MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of

LITTLEFIELDS BRIDGE
OVER LITTLE ANDROSCOGGIN RIVER
HOTEL ROAD
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 Littlefields Bridge over Little Androscoggin River in Auburn, Maine. The proposed bridge replacement will consist of an approximately 144 foot long, single-span, steel welded plate girder superstructure founded on H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-Piles – The use of stub abutments founded on a single row of driven integral H-piles 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. 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 proposed integral 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 abutment 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.

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. In designing integral abutments for passive earth pressure, the Rankine earth pressure coefficient (K_p) of 3.25 is allowed if the displacement of the abutment is less than 0.5 percent of the abutment height. All abutment designs shall include a drainage system to intercept any water. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted.

Prefabricated Concrete Modular Block Gravity Wall - Precast Concrete Modular Gravity (PCMG) walls will be constructed on the upstream side of 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 AASHTO LRFD Bridge Design Specifications 6th Edition (LRFD), Special Provision 635 and plan notes.

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 roadway profile will be raised approximately 3.4 feet at Abutment No. 1 and approximately 2.1 feet at Abutment No. 2. Potential settlement due the placement of the proposed fill is estimated as less than 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Frost Protection - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Foundations placed on granular soils should be founded a minimum of 5.5 feet below finished exterior grade for frost protection.

Seismic Design Considerations – A seismic analysis is not required for single-span bridges regardless of seismic zone. Littlefields Bridge is on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criterion eliminates the MaineDOT Bridge Design Guide (BDG) requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

Construction Considerations – There is potential for boulders, cobbles and wood to impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations and installation of sheet piles for cofferdams. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling, or as approved by the Resident. The potential for these obstructions to slow construction activities should be considered if accelerated bridge construction methods are proposed for the project.

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

1.0 INTRODUCTION

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

The existing Littlefields Bridge carries Hotel Road over Little Androscoggin River and was constructed in 1937. The bridge consists of a single span, riveted steel, through truss superstructure founded on mass concrete abutments. The south abutment is believed to be a cast-in-place concrete abutment on a spread footing founded on soil and the north abutment is believed to be a cast-in-place concrete abutment on a spread footing founded on bedrock. The existing structure has a total length of approximately 115 feet. The 2010 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and substructure are in satisfactory condition (rating of 6) and the superstructure is in fair condition (rating of 5). The Bridge Sufficiency Rating is 46.0. The structure has a scour critical rating of "8 – Stable Above Footing" meaning that the foundations have been determined to be stable for the assessed or calculated scour condition. The scour is determined to be above the top of the footings. Inspection records note that the bridge substructure is spalled and has cracking in two places. The 2011 MaineDOT Underwater Dive Inspection Report shows no undermining of the abutments. There is a concrete arch bridge and a steel truss railroad bridge located immediately downstream of the bridge. The downstream wingwalls of the bridge connect to the wingwalls of the adjacent concrete arch bridge.

The MaineDOT Bridge Program is currently proposing to replace Littlefields Bridge with a single-span, steel welded plate girder superstructure founded on H-pile supported integral abutments constructed behind the location of the existing abutments. Precast Concrete Modular Gravity (PCMG) walls will be used for wingwalls as necessary. The span of the proposed replacement structure will be approximately 144 feet. The roadway centerline will remain on the existing alignment. The roadway profile will be raised approximately 3.4 feet at the south abutment and approximately 2.1 feet at the north abutment due to the loss of freeboard resulting from the switch from through truss to steel girder superstructure. The existing abutments will be capped above the Q50 elevation and left in place. The bridge will be closed for approximately 30 days for the replacement of the structure.

2.0 GEOLOGIC SETTING

Littlefields Bridge in Auburn carries Hotel Road over the Little Androscoggin River 1.5 miles south of Route 11 as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic map entitled Minot Quadrangle, Maine, Open File No. 02-231 (2002) published by the Maine Geological Survey the surficial soils in the vicinity of the site consist of stream alluvium and stream terrace deposits. The stream alluvium consists of sand, silt, gravel, and muck deposited in flood plains along present rivers and streams. The stream terrace deposits consist of sand, silt, gravel, and occasional muck deposited on terraces cut into glacial deposits. These terraces formed in part during the late-glacial time as sea level regressed.

According to the Bedrock Geologic Map of Maine (1985) published by the Maine Geologic Survey, the bedrock in the vicinity of the site consists of interbedded pelite and limestone and/or dolostone. Bedrock cores obtained from the 100-series borings are identified as medium to coarse grained gneiss of the Sangerville Formation.

3.0 SUBSURFACE INVESTIGATION

Two sets of borings were drilled at the site. Preliminary test borings, CM-30-94 and CM-32-94, were drilled in 1994. Final test borings, BB-ALAR-101, BB-ALAR-101A, BB-ALAR-101B, BB-ALAR-102 and BB-ALAR-103, were drilled in 2011.

