MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

ROYAL RIVER BRIDGE
OVER ROYAL RIVER
AUBURN, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Royal River Bridge over the Royal River in Auburn, Maine. The MaineDOT Bridge Program has selected the Royal River Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure. The proposed 38 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 arch stem wall/pile caps founded on driven H-piles is a viable foundation system for use at the site. The piles will be driven to bedrock. Piles should be fitted with driving points to protect the tips and improve penetration. 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 abutment. 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 maximum factored axial 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. 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 PCMG wall shall be constructed with a 1.5 foot thick layer of crushed stone placed vertically along the inside face of the wall units. The PCMG walls shall consist of Class "LP" concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor. 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 5.5 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 the Geotechnical Design Report is to present geotechnical recommendations for the replacement of the Royal River Bridge over the Royal River in Auburn, 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 Royal River Bridge was constructed in 1970 and consists of a 24 foot long, single span, steel girder superstructure supported on concrete abutments. There is no available information about the existing abutment foundations. The 2007 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge superstructure is in "satisfactory" condition (rating of 6), the substructure is in "poor" condition (rating of 4) and the deck is in "fair" condition (rating of 5). The Bridge Sufficiency Rating is 59.4. The bridge has a scour critical rating of "U" meaning that the bridge has unknown foundations that have not been evaluated for scour. A gabion wall supports the roadway and abutment on the southeast corner of the structure. This wall is out of plumb and the gabions have shifted. A stacked granite block wall supports the roadway and abutment on the northeast corner of the structure. Inspection records note that the footings are undermined.

The MaineDOT Bridge Program has selected the Royal River Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure developed by the University of Maine's Advance Engineering Wood Composites (AEWC) Center in Orono, Maine. The tubes are inflated at the site of the bridge and then infused with resin. After hardening, the tubes are lowered into place and filled with concrete. The proposed arch structure will have a span length of approximately 38 feet and will be founded on an arch stem wall/pile cap on driven H-piles. The proposed bridge alignment will match into the existing with a minor lateral shift to the east in order to accommodate the wider road section. The roadway grade will match the existing grade behind both arch stem wall/pile caps. The bridge will be closed to traffic during the replacement.

2.0 GEOLOGIC SETTING

The Royal River Bridge in Auburn carries Old Danville Road over the Royal River approximately 1.8 miles north of Routes 202, 100 and 4 as shown on Sheet 1 - Location Map found at the end of this report. The Royal River flows in a southerly direction into Casco Bay.

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 Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as carboniferous muscovite-biotite granite with abundant metasedimentary inclusions. This intrusive plutonic rock is identified as the Sebago Pluton.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling two (2) test borings at the site. Test boring BB-ARR-101 was drilled at the location of existing Abutment No. 1 (south). Test boring BB-ARR-102 was drilled at the location of existing Abutment No. 2 (north).

The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The borings were drilled between August 12 and September 3, 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 4 - Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. 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 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 eight (8) standard grain size analyses, six (6) grain size analyses with hydrometer and one (1) Atterberg Limits test. 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 4 - 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 silt, underlain by native sand, underlain by granite bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 – 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 behind both of the existing abutments. The thickness of the layer was approximately 4.0 feet in both borings. The soil generally consisted of brown, moist, fine to coarse sand with little to some gravel and little to trace silt. Two corrected SPT N-values in the fill sand were both 20 blows per foot (bpf) indicating that the soil is medium dense in consistency. Water contents from two (2) samples obtained within the fill sand layer range from approximately 4% to 6%. Two (2) grain size analyses conducted on samples of the fill sand indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SW-SM or SM by the Unified Soil Classification System.

5.2 Silt

Silt was encountered beneath the fill sand behind the existing abutments. The thickness of the silt layer was approximately 6.0 feet in both borings. The silt generally consisted of olive, moist, silt, with some sand, some clay and trace gravel. Corrected SPT N-values in the silt layer ranged from 8 to 11 bpf indicating that the silt is medium stiff in consistency. Water contents from two (2) samples obtained within the silt layer range from approximately 15% to 16%. Two (2) grain size analyses with hydrometer conducted on samples from the silt layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML by the Unified Soil Classification System.

5.3 Native Sand

A layer of native sand was encountered beneath the silt. The thickness of the sand layer ranged from approximately 30.0 feet in boring BB-ARR-102 to approximately 35.2 feet thick boring BB-ARR-101. The native sand generally consisted of olive, moist, fine to coarse sand, some silt, little clay, little clay; grey, wet, silty fine to medium sand with trace organics; and olive, brown and grey, wet, fine to coarse sand with trace to some silt, little to some gravel, and trace to little clay. Corrected SPT N-values in the native sand layer ranged from weight of hammer (WOH) to 45 bpf indicating that the soil is very loose to dense in consistency. Water contents from nine (9) samples obtained within the native sand layer range from approximately 6% to 26%. Nine (9) grain size analyses conducted on samples from the native sand layer indicate that the soil is classified as an A-4, A-1-b or A-2-4 by the AASHTO Classification System and a SC-SM, SM, SW or SW-SM by the Unified Soil Classification System.

5.4 Clayey Silt

In boring BB-ARR-101 the native sand layer is intersected by a layer of clayey silt at a depth of approximately 19.5 feet below ground surface. The thickness of the clayey silt layer was approximately 4.5 feet. This layer was found to be grey, wet, clayey silt, with trace fine sand. A corrected SPT N-value in the clayey silt layer was 3 bpf indicating that the soil is soft in consistency. One (1) water content from a sample obtained within the clayey layer was approximately 29%. One (1) grain size analysis with hydrometer conducted on a sample from this layer indicates that the soil is classified as an A-4 by the AASHTO Classification System and a CL by the Unified Soil Classification System.

Table 5-1 below summarizes the results of the Atterberg Limits test from one (1) sample of the clayey silt:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ARR-101 5D	28.7	31	19	12	0.81

Table 5-1 - Summary of Atterberg Limits Testing Results for Clayey Silt Sample

Interpretation of these results indicates that the clayey silt is normally consolidated.

5.5 Bedrock

Bedrock was encountered and cored in both of the borings. 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- ARR -101/ Abutment No. 1	45.2 feet	136.8 feet	15 – 33%
BB- ARR -102/ Abutment No. 2	41.0 feet	143.8 feet	0 – 70%

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

The bedrock is identified as white, brown, grey and black, coarse grained, GRANITE with mica and pyrite. The rock quality designation (RQD) of the bedrock was determined to range from 0 to 70 percent indicating a rock mass quality of very poor to fair quality.

5.6 Groundwater

Groundwater was observed at a depths ranging from approximately 6.0 feet to 17.0 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 the Royal River Bridge site as a location to install a rigidified, inflatable, composite, tubular arch bridge structure developed by the University of Maine's AEWC Advanced Structures & Composites Center in Orono, Maine. AEWC's tubular arches are made of Fiber Reinforced Plastic (FRP) composite materials. The tubes are inflated at the site of the bridge and then infused with resin. After hardening, the tubes are 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 AEWC 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 strip footings 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-ARR-101	162.5 feet	45.2 feet	136.8 feet	26 feet
Abutment No.2 BB-ARR-103	162.5 feet	41.0 feet	143.8 feet	19 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 4th 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 column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. 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 0.60 (good driving conditions) and a λ of 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 φ_{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* ϕ_c=0.60 λ=0	Geotechnical Resistance φ_{stat}=0.45	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	465	357	270	270
HP 12x74	654	498	417	417
HP 14x73	642	444	410	410
HP 14x89	783	539	484	484
HP 14x117	1032	706	506	506

* based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

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 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 strength limit state should not exceed the factored drivability resistance shown in 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 ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LFRD 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. For preliminary analysis, the H-piles can be assumed fully embedded and λ can be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances.

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* $\phi=1.0$ $\lambda=0$	Geotechnical Resistance $\varphi=1.0$	Drivability Resistance $\varphi=1.0$	Governing Resistance
HP 12x53	775	793	415	415
HP 12x74	1090	1106	642	642
HP 14x73	1070	986	631	631
HP 14x89	1305	1198	744	744
HP 14x117	1720	1568	779	779

*based on preliminary assumption of λ=0 for the lower portion of the pile in only axial compression (no flexure)

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 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. In light of the short pile indicated at Abutment No. 2 (approximately 19 feet), the designer should verify that the piles at this abutment achieve a fixed condition at the pile tip.

