C

Calculations

LIQUIDITY INDEX (LI):

$$\text{Liquidity Index} = \frac{\text{natural water content} - \text{Plastic Limit}}{\text{Liquid Limit} - \text{Plastic Limit}}$$

wc is close to LL Soil is normally consolidated
 wc is close to PL Soil is some-to-heavily over consolidated
 wc is intermediate Soil is over consolidated
 wc is greater than LL Soil is on the verge of being a viscous liquid when remolded

Sample	WC	LL	PL	PI	LI	
BB-WMB-101/1U	29.2	22	17	5	2.44	viscous liquid when remolded
BB-WMB-102/2D	33.7	28	18	10	1.57	viscous liquid when remolded

CONSOLIDATION TEST RESULTS:

BB-WMB-101 Sample 1U

$$\text{tsf} := g \cdot \left(\frac{\text{ton}}{\text{ft}^2} \right)$$

Determine in-situ over burden stress: Sample depth = 31.0 ft below ground surface

Groundwater table at 20.0 ft below ground surface Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.02$ Silt is overlain by:
 11.5 ft of sand fill at 125 pcf
 14.5 ft of sand at 125 pcf
 5.0 ft of clay/silt at 115 pcf

$$\sigma'_{v0} := 20.0 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 6.0 \cdot \text{ft} \cdot (125 - 62.4) \cdot \text{pcf} + 5.0 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$

$$\sigma'_{v0} = 3139 \cdot \text{psf} \quad \text{or} \quad \sigma'_{v0} = 1.569 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.3 \cdot \text{tsf}$

Determine OCR: $\text{OCR} := \frac{\sigma'_p}{\sigma'_{v0}}$ OCR = 1.4656 *over consolidated*

Determine C_c : from consolidation curve and lab results:

$$\begin{aligned}
 p_1 &:= 3.25 \cdot \text{tsf} & e_1 &:= 0.870 \\
 p_2 &:= 7 \cdot \text{tsf} & e_2 &:= 0.736 \\
 C_c &:= \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} & C_c &= 0.4021
 \end{aligned}$$

Determine C'_c : from consolidation curve and lab results:

$$\begin{aligned}
 \varepsilon_1 &:= \frac{7.41}{100} & \varepsilon_2 &:= \frac{14.01}{100} & \text{strain is given in percent} \\
 C'_c &:= \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} & C'_c &= 0.1981 \quad \text{or:} & C'_c &:= \frac{C_c}{1 + e_0} & C'_c &= 0.1991
 \end{aligned}$$

Determine C_r :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 0.683 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.652$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0463$$

Arch Foundations: Driven H-piles

Ref: AASHTO LRFD Bridge Design
Specifications 5th Edition 2010

Axial Structural Resistance of H-piles

Look at the following piles:

HP 12 x 53

HP 12 x 74

HP 14 x 73

HP 14 x 89

HP 14 x 117

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2$ yield strength: $F_y := 50 \cdot \text{ksi}$

Determine equivalent yield resistance $P_o = QF_yA_s$ LRFD Article 6.9.4.1.1

$Q := 1.0$ LRFD Article 6.9.4.2 $F_y = 50 \cdot \text{ksi}$

$P_o := Q \cdot F_y \cdot A_s$

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

Determine elastic critical buckling resistance: $P_e = \pi^2 EA_s / (Kl/r_s)^2$ LRFD eq. 6.9.4.1.2-1

$E = \text{steel modulus}$ $E := 29000 \cdot \text{ksi}$

$K = \text{effective length factor}$ $K_{\text{eff}} := 1.0$ LRFD Table C4.6.2.5-1 (assume fixed head)

$l = \text{unbraced length}$ $l_{\text{unbraced}} := 48 \cdot \text{in}$

$r_s = \text{radius of gyration}$ $r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot \text{in}$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

LRFD Article C6.9.4.1.2 states that the critical flexural buckling resistances be calculated about the x- and y-axes with the smaller value taken as P_e .

Use y-axis as this results in the smaller value.

LRFD eq. 6.9.4.1.2-1

$$P_e := \left[\frac{\pi^2 \cdot E}{\left(\frac{K_{\text{eff}} \cdot l_{\text{unbraced}}}{r_s} \right)^2} \cdot A_s \right]$$

$$P_e = \begin{pmatrix} 15750 \\ 23091 \\ 32380 \\ 40402 \\ 55076 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 20.3225 \\ 21.1841 \\ 30.2619 \\ 30.9596 \\ 32.0209 \end{pmatrix}$$

If $P_e/P_o \geq 0.44$ then:

LRFD Equation 6.9.4.1.1-1

$$P_n := \left[\left[0.658 \left(\frac{P_o}{P_e} \right) \right] \cdot P_o \right]$$

$$P_n = \begin{pmatrix} 759 \\ 1069 \\ 1055 \\ 1287 \\ 1698 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "severe" due to presence of cobbles and boulders.

Strength Limit State Axial Resistance factor for piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$$P_r := \phi_c \cdot P_n$$

$$P_r = \begin{pmatrix} 380 \\ 534 \\ 528 \\ 644 \\ 849 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_r := \phi \cdot P_n$$

$$P_r = \begin{pmatrix} 759 \\ 1069 \\ 1055 \\ 1287 \\ 1698 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Geotechnical Resistance

Assume piles will be end bearing on bedrock driven through overlying sand and silt.

Bedrock Type:

Sandstone RQD ranges from 58 to 67%

Use RQD = 60% and $\phi = 27$ to 34 deg (LRFD Table C10.4.6.4-1)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design
Specifications 4th Edition 2007

Look at these piles:

HP 12 x 53

HP 12 x 74

HP 14 x 73

HP 14 x 89

HP 14 x 117

Note: All matrices set up in this order

$$\begin{array}{lll} \text{Steel area:} & A_s = \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2 & \text{Pile depth:} & d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot \text{in} & \text{Pile width:} & b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot \text{in} \end{array}$$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for sandstone compressive strength ranges from 9700 to 25000 psi

$$\text{use } \sigma_c := 18000 \cdot \text{psi}$$

Determine K_{sp} : From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 48 \cdot \text{in}$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot \text{in}$ joints are tight

$$\begin{array}{ll} \text{Footing width, } b: & b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot \text{in} & \begin{array}{l} \text{HP 12 x 53} \\ \text{HP 12 x 74} \\ \text{HP 14 x 73} \\ \text{HP 14 x 89} \\ \text{HP 14 x 117} \end{array} \end{array}$$

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \quad K_{sp} = \begin{pmatrix} 0.6667 \\ 0.6614 \\ 0.6005 \\ 0.5981 \\ 0.5941 \end{pmatrix} \quad K_{sp} \text{ includes a factor of safety of 3}$$

Length of rock socket, L_s : $L_s := 0 \cdot \text{in}$ Pile is end bearing on rock

Diameter of socket, B_s : $B_s := 1 \cdot \text{ft}$

depth factor, d_f : $d_f := 1 + 0.4 \left(\frac{L_s}{B_s} \right)$ $d_f = 1$ should be $< \text{ or } = 3$ OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f$$

$$q_a = \begin{pmatrix} 1728 \\ 1714 \\ 1556 \\ 1550 \\ 1540 \end{pmatrix} \cdot \text{ksf}$$

Nominal Geotechnical Tip Resistance, R_p :

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{(3q_a \cdot A_s)}$$

$$R_p = \begin{pmatrix} 558 \\ 779 \\ 694 \\ 843 \\ 1104 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p$$

$$R_f = \begin{pmatrix} 251 \\ 350 \\ 312 \\ 379 \\ 497 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.5.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p$$

$$R_{fse} = \begin{pmatrix} 558 \\ 779 \\ 694 \\ 843 \\ 1104 \end{pmatrix} \cdot \text{kip}$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme
Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension

$$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y \text{ (eq. 10.7.8-1)}$$

$$f_y := 50 \cdot \text{ksi} \quad \text{yield strength of steel}$$

$$\phi_{da} := 1.0 \quad \begin{array}{l} \text{resistance factor from LRFD Table 10.5.5.2.3-1} \\ \text{Pile Drivability Analysis, Steel piles} \end{array}$$

$$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y \quad \sigma_{dr} = 45 \cdot \text{ksi} \quad \text{driving stresses in pile can not exceed 45 ksi}$$

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs) divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-45 gives resistance factor for dynamic test, ϕ_{dyn} :

$$\phi_{dyn} := 0.65$$

Pile Size = 12 x 53 Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation Westbrook Mill Brook Bridge 17092.04				31-Aug-2010 GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
488.0	44.83	6.60	10.4	8.88	20.01
489.0	44.85	6.60	10.5	8.89	20.00
490.0	44.95	6.61	10.5	8.90	20.05
491.0	45.00	6.61	10.6	8.91	20.04
492.0	45.07	6.62	10.6	8.92	20.09
493.0	45.11	6.62	10.7	8.92	20.08
494.0	45.17	6.63	10.8	8.93	20.13
495.0	45.21	6.63	10.8	8.94	20.12
496.0	45.26	6.63	10.9	8.95	20.11
497.0	45.34	6.64	11.0	8.96	20.17

Limited driving stress to 45 ksi

DELMAG D 19-42

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_12x53_strength} := 491 \cdot \text{kip} \cdot \phi_{dyn}$

$R_{dr_12x53_strength} = 319 \cdot \text{kip}$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_12x53_servext} := 491 \cdot \text{kip}$

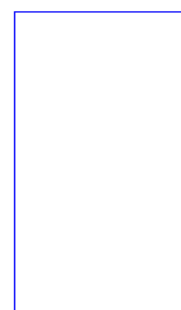
Efficiency 0.800

Helmet 3.20 kips
Hammer Cushion 109975 kips/in

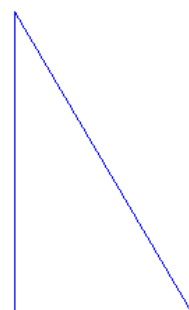
Skin Quake 0.100 in
Toe Quake 0.040 in
Skin Damping 0.050 sec/ft
Toe Damping 0.150 sec/ft

Pile Length 50.00 ft
Pile Penetration 50.00 ft
Pile Top Area 15.50 in²

Pile Model



Skin Friction
Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74 Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation Westbrook Mill Brook Bridge 17092.04				31-Aug-2010 GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
590.0	40.77	5.32	14.7	8.79	18.90
591.0	40.82	5.33	14.7	8.79	18.92
592.0	40.81	5.35	14.8	8.79	18.91
593.0	40.83	5.36	14.9	8.79	18.93
594.0	40.88	5.39	15.0	8.80	18.95
595.0	40.90	5.38	15.1	8.80	18.93
596.0	40.93	5.41	15.2	8.81	18.95
597.0	40.97	5.43	15.3	8.81	18.97
598.0	40.98	5.45	15.3	8.82	19.00
599.0	41.03	5.45	15.5	8.82	18.98

Limited to blow count to 15 blows per inch

DELMAG D 19-42

Strength Limit State: $\phi_{dyn} = 0.65$

Efficiency 0.800

$R_{dr_12x74_strength} := 594 \cdot \text{kip} \cdot \phi_{dyn}$

Helmet 3.20 kips
Hammer Cushion 109975 kips/in

$R_{dr_12x74_strength} = 386 \cdot \text{kip}$

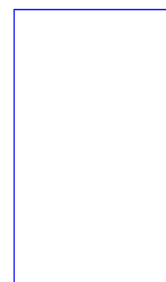
Skin Quake 0.100 in
Toe Quake 0.040 in
Skin Damping 0.050 sec/ft
Toe Damping 0.150 sec/ft

Service and Extreme Limit States: $\phi := 1.0$

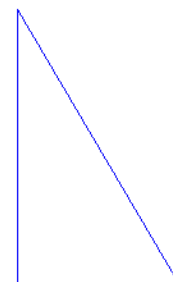
Pile Length 50.00 ft
Pile Penetration 50.00 ft
Pile Top Area 21.80 in²