Test borings CM-32-94, BB-ALAR-101, BB-ALAR-101A and BB-ALAR-101B were conducted behind the southwest abutment and test borings CM-30-94 and BB-ALAR-103 were conducted behind the northeast abutment. Boring BB-ALAR-102 was drilled at the location of a possible center pier. The exploration locations and an interpretive subsurface profile depicting the soil stratigraphy across the site are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The preliminary borings were drilled in 1994 by the MaineDOT drill crew. The final borings were drilled between October 5 and 11, 2011 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 Sheets 3 and 4 – Boring Logs found end of this report.

No information regarding the drilling methods used to conduct the 1994 borings is available beyond rough boring logs. The 1994 borings are included in this report for informational purposes only.

The 2011 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. The preliminary borings were drilled using a rope and cathead system to drive the split spoon. The final borings were drilled using an automatic hammer to drive the split spoon. The automatic hammer was calibrated in March of 2010 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 (N_{60}) are shown on the boring logs. The bedrock was cored in the borings using an NQ-2 inch 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 New England 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 exploration programs.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of fourteen (14) standard grain size analyses with water content. 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 Sheets 3 and 4 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of fill with frequent cobbles and boulders, underlain by sand, sandy gravel and gravelly sand with occasional cobbles and boulders, underlain by bedrock. The exploration locations and an interpretive subsurface profile depicting the generalized soil stratigraphy across the site are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

5.1 Fill Material

Fill material was encountered beneath the pavement in the borings conducted behind the existing abutments (BB-ALAR-101, BB-ALAR-101A, BB-ALAR-101B and BB-ALAR-103). The fill material consisted of:

- Brown, dry to wet, fine to coarse sand, trace to some silt, trace to some gravel, trace organics, frequent cobbles and boulders; and
- Brown, dry, gravelly fine to coarse sand, trace silt, occasional cobbles and boulders;

A layer of concrete was encountered at the bottom of the fill in boring BB-ALAR-101A. Concrete was also encountered in boring CM-30-94. This concrete is thought to be part of the footing for the existing abutments. A 4 inch thick layer of wood was encountered at a depth of 20 feet below ground surface in boring CM-32-94.

The thickness of the fill was approximately 27.2 feet in boring BB-ALAR-101A and approximately 18.5 feet in boring BB-ALAR-103. Corrected SPT N-values in the fill ranged from 8 to 29 blows per foot (bpf) indicating that the fill is loose to medium dense in consistency. One corrected N-value in boring BB-ALAR-101A was greater than 50 bpf. This value was influenced by the presence of cobbles and boulders and is not indicative of the actual density of the fill layer. Water contents obtained from fill samples ranged from approximately 2% to 18%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-2-4, A-1-a, A-1-b or A-3 by the AASHTO Classification System and an SM, SW-SM, or SP-SM by the Unified Soil Classification System.

5.2 Native Sand and Gravel

A native sand and gravel layer was encountered beneath the fill in both of the abutment borings and in the pier boring. The native sand and gravel consisted of:

- Grey, wet, fine to coarse sandy gravel, trace silt;
- Grey, wet, fine to coarse sand, some gravel, little silt, occasional cobbles and boulders;
- Grey, wet, fine to coarse sand, trace silt, occasional cobbles and boulders; and
- Grey-brown, wet, gravelly fine to coarse sand, trace silt.

The thickness of the sand and gravel layer ranged from approximately 8.7 feet in boring BB-ALAR-101A and approximately 11.4 feet in boring BB-ALAR-102. Corrected SPT N-values in the sand and gravel ranged from 18 to 50 bpf indicating that the layer is medium dense to very dense in consistency. One (1) SPT N-value in the sand and gravel layer was greater 50 bpf. This value was influenced by the presence of cobbles within the soil matrix. Water contents from samples obtained within the layer range from approximately 9% to 11%. Grain size analyses conducted on samples of the sand and gravel indicate that the soil is classified as an A-1-a or A-1-b by the AASHTO Classification System and an SW-SM or SM by the Unified Soil Classification System.

5.3 Bedrock

Bedrock was encountered and cored in five (5) of the borings. The Table 5-1 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD for both series of borings:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB-ALAR-101A	35.9 feet	192.6 feet	40-80%
CM-32-94	33.1 feet	196.5 feet	N/A
BB-ALAR-102	11.4 feet	195.1 feet	43-67%
BB-ALAR-103	28.7 feet	200.4 feet	60-89%
CM-30-94	30.2 feet	199.4 feet	N/A

Table 5-1 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as banded black and greenish white, medium to coarse grained, gneiss, with biotite and muscovite mica, quartz, feldspar, plagioclase, and garnet, with iron staining, joints dipping at approximately 15 to 90 degrees. The RQD of the bedrock was determined to range from 40 to 89 percent indicating a rock mass quality of poor to good.

5.4 Groundwater

Groundwater depth was inferred from the soil samples taken in the boring to be at a depth of approximately 10.0 to 15.0 feet below the existing ground surface. Note that water was introduced into the boreholes during the drilling operations. It is likely that the stabilized groundwater conditions differ from this estimate. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The following foundation alternatives were considered for the bridge replacement:

- Cantilever-type abutments founded on spread footings on soil or bedrock,
- Cantilever-type abutments on driven H-pile groups, and
- Integral, driven H-pile supported stub abutments.