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)
Silt	178 - 172	181 - 175	Above	0.0667 (115)
Upper Native Sand	172 - 162	-	Above	0.0694 (120)
Clayey Silt	162 - 158	-	Below	0.0307 (53)
Lower Native Sand	158 - 136	175 - 145	Below	0.0307 (53)

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
Silt	500	1500	0.005	-
Upper Native Sand	25	-	-	30°
Clayey Silt	30	375	0.020	-
Lower Native Sand	60	-	-	32°

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 8 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 ϕ= 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. 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, γ_{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: ϕ = 32 degrees, γ = 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 backface to edge of traffic = 0 feet	Distance from wall backface 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, $\phi_{b,}$ 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 ($1/4^{th}$) 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 30º at the foundation soil to soil infill interface and a maximum frictional coefficient of 0.8x(tan 30º) 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 PCMG wall shall be constructed with a 1.5 foot thick layer of crushed stone placed vertically along the inside face of the wall units. The layer of crushed stone shall extend vertically from the bottom course of the wall units to the top course of the wall units. The crushed stone shall be separated from the surrounding backfill with erosion control geotextile. The PCMG walls shall consist of Class “LP” concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor.

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:

At Abutment No. 1

- Average diameter of particle at 50 percent passing, D_{50} = 0.077 mm
- Average diameter of particle at 95 percent passing, D_{95} = 0.056 mm
- Soil Classification AASHTO Soil Type A-4

At Abutment No. 2

- Average diameter of particle at 50 percent passing, D_{50} = 0.80 mm
- Average diameter of particle at 95 percent passing, D_{95} = 12.7 mm
- Soil Classification AASHTO Soil Type A-1-b

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 1224 F-degree days. In a granular soil with a water content of approximately 15%, this correlates to a frost depth of approximately 5.5 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.5 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 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the Maine DOT BDG, the Royal River Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak Ground Acceleration coefficient (PGA) = 0.088g
- Site Class D (stiff soils with an N-value between 15 and 50 bpf)
- Acceleration coefficient (A_s) = 0.141
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.283g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.112g
- Seismic Zone 1 (based on S_{D1} less than or equal to 0.15g)

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

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.

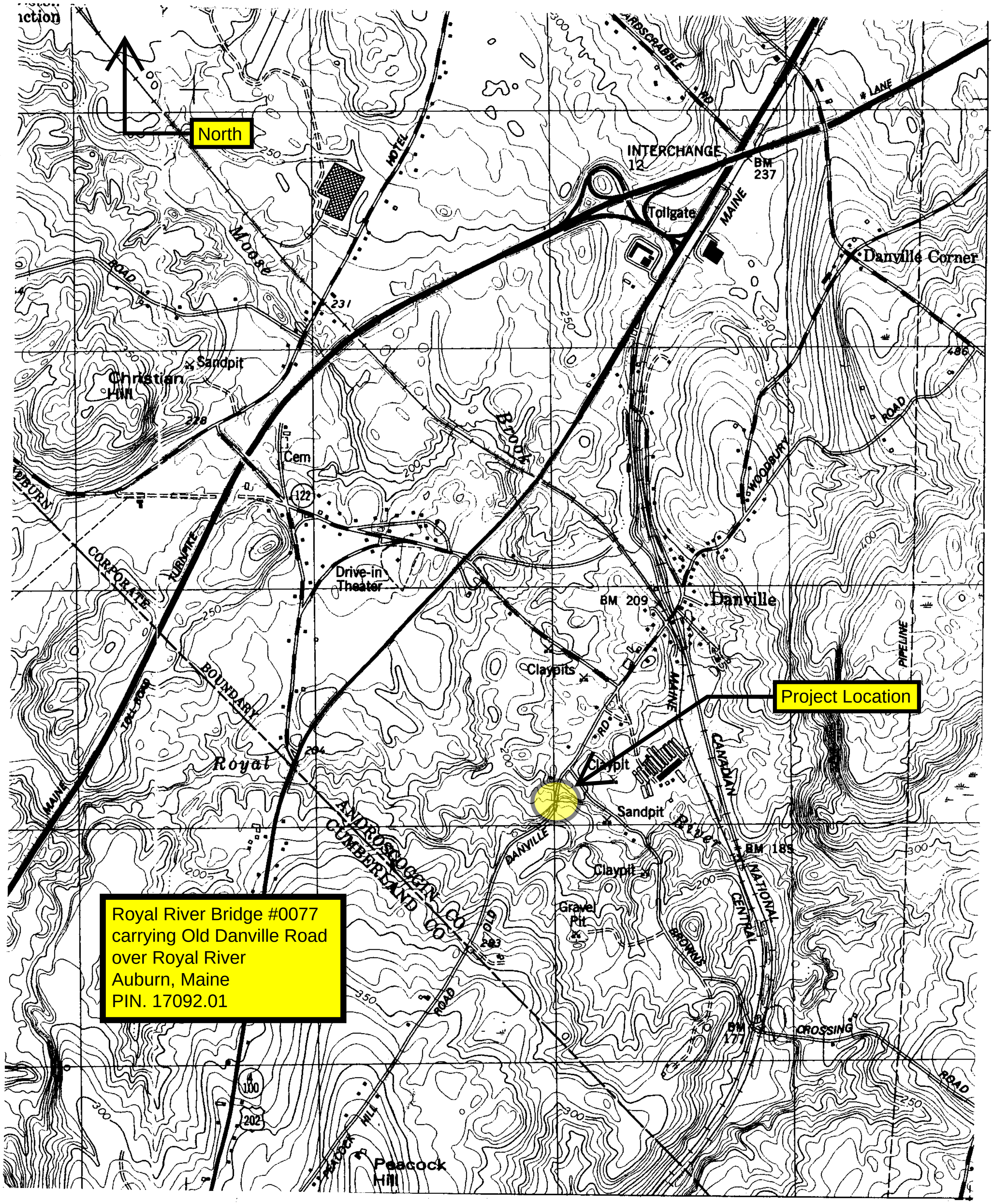
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements 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 the Royal River Bridge in Auburn in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets



MINOT QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
SE/4 POLAND 15' QUADRANGLE

SCALE 1:24 000

CONTOUR INTERVAL 10 FEET

Filename: ...\01\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:12/16/2009

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1709(201)X

BRIDGE NO. 0077 | PIN 17092.01 | BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	OCT 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