$R_{dr_12x74_servext} := 594 \cdot \text{kip}$

Pile Model



Skin Friction
Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73 Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation Westbrook Mill Brook Bridge 17092.04				31-Aug-2010 GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
590.0	41.16	5.14	14.9	8.82	19.01
591.0	41.16	5.16	15.0	8.81	19.00
592.0	41.20	5.18	15.1	8.82	19.02
593.0	41.23	5.19	15.2	8.83	19.04
594.0	41.25	5.21	15.2	8.83	19.06
595.0	41.27	5.22	15.4	8.83	19.05
596.0	41.33	5.21	15.5	8.84	19.02
597.0	41.37	5.23	15.6	8.85	19.04
598.0	41.38	5.27	15.7	8.85	19.08
599.0	41.41	5.28	15.7	8.85	19.10

Limited to blow count to 15 blows per inch

DELMAG D 19-42

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_14x73_strength} := 591 \cdot \text{kip} \cdot \phi_{dyn}$

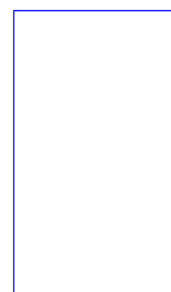
$R_{dr_14x73_strength} = 384 \cdot \text{kip}$

Service and Extreme Limit States: $\phi := 1.0$

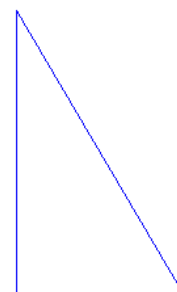
$R_{dr_14x73_servext} := 591 \cdot \text{kip}$

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	50.00 ft
Pile Penetration	50.00 ft
Pile Top Area	21.40 in ²

Pile Model



Skin Friction
Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89 Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation Westbrook Mill Brook Bridge 17092.04				31-Aug-2010 GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
685.0	42.00	6.80	14.7	9.82	21.73
686.0	42.02	6.80	14.9	9.82	21.69
687.0	42.02	6.81	15.0	9.82	21.70
688.0	42.07	6.83	15.0	9.83	21.71
689.0	42.10	6.85	15.1	9.83	21.72
690.0	42.13	6.86	15.2	9.84	21.73
691.0	42.14	6.88	15.3	9.84	21.74
692.0	42.17	6.90	15.2	9.85	21.80
693.0	42.22	6.93	15.3	9.85	21.81
694.0	42.25	6.94	15.4	9.86	21.82

Limit blow count to 15 bows per inch

DELMAG D 19-42

Strength Limit State: $\phi_{dyn} = 0.65$

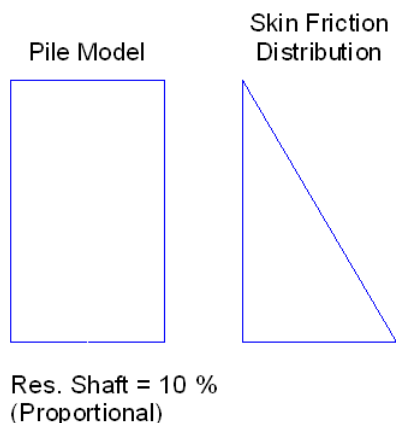
$R_{dr_14x89_strength} := 688 \cdot \text{kip} \cdot \phi_{dyn}$

$R_{dr_14x89_strength} = 447 \cdot \text{kip}$

Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_14x89_servext} := 688 \cdot \text{kip}$

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	50.00 ft
Pile Penetration	50.00 ft
Pile Top Area	26.10 in ²



Pile Size = 14 x 117 Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation				31-Aug-2010	
Westbrook Mill Brook Bridge 17092.04				GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
725.0	36.64	5.23	13.4	9.41	20.03
730.0	36.77	5.26	13.6	9.42	20.06
735.0	36.90	5.31	13.9	9.43	20.09
740.0	37.01	5.33	14.1	9.45	20.12
745.0	37.12	5.38	14.3	9.47	20.19
750.0	37.22	5.41	14.5	9.48	20.23
755.0	37.31	5.45	14.8	9.50	20.25
760.0	37.46	5.48	15.0	9.51	20.28
765.0	37.55	5.53	15.3	9.52	20.31
770.0	37.67	5.55	15.5	9.54	20.34

Limit to blow count to 15 blows per inch

DELMAG D 19-42

Strength Limit State: $\phi_{\text{dyn}} = 0.65$

$R_{\text{dr}_{14 \times 117} \text{ strength}} := 760 \cdot \text{kip} \cdot \phi_{\text{dyn}}$

Efficiency 0.800

$R_{\text{dr}_{14 \times 117} \text{ strength}} = 494 \cdot \text{kip}$

Helmet 3.20 kips
Hammer Cushion 109975 kips/in

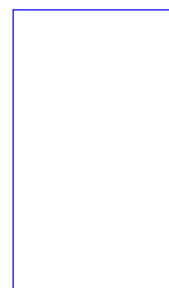
Service and Extreme Limit States: $\phi := 1.0$

Skin Quake 0.100 in
Toe Quake 0.040 in
Skin Damping 0.050 sec/ft
Toe Damping 0.150 sec/ft

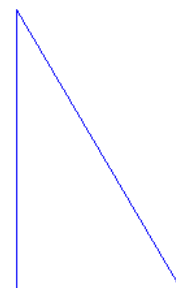
$R_{\text{dr}_{14 \times 117} \text{ servext}} := 760 \cdot \text{kip}$

Pile Length 50.00 ft
Pile Penetration 50.00 ft
Pile Top Area 34.40 in²

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Earth Pressure:

Passive Earth Pressure - Rankine Theory from Bowles 5th Edition Section 11-5 pg 602

Angle of backfill to the horizontal $\beta := 0 \cdot \text{deg}$

Angle of internal soil friction: $\phi := 32 \cdot \text{deg}$

$$K_{p_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}} \quad K_{p_rank} = 3.25$$

Bowles does not recommend the use of the Rankine Method for K_p when $\beta > 0$.

Passive Earth Pressure - Coulomb Theory from Maine DOT Bridge Design Guide Section 3.6.6 pg 3-8

Angle of back face of wall to the horizontal: $\alpha := 90 \cdot \text{deg}$

Angle of internal soil friction: $\phi := 32 \cdot \text{deg}$

Friction angle between fill and wall:

From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 19.5 \cdot \text{deg}$

Angle of backfill to the horizontal $\beta := 0 \cdot \text{deg}$

$$K_p := \frac{\sin(\alpha - \phi)^2}{\sin(\alpha)^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)}} \right)^2}$$

$$K_p = 6.73$$

Bearing Resistance - Native Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 4th Edition
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: Coarse to medium sand, with little gravel (SW, SP)

Based on corrected N-values ranging from 13 to 34 - Soils are medium dense to dense

Consistency In Place: Medium dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use: 6 ksf

$$\text{tsf} := g \cdot \left(\frac{\text{ton}}{\text{ft}^2} \right)$$

Recommended Value: $6 \cdot \text{ksf} = 3 \cdot \text{tsf}$

Therefore: $q_{\text{nom}} := 3 \cdot \text{tsf}$

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

$$q_{\text{factored_bc}} := 3 \cdot \text{tsf} \quad \text{or} \quad q_{\text{factored_bc}} = 6 \cdot \text{ksf}$$

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on native soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. Footings will be embedded 5.0 feet for frost protection. $D_f := 5.0 \cdot \text{ft}$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)
 - Saturated unit weight: $\gamma_s := 125 \cdot \text{pcf}$
 - Dry unit weight: $\gamma_d := 120 \cdot \text{pcf}$
 - Internal friction angle: $\phi_{\text{ns}} := 32 \cdot \text{deg}$
 - Undrained shear strength: $c_{\text{ns}} := 0 \cdot \text{psf}$
3. Use Terzaghi strip equations as $L > B$
4. Effective stress analysis footing on ϕ -c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 0 \cdot \text{ft}$ Based on boring logs

Unit Weight of water: $\gamma_w := 62.4 \cdot \text{pcf}$

Look at several footing widths

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot \text{ft}$$

Terzaghi Shape factors from Table 4-1

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

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For $\phi=32$ deg

$$N_c := 35.47 \quad N_q := 23.2 \quad N_\gamma := 22.0$$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q := D_f \cdot (\gamma_s - \gamma_w) \quad q = 0.1565 \cdot \text{tsf}$$

$$q_{\text{nominal}} := c_{ns} \cdot N_c \cdot s_c + q \cdot N_q + 0.5(\gamma_s - \gamma_w)B \cdot N_\gamma \cdot s_\gamma$$

$$q_{\text{nominal}} = \begin{pmatrix} 5.4 \\ 6.4 \\ 7.1 \\ 7.8 \\ 8.8 \end{pmatrix} \cdot \text{tsf}$$

Resistance Factor:

$$\phi_b := 0.45$$

AASHTO LRFD Table 10.5.5.2.2-1

$$q_{\text{factored}} := q_{\text{nominal}} \cdot \phi_b$$

$$q_{\text{factored}} = \begin{pmatrix} 2.4 \\ 2.9 \\ 3.2 \\ 3.5 \\ 4 \end{pmatrix} \cdot \text{tsf}$$

Based on these footing widths

$$q_{\text{factored}} = \begin{pmatrix} 4.8 \\ 5.7 \\ 6.4 \\ 7 \\ 7.9 \end{pmatrix} \cdot \text{ksf}$$

$$B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot \text{ft}$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 5 ksf for walls less than 8 feet wide.

Recommend a limiting factored bearing resistance of 7 ksf for walls between 8.5 and 12 feet wide.

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 = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1200 and wc =10%
Frost Penetration = 73.1 inches

$$\text{Frost_depth} := 73.1 \text{ in} \quad \text{Frost_depth} = 6.1 \cdot \text{ft}$$

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	59.6	10.0	120.0	26	32	1.7	1.5	1,728

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 = 4.97 ft = 59.6 in.

$$\text{Frost_depth}_{\text{modberg}} := 59.6 \cdot \text{in}$$

$$\text{Frost_depth}_{\text{modberg}} = 4.9667 \text{ ft}$$

Use Frost Depth = 5.0 feet for design

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

SPECIAL PROVISION
SECTION 635
PREFABRICATED CONCRETE MODULAR GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are: all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessary to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Pre-cast Concrete Units	712.061
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) $\pm 3/16$ inch.

2. Squareness. The length differences between the two diagonals shall not exceed 5/16 inch.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 inch in 4 feet will be rejected. For textured surfaces, any surface defects in excess of 5/16 inch in 5 feet shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 inch wide, by 0.5 inch thick preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.375 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 inches wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 inches minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 inch in 10 feet. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 inches in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a licensed Professional Engineer registered in accordance with the laws

of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:

1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.
2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH + ES)$
Where: R_R = Factored Sliding Resistance
 $\gamma_{p(max)}$ = Maximum Load Factor
EH = Horizontal Earth Pressure
ES = Earth Surcharge (as applicable)
4. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure
Where: q_R = Factored Bearing Resistance, as shown on the plans
Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
5. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Section 11, where Article 11.10.10.2 is modified such that the upper 3.5 feet of concrete modular units shall be designed for an additional horizontal load of γP_{HI} , where γP_{HI} = 300 lbs per linear foot of wall.

- B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For overturning, the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

- C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout

- resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikis Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 feet. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.
- D. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.
 - E. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.
 - F. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.
 - G. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.
 - H. Design Life. The wall design life shall be a minimum of 75 years.
 - I. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

- A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.
- B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

- C. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.
- D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.
- E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.
- F. Four weeks prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.05 Construction Requirements

Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity Wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any pre-cast units that are damaged due to handling will be replaced at the Contractor's expense.