After consideration of all of the alternatives, H-pile supported integral abutments located behind the existing abutments were selected because they require minimal future maintenance. This report addresses only this foundation type.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for stub abutments founded on a single row of integral H-piles driven to bedrock which have been identified as the optimal substructure for the project.

7.1 Integral Abutment H-Piles

The use of stub abutments founded on a single row of driven integral H-piles 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 factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with pile tips to protect the tips and improve penetration.

Pile lengths at the proposed abutments may be estimated based on Table 7-1 below:

Location/ Relevant Borings	Estimated Pile Cap Bottom Elevation	Approximate Depth to Bedrock From Ground Surface	Approximate Top of Rock Elevation	Estimated Pile Length
Abutment #1 BB-ALAR-101A CM-32-94	219.0 feet	 35.9 feet 33.1 feet	 192.6 feet 199.5 feet	28 feet
Abutment #2 BB-ALAR-103 CM-30-94	218.0 feet	 28.7 feet 30.2 feet	 200.4 feet 199.4 feet	20 feet

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

These pile lengths do not take into account the length of pile embedded in the pile cap, the additional two (2) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate damaged pile lengths, bedrock deeper than that encountered in the borings and the Contractor's leads and driving equipment.

7.1.1 Strength Limit State Design

The design of pile foundations bearing on or within the bedrock at the strength limit state shall consider:

- structural resistance of individual piles in axial compression
- structural resistance of individual piles in combined axial loading and flexure
- compressive axial geotechnical resistance of individual piles bearing on rock

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred trough the pile caps. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

Since the H-piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribed in AASHTO LRFD Bridge Design Specifications 6th Edition (LRFD) Articles 6.9.2.2 and 6.15.2. The analysis shall assign a fixed condition at the pile tip. The H-piles shall also be checked for fixity and combined axial and flexure using LPile® software.

Structural Resistance. The nominal axial 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. Preliminary estimates of the factored axial structural compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.5 (severe driving conditions) and an unbraced length (ℓ) of 1 inch and an effective length factor (K) of 1.2. This factored axial structural compressive resistance is presented in Table 7-2 below. It is the responsibility of the structural engineer to recalculate the nominal axial structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e".

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

The nominal axial geotechnical compressive resistance in the strength limit state was also calculated using the guidance in LRFD Article 10.7.3.2.3 which states that "The nominal bearing 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. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving." This limiting nominal bearing resistance is subsequently factored by a resistance factor, $\varphi_{dyn,}$ of 0.65 considering a pile resistance determination method of dynamic pile testing with signal matching for at least two (2) piles.

Both of these factored axial geotechnical compressive resistances are presented in Table 7-2 below.

Drivability Resistance. 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. This factored drivability resistance is presented in Table 7-2 below.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections fro the strength limit state is presented 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[1] ϕ_c=0.50	Geotechnical Resistance by Canadian Method φ_{stat}=0.45	Controlling Geotechnical Resistance[2] φ_{dyn}=0.65	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	387	349	252	279	279
HP 12x74	545	487	354	406	406
HP 14x73	535	434	348	395	395
HP 14x89	652	527	424	527	527
HP 14x117	860	690	559	651	651

1 Based on preliminary assumption of ℓ=1" and K=1.2
2 Calculated using LRFD Article 10.7.3.2.3

Table 7-2 - Factored Axial Resistances for Abutment 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 is controlled by the structural limit state with a factor for severe driving conditions (ϕ_c=0.50) applied. However, local experience supports the slightly higher estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the last column of Table 7-2 above.

The piles shall also be checked for resistance against combined axial compression and flexure accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2. This design axial load may govern the design. 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 (LRFD Eq. 6.9.2.2-1 or -2).

7.1.2 Service and Extreme Limit State Design

The design of the H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall stability of the pile group and pile group movements/stability considering changes in foundation conditions due to scour at the design flood event.

Extreme limit state design checks for the H-piles shall include pile axial bearing resistance, failure of the pile group by over turning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events. 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 LRFD Articles 2.6.4.4.2 and 3.7.5.

For the service and extreme limit states resistance factors, ϕ, of 1.0 are recommended for structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.1 and 10.5.5.3. It is the responsibility of the structural engineer to recalculate P_n based on refined elastic critical buckling resistance (P_e) evaluations. The nominal axial geotechnical resistance in the service and extreme limit states was calculated using Canadian Foundation Engineering Manual and the guidance in LRFD Article 10.7.3.2.3.