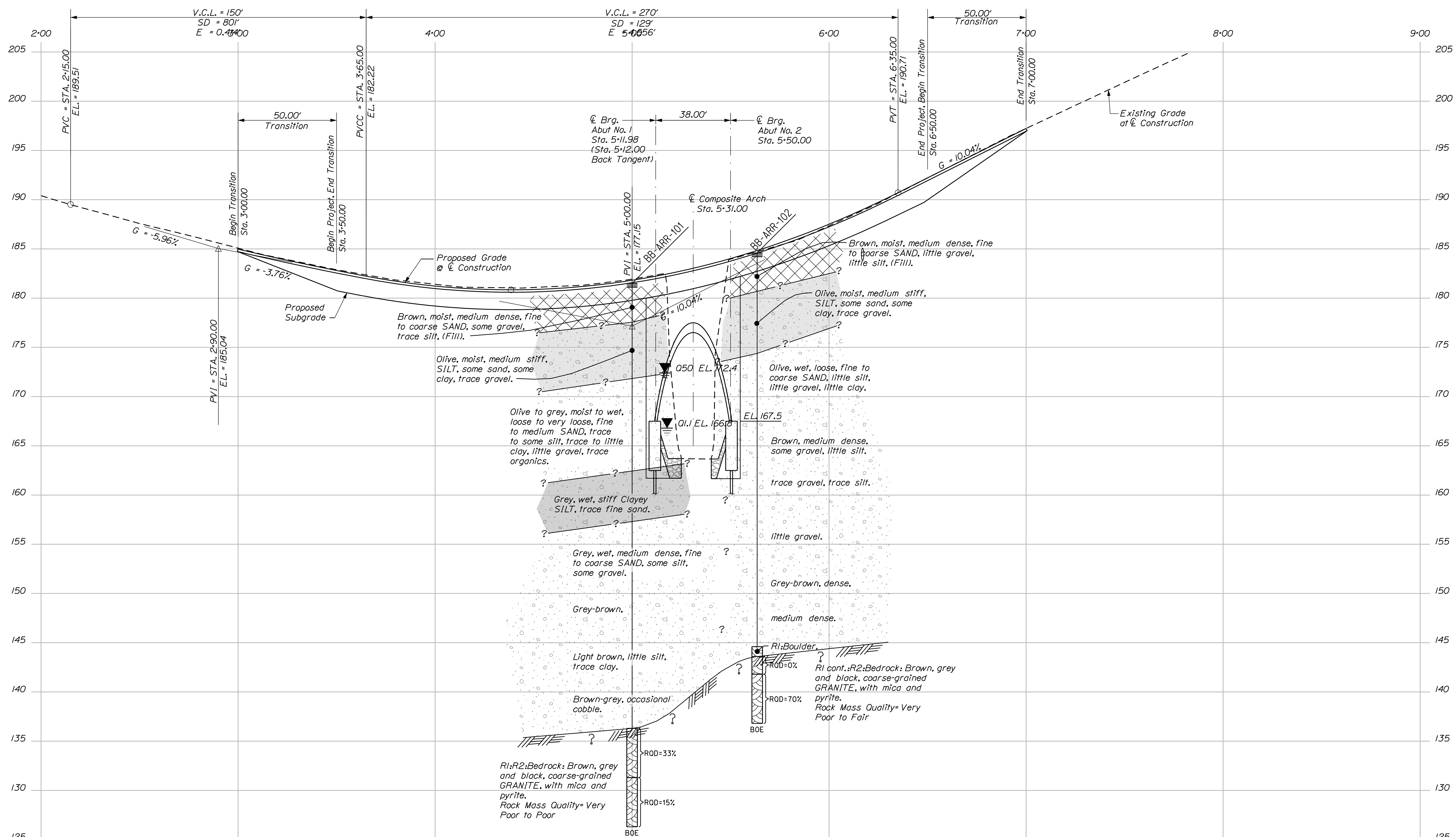
SIGNATURE | P.E. NUMBER | DATE

ROYAL RIVER BRIDGE
ROYAL RIVER
AUBURN ANDROSCOGGIN COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4

Filename: ...\01\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:12/16/2009



PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

Boring No., Offset, if shown

Strata Interface

Boring

RQD: Rock Quality Designation for Rock Core Sample

BOE: Bottom Of Exploration

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1709(201)X

PIN 17092.01

BRIDGE PLANS

BRIDGE NO. 0077

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K. MAGUIRE	
CHECKED-REVIEWED	T. WHITE	OCT 2009
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

ROYAL RIVER BRIDGE
ROYAL RIVER
AUBURN ANDROSCOGGIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 4

Filename: ...\msta\008_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:12/16/2009

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Bridge #0077 on Old Danville Road over Royal River.
Location: Auburn, Maine

Boring No.: BB-ARR-101
PIN: 17092.01

Driller:	MaineDOT	Elevation (ft.)	181.5	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:	8/12/09; 07:00-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+00, 8.1 Lt.	Casing ID/OD:	HW	Water Level*:	17.0' 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
	1D	24/16	1.00 - 3.00	6/7/7/8	14	20	
5	2D	24/15	5.00 - 7.00	2/3/3/3	6	8	
10	3D	24/14	10.00 - 12.00	2/3/3/5	6	8	
15	4D	24/22	15.00 - 17.00	WOH/WOH/WOH/WOH	---		57
							43
							53
							58
							69
20	5D	24/18	20.00 - 22.00	6/1/1/1	2	3	47
							33
							33
							31
							41
25	6D	24/14	25.00 - 27.00	6/6/9/21	15	21	49
							93
							64
							66
							68
30	7D	24/6	30.00 - 32.00	16/13/6/6	19	27	45
							72
							68
							85
							83
35	8D	24/10	35.00 - 37.00	5/5/6/7	11	15	79
							88
							90
							122
							135
40	9D	24/5	40.00 - 42.00	11/5/8/7	13	18	113
							129
							128
							166
							238
45	10D R1	2.4/2.4 60/57	45.00 - 45.20 45.20 - 50.20	30(2.4") RQD = 33%	---		NQ-2
50	R2	60/60	50.20 - 55.20	RQD = 15%			
55							
60							
65							
70							
75							

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
181.10	0.40	Pavement.	
177.50	4.00	Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#246321 A-1-b, SW-SM WC=3.9%
171.50	10.00	Olive, moist, medium stiff, SILT, some sand, some clay, trace gravel.	G#246322 A-4, CL-ML WC=15.7%
		Olive, moist, loose, fine to coarse SAND, some silt, little clay, little gravel.	G#246323 A-4, SC-SM WC=11.0%
162.00	19.50	Grey, wet, very loose, Silty fine to medium SAND, trace organics.	G#246324 A-4, SM WC=26.1%
157.50	24.00	Grey, wet, soft, Clayey SILT, trace fine sand.	G#246325 A-4, CL WC=28.7% LL=31 PL=19 PI=12
		Grey, wet, medium dense, fine to coarse SAND, some silt, some gravel.	G#246326 A-2-4, SM WC=13.5%
		Grey-brown, wet, medium dense, fine to coarse SAND, some silt, some gravel.	
		Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#246327 A-1-b, SW WC=11.3%
136.30		Brown-grey, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay, occasional cobble.	
	45.20	Brown, wet, fine to coarse SAND, some gravel, little silt, trace clay.	G#246328 A-2-4, SC-SM WC=11.9%
126.30	55.20	Top of Bedrock at Elev. 136.3'. Bedrock: Brown, grey and black, coarse- grained GRANITE with mica and pyrite. Rock Mass Quality = Very Poor to Poor. R1:Core Times (min:sec) 45.2-46.2' (2:04) 46.2-47.2' (2:04) 47.2-48.2' (2:00) 48.2-49.2' (2:00) 49.2-50.2' (2:30) 95% Recovery R2:Core Times (min:sec) 50.2-51.2' (2:40) 51.2-52.2' (2:15) No water flow 52.2-53.2' (2:00) 53.2-54.2' (2:00) 54.2-55.2' (1:42) 100% Recovery	
		Bottom of Exploration at 55.20 feet below ground surface.	

Remarks:

400-500 lbs down pressure on Core Barrel.

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

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

Page 1 of 1

Boring No.: BB-ARR-101

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Bridge #0077 on Old Danville Road over Royal River.
Location: Auburn, Maine

Boring No.: BB-ARR-102
PIN: 17092.01

Driller:	MaineDOT	Elevation (ft.)	184.6	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:	9/3/09; 07:30-?	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+63.5, 3.6 Rt.	Casing ID/OD:	HW	Water Level*:	6.0' 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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows
0							SSA
	1D	24/19	1.00 - 3.00	11/8/6/6	14	20	
5	2D	24/16	5.00 - 7.00	3/4/4/5	8	11	
10	3D	24/6	10.00 - 12.00	WOH/WOH/WOH/WOH	---		2
							2
							6
							24
							42
15	4D	24/8	15.00 - 17.00	12/11/9/11	20	28	7
							30
							43
							43
							42
20	5D	24/10	20.00 - 22.00	9/8/7/7	15	21	16
							30
							38
							30
							33
25	6D	24/11	25.00 - 27.00	13/7/7/10	14	20	19
							31
							40
							52
							67
30	7D	24/12	30.00 - 32.00	21/14/18/12	32	45	30
							48
							62
							50
							64
35	8D	24/12	35.00 - 37.00	22/12/14/11	26	36	37
							66
							49
							36
							100
40	R1	33.6/ 33.6	40.00 - 42.80	RQD = 0%			NQ-2
	R2	60/58	42.80 - 47.80	RQD = 70%			
45							
50							

Elevation (ft.)	Depth (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
184.20	0.40	Pavement.	
180.60	4.00	Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#246329 A-1-b, SM WC=6.0%
174.60	10.00	Olive, moist, medium stiff, SILT, some sand, some clay, trace gravel.	G#246330 A-4, CL-ML WC=15.3%
		Olive, wet, very loose, fine to coarse SAND, little silt, little gravel, little clay.	G#246331 A-4, SC-SM WC=12.8%
		Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt.	G#246332 A-1-b, SM WC=5.5%
		Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#246333 A-1-b, SW-SM WC=12.8%
		Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#246334 A-1-b, SW WC=14.4%
		Grey-brown, wet, dense, fine to coarse SAND, little gravel, trace silt.	
144.60	40.00	Similar to above, medium dense.	
143.60	41.00	R1:Boulder from 40.0-41.0' bgs. R1:Core Times (min:sec) 40.0-41.0' (1:50)	
136.80	47.80	Top of Bedrock at Elev. 143.6'. Bedrock: Brown, grey and black, coarse grained GRANITE with mica and pyrite. Rock Mass Quality = Very Poor to Fair. 41.0-42.0' (2:55) 42.0-42.8' (5:30) 100% Recovery Core Blocked R2:Core Times (min:sec) 42.8-43.8' (3:00) No water return 43.8-44.8' (3:00) 44.8-45.8' (1:50) 45.8-46.8' (2:00) 46.8-47.8' (1:40) 97% Recovery	
		Bottom of Exploration at 47.80 feet below ground surface.	