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 inch. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 inch per 10 feet of wall height. The

prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square meter of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square meter complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot

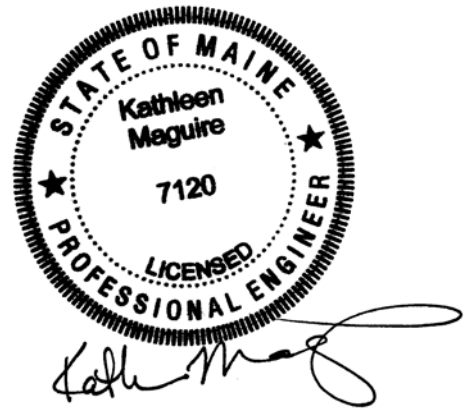
**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE**

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**MILL BROOK BRIDGE
OVER MILL BROOK
WESTBROOK, MAINE**

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



Reviewed by:
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Cumberland County
PIN 17092.04

Soils Report No. 2010-31
Bridge No. 3467

Fed No. NH-1709(204)E
November 23, 2010

MaineDOT TEDOCS No. 1083475

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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 Mill Brook Bridge which carries Route 302 over Mill Brook in Westbrook, Maine. Mill Brook Bridge was built in 1936 and consists of twin concrete box culverts with concrete wingwalls each end skewed at approximately 45 degrees. The culverts kink near the center of the structure. The proposed replacement structure will be a precast concrete double box culvert. The proposed structure will have an over all length of approximately 140 feet. The following design recommendations are discussed in detail in this report.

Precast Concrete Box Culvert Design and Construction– The precast concrete box culvert shall be design by the Manufacturer in accordance with Special Provision 534 and AASHTO LRFD Bridge Design Specifications 5th Edition 2010 (LRFD) specifications. The loading specified for the structure should be Modified HL-93 Strength 1. The precast concrete box culverts shall be designed for all relevant strength and service limit states and load combinations. The box culverts shall be constructed with concrete inlet and outlet toe walls.

Precast Concrete Box Culvert Headwall Design – Concrete headwalls should be specified to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. A minimum 1 foot by 1 foot concrete headwall is recommended. Precast concrete box culvert headwalls that are any larger than the nominal 1 foot by 1 foot shall be designed for all relevant strength and service limit states and load combinations. The head walls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culvert. The headwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}).

Bearing Resistance - The factored bearing resistance for at the strength limit state for the box culvert on compacted fill shall not exceed 8 ksf, however, the service limit state will control. A factored bearing resistance of 8 ksf shall be used to control settlement when analyzing the service limit state. In no instance shall the bearing stress exceed the nominal resistance of the structural concrete which may be taken as $0.3f'_c$.

Scour and Riprap - The proposed box culvert will have beveled ends eliminating the need for wingwalls. The proposed roadway side slopes will be 1.5:1 at the bridge to minimize additional bridge length. Nominal 1 foot by 1 foot concrete headwalls should be specified to retain riprap slopes. The slopes shall be armored with a 4-foot thick layer of heavy riprap conforming to MaineDOT 703.28 Heavy Riprap of Special Provision 703 - Aggregates. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation. The box culverts shall be fitted with inlet and outlet cutoff walls that extend below the maximum depth of scour.

Frost Protection - Any foundation placed on granular subgrade 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 buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

Construction Considerations – Construction of the proposed precast concrete box culverts will require soil excavation. Earth support systems will be required. The fill and native soils at the site will be susceptible to disturbance and rutting as a result of exposure to water and construction traffic. All subgrade surfaces should be protected from any unnecessary construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace disturbed areas with compacted gravel borrow. Any cobbles or boulder encountered in excess of 6 inches shall be removed and replaced with compacted gravel borrow.

The Contractor shall control groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Mill Brook Bridge which carries Route 302 over Mill Brook, in Westbrook, Maine. A subsurface investigation has been completed at the site. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement this report presents the soils information obtained at the site during the subsurface investigation, foundation design recommendations and geotechnical design parameters for the bridge replacement.

Mill Brook Bridge was built in 1936 and consists of buried twin concrete box culverts (9 foot by 6 foot) with integral concrete headwalls and wingwalls. The existing box culvert wingwalls are skewed at approximately 45 degrees. The existing culverts kink near the center with a maximum skew of 20 degrees. The stream flow is controlled by a dam at the south end of Highland Lake located approximately 0.25 miles upstream from the bridge. Gabion walls were installed above the structure in 1997 in order to widen Route 302. Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection reports for the bridge indicate the structure is in poor but serviceable condition. Areas of the wingwalls and center wall have soft deteriorating concrete. The ends of the slab have heavy efflorescence, but the interior is in satisfactory condition. There is spalling and scaling of the concrete at the center wall ends and downstream wingwalls, but the interior of the culvert is reported as being in fair condition. There is undermining of the center wall. 2008 MaineDOT Bridge Maintenance inspection reports assign the culvert a condition rating of 4 – considerable damage, and the channel protection a rating of 6 – bank slumping. The bridge has a Bridge Sufficiency Rating of 54.6.

The MaineDOT Bridge Program is currently proposing a replacement structure consisting of precast concrete double box culvert skewed at approximately 16 degrees. The proposed box culverts will have inlet and outlet toe walls and beveled ends which will eliminate the need for wingwalls. The overall length of the culverts will be increased from the existing 92 feet to 140 feet. Proposed roadway side slopes at the bridge are 1.5:1. In order to allow for fish passage at the structure location the box culverts will be offset with one box culvert founded at a lower elevation. The roadway profile will be raised approximately 4 feet at the bridge. Staged construction with two lanes of traffic will be utilized in the replacement of the structure.

2.0 GEOLOGIC SETTING

Mill Brook Bridge is located on Route 302 in Westbrook, Maine and crosses Mill Brook approximately 0.9 miles south of the Westbrook/Windham town line as shown on Sheet 1 - Location Map presented at the end of this report.

The Maine Geologic Survey “Surficial Geology of Portland West Quadrangle, Maine, Open-file No. 97-51”, 1997, indicates that the surficial soils at the Mill Brook Bridge site consist of end moraine deposits. End moraine deposits were deposited at the receding margin of the last

glacial ice sheet. They are composed of glacial till and/or sand and gravel. In the Westbrook area, some of the end moraine complexes include ice-margin submarine fans. Glacial till is typically loose to very compact, poorly sorted, mostly non-stratified mixture of sand, silt and gravel, but may contain lenses of water laid sediment. Submarine fan deposits are water-laid deposits and are well layered units of sand and gravel deposited on the sea floor at the glacial margin.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, and the Bedrock Geology of the Portland Quadrangle, Maine and New Hampshire, Maine Geological Survey, Open-File No. 98-1 1998, the site is underlain by carboniferous, muscovite granite. The formation is unnamed.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings. Test borings BB-WMB-101 and BB-WMB-102 were drilled on either side of the existing twin boxes. The borings were drilled on July 15, 27 and 28, 2009, using the MaineDOT drill rig. The boring locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. 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 end of this report.

The borings were drilled using solid stem auger and cased wash boring techniques. Soil samples were typically obtained at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with a Central Mine Equipment (CME) automatic hammer to drive the spilt spoon. The hammer was calibrated in February of 2009 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. These hammer efficiency factor, 0.84, and both the raw field N-value and the corrected N-value are shown on the boring logs. The bedrock was cored in two borings using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated for the NQ cores.

The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team Member or a New England Transportation Technical Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of seven (7) standard grain size analyses, eight (8) grain size analyses with hydrometer, and fifteen (15) natural water contents. The results of soil laboratory tests are included as Appendix B - Laboratory Test Results at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at test borings generally consisted of granular fill, and glacial margin deposits, underlain by glacial till and igneous bedrock. An interpretive subsurface profile depicting the detailed soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The boring logs are provided in Appendix A – Boring Logs at the end of this report. A brief summary description of the strata encountered follows:

5.1 Fill

A layer of granular fill was encountered in borings. The encountered layer is approximately 24.0 feet thick. The deposit generally consisted of brown, damp to wet, sand, some to trace gravel, little to some silt, little to trace clay, trace of pavement fragments and gravelly sand, some to little silt.

Corrected SPT N-values in the fill unit ranged from 7 to 20 blows per foot (bpf), indicating a soil that is loose to medium dense in consistency.

Grain size analyses were conducted on seven (7) samples from the fill unit. Grain size analyses resulted in the soil being classified as A-1-b, A-4, and A-2-4 under the AASHTO Soil Classification System and SM and SC-SM under the Unified Soil Classification System. Measured natural water contents of samples tested ranged from approximately 4 to 13 percent.

5.2 Glacial Margin Deposits

Glacial margin deposits were encountered below the fill unit. The encountered thickness of the unit was approximately 15.0 feet thick. The glacial margin deposits consisted of:

- Grey or brown, wet, sand, some silt, trace to little clay, trace gravel,
- grey, wet, silt, some clay, trace sand, and
- grey, wet, gravelly sand, some silt.

Within this unit, an isolated cobble was encountered in boring BB-WMB-102 at a depth of approximately 36.8 feet bgs and an isolated boulder was encountered in boring BB-WMB-101 at a depth of approximately 24.0 feet bgs.

Corrected SPT N-values in glacial margin deposits ranged from 13 to 48 bpf indicating that the soil deposit is medium dense to dense in consistency.

Grain size analyses were conducted on four (4) samples from the glacial margin deposits unit. Grain size analyses resulted in the soil being classified as an A-4 and A-1-b under the AASHTO Soil Classification System and SP, SC-SM and CL-ML under the Unified Soil Classification System. Measured natural water contents of samples tested ranged from approximately 10 to 16 percent.

5.3 Glacial Till

Glacial till was encountered underlying the glacial margin soils in the borings. The encountered thickness of the deposit was approximately 10.0 feet at the boring locations. The glacial till generally consisted of grey, wet, sand, some to little gravel, some to little silt, little to trace clay, and sandy silt, little gravel, little clay.

Corrected SPT N-values in the glacial till were greater than 50 bpf indicating that the deposit is very dense in consistency.

Grain size analyses were conducted on four (4) samples from the glacial till unit. Grain size analyses resulted in the soil being classified as an A-1-b, A-2-4, and A-4 under the AASHTO Soil Classification System and SC-SM under the Unified Soil Classification System. Measured natural water contents of samples tested ranged from approximately 8 to 10 percent.

5.4 Bedrock

Bedrock was encountered and cored beginning at depths of approximately 49.0 feet bgs and approximate Elevation 139 feet in both of the borings.

The bedrock at the site is identified as light grey to white, medium grained, muscovite granite, hard, fresh, with no apparent jointing, with occasional bands of biotite gneiss. The RQD of the bedrock was determined to range from 78 to 97 percent, correlating to a rock mass quality of good to excellent.

Table 1 summarizes approximate top of bedrock elevations at the exploration locations.

Boring	Station	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
BB-WMB-102	9+17.8	48.6	139.4
BB-WMB-101	9+48.3	49.0	139.2

Table 1 - Approximate Elevation of Bedrock Surface at Exploration Locations

5.5 Groundwater

The water level in borings ranged from 18 to 19 feet bgs. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, runoff and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

Based on the subsurface conditions encountered during the exploration program, the following foundation alternatives, with varying levels of risk and durability, were considered for the bridge replacement:

- Complete structure replacement with a composite tubular arch bridge structure and
- Complete structure replacement with double precast concrete box culverts.

Due to the existing deep roadway embankments at the site replacement with a conventional bridge was ruled out. This site was initially a candidate for replacement with a composite tubular arch bridge structure but a more suitable location was identified during the design phase. The Preliminary Design Report (PDR) for this project recommends that the replacement structure be double precast concrete box culverts. This report addresses only this replacement structure.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for design of double precast concrete box culverts which will make up the replacement structure. The proposed replacement structure will consist of one 12 foot wide by 7 foot high precast concrete box culvert and one 12 foot wide by 5 foot high precast concrete box culvert. In order to allow for fish passage at the structure location, the box culverts will be offset with the taller box culvert founded at a lower elevation. A natural stream bottom will be specified for the lower box culvert.