For the service and extreme limit states, the calculated factored axial compressive 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 States Factored Axial Pile Resistance (kips)				
	Structural Resistance[1] ϕ=1.0	Geotechnical Resistance by Canadian Method ϕ=1.0	Controlling Geotechnical Resistance[2] ϕ=1.0	Drivability Resistance ϕ=1.0	Governing Resistance
HP 12x53	775	775	387	429	429
HP 12x74	1090	1081	545	624	624
HP 14x73	1070	964	535	608	608
HP 14x89	1305	1171	652	811	811
HP 14x117	1720	1533	860	1002	1002

1 Based on preliminary assumption of ℓ=1" and K=1.2
2 Calculated using LRFD Article 10.7.3.2.3

Table 7-3 - Factored Axial Resistances for Abutment 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 is controlled by the structural limit state with a factor for severe driving conditions (ϕ_c=0.50) applied. However, local experience supports the slightly higher estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the last column of Table 7-3 above.

7.1.3 Lateral Pile Resistance

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include explicit consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should also be confirmed with soil-structure interaction analyses.

Lateral loads will be reacted by plumb piles. It is recommended that the structural designer or geotechnical engineer perform a series of lateral pile resistance analyses to evaluate the pile top defections and bending stresses under strength limit state design lateral loads using L-Pile® software or FB-Pier software. These software programs analyze pile response under lateral loads where the nonlinear soil behavior is modeled using soil resistance (p-y) curves. A secondary lateral pile analysis to determine maximum factored lateral loads permissible based on the allowable displacement criteria may be used. The structural designer should evaluate the associated pile stresses under factored lateral loads.

Recommended geotechnical parameters for generation of soil-interaction (p-y) curves in lateral pile analyses are provided in Table 7-4 below. In general, the model developed should emulate the soils at the site by using the soil layers (referenced in Table 7-4 by elevation), 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	Approx. Elevation of Soil Layer- feet	Water Table Condition	Effective Unit Weight lb/in^3 (lb/ft^3)	k_s lb/in^3	Cohesion lb/in^2 (lb/ft^2)	E_{50} for clays	Friction Angle
Fill	229 to 221	Above	0.0723 (125)	90	-	-	34°
Fill	221 to 201	Below	0.036 (63)	60	-	-	34°
Sand and Gravel	201 to 192	Below	0.036 (63)	125	-	-	32°

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

7.1.4 Driven 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 with signal matching at each integral abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the Contractor in the wave equation analysis. Restrikes will not be required as a part of the field quality control program unless pile behavior indicates the pile is not seated firmly on bedrock or if piles "walk" out of position. 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 maximum factored axial 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 and verified by dynamic pile test measurements. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Stub Abutment Design

Integral abutment sections 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. Stub abutments shall be designed to resist all lateral loads, vehicular loads, dead and live loads and lateral forces transferred through the integral structure. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider changes in foundation conditions and pile group resistance after scour due to the design flood.

A resistance factor of ϕ= 1.0 shall be used to assess abutment design at the service limit state including: settlement, excessive horizontal movement and movement resulting from 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 abutments supported on 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.

The Designer may assume Soil Type 4 (MaineDOT Bridge Design Guide [BDG] Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction angle of 20 degrees. Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive earth pressure state. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, K_p, of 3.25 anticipating that integral abutments will experience some movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.5 percent of the abutment height, then the calculation of lateral earth pressure should assume a Coulomb passive earth pressure coefficient, K_p, of 6.89. For designing the integral abutment backwall reinforcing steel, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 7-5 below:

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

Table 7-5 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Weep holes should be constructed approximately 6 inches above the Q1.1 elevation (normal high water). The approach slab should be positively connected to the integral abutment. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.4.

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

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V unless project specific slope stability analyses are performed.

7.3 Precast Concrete Modular Block Retaining Wall

Precast Concrete Modular Gravity (PCMG) walls will be constructed on the upstream side of the roadway adjacent to both abutments 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 7-5 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 soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 6 ksf for wall system bases less than 8 feet wide and 7 ksf for bases from 8.5 to 14 feet wide. The bearing resistance factor, ϕ_b, for spread footings on soil is 0.45. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

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

The designer shall apply a sliding resistance factor φ_T 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 (1/4th) 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 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 analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 2.4 mm
- Average diameter of particle at 95 percent passing, D_{95} = 19 mm
- Soil Classification AASHTO Soil Type A-1-a

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.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap placed at a maximum slope of 1.75H:1V. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Bridge approach slopes and slopes at wingwalls shall be armored with 3 feet of riprap. Stone riprap shall conform to item number 703.26 of MaineDOT Special Provision 703 and shall be placed at a maximum slope of 2H:1V (MaineDOT Standard Detail 601(02) August 2011). 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).

7.5 Settlement

The roadway profile will be raised approximately 3.4 feet at Abutment No. 1 and approximately 2.1 feet at Abutment No. 2. Potential settlement due the placement of the proposed fill is estimated as less than 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible. See Appendix C - Calculations for supporting documentation.

7.6 Frost Protection

Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG.

Foundations placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Auburn has a design freezing index of approximately 1400 F-degree days. In granular soils with an assumed water content of approximately 15%, this correlates to a frost depth approximately 6.0 feet.

An analysis performed using Modberg Software by the US Army Cold Regions Research and Engineering Laboratory showed 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.