Remarks:

100-200 lbs down pressure on Core Barrel.

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

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

Page 1 of 1

Boring No.: BB-ARR-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1709(201)X

BRIDGE NO. 0077   PIN 17092.01   BRIDGE PLANS

		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	OCT 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

ROYAL RIVER BRIDGE
ROYAL RIVER
AUBURN   ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Bridge #0077 on Old Danville Road over Royal River.
Location: Auburn, Maine

Boring No.: BB-ARR-101
PIN: 17092.01

Driller:	MaineDOT	**Elevation (ft.)**	181.5	**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:	8/12/09; 07:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+00, 8.1 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	17.0' 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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	181.10		Pavement. 0.40	
	1D	24/16	1.00 - 3.00	6/7/7/8	14	20				Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	G#246321 A-1-b, SW-SM WC=3.9%
								177.50		4.00	
5	2D	24/15	5.00 - 7.00	2/3/3/3	6	8				Olive, moist, medium stiff, SILT, some sand, some clay, trace gravel.	G#246322 A-4, CL-ML WC=15.7%
10	3D	24/14	10.00 - 12.00	2/3/3/5	6	8		171.50		10.00 Olive, moist, loose, fine to coarse SAND, some silt, little clay, little gravel.	G#246323 A-4, SC-SM WC=11.0%
15	4D	24/22	15.00 - 17.00	WOH/WOH/WOH/ WOH	---		57			Grey, wet, very loose, Silty fine to medium SAND, trace organics.	G#246324 A-4, SM WC=26.1%
							43				
							53				
							58				
							69	162.00		19.50	
20	5D	24/18	20.00 - 22.00	6/1/1/1	2	3	47			Grey, wet, soft, Clayey SILT, trace fine sand.	G#246325 A-4, CL WC=28.7% LL=31 PL=19 PI=12
							33				
							33				
							31				
							41	157.50		24.00	
25											

Remarks:

400-500 lbs down pressure on Core Barrel.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Bridge #0077 on Old Danville Road over Royal River.	**Boring No.:**	BB-ARR-101		
Location:	Auburn, Maine	**PIN:**	17092.01		
Driller:	MaineDOT	**Elevation (ft.)**	181.5	**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:	8/12/09; 07:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+00, 8.1 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	17.0' 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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/14	25.00 - 27.00	6/6/9/21	15	21	49			Grey, wet, medium dense, fine to coarse SAND, some silt, some gravel.	G#246326 A-2-4, SM WC=13.5%
							93				
							64				
							66				
							68				
30	7D	24/6	30.00 - 32.00	16/13/6/6	19	27	45			Grey-brown, wet, medium dense, fine to coarse SAND, some silt, some gravel.	
							72				
							68				
							85				
							83				
35	8D	24/10	35.00 - 37.00	5/5/6/7	11	15	79			Light brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt.	G#246327 A-1-b, SW WC=11.3%
							88				
							90				
							122				
							135				
40	9D	24/5	40.00 - 42.00	11/5/8/7	13	18	113			Brown-grey, wet, medium dense, fine to coarse SAND, some gravel, little silt, trace clay, occasional cobble.	
							129				
							128				
							166				
							238				
45	10D R1	2.4/2.4 60/57	45.00 - 45.20 45.20 - 50.20	30(2.4") RQD = 33%	---		NQ-2	136.30		Brown, wet, fine to coarse SAND, some gravel, little silt, trace clay. 45.20 Top of Bedrock at Elev. 136.3'. Bedrock: Brown, grey and black, coarse- grained GRANITE with mica and pyrite. Rock Mass Quality = Very Poor to Poor. R1:Core Times (min:sec) 45.2-46.2' (2:04) 46.2-47.2' (2:04) 47.2-48.2' (2:00) 48.2-49.2' (2:00)	G#246328 A-2-4, SC-SM WC=11.9%
50											

Remarks:

400-500 lbs down pressure on Core Barrel.

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Bridge #0077 on Old Danville Road over Royal River. **Location:** Auburn, Maine	**Boring No.:** BB-ARR-101 **PIN:** 17092.01

Driller:	MaineDOT	**Elevation (ft.)**	181.5	**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:	8/12/09; 07:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+00, 8.1 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	17.0' 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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	R2	60/60	50.20 - 55.20	RQD = 15%						49.2-50.2' (2:30) 95% Recovery R2:Core Times (min:sec) 50.2-51.2' (2:40) 51.2-52.2' (2:15) No water flow 52.2-53.2' (2:00) 53.2-54.2' (2:00) 54.2-55.2' (1:42) 100% Recovery	
55								126.30		55.20 **Bottom of Exploration at 55.20 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

400-500 lbs down pressure on Core Barrel.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Bridge #0077 on Old Danville Road over Royal River.
Location:	Auburn, Maine
Boring No.:	BB-ARR-102
PIN:	17092.01

Driller:	MaineDOT	**Elevation (ft.)**	184.6	**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:	9/3/09; 07:30-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+63.5, 3.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	6.0' 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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	184.20		Pavement. 0.40	
	1D	24/19	1.00 - 3.00	11/8/6/6	14	20				Brown, moist, medium dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#246329 A-1-b, SM WC=6.0%
								180.60		4.00	
5	2D	24/16	5.00 - 7.00	3/4/4/5	8	11				Olive, moist, medium stiff, SILT, some sand, some clay, trace gravel.	G#246330 A-4, CL-ML WC=15.3%
10	3D	24/6	10.00 - 12.00	WOH/WOH/WOH/ WOH	---		2	174.60		10.00 Olive, wet, very loose, fine to coarse SAND, little silt, little gravel, little clay.	G#246331 A-4, SC-SM WC=12.8%
							2				
							6				
							24				
							42				
15	4D	24/8	15.00 - 17.00	12/11/9/11	20	28	7			Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt.	G#246332 A-1-b, SM WC=5.5%
							30				
							43				
							43				
							42				
20	5D	24/10	20.00 - 22.00	9/8/7/7	15	21	16			Brown, wet, medium dense, fine to coarse SAND, trace gravel, trace silt.	G#246333 A-1-b, SW-SM WC=12.8%
							30				
							38				
							30				
25							33				

Remarks:

100-200 lbs down pressure on Core Barrel.

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Bridge #0077 on Old Danville Road over Royal River. **Location:** Auburn, Maine	**Boring No.:** BB-ARR-102 **PIN:** 17092.01

Driller:	MaineDOT	**Elevation (ft.)**	184.6	**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:	9/3/09; 07:30-?	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+63.5, 3.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	6.0' 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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/11	25.00 - 27.00	13/7/7/10	14	20	19			Brown, wet, medium dense, fine to coarse SAND, little gravel, trace silt.	G#246334 A-1-b, SW WC=14.4%
							31				
							40				
							52				
							67				
30	7D	24/12	30.00 - 32.00	21/14/18/12	32	45	30			Grey-brown, wet, dense, fine to coarse SAND, little gravel, trace silt.	
							48				
							62				
							50				
							64				
35	8D	24/12	35.00 - 37.00	22/12/14/11	26	36	37			Similar to above, medium dense.	
							66				
							49				
							36				
							100				
40	R1	33.6/33.6	40.00 - 42.80	RQD = 0%			NQ-2	144.60		40.00 R1:Boulder from 40.0-41.0' bgs. R1:Core Times (min:sec) 40.0-41.0' (1:50)	
								143.60		41.00 Top of Bedrock at Elev. 143.6'. Bedrock: Brown, grey and black, coarse grained GRANITE with mica and pyrite. Rock Mass Quality = Very Poor to Fair. 41.0-42.0' (2:55) 42.0-42.8' (5:30) 100% Recovery Core Blocked	
	R2	60/58	42.80 - 47.80	RQD = 70%						R2:Core Times (min:sec) 42.8-43.8' (3:00) No water return 43.8-44.8' (3:00) 44.8-45.8' (1:50) 45.8-46.8' (2:00) 46.8-47.8' (1:40) 97% Recovery	
45											
								136.80		47.80	
50										**Bottom of Exploration at 47.80 feet below ground surface.**	

Remarks:

100-200 lbs down pressure on Core Barrel.