7.1 Precast Concrete Box Culvert Design and Construction

Precast concrete box culverts are typically detailed on the contract plans with only the basic layout and require hydraulic opening so that the Contractor may choose the appropriate structure. The Manufacturer is responsible for the design of the structure including determination of the wall thickness, haunch thickness and reinforcement in accordance with Special Provision 534 Precast Concrete Arches, Box Culverts, which is included in Appendix D of this report. The loading specified for the structure should be Modified HL-93 Strength 1 in which the HL-93 wheel loads are increased by a factor of 1.25. The designer should use Soil Type 4 as presented in the Maine DOT Bridge Design Guide (BDG) Section 3.6 to

design earth loads from the soil envelope. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

The precast concrete box shall include accommodations for toe walls at both the inlet and outlet ends to prevent undermining per MaineDOT BDG Section 8.3.1. The cutoff walls should extend below the maximum depth of scour.

The precast concrete box culverts will be supplier-designed in accordance with AASHTO LRFD Bridge Design Specifications 5th Edition 2010 (LRFD) specifications. The precast concrete box culverts shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The precast concrete box culverts shall be constructed in conformance with MaineDOT BDG Section 8 and Special Provision 534. The soil envelop and backfill shall consist of Standard Specification 703.19 - Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The crushed stone bedding should be placed in 12 inch maximum thick lifts and compacted with a minimum of four passes of a large walk behind compactor. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Precast Concrete Box Culvert Headwall Design

Concrete headwalls should be included in the culvert design to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. A nominal 1 foot by 1 foot concrete headwall is recommended.

Larger precast or cast-in-place concrete box culvert headwalls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The head walls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culvert. The headwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 2 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall pressure surface to edge of traffic = 0 feet	Distance from wall pressure surface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥ 20	2.0	2.0

Table 2 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Culvert headwall sections that are fixed to the box culverts to resist movement should be designed using an at-rest earth pressure coefficient, K_o , of 0.5. Headwall sections that are

independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a , of 0.31 assuming a level back slope. The active earth pressure coefficient may change if the back slope conditions are different. See Appendix C - Calculations for supporting documentation.

7.3 Bearing Resistance

The factored bearing resistance for at the strength limit state for the box culvert on compacted fill shall not exceed 8 ksf, however, the service limit state bearing resistance will govern. A factored bearing resistance of 8 ksf shall be used to control settlement when analyzing the service limit state as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the structural concrete which may be taken as $0.3f'_c$. See Appendix C - Calculations for supporting documentation.

7.4 Scour and Riprap

The proposed box culvert will have beveled ends eliminating the need for wingwalls. The proposed roadway side slopes will be 1.5:1 at the bridge to minimize additional bridge length. Concrete headwalls with nominal dimensions of 1 foot by 1 foot should be included in the design to retain the oversteepened riprap slopes. The slopes shall be armored with a 4-foot thick layer of heavy riprap conforming to MaineDOT 703.28 Heavy Riprap of Special Provision 703 – Aggregates provided in Appendix D of this report. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation. The riprap slopes should also be constructed in accordance with Special Provision 610 – Stone Fill, Riprap, Stone Blanket and Stone Ditch Protection provided in Appendix D of this report.

The box culverts shall be fitted with inlet and outlet cutoff walls that extend below the maximum depth of scour.

7.5 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1195 F-degree days. In a granular soil with a water content of approximately 10%, this correlates to a frost depth of approximately 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. See Appendix C- Calculations at the end of this report for supporting documentation.

7.6 Seismic Design Considerations

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

7.8 Construction Considerations

Construction of the proposed precast concrete box culverts will require soil excavation. Earth support systems will be required. The fill and native soils at the site will be susceptible to disturbance and rutting as a result of exposure to water and construction traffic. All subgrade surfaces should be protected from any unnecessary construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace disturbed areas with compacted gravel borrow. Any cobbles or boulder encountered in excess of 6 inches shall be removed and replaced with compacted gravel borrow.

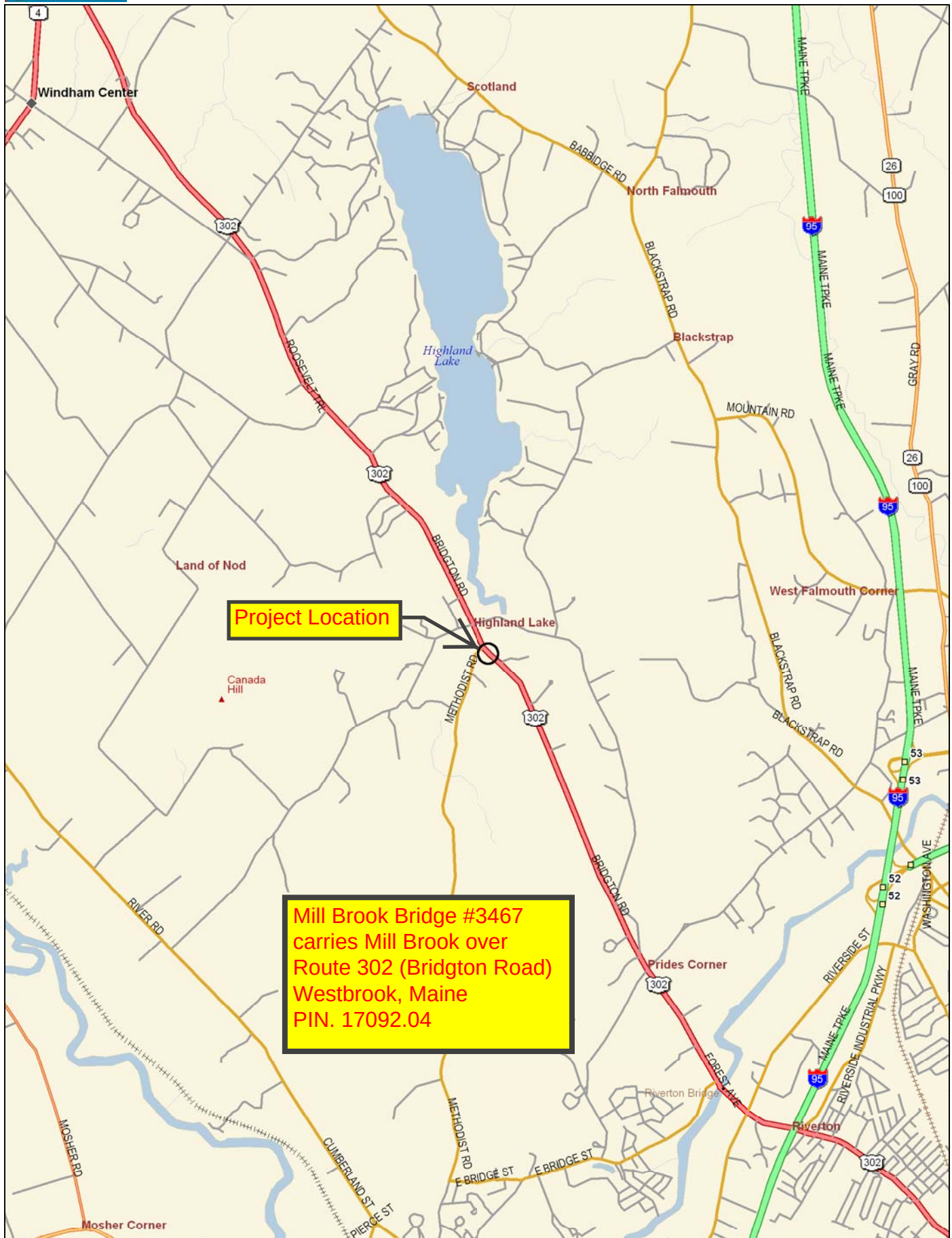
The Contractor shall control groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

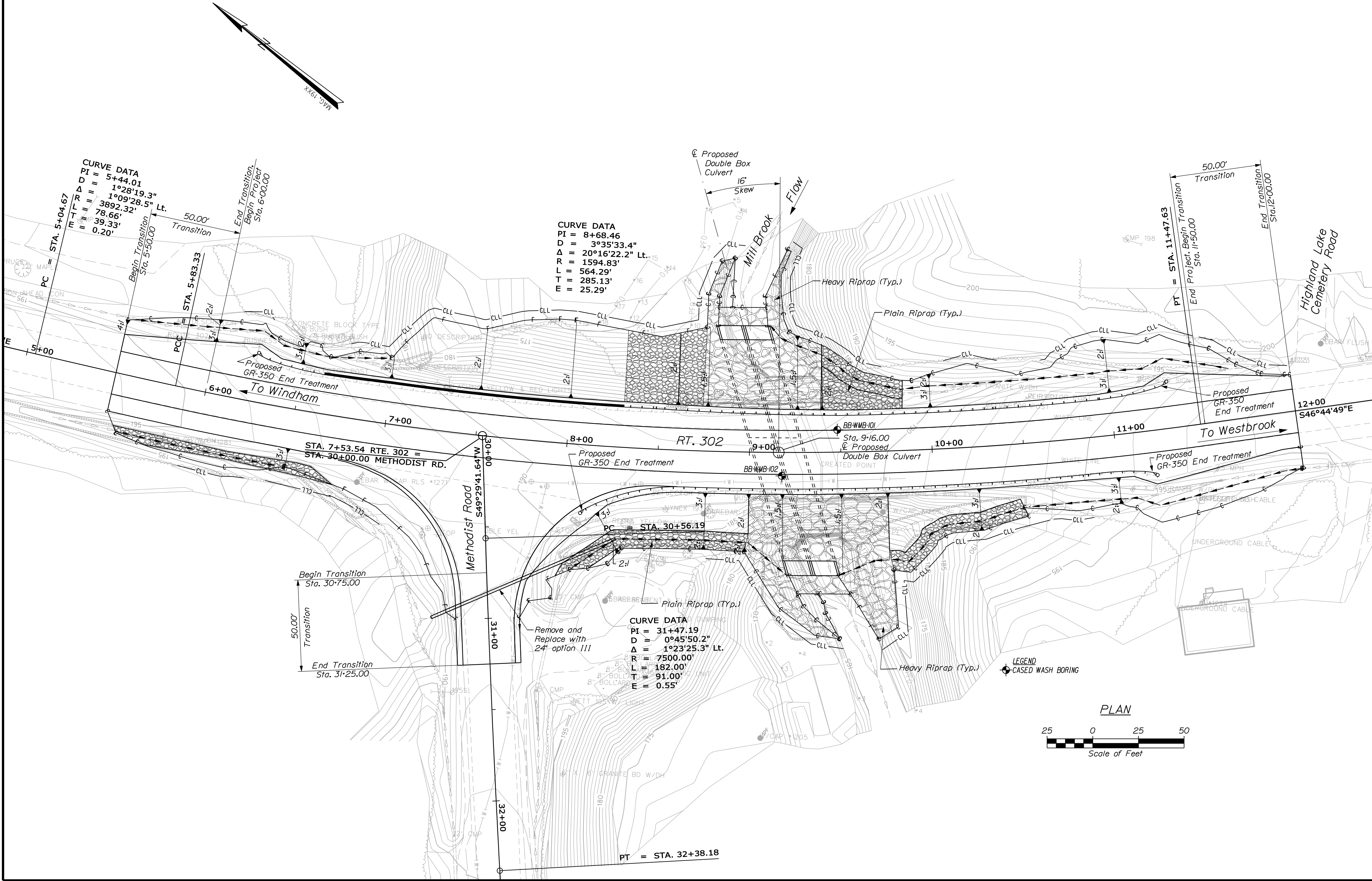
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Mill Brook Bridge 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 plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