It is recommended that 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

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 soil with 15 < average N-value < 50 blows per foot)
- 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: $S_{D1} < 0.15g$ (LRFD Table 3.10.6-1)

In conformance with LRFD Table 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, Littlefields Bridge is not the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criterion eliminates the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

7.8 Construction Considerations

Boulders and cobbles were encountered within the fill layer in all of the borings. A layer of wood was also encountered in one boring in the area of proposed Abutment No. 1 within the fill layer. It is likely that these obstructions will impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations and installation of sheet piles for cofferdams. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident. The potential for these obstructions to slow construction activities should be considered if accelerated bridge construction methods are proposed for the project.

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure 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.

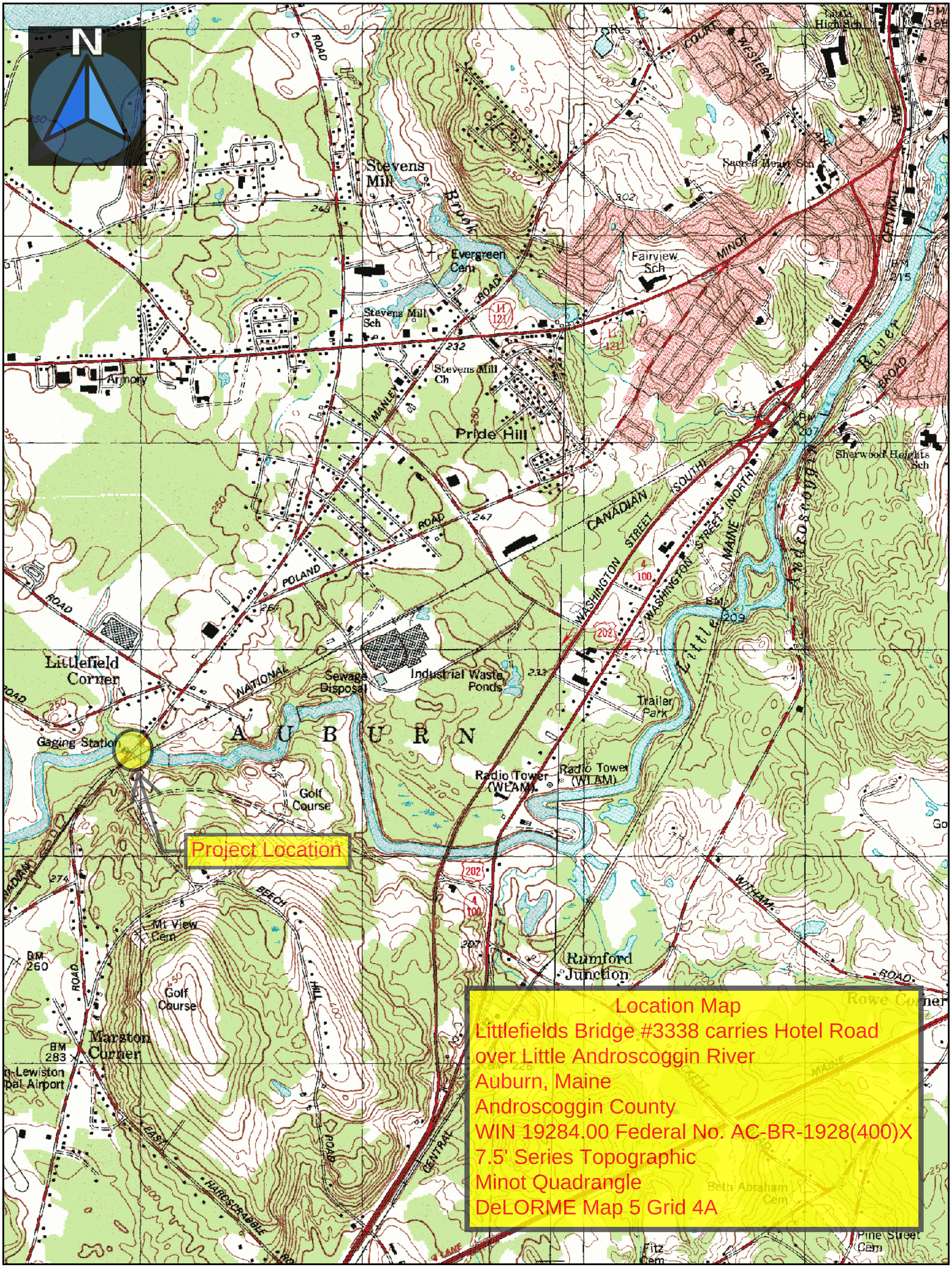
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements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