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Auburn Project Number: 17092.01

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-ARR-101, 1D	5+00	8.1 Lt.	1.0-3.0	246321	1	3.9			SW-SM	A-1-b	0
BB-ARR-101, 2D	5+00	8.1 Lt.	5.0-7.0	246322	1	15.7			CL-ML	A-4	IV
BB-ARR-101, 3D	5+00	8.1 Lt.	10.0-12.0	246323	1	11.0			SC-SM	A-4	III
BB-ARR-101, 4D	5+00	8.1 Lt.	15.0-17.0	246324	1	26.1			SM	A-4	III
BB-ARR-101, 5D	5+00	8.1 Lt.	20.0-22.0	246325	2	28.7	31	12	CL	A-4	III
BB-ARR-101, 6D	5+00	8.1 Lt.	25.0-27.0	246326	2	13.5			SM	A-2-4	II
BB-ARR-101, 8D	5+00	8.1 Lt.	35.0-37.0	246327	2	11.3			SW	A-1-b	0
BB-ARR-101, 10D	5+00	8.1 Lt.	45.0-45.2	246328	2	11.9			SC-SM	A-2-4	II
BB-ARR-102, 1D	5+63.5	3.6 Rt.	1.0-3.0	246329	3	6.0			SM	A-1-b	II
BB-ARR-102, 2D	5+63.5	3.6 Rt.	5.0-7.0	246330	3	15.3			CL-ML	A-4	IV
BB-ARR-102, 3D	5+63.5	3.6 Rt.	10.0-12.0	246331	3	12.8			SC-SM	A-4	III
BB-ARR-102, 4D	5+63.5	3.6 Rt.	15.0-17.0	246332	3	5.5			SM	A-1-b	II
BB-ARR-102, 5D	5+63.5	3.6 Rt.	20.0-22.0	246333	3	12.8			SW-SM	A-1-b	0
BB-ARR-102, 6D	5+63.5	3.6 Rt.	25.0-27.0	246334	3	14.4			SW	A-1-b	0

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.

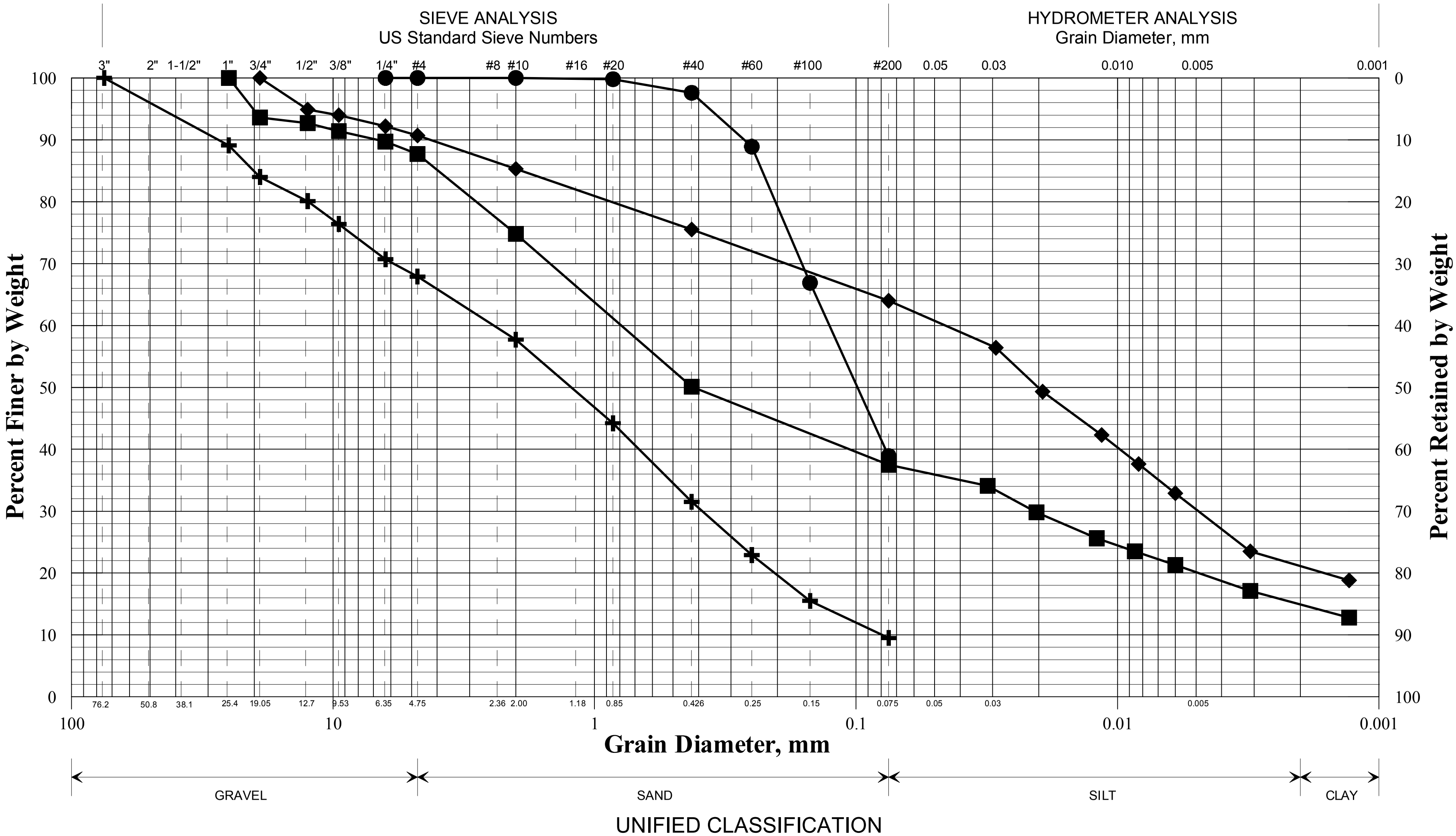
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



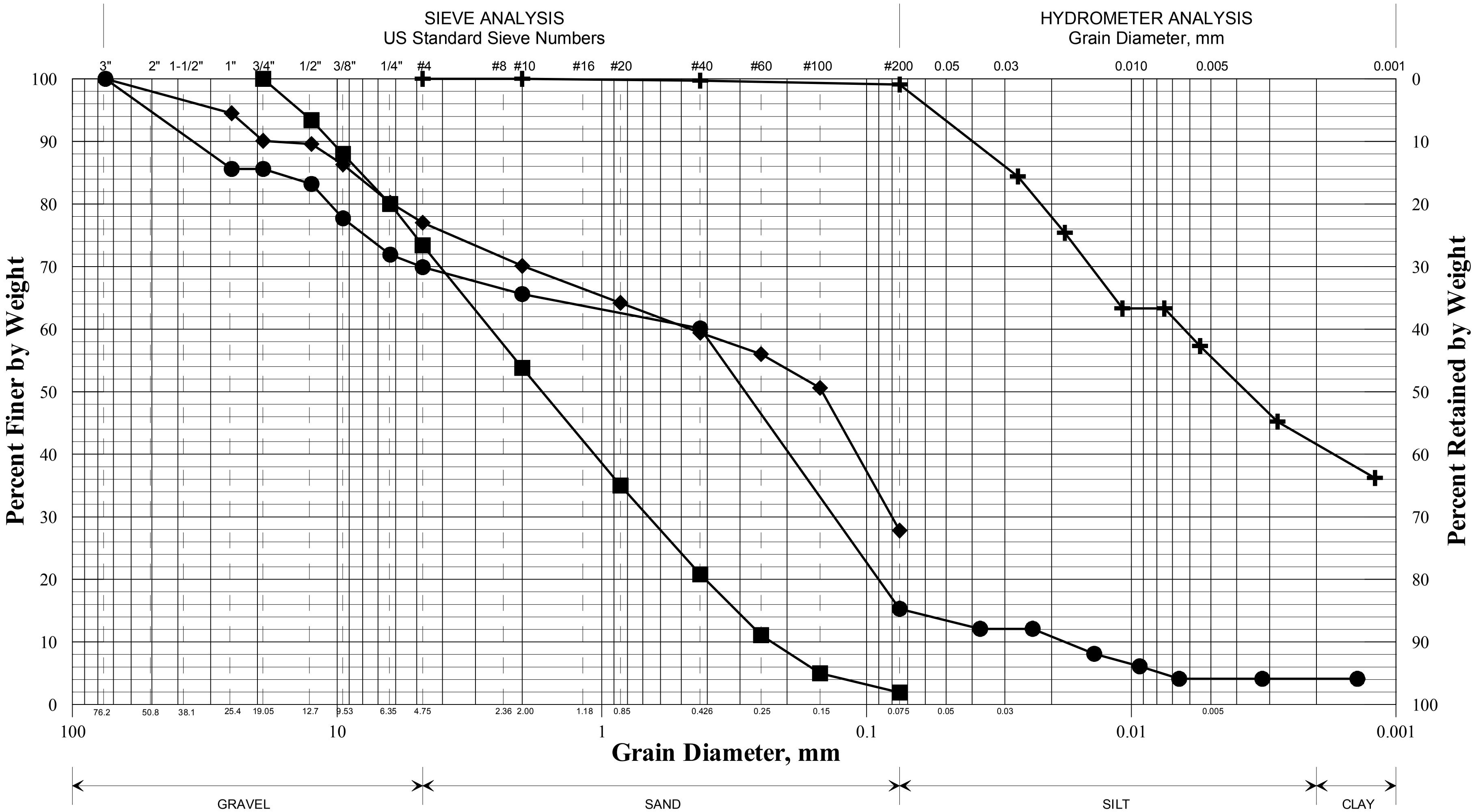
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ARR-101/1D	5+00	8.1 LT	1.0-3.0	SAND, some gravel, trace silt.	3.9			
◆	BB-ARR-101/2D	5+00	8.1 LT	5.0-7.0	SILT, some sand, some clay, trace gravel.	15.7			
■	BB-ARR-101/3D	5+00	8.1 LT	10.0-12.0	SAND, some silt, little clay, little gravel.	11.0			
●	BB-ARR-101/4D	5+00	8.1 LT	15.0-17.0	Silty SAND.	26.1			
▲									
✖									