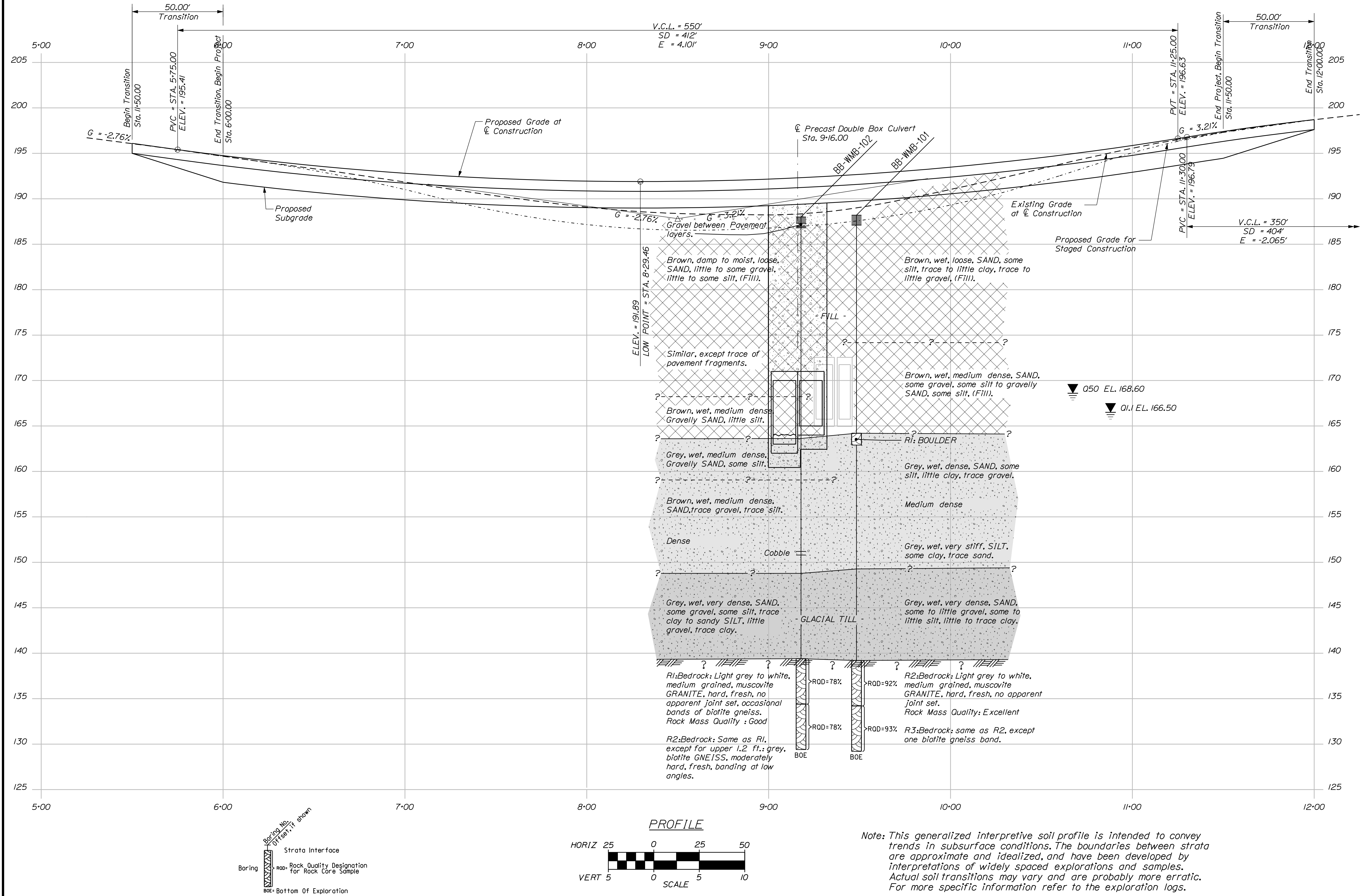
Sheets





PROJ. MANAGER	BY	DATE	SIGNATURE
CHECKED-REVIEWED	T. WHITE	OCT 2009	
DESIGNS-DETAILED			
DESIGNS-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MILL BROOK BRIDGE MILL BROOK WESTBROOK CUMBERLAND COUNTY	BORING LOCATION PLAN
--	----------------------



OF 4		3		SHEET NUMBER	
WESTBROOK		CUMBERLAND COUNTY		MILL BROOK BRIDGE	
MILL BROOK					
INTERPRETIVE SUBSURFACE PROFILE		DESIGN3-DETAILED2		DESIGN-DETAILED	
		DESIGN3-DETAILED3		CHECKED-REVIEWED	
		REVISIONS 1		K. MACUIRE	
		REVISIONS 2		T. WHITE	
		REVISIONS 3			
		REVISIONS 4			
		FIELD CHANGES			
				DATE	
				P.E. NUMBER	
				SIGNATURE	
				OCT 2009	
				DATE	

Maine Department of Transportation				Project: Mill Brook Bridge #2467 over Mill Brook on Route 302 Location: Westbrook, Maine		Boring No.: BB-WMB-102	
Soil/Rock Exploration Log				P/N:		17092_04	
US CUSTOMARY UNITS							
Driller: MainDOT		Elevation (ft.): 188.0		Auger 10/00:		5' Solid Stem	
Operator: Giguere/Giles/Wright		Datum: NAVD 88		Sampler:		Standard Split Spoon	
Logged By: B. Wilder		Rig Types: CME 45C		Hammer Wt./Fall:		140#/30"	
Date Start/Finish: 9/27/28/09 7.5 hours		Drilling Method: Cased Wash Boring		Core Barrel:		NO-2"	
Boring Location: 9+17.8; 12.3 RT.		Casing ID(s): NW		Water Level#:		19.0' bgs.	
Notes: Efficiency Factor: 0.84		Hammer Type: Automatic ☒ Hydraulic ☐		Pipe & Collared ☐			
Definitions: S = Split Spoon Sample M = unconsolidated Split Spoon Sample attempt U = Thin Wall Tube Sample W = unconsolidated Thin Wall Tube Sample attempt V = In situ vane shear Test H = Pocket Penetrometer Test R = weight of rods or casing A = Blowfall at top of core B = Rock Core Sample SA = Solid Stem Auger HA = Hollow Stem Auger BA = Baller Core WS = Weight of hole - hammer WT = Hammer Efficiency Factor CG = Correlation value SG = Unconfined Compressive Strength (psi) T _c = Point Load Shear Strength (psi) S _c = Unconfined Compressive Strength (ksi) U _c = Unconfined Compressive Strength (ksi) E = Elastic Limit P = Plasticity Index N _p = SFH corrected for hammer efficiency G = Grain Size Analysis C = Consolidation Test				Soils: 1.5k. Fine Shear Strength (psi) WC = water content, percent L = Liquid Limit P = Plasticity Index R = Plasticity Index N _p = SFH corrected for hammer efficiency G = Grain Size Analysis C = Consolidation Test			
Sample Information				Visual Description and Remarks			
Depth (feet)	Sample No.	Pen./Ref. (in.)	Sample Depth (ft.)	Blow / S.F. (in.)	Shear / S.F. (in.)	Length of Rod (ft.)	Laboratory Testing Results/ASHTO and Unified Class
0							
10	2-6/2-4	1.00 - 1.20	50/2-4"	---			Pavement Layer of Gravel Old Pavement
20	24/13	5.00 - 7.00	2/3/2/1	5	7		Brown, damp, loose, fine to coarse SAND, little gravel, little silt, (fossil).
30	24/17	10.00 - 12.00	2/3/2/3	5	7		Brown, moist, loose, fine to coarse SAND, some silt, little gravel, (fossil).
40	24/19	15.00 - 17.00	2/3/4/4	7	10		Brown, moist, loose, fine to coarse SAND, some gravel, some silt, trace of fragments of pavement, (fossil).
50	24/15	20.00 - 22.00	7/6/8/6	14	20		Brown, wet, medium dense, Gravely, fine to coarse SAND, little silt.
60	24/9	25.00 - 27.00	12/21/6/6	27	38		Grey, wt., dense, Gravely, fine to coarse SAND, some silt.
70	24/16	30.00 - 32.00	4/4/5/5	9	13		Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.
80	21-4/14	35.00 - 36.78	8/10/15/50/3-4"	35	35		Similar to above, dense. COBBLE from 36.8-37.2' bgs. Roller Cone ended of Casing from 36.0-48.6'.
90	24/15	40.00 - 42.00	56/42/23/22	65	91		Grey, wt., very dense, fine to coarse SAND, some gravel, some silt, trace clay, (Glacial Till).
100	24/19	45.00 - 47.00	18/22/29/31	51	71		Grey, wt., very dense, Sandy SILT, little gravel, little clay.
110	60/60	49.60 - 53.60	ROD = 78%				9100 blows for 0.2". Roller Cone ended to 48.6' bgs. Top of Bedrock at Elev. 139.4'. Bedrock Light grey to white, medium grained, massive GRANITE, hard, fresh no apparent joint set, banding of biotite gneiss in the upper 1.3 ft. and at 2.3 ft. Rock Mass Quality Fair Good. R1 Core Times (min/sec) 48.0-49.6' (1230) 49.6-50.6' (1200) 50.6-51.6' (1218) 51.6-52.6' (1187) 52.6-53.6' (1245) 100% Recovery R2 Bedrock Grey, biotite GNEISS, for 1.2 ft, moderately hard, fresh, slightly fractured, banding of low angles, thin light grey to white, Rutovite GRANITE, hard fresh, no apparent joint set. Rock Mass Quality Good. R2 Core Times (min/sec) 53.6-54.6' (1145) 54.6-55.6' (1135) 55.6-56.6' (1129) 56.6-57.6' (1156) 57.6-58.6' (1155) 92% Recovery
120	60/55	53.60 - 58.60	ROD = 78%				Bottom of Exploration at 58.60 feet below ground surface.
Remarks: 500 lbs foot pressure on Core Barrel.							
Stratification lines represent approximate boundaries between soil types. Transitions may be gradual.							
* Aster level readings have been read of times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present in the time measurements were made.							
Boring No.: BB-WMB-102							

[illegible]

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					TERMS DESCRIBING DENSITY/CONSISTENCY																														
MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES																															
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	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 <table><tr><th>Descriptive Term</th><th>Portion of Total</th></tr><tr><td>trace</td><td>0% - 10%</td></tr><tr><td>little</td><td>11% - 20%</td></tr><tr><td>some</td><td>21% - 35%</td></tr><tr><td>adjective (e.g. sandy, clayey)</td><td>36% - 50%</td></tr></table> <table><tr><th>Density of Cohesionless Soils</th><th>Standard Penetration Resistance N-Value (blows per foot)</th></tr><tr><td>Very loose</td><td>0 - 4</td></tr><tr><td>Loose</td><td>5 - 10</td></tr><tr><td>Medium Dense</td><td>11 - 30</td></tr><tr><td>Dense</td><td>31 - 50</td></tr><tr><td>Very Dense</td><td>> 50</td></tr></table>				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					
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(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines																																	
GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.																																	
SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines																																
	(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																																
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.	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. <table><tr><th>Consistency of Cohesive soils</th><th>SPT N-Value blows per foot</th><th>Approximate Undrained Shear Strength (psf)</th><th>Field Guidelines</th></tr><tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily Penetrates</td></tr><tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr><tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr><tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr><tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr><tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr></table>				Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines	Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates	Soft	2 - 4	250 - 500	Thumb easily penetrates	Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort	Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	Hard	>30	over 4000	Indented by thumbnail with difficulty
		Consistency of Cohesive soils	SPT N-Value blows per foot					Approximate Undrained Shear Strength (psf)	Field Guidelines																										
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Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail																																
Hard	>30	over 4000	Indented by thumbnail with difficulty																																
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.	Rock Quality Designation (RQD): RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table><tr><th>Rock Mass Quality</th><th>RQD</th></tr><tr><td>Very Poor</td><td><25%</td></tr><tr><td>Poor</td><td>26% - 50%</td></tr><tr><td>Fair</td><td>51% - 75%</td></tr><tr><td>Good</td><td>76% - 90%</td></tr><tr><td>Excellent</td><td>91% - 100%</td></tr></table> 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				Rock Mass Quality	RQD	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%																
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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					Sample Container Labeling Requirements: <table><tr><td>PIN</td><td>Blow Counts</td></tr><tr><td>Bridge Name / Town</td><td>Sample Recovery</td></tr><tr><td>Boring Number</td><td>Date</td></tr><tr><td>Sample Number</td><td>Personnel Initials</td></tr><tr><td>Sample Depth</td><td></td></tr></table>				PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth																		
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Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information																																			