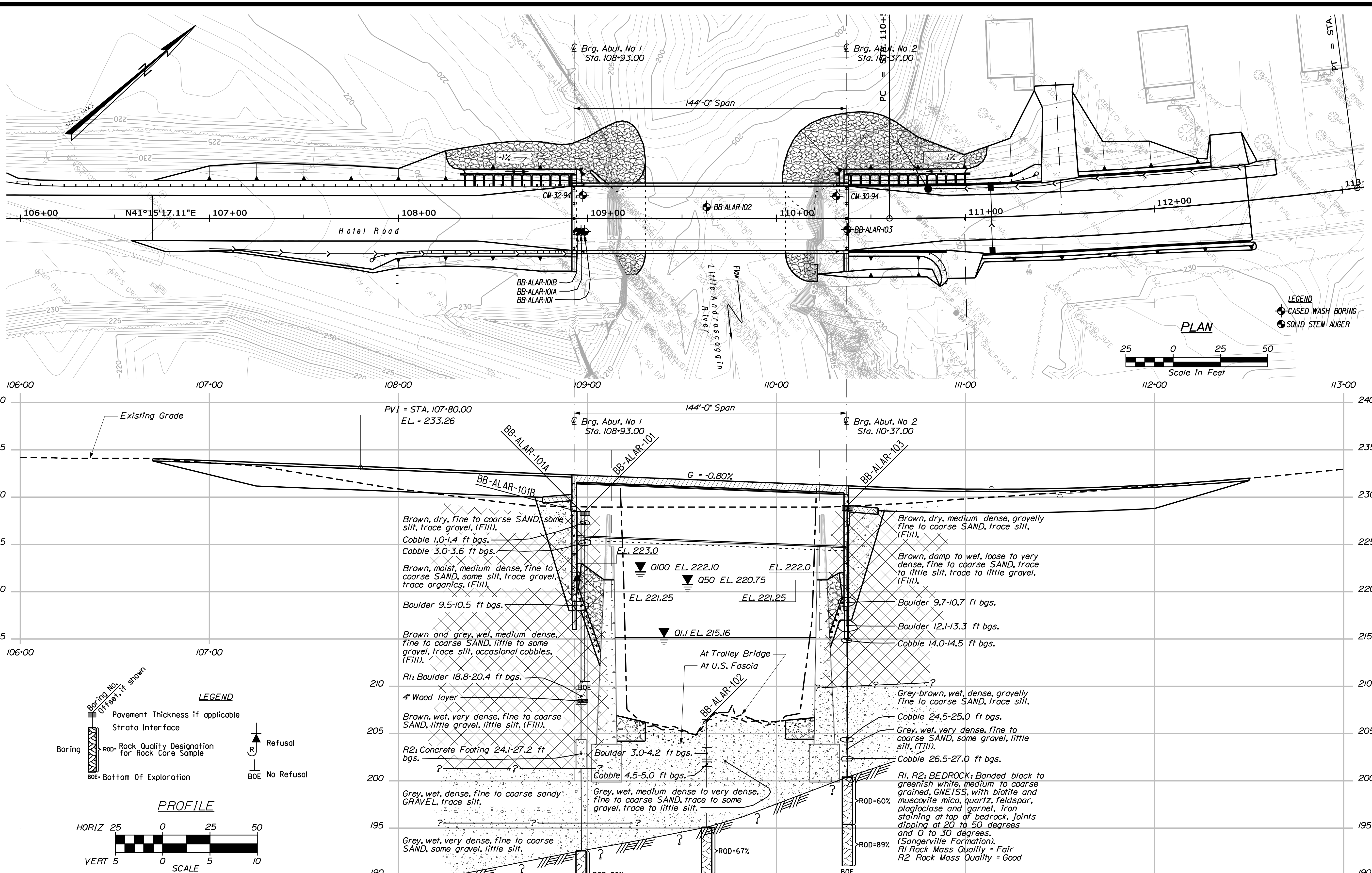
It is also recommended that the geotechnical engineer be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



Map Scale 1:24000

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



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

CM-30-94 and CM-32-94 are not shown on the Interpretive Subsurface Profile for clarity purposes. See Boring Log Sheet for details.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1928(400)X

WIN 19284.00

BRIDGE PLANS

BRIDGE NO. 3338

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

SIGNATURE

P.E. NUMBER

DATE

LITTLEFIELDS BRIDGE
LITTLE ANDROSCOGGIN RIVER
AUBURN ANDROSCOGGIN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 4

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:3/28/2012

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:3/28/2012

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: BB-ALAR-101B
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	228.5	Auger ID/OD:	5" Dia.
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	N/A
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	N/A
Date Start/Finish:	10/6/11; 14:00-14:30	Drilling Method:	Solid Stem	Core Barrel:	N/A
Boring Location:	108+94.6, 7.0 ft Rt.	Casing ID/OD:	N/A	Water Level*:	None Observed
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			Cobble at 1.2 ft bgs.	
6.50								222.00		Bottom of Exploration at 6.50 feet below ground surface. BOULDER REFUSAL? Moved to BB-ALAR-101A.	