PIN
017092.01
Town
Auburn
Reported by/Date
WHITE, TERRY A 10/15/2009

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



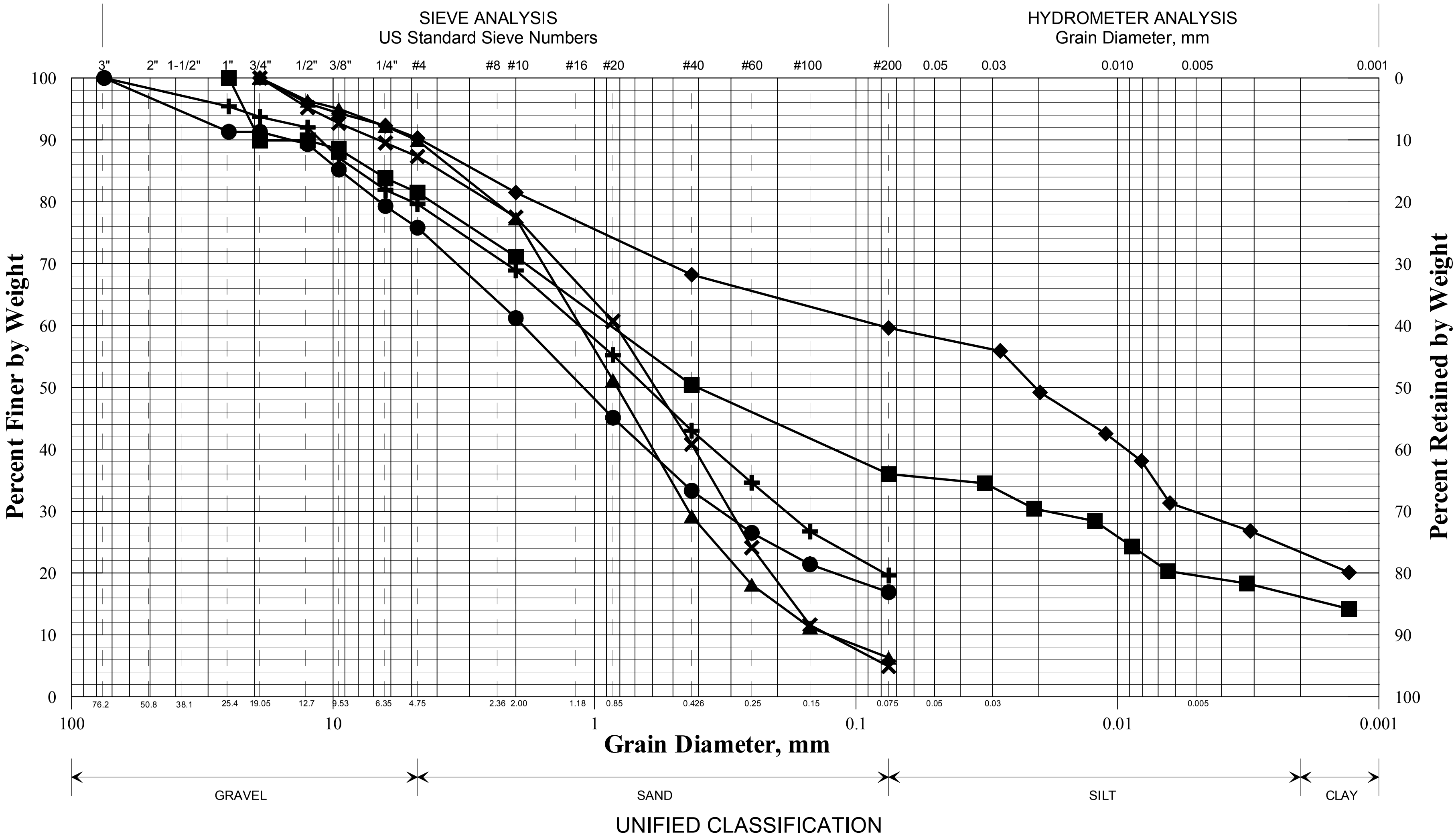
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ARR-101/5D	5+00	8.1 LT	20.0-22.0	Clayey SILT, trace sand.	28.7	31	19	12
◆	BB-ARR-101/6D	5+00	8.1 LT	25.0-27.0	SAND, some silt, some gravel.	13.5			
■	BB-ARR-101/8D	5+00	8.1 LT	35.0-37.0	SAND, some gravel, trace silt.	11.3			
●	BB-ARR-101/10D	5+00	8.1 LT	45.0-45.2	SAND, some gravel, little silt, trace clay.	11.9			
▲									
✖									

PIN	
017092.01	
Town	
Auburn	
Reported by/Date	
WHITE, TERRY A	10/15/2009

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



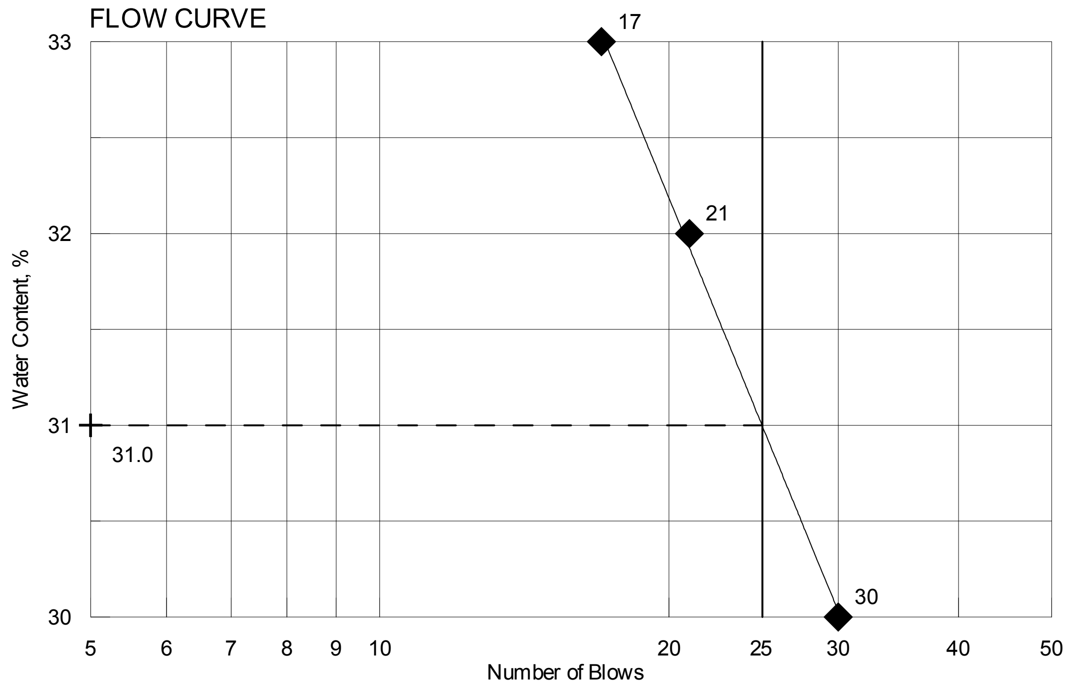
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ARR-102/1D	5+63.5	3.6 RT	1.0-3.0	SAND, little gravel, little silt.	6.0			
◆	BB-ARR-102/2D	5+63.5	3.6 RT	5.0-7.0	SILT, some sand, some clay, trace gravel.	15.3			
■	BB-ARR-102/3D	5+63.5	3.6 RT	10.0-12.0	SAND, little silt, little garvel, little clay.	12.8			
●	BB-ARR-102/4D	5+63.5	3.6 RT	15.0-17.0	SAND, some gravel, little silt.	5.5			
▲	BB-ARR-102/5D	5+63.5	3.6 RT	20.0-22.0	SAND, trace gravel, trace silt.	12.8			
✖	BB-ARR-102/6D	5+63.5	3.6 RT	25.0-27.0	SAND, little gravel, trace silt.	14.4			