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Mill Brook Bridge #3467 over Mill Brook on Route 302 Location: Westbrook, Maine		Boring No.: BB-WMB-101 PIN: 17092.04																																																																																																																																																																																																																																																														
Driller: MaineDOT			Elevation (ft.) 188.2		Auger ID/OD: 5" Solid Stem																																																																																																																																																																																																																																																															
Operator: Giguere/Giles/Wright			Datum: NAVD 88		Sampler: Standard Split Spoon																																																																																																																																																																																																																																																															
Logged By: B. Wilder			Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																															
Date Start/Finish: 7/15/09, 7/27/09 11 hours			Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"																																																																																																																																																																																																																																																															
Boring Location: 9+48.3, 12.5 Lt.			Casing ID/OD: HW & NW		Water Level*: 18.0' bgs.																																																																																																																																																																																																																																																															
Hammer Efficiency Factor: 0.84			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																																																																																																																																																																																																																																																																	
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt			R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR/C = weight of rods or casing WO1P = Weight of one person		S _u = Insitu Field Vane Shear Strength (psf) T _v = Pocket Torvane Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N _{uncorrected} = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N ₆₀ = SPT N-uncorrected corrected for hammer efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N _{uncorrected}		S _{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test																																																																																																																																																																																																																																																													
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="8">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>187.10</td><td rowspan="20"></td><td>Pavement</td><td rowspan="20">G#212324 A-1-b, SM WC=5.0%</td></tr><tr><td></td><td>1D</td><td>24/17</td><td>2.00 - 4.00</td><td>4/8/5/9</td><td>13</td><td>18</td><td></td><td></td><td>Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>5</td><td>2D</td><td>24/18</td><td>5.00 - 7.00</td><td>3/3/3/3</td><td>6</td><td>8</td><td></td><td></td><td>Brown, wet, loose, fine to coarse SAND, some silt, little clay, trace gravel, (Fill).</td><td>G#212325 A-4, SC-SM WC=13.0%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>10</td><td>3D</td><td>24/12</td><td>10.00 - 12.00</td><td>3/3/3/2</td><td>6</td><td>8</td><td>36</td><td></td><td>Brown, wet, loose, fine to coarse SAND, some silt, little gravel.</td><td>G#209251 A-2-4, SM WC=10.3%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>39</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>23</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>17</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>23</td><td></td><td></td><td></td></tr><tr><td>15</td><td>4D</td><td>24/3</td><td>15.00 - 17.00</td><td>6/5/5/6</td><td>10</td><td>14</td><td>15</td><td></td><td>Brown, wet, medium dense, fine to coarse SAND, some gravel, some silt.</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>28</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>36</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>46</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>65</td><td></td><td></td><td></td></tr><tr><td>20</td><td>5D</td><td>24/8</td><td>20.00 - 22.00</td><td>10/9/3/4</td><td>12</td><td>17</td><td>27</td><td></td><td>Brown, wet, medium dense, gravelly fine to coarse SAND, some silt.</td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>28</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>48</td><td></td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>48</td><td></td><td></td><td></td></tr><tr><td>25</td><td>R1</td><td></td><td>24.00 - 25.30</td><td></td><td></td><td></td><td>300</td><td>164.20</td><td>R1:BOULDER.</td><td>24.00</td></tr></table>										Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	0							SSA	187.10		Pavement	G#212324 A-1-b, SM WC=5.0%		1D	24/17	2.00 - 4.00	4/8/5/9	13	18			Brown, moist, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).												5	2D	24/18	5.00 - 7.00	3/3/3/3	6	8			Brown, wet, loose, fine to coarse SAND, some silt, little clay, trace gravel, (Fill).	G#212325 A-4, SC-SM WC=13.0%																							10	3D	24/12	10.00 - 12.00	3/3/3/2	6	8	36		Brown, wet, loose, fine to coarse SAND, some silt, little gravel.	G#209251 A-2-4, SM WC=10.3%								39											23											17											23				15	4D	24/3	15.00 - 17.00	6/5/5/6	10	14	15		Brown, wet, medium dense, fine to coarse SAND, some gravel, some silt.									28											36											46											65				20	5D	24/8	20.00 - 22.00	10/9/3/4	12	17	27		Brown, wet, medium dense, gravelly fine to coarse SAND, some silt.									28											48											48				25	R1		24.00 - 25.30				300	164.20	R1:BOULDER.	24.00
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Date Start/Finish: 7/15/09, 7/27/09 11 hours				Drilling Method: Cased Wash Boring				Core Barrel: NQ-2"																																																																																																																																																																																																																										
Boring Location: 9+48.3, 12.5 Lt.				Casing ID/OD: HW & NW				Water Level*: 18.0' bgs.																																																																																																																																																																																																																										
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Mill Brook Bridge #3467 over Mill Brook on Route 302 Location: Westbrook, Maine		Boring No.: BB-WMB-102 PIN: 17092.04																																																																																																																																																																																																																																					
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Mill Brook Bridge #3467 over Mill Brook on Route 302 Location: Westbrook, Maine		Boring No.: BB-WMB-102 PIN: 17092.04							
Driller: MaineDOT		Elevation (ft.) 188.0		Auger ID/OD: 5" Solid Stem									
Operator: Giguere/Giles/Wright		Datum: NAVD 88		Sampler: Standard Split Spoon									
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"									
Date Start/Finish: 7/27,28/09 7.5 hours		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"									
Boring Location: 9+17.8, 12.5 Rt.		Casing ID/OD: NW		Water Level*: 19.0' bgs.									
Hammer Efficiency Factor: 0.84		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample attempt V = Insitu Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Insitu Vane Shear Test attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = weight of 140lb. hammer WOR/C = weight of rods or casing WO1P = Weight of one person S_u = Insitu Field Vane Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N_{uncorrected} = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N₆₀ = SPT N-uncorrected corrected for hammer efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N_{uncorrected} S_{u(lab)} = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test													
Depth (ft.)	Sample Information							Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.			
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				Elevation (ft.)		
25	6D	24/9	25.00 - 27.00	12/21/6/6	27	38	39	159.50		Grey, wet, dense, Gravelly, fine to coarse SAND, some silt.	G#246339 A-1-b, SP WC=15.2%		
							32						
							42						
							57						
30							66					Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	
	7D	24/16	30.00 - 32.00	4/4/5/5	9	13	14						
							22						
							37						
35							53						Similar to above, dense.
							67						
	8D	21.4/14	35.00 - 36.78	8/10/15/50(3.4")	25	35	29						
							100						
40							63		COBBLE from 36.8-37.2' bgs. Roller Coned ahead of Casing from 36.0-48.6'.				
							71						
							114						
							149.00						
45	9D	24/15	40.00 - 42.00	56/42/23/22	65	91	52			Grey, wet, very dense, fine to coarse SAND, some gravel, some silt, trace clay, (Glacial Till).			
							45						
							41						
							47						
50							153			Grey, wet, very dense, Sandy SILT, little gravel, little clay.			
	10D	24/19	45.00 - 47.00	18/22/29/31	51	71	89						
							109						
							188						
	R1	60/60	48.60 - 53.60	RQD = 78%			a100 NQ-2	139.40		a100 blows for 0.2'. Roller Coned ahead to 48.6' bgs.			
										Top of Bedrock at Elev. 139.4'.			
										R1:Bedrock: Light grey to white, medium grained, muscovite			
Remarks: 500 lbs down pressure on Core Barrel.													
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 of 3			
* 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-WMB-102			

Maine Department of Transportation <u>Soil/Rock Exploration Log</u> US CUSTOMARY UNITS				Project: Mill Brook Bridge #3467 over Mill Brook on Route 302 Location: Westbrook, Maine				Boring No.: BB-WMB-102 PIN: 17092.04																																																																																																												
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Boring No.: BB-WMB-102																																																																																																																				

Appendix B

Laboratory Test Results

Laboratory Testing Summary Sheet

Town(s): Westbrook

Project Number: 17092.04

[illegible]

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible).

The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

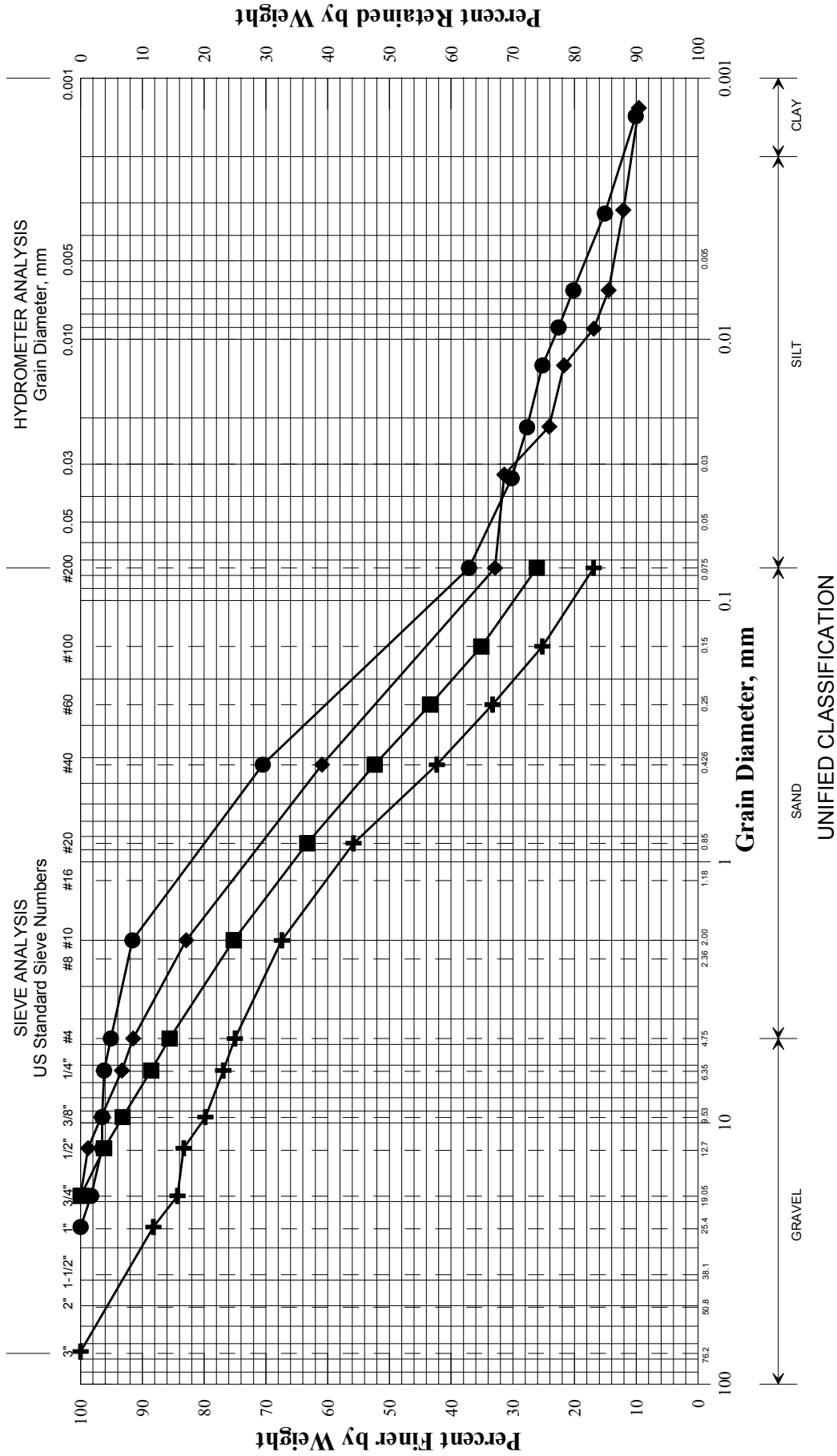
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