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ALAR-101B

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: BB-ALAR-101A
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	228.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/6/11-10/7/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	108+96.6, 7.0 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	228.08		5" Pavement	
0.42										Similar to BB-ALAR-101.	
10							SPUN CASE AHEAD			Boulder from 9.5-10.5 ft bgs.	
	1D	21.6/14	14.00 - 15.80	3/4/3/50(3.6")	7	10				Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles, (Fill). Changed to NW Casing at 15.0 ft bgs.	G#261843 A-1-b, SW-SM WC=11.5%
18.80	R1	19.2/ 19.2	18.80 - 20.40				NQ-2	209.70		R1: BOULDER	
20.40	2D	15.6/14	20.50 - 21.80	11/39/50(3.6")	---			208.10		Brown, wet, very dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#261844 A-3, SP-SM WC=16.1%
24.10	R2	37.2/ 37.2	24.10 - 27.20				NQ-2	204.40		R2: CONCRETE footing	
27.20	3D	24/16	27.50 - 29.50	14/15/15/14	30	42		201.30		Grey, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#261845 A-1-a, SW-SM WC=11.3%
33.00								195.50		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt. Roller Coned ahead from 35.25-35.9 ft bgs.	
35	4D R3	6/6 60/56	35.00 - 35.50 35.90 - 40.90	50 RQD = 80%	---		NQ-2				
35.90								192.60		Top of Bedrock at Elev. 192.6 ft. R3:Bedrock: Banded black to greenish white, medium to coarse grained, GNEISS, with biotite and muscovite mica, quartz, feldspar, plagioclase and garnet, breaks along mica layers, cleavage sub-horizontal (0 to 20 degrees) with iron staining, (Sangerville Formation). Rock Mass Quality = Good. R3:Core Times (min:sec) 35.9-36.9 ft (2:45) 36.9-37.9 ft (1:45) 37.9-38.9 ft (2:20) 38.9-39.9 ft (2:25) 39.9-40.9 ft (3:00) 93% Recovery	
40	R4	60/60	40.90 - 45.90	RQD = 40%						R4:Bedrock: Silmiar to above with one joint dipping at 80 to 90 degrees and others at 50 degrees, (Sangerville Formation). Rock Mass Quality = Poor. R4:Core Times (min:sec) 40.9-41.9 ft (2:00) 41.9-42.9 ft (2:20) 42.9-43.9 ft (2:40) 43.9-44.9 ft (2:20) 44.9-45.9 ft (2:10) 100% Recovery	
45.90								182.60		Bottom of Exploration at 45.90 feet below ground surface.	

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ALAR-101A

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: CM-32-94
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	228.6	Auger ID/OD:	N/A
Operator:	Clyde Mann (Ret).	Datum:		Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1994	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	108+97.5, 12.3 ft Lt.	Casing ID/OD:	NW	Water Level*:	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SPUN AHEAD				
5	1D		5.00 - 7.00		17				Medium dense, brown, gravelly silty SAND, with cobbles and boulders.	
11.50	2D		11.50 - 13.50		25		217.10			
15	MD		15.00 - 17.00		10					
20.00							208.60		4" Wood layer.	
20.30	3D		20.00 - 22.00		4		208.30		Loose to medium, brown, gravelly, medium to fine SAND with cobbles and boulders.	
25	4D		25.00 - 27.00		13					
30.10	R1	36/36	30.10 - 33.10			CORE	198.50		CONCRETE	
33.10	R2	24/24	33.10 - 35.10			CORE	195.50		BEDROCK	
34.10							194.50		Bottom of Exploration at 34.10 feet below ground surface.	

Remarks:

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

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

Page 1 of 1
Boring No.: CM-32-94

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: BB-ALAR-101
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	228.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/6/11; 07:30-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	108+98.6, 7.0 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
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
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SSA = Solid Stem Auger
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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)
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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	228.08		5" Pavement	
0.42										Brown, dry, fine to coarse SAND, some silt, trace gravel, (description from soil on auger flights), (Fill). Cobble from 1.0-1.4 ft bgs. Cobble from 3.0-3.6 ft bgs.	
5	1D	24/12	5.00 - 7.00	5/5/5/6	10	14				Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel, trace organics, (Fill).	G#261840 A-2-4, SM WC=14.3%
10	2D	24/14	10.00 - 12.00	8/9/10/9	19	27	46			Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#261841 A-2-4, SM WC=7.7%
							58				
							90				
							92				
14.00							88	214.50			
15	3D	24/15	15.00 - 17.00	15/13/7/8	20	28	59			Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt. Changed to NW Casing at 15.0 ft bgs.	G#261842 A-1-b, SP-SM WC=15.1%
							70				
							33				
18.00							208	210.50		Bottom of Exploration at 18.00 feet below ground surface. Casing was too crooked to work boring, abandon hole, moved to BB-ALAR-101A.	