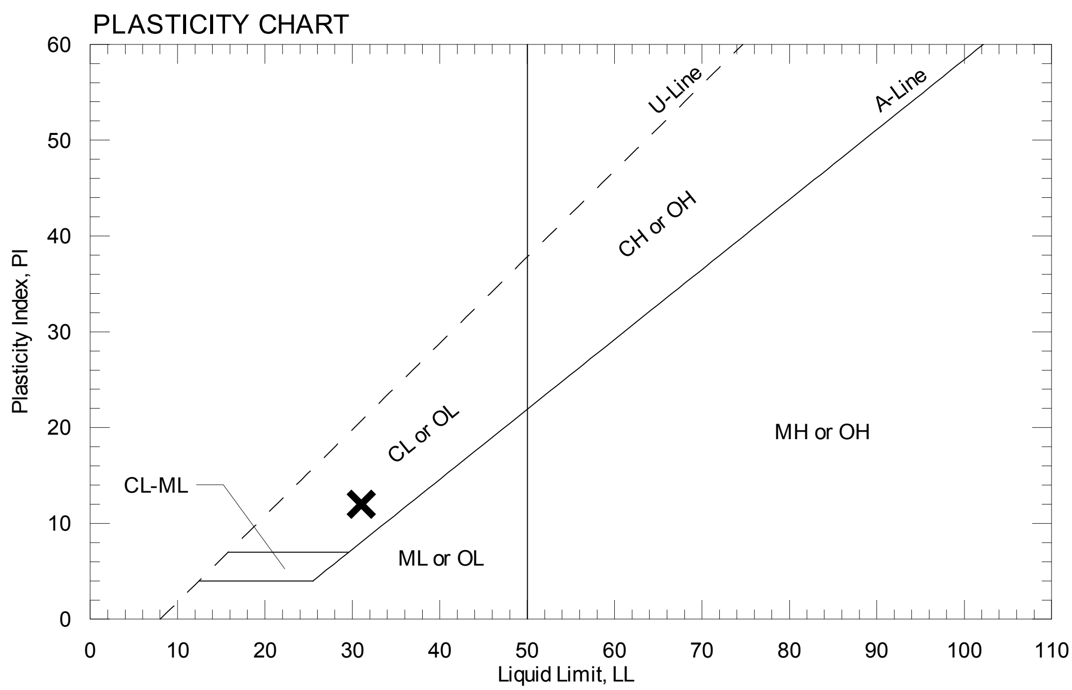
PIN	
017092.01	
Town	
Auburn	
Reported by/Date	
WHITE, TERRY A	10/15/2009

SHEET 3

State of Maine-Department of Transportation
Atterberg Limits Test Summary Sheet

TOWN	Auburn	Reference No.	246325
PIN	017092.01	Water Content, %	28.7
Sampled	8/12/2009	Plastic Limit	19
Boring No./Sample No.	BB-ARR-101/5D	Liquid Limit	31
Station	5+00	Plasticity Index	12
Depth	20.0-22.0	Tested By	BBURR





Appendix C

Calculations

LIQUIDITY INDEX (LI):

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

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-ARR-101/5D	28.7	31	19	12	0.81	Normally Consolidated

Arch Foundations: Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007 with Interims through 2009

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: $A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

yield strength: $F_y := 50 \cdot ksi$

Nominal Compressive Resistance $P_n=0.66^{\lambda}*F_y*A_s$: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

$\lambda=(Kl/r_s\pi)2*F_y/E$ eq. 6.9.4.1-3

$\lambda := 0$ as l = unbraced length = 0

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$

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

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 "good".

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: eq. 6.9.2.1-1

$P_f := \phi_c \cdot P_n$

$$P_f = \begin{pmatrix} 465 \\ 654 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$$

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

Nominal Compressive Resistance P_n=0.66$^\lambda$*F_y*A_s: eq. 6.9.4.1-1

Where λ=normalized column slenderness factor

λ=(Kl/$r_s\pi$)2*F_y/E eq. 6.9.4.1-3

$\lambda := 0$ as l unbraced length is 0

$P_n := 0.66^\lambda \cdot F_y \cdot A_s$

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

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

Resistance Factors for Service and Extreme Limit States ϕ = 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

$P_f := \phi \cdot P_n$

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

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:

Granite RQD ranges from 0 to 70%

Use RQD = 30% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

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

Steel area: $A_s = \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

Pile depth: $d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

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 granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 25000 \cdot psi$

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

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

Aperture of discontinuities: $\delta := \frac{1}{128} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.6821 \\ 0.6766 \\ 0.6143 \\ 0.6119 \\ 0.6078 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

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

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

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

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2455 \\ 2436 \\ 2211 \\ 2203 \\ 2188 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

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

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 793 \\ 1106 \\ 986 \\ 1198 \\ 1568 \end{pmatrix} \cdot kip$$

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

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 357 \\ 498 \\ 444 \\ 539 \\ 706 \end{pmatrix} \cdot kip$$

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 ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 793 \\ 1106 \\ 986 \\ 1198 \\ 1568 \end{pmatrix} \cdot kip$$

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

$f_y := 50 \cdot ksi$ yield strength of steel

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1
Pile Drivability Analysis, Steel piles

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ 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 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Assuming there are at least 5 piles at each arch footing. No reduction of Φ_{dyn} is necessary.

Pile Size = 12 x 53

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

State of Maine Dept. Of Transportation — 29-Oct-2009
Auburn - HP 12x53 Delmag 19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
410.0	44.68	4.57	5.1	8.64	19.18
411.0	44.72	4.56	5.1	8.65	19.18
412.0	44.86	4.58	5.1	8.65	19.23
413.0	44.88	4.57	5.1	8.66	19.23
414.0	44.92	4.56	5.2	8.67	19.23
415.0	44.95	4.55	5.2	8.68	19.23
416.0	45.13	4.57	5.2	8.69	19.29
417.0	45.15	4.55	5.2	8.70	19.28
418.0	45.19	4.55	5.3	8.70	19.28
419.0	45.23	4.55	5.3	8.71	19.29

Limited driving stress to 45 ksi

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

$R_{dr\_12x53\_factored} := 415 \cdot kip \cdot \phi_{dyn}$

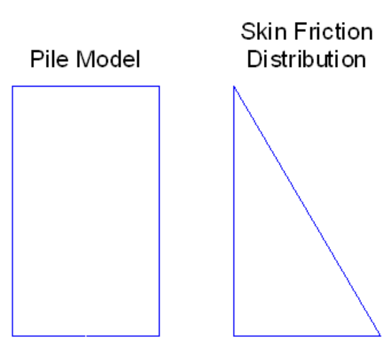
$R_{dr\_12x53\_factored} = 270 \cdot kip$

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

$R_{dr\_12x53\_servext} := 415 \cdot kip$

DELMAG D 19-42

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	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

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

State of Maine Dept. Of Transportation 29-Oct-2009
Auburn HP 12x74 Delmag 19-42 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
640.0	44.92	5.42	10.9	9.98	20.51
641.0	44.95	5.43	11.0	9.99	20.53
642.0	44.99	5.42	11.1	9.99	20.49
643.0	45.03	5.42	11.1	10.00	20.51
644.0	45.05	5.44	11.2	10.01	20.53
645.0	45.08	5.45	11.2	10.01	20.55
646.0	45.13	5.45	11.2	10.02	20.57
647.0	45.17	5.46	11.3	10.03	20.58
648.0	45.20	5.47	11.3	10.03	20.61
649.0	45.23	5.48	11.3	10.04	20.63