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

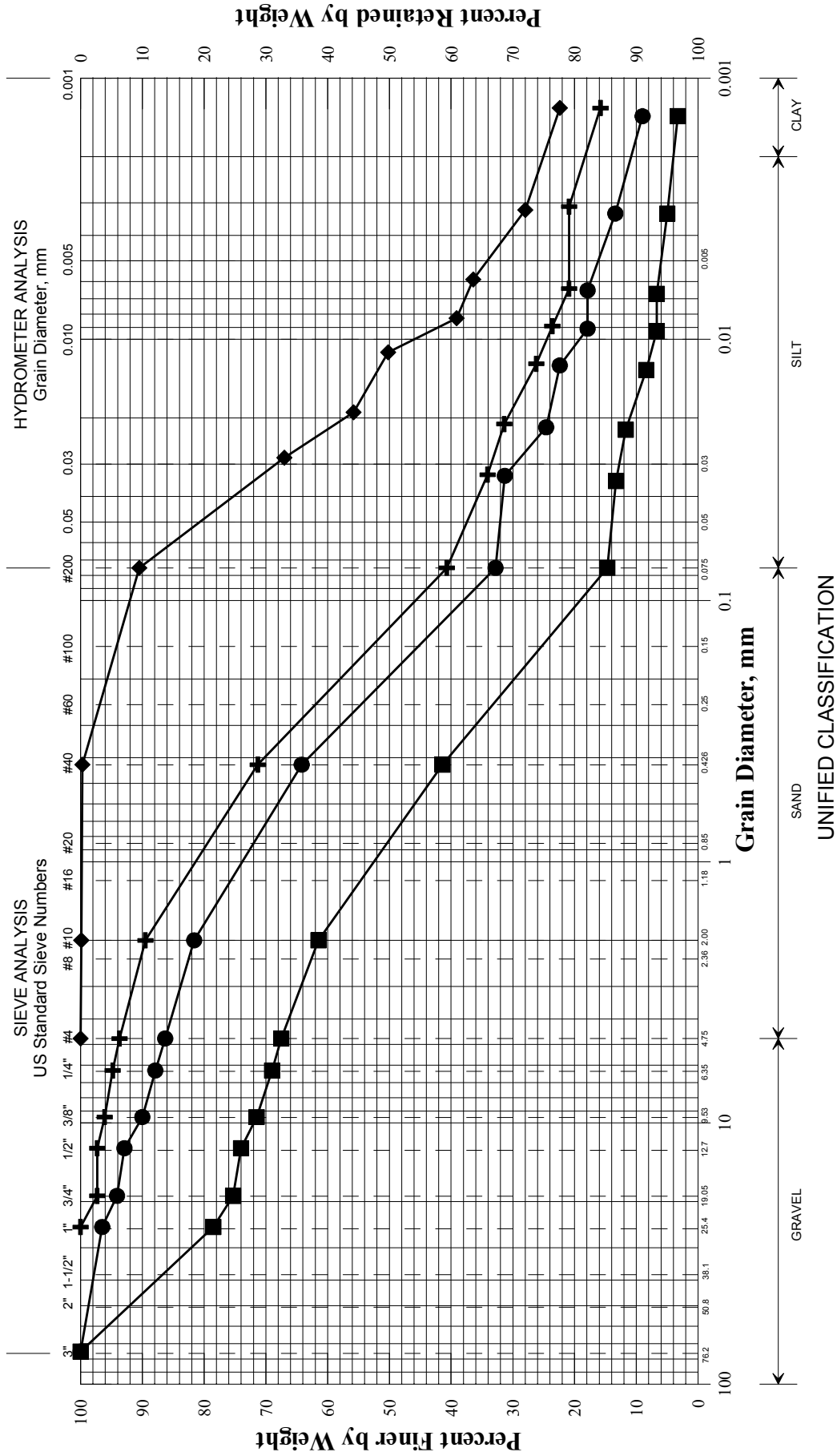
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	PIN
017092.04	
	Town
Westbrook	
	Reported by/Date
WHITE, TERRY A	9/2/2009

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-101/1D	9+48.3	12.5 LT	2.0-4.0	SAND, some gravel, little silt.	5.0			
◆	BB-WMB-101/2D	9+48.3	12.5 LT	5.0-7.0	SAND, some silt, little clay, trace gravel.	13.0			
■	BB-WMB-101/3D	9+48.3	12.5 LT	10.0-12.0	SAND, some silt, little gravel.	10.3			
●	BB-WMB-101/6D	9+48.3	12.5 LT	26.0-28.0	SAND, some silt, little clay, trace gravel.	9.4			
▲									
×									

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
BB-WMB-101/7D	9+48.3	12.5 LT	30.0-32.0	SAND, some silt, little clay, trace gravel.	9.4			
BB-WMB-101/8D	9+48.3	12.5 LT	35.0-37.0	SILT, some clay, trace sand.	15.6			
BB-WMB-101/9D	9+48.3	12.5 LT	40.0-41.5	SAND, some gravel, little silt, trace clay.	10.4			
BB-WMB-101/10D	9+48.3	12.5 LT	45.0-47.0	SAND, some silt, little gravel, little clay.	8.5			

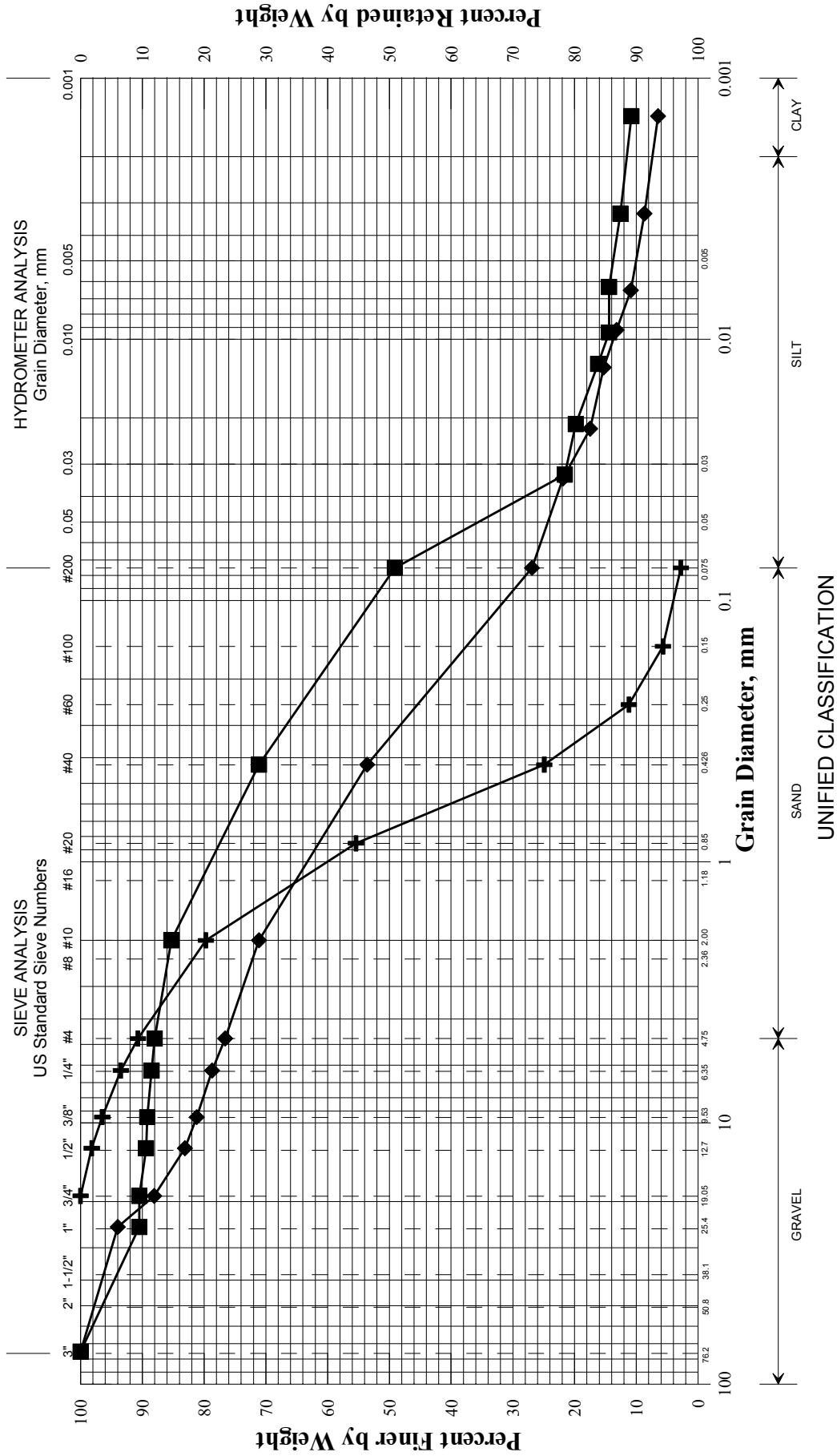
PIN	017092.04
Town	Westbrook
Reported by/Date	WHITE, TERRY A 9/2/2009

GRAIN SIZE DISTRIBUTION CURVE



PIN
017092.04
Town
Westbrook
Reported by/Date
WHITE, TERRY A 10/15/2009

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-WMB-102/7D	9+17.8	12.5 RT	30.0-32.0	SAND, trace gravel, trace silt.	15.2			
◆	BB-WMB-102/9D	9+17.8	12.5 RT	40.0-42.0	SAND, some gravel, some silt, trace clay.	7.5			
●	BB-WMB-102/10D	9+17.8	12.5 RT	45.0-47.0	Sandy SILT, little gravel, little clay.	7.9			
■									
▲									
×									

PIN	017092.04	Reported by/Date
Town	Westbrook	WHITE, TERRY A 10/15/2009

Appendix C

Calculations

At-Rest and Active Earth Pressure:

At-Rest Lateral Earth Pressure from LRFD Article 3.11.5.2 pg 3-71

Effective friction angle of soil $\phi_f := 30\text{-deg}$

$$K_o := 1 - \sin(\phi_f) \quad K_o = 0.5$$

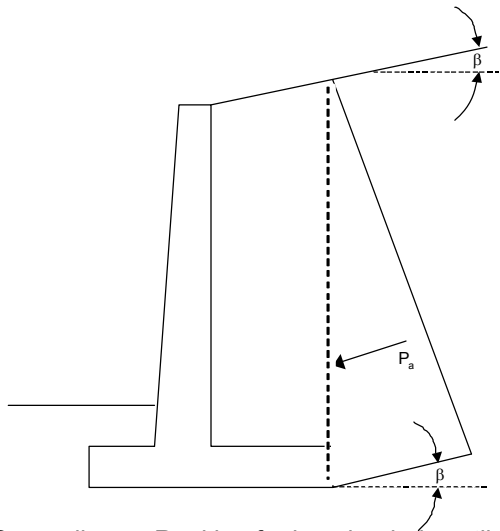
Active Earth Pressure - Rankine Theory from MaineDOT Bridge Design Guide Section 3.6.5.2 pg 3-7

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

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

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

Cohesion: $c_{\text{sand}} := 0\text{-psf}$



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

$\beta := 0\text{-deg}$ assume horizontal backfill surface

$$K_{a_rankine_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{\text{type4}})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{\text{type4}})^2}} \quad K_{a_rankine_slope} = 0.31$$

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

Bearing Resistance - Native Granular Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - box culvert on granular soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 4th Edition
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: Fine to coarse sand (SC-SM)

Based on N-values ranging from 38 to 48 - Soils are dense

Consistency In Place: dense

Bearing Resistance: Ordinary Range (ksf) 8 to 12

Recommended Value of Use: 8 ksf

$$\text{tsf} := \text{g} \cdot \left(\frac{\text{ton}}{\text{ft}^2} \right)$$

Recommended Value:

$$8 \cdot \text{ksf} = 4 \cdot \text{tsf}$$

Therefore: $q_{\text{nom}} := 4 \cdot \text{tsf}$

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

$$q_{\text{factored_bc}} := 4 \cdot \text{tsf} \quad \text{or} \quad q_{\text{factored_bc}} = 8 \cdot \text{ksf}$$

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - box culvert on native soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. The box culverts will be founded at ~ Elev 160 to 163
25 to 28 ft below roadway surface $D_{\text{box}} := 2.0 \cdot \text{ft}$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)
 - Saturated unit weight: $\gamma_s := 125 \cdot \text{pcf}$
 - Dry unit weight: $\gamma_d := 120 \cdot \text{pcf}$
 - Internal friction angle: $\phi_{\text{ns}} := 32 \cdot \text{deg}$
 - Undrained shear strength: $c_{\text{ns}} := 0 \cdot \text{psf}$
3. Use Terzaghi strip equations as $L > B$
4. Effective stress analysis footing on ϕ -c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth the water table: $D_w := 0 \cdot \text{ft}$

Unit Weight of water: $\gamma_w := 62.4 \cdot \text{pcf}$

Effective stress at box bearing level:

$$q_{\text{eff}} := D_w \cdot \gamma_s + (D_{\text{box}} - D_w) \cdot (\gamma_s - \gamma_w) \quad q_{\text{eff}} = 0.125 \cdot \text{ksf}$$

Look at 2 widths: $B := \left(\frac{12}{24} \right) \cdot \text{ft}$ One culvert
Two culverts

Terzaghi Shape factors from Table 4-1 For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors - Bowles 5th Ed. table 4-4 pg 223

For $\phi = 32$ deg $N_c := 35.47$ $N_q := 23.2$ $N_\gamma := 22.0$

Nominal Bearing Resistance per Terzaghi equation (Bowles 5th Ed. Table 4-1 pg 220)

$$q_{\text{nominal}} := c_{\text{ns}} \cdot N_c \cdot s_c + q_{\text{eff}} \cdot N_q + 0.5(\gamma_s) B \cdot N_\gamma \cdot s_\gamma$$

$$q_{\text{nominal}} = \left(\frac{9.7}{18} \right) \cdot \text{tsf}$$

Resistance Factor: $\phi_b := 0.45$ AASHTO LRFD Table 10.5.5.2.2-1

$$q_{\text{factored}} := q_{\text{nominal}} \cdot \phi_b$$

$$q_{\text{factored}} = \left(\frac{4.4}{8.1} \right) \cdot \text{tsf} \quad B = \left(\frac{12}{24} \right) \cdot \text{ft}$$

$$q_{\text{factored}} = \left(\frac{8.7}{16.2} \right) \cdot \text{ksf} \quad B = \left(\frac{12}{24} \right) \cdot \text{ft}$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 8 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 = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1200 and wc =10%
Frost Penetration = 73.1 inches

Frost_depth := 73.1in Frost_depth = 6.1·ft

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	t	w%	d	Cf	Cu	Kf	Ku	L
#.Type								
1-Coarse	59.6	10.0	120.0	26	32	1.7	1.5	1,728

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 = 4.97 ft = 59.6 in.								

Frost_depth_{modberg} := 59.6·in

Frost_depth_{modberg} = 4.967·ft

Use Frost Depth = 5.0 feet for design

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 534
PRECAST STRUCTURAL CONCRETE
(Precast Structural Concrete Arches, Box Culverts)

534.10 Description The Contractor shall design, manufacture, furnish, and install elements, precast structural concrete structures, arches, or box culverts and associated wings, headwalls, and appurtenances, in accordance with the contract documents.

534.20 Materials Structural precast elements for the arch or box culvert and associated precast elements shall meet the requirements of the following Subsection:

Structural Precast Concrete Units

712.061

Grout, concrete patching material, and geotextiles shall be one of the products listed on the Department's list of prequalified materials, unless otherwise approved by the Department.

Box culvert bedding and backfill material shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill, with the additional requirement that the maximum particle size be limited to 4 inches.

534.30 Design Requirements The Contractor shall design the precast structural concrete structure in accordance with the AASHTO LRFD Bridge Design Specifications 5th Edition 2010. The design live load shall be as follows: *modified HL-93 Strength I for LRFD method. *(modify HL-93 by increasing all wheel loads by a factor of 1.25)

The Contractor shall submit design calculations and shop drawings for the precast structure to the Department for approval. A Registered Professional Engineer, licensed in accordance with State of Maine laws, shall sign and seal all design calculations and drawings. The Contractor shall submit a bridge rating on the Department's Standard Bridge Rating Summary Sheet with the design calculations. Drawings shall conform with Section 105.7 - Working Drawings.

The Contractor shall submit the following items for review by the Resident at least ten working days prior to production:

- A) The name and location of the manufacturer.
- B) Method of manufacture and material certificates.
- C) Description of method of handling, storing, transporting, and erecting the members.
- D) Shop Drawings with the following minimum details:
 - 1) Fully dimensioned views showing the geometry of the members, including all projections, recesses, notches, openings, block outs, and keyways.
 - 2) Details and bending schedules of reinforcing steel including the size, spacing, and location. Reinforcing provided under lifting devices shall be shown in detail.
 - 3) Details and locations of all items to be embedded.
 - 4) Total mass (weight) of each member.

534.40 Construction Requirements The applicable provisions of Subsection 535.10 - Forms and Casting Beds and Subsection 535.20 – Finishing Concrete and Repairing Defects shall be met.

Manufacture of Precast Units The internal dimensions shall not vary by more than 1 percent from the design dimensions or 1 ½ inches, whichever is less. The haunch dimensions shall not vary by more than ¾ inch from the design dimension. The dimension of the legs shall not vary by more than ¼ inch from the dimension shown on the approved shop drawings.

The slab and wall thickness shall not be less than the design thickness by more than ¼ inch. A thickness greater than the design thickness shall not be cause for rejection.

Variations in laying lengths of two opposite surfaces shall not be more than ⅝ inch in any section, except where beveled ends for laying of curves are specified.

The under-run in length of any section shall not be more than ½ inch.

The cover of concrete over the outside circumferential reinforcement shall be 2 inch minimum. The concrete cover over the inside reinforcement shall be 1 ½ inch minimum. The clear distance of the end of circumferential wires shall not be less than 1 inch or more than 2 inch from the end of the sections. Reinforcement shall be single or multiple layers of welded wire fabric or a single layer of deformed billet steel bars.

Welded wire fabric shall meet the space requirements and contain sufficient longitudinal wires extending through the section to maintain the shape and position of the reinforcement. Longitudinal distribution reinforcement may be welded wire fabric or deformed billet steel bars which meet the spacing requirements. The ends of the longitudinal distribution reinforcement shall be not more than 3 inches from the ends of the sections.

The inside circumferential reinforcing steel for the haunch radii or fillet shall be bent to match the radii or fillets of the forms.

Tension splices in the reinforcement will not be permitted. For splices other than tension splices, the overlap shall be a minimum of 12 inches for welded wire fabric or billet steel bars. The spacing center to center of the circumferential wires in a wire fabric sheet shall be not less than 2 inches or more than 4 inches. For the wire fabric, the spacing center to center of the longitudinal wires shall not be more than 8 inches. The spacing center to center of the longitudinal distribution steel for either line of reinforcing in the top slab shall be not more than 15 inches.

The members shall be free of fractures. The ends of the members shall be normal to the walls and centerline of the section, within the limits of variation provided, except where beveled ends are specified. The surfaces of the members shall be a smooth steel form or troweled surface finish, unless a form liner is specified. The ends and interior of the assembled structure shall make a continuous line of members with a smooth interior surface.

Defects which may cause rejection of precast units include the following:

- 1) Any discontinuity (crack or rock pocket etc.) of the concrete which could allow moisture to reach the reinforcing steel.
- 2) Rock pockets or honeycomb over 6 inch² in area or over 1 inch deep.
- 3) Edge or corner breakage exceeding 12 inches in length or 1 inch in depth.
- 4) Extensive fine hair cracks or checks.
- 5) Any other defect that clearly and substantially impacts the quality, durability, or maintainability of the structure as measured by accepted industry standards.

The Contractor shall store and transport members in a manner to prevent cracking or damage. The Contractor shall not place precast members in an upright position until a compressive strength of at least 4350 psi is attained.

Installation of Precast Units The Contractor shall not ship precast members until sufficient strength has been attained to withstand shipping, handling and erection stresses without cracking, deformation, or spalling (but in no case less than 4350 psi).

The Contractor shall set precast members on ½ inch neoprene pads during shipment to prevent damage to the section legs. The Contractor shall repair any damage to precast members resulting from shipping or handling by saw cutting a minimum of ½ inch deep around the perimeter of the damaged area and placing a polymer-modified cementitious patching material.

When footings are required, the Contractor shall install the precast members on concrete footings that have reached a compressive strength of at least 2900 psi. The Contractor shall construct the completed footing surface to the lines and grades shown on the plans. When checked with a 10 feet straightedge, the surface shall not vary more than ¼ inch in 10 feet. The footing keyway shall be filled with a non-shrink flowable cementitious grout with a design compressive strength of at least 5075 psi.

The Contractor shall fill holes that were cast in the units for handling, with either Portland cement mortar, or with precast plugs secured with Portland cement mortar or other approved adhesive. The Contractor shall completely fill the exterior face of joints between precast members with an approved material and cover with a minimum 12 inch wide joint wrap. The surface shall be free of dirt and deleterious materials before applying the filler material and joint wrap. The Contractor shall install the external wrap in one continuous piece over each member joint, taking care to keep the joint wrap in place during backfilling. The Contractor shall seal the joints between the end unit and attached elements with a non-woven geotextile. The Contractor shall install and tighten the bolts fastening the connection plate(s) between the elements that are designed to be fastened together as designated by the manufacturer. Final assembly shall be approved by the manufacturer's representative prior to backfilling.

The Contractor shall place and compact the bedding material as shown on the plans prior to lifting and setting the box culvert sections. The Contractor shall backfill the structure in accordance with the manufacturer's instructions and the Contract Documents. The Contractor shall uniformly distribute backfill material in layers of not more than 8 inches in depth, loose measure, and thoroughly compact each layer using approved compactors before successive layers are placed. The Contractor shall compact the Granular Borrow bedding

and backfill in accordance with Section 203.12 - Construction of Earth Embankment with Moisture and Density Control, except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180, Method C or D. The Contractor shall place and compact backfill without disturbance or displacement of the wall units, keeping the fill at approximately the same elevation on both sides of the structure. Whenever a compaction test fails, the Contractor shall not place additional backfill over the area until the lift is re-compacted and a passing test achieved.

The Contractor shall use hand-operated compactors within 5 feet of the precast structure as well as over the top until it is covered with at least 12 inches of backfill. Equipment in excess of 12 ton shall not use the structure until a minimum of 24 inches of backfill cover is in place and compacted.

534.50 Method of Measurement The Department will measure Precast Structural Concrete Arch or Box Culvert for payment per Lump Sum each, complete in place and accepted.

534.60 Basis of Payment The Department will pay for the accepted quantity of Precast Structural Concrete Arch or Box Culvert at the Contract Lump Sum price, such payment being full compensation for all labor, equipment, materials, professional services, and incidentals for furnishing and installing the precast concrete elements and accessories. Falsework, reinforcing steel, jointing tape, grout, cast-in-place concrete fill or grout fill for anchorage of precast wings and/or other appurtenances is incidental to the Lump Sum pay item. Cast-in-place concrete, reinforcing steel in cast-in-place elements, excavation, backfill material, and membrane waterproofing will be measured and paid for separately under the provided Contract pay items. Pay adjustments for quality level will not be made for precast concrete.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
534.71 Precast Concrete Box Culvert	Lump Sum

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