Limited to driving stress to 45 ksi

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

$R_{dr\_12x74\_factored} := 642 \cdot kip \cdot \phi_{dyn}$

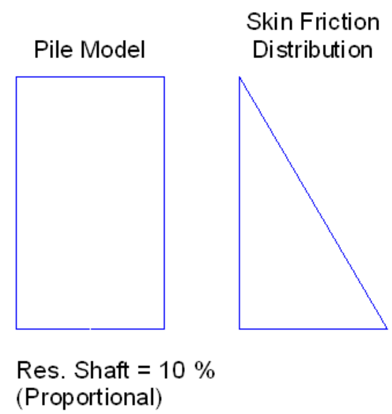
$R_{dr\_12x74\_factored} = 417 \cdot kip$

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

$R_{dr\_12x74\_servext} := 642 \cdot kip$

DELMAG D 19-42

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	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	21.80	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

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

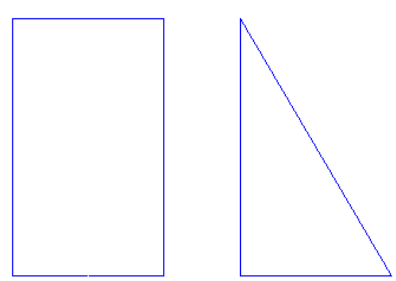
State of Maine Dept. Of Transportation — 29-Oct-2009
Auburn HP 14x73 Delmag 19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
630.0	44.96	5.51	10.7	9.94	20.45
631.0	44.99	5.52	10.7	9.94	20.47
632.0	45.02	5.53	10.8	9.95	20.49
633.0	45.06	5.53	10.8	9.96	20.51
634.0	45.10	5.54	10.8	9.96	20.54
635.0	45.14	5.55	10.9	9.97	20.56
636.0	45.19	5.55	10.9	9.98	20.56
637.0	45.21	5.56	10.9	9.99	20.59
638.0	45.25	5.57	11.0	9.99	20.61
639.0	45.29	5.58	11.0	10.00	20.63

DELMAG D 19-42

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	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Limit to driving stress to 45 ksi

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

$R_{dr\_14x73\_factored} := 631 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x73\_factored} = 410 \cdot kip$

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

$R_{dr\_14x73\_servext} := 631 \cdot kip$

Pile Size = 14 x 89

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

State of Maine Dept. Of Transportation — 29-Oct-2009
Auburn HP 14x89 Delmag 19-42 — GRLWEAP (TM) Version 2003

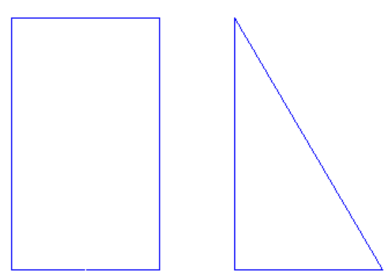
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
740.0	43.89	5.02	14.8	10.22	20.72
741.0	43.92	5.02	14.9	10.23	20.73
742.0	43.95	5.03	14.9	10.23	20.74
743.0	43.98	5.03	15.0	10.24	20.76
744.0	44.02	5.05	15.0	10.24	20.77
745.0	44.05	5.05	15.1	10.24	20.78
746.0	44.08	5.06	15.2	10.25	20.79
747.0	44.11	5.06	15.3	10.25	20.80
748.0	44.13	5.06	15.3	10.26	20.81
749.0	44.15	5.08	15.4	10.26	20.82

DELMAG D 19-42

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	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	26.10	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Limit blow count to 15 bows per inch

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

$R_{dr\_14x89\_factored} := 744 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x89\_factored} = 484 \cdot kip$

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

$R_{dr\_14x89\_servext} := 744 \cdot kip$

Pile Size = 14 x 117

Assume Contractor will use a Delmag D19-42 hammer on highest fuel setting to install 14 x 73 piles

State of Maine Dept. Of Transportation — 29-Oct-2009
Auburn HP 14x117 Delmag 19-42 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
775.0	38.22	4.58	14.9	9.73	19.62
776.0	38.27	4.58	14.9	9.74	19.62
777.0	38.28	4.59	15.0	9.74	19.62
778.0	38.30	4.60	15.0	9.74	19.62
779.0	38.30	4.61	15.0	9.75	19.66
780.0	38.31	4.63	15.1	9.75	19.65
781.0	38.39	4.64	15.2	9.75	19.66
782.0	38.39	4.64	15.2	9.76	19.66
783.0	38.43	4.67	15.2	9.76	19.70
784.0	38.42	4.67	15.3	9.77	19.69

Limit to blow count to 15 blows per inch

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

$R_{dr\_14x117\_factored} := 779 \cdot kip \cdot \phi_{dyn}$

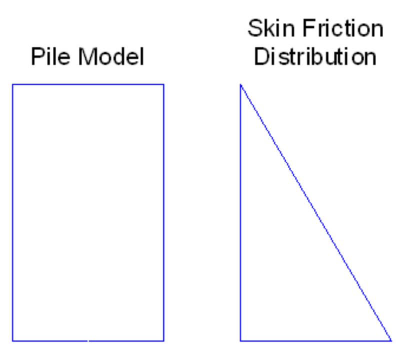
$R_{dr\_14x117\_factored} = 506 \cdot kip$

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

$R_{dr\_14x117\_servext} := 779 \cdot kip$

DELMAG D 19-42

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	35.00	ft
Pile Penetration	35.00	ft
Pile Top Area	34.40	in2



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 $\beta := 0 \cdot deg$

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

$$K_{p\_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

$K_{p\_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when β>0.

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 8 to 20 - Soils are loose to dense

Consistency In Place: Medium dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use: 6 ksf

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

Recommended Value: $6 \cdot ksf = 3 \cdot tsf$

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

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

$q_{factored\_bc} := 3 \cdot tsf$ or $q_{factored\_bc} = 6 \cdot ksf$

Note: This bearing resistance is settlement limited (1 inch) and applies only a 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 5.5 feet for frost protection. $D_f := 5.5 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)
   - Saturated unit weight: $\gamma_s := 125 \cdot pcf$
   - Dry unit weight: $\gamma_d := 120 \cdot pcf$
   - Internal friction angle: $\phi_{ns} := 32 \cdot deg$
   - Undrained shear strength: $c_{ns} := 0 \cdot psf$
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: $D_w := 17 \cdot ft$ Based on boring logs

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

Look at several footing widths

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

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 ϕ=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)

$q := D_f \cdot (\gamma_s - \gamma_w)$ $q = 0.1722 \cdot tsf$

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

$$q_{nominal} = \begin{pmatrix} 5.7 \\ 6.7 \\ 7.4 \\ 8.1 \\ 9.2 \end{pmatrix} \cdot tsf$$

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

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

$$q_{factored} = \begin{pmatrix} 2.6 \\ 3 \\ 3.3 \\ 3.7 \\ 4.1 \end{pmatrix} \cdot tsf$$

Based on these footing widths

$$q_{factored} = \begin{pmatrix} 5.1 \\ 6.1 \\ 6.7 \\ 7.3 \\ 8.2 \end{pmatrix} \cdot ksf \qquad B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

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 betweeh 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:
Auburn, Maine
DFI = 1400 degree-days

From the lab testing: soils are coarse grained with a water content = ~15%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1400 and wc =15%
Frost Penetration = 72.4 inches

$Frost\_depth := 72.4in \qquad Frost\_depth = 6 \cdot ft$

Note: The final depth of footing embedment may be controlled by the scour susceptibility of the foundation material and may, in fact, be deeper than the depth required for frost protection.

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Lewiston

ModBerg Results

Project Location: Lewiston, Maine

Air Design Freezing Index	=	1224 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	979 F-days
Mean Annual Temperature	=	46.4 deg F
Design Length of Freezing Season	=	118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	66.6	15.0	125.0	31	40	2.9	1.8	2,700

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

****

Total Depth of Frost Penetration = 5.55 ft = 66.6 in.

****

$Frost\_depth_{modberg} := 66.6 \cdot in$

$Frost\_depth_{modberg} = 5.55\,ft$ Use Frost Depth = 5.5 feet for design

Seismic:

Auburn Royal River Bridge PIN 17092.01
Date and Time: 10/29/2009 1:55:22 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04210
Zip Code Latitude = 44.097300
Zip Code Longitude = -070.240100
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.088	PGA - Site Class B
0.2	0.177	Ss - Site Class B
1.0	0.047	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04210
Zip Code Latitude = 44.097300
Zip Code Longitude = -070.240100
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.141	As - Site Class D
0.2	0.283	SDs - Site Class D
1.0	0.112	SD1 - Site Class D

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

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

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

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

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

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

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 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 necessity 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) ±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 0.25 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_{H1}, where γP_{H1}=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 Jumikus 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:

Pay Item	Pay Unit
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot

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