Remarks:

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

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

Page 1 of 1
Boring No.: BB-ALAR-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

AC-BR-1928(400)X

WIN 19284.00

BRIDGE PLANS

BRIDGE NO. 3338

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	K.MAGUIRE T.WHITE	MAR 2012
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

LITTLEFIELDS BRIDGE
LITTLE ANDROSCOGGIN RIVER
AUBURN ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER
3
OF 4

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: BB-ALAR-102
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	206.5	Auger ID/OD:	N/A
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/7/11, 10/11/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	109+63.1, 6.0 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	River Boring
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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/14	0.00 - 2.00	WOH/4/9/50	13	18	SPUN CASE		Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt. Roller Coned ahead to 6.0 ft bgs. Changed to NW Casing at 2.0 ft bgs. Boulder from 3.0-4.2 ft bgs. Cobble from 4.5-5.0 ft bgs.	G#261846 A-1-a, SW-SM WC=10.4%
5	2D	24/20	6.50 - 8.50	7/15/16/23	31	43			Grey, wet, dense, fine to coarse SAND, some gravel, little silt .	G#261847 A-1-b, SW-SM WC=11.0%
10	3D	16.8/13	10.00 - 11.40	30/30/50(4.8")	---				Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	
	R1	60/60	11.40 - 16.40	RQD = 67%			NQ-2	195.10	11.40 Top of Bedrock at Elev. 195.1 ft. R1:Bedrock: Greenish white, medium to coarse grained, GNEISS, with fine grained black specks, quartz, feldspar, muscovite mica and garnet, iron staining at joints, joints dipping at 60 degrees, (Sangerville Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 11.4-12.4 ft (1:34) 12.4-13.4 ft (2:20) 13.4-14.4 ft (2:30) 14.4-15.4 ft (2:30) 15.4-16.4 ft (4:20) 100% Recovery	
15	R2	60/60	16.40 - 21.40	RQD = 43%					R2:Bedrock: Similar to above, with joints at 60 to 90 degrees, (Sangerville Formation). Rock Mass Quality = Poor. R2:Core Times (min:sec) 16.4-17.4 ft (2:00) 17.4-18.4 ft (2:30) 18.4-19.4 ft (2:25) 19.4-20.4 ft (2:30) 20.4-21.4 ft (4:10) 100% Recovery	
20								185.10	21.40 Bottom of Exploration at 21.40 feet below ground surface.	
25										

Remarks:
11.0 " Concrete Deck.
26.8 ft from Bridge Deck to Ground.

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-ALAR-102

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: CM-30-94
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	228.6	Auger ID/OD:	N/A
Operator:	Clyde Mann (Ret).	Datum:		Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	1994	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	110+31.8, 11.7 ft Lt.	Casing ID/OD:	NW	Water Level*:	None Observed

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						SPUN AHEAD			
5	1D		5.00 - 7.00		5			Loose, brown, silty fine to meduim SAND, some gravel.	
10	2D		10.00 - 12.00		>50		218.60	10.00	
15	3D		15.00 - 17.00		>50			Dense, brown, gravelly SAND with cobbles and boulders.	
20	4D		20.00 - 21.08		>50				
25	MD R1	1/1 60.2/60	25.00 - 25.08 25.08 - 30.10		>50	CORE	203.52	25.08 CONCRETE	
30	R2		30.20 - 34.20			CORE	198.40	30.20 BEDROCK	
							194.40	34.20 Bottom of Exploration at 34.20 feet below ground surface.	
35									
40									
45									
50									

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: CM-30-94

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little
Location: Auburn, Maine

Boring No.: BB-ALAR-103
WIN: 19284.00

Driller:	MaineDOT	Elevation (ft.)	229.1	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/5/11; 07:30-13:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	110+37.5, 5.9 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	228.60	6" Pavement 0.50	
	1D	24/18	1.00 - 3.00	12/13/8/9	21	29			Brown, dry, medium dense, Gravelly fine to coarse SAND, trace silt.	G#261848 A-1-a, SW-SM WC=1.9%
5	2D	24/13	5.00 - 7.00	2/3/3/3	6	8			Brown, damp, loose, fine to coarse SAND, little silt, little gravel.	G#261849 A-2-4, SM WC=7.1%
							45			
10							10		Boulder from 9.7-10.7 ft bgs.	
	3D	4.8/4.8	11.70 - 12.10	50	---		12		Brown, damp, very dense, fine to coarse SAND, little silt, trace gravel. Boulder from 12.1-13.3 ft bgs. Roller Coned ahead to 14.5 ft bgs.	G#261850 A-3, SP-SM WC=15.1%
							16			
							38		Cobble from 14.0-14.5 ft bgs.	
15	4D	24/20	14.50 - 16.50	5/9/9/8	18	25	30		Brown, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#261851 A-3, SP-SM WC=18.4%
							42			
							30			
							32			
							36	210.60	18.50	
							50			
20	5D	24/14	20.00 - 22.00	6/24/12/30	36	50	24		Grey-brown, wet, dense, Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 24.0 ft bgs.	G#261852 A-1-a, SW-SM WC=10.5%
							73			
							69			
							107			
							116		Cobble from 24.5-25.0 ft bgs.	
25	6D	12/12	25.50 - 26.50	40/60	---		96		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, (Till). Cobble from 26.5-27.0 ft bgs.	G#261853 A-1-b, SM WC=8.7%
							58			
							82		Roller Coned ahead to 28.7 ft bgs.	
	R1	60/57	28.70 - 33.70	RQD = 60%			100 NQ-2	200.40	28.70 Top of Bedrock at Elev. 200.4 ft. R1:Bedrock: Banded black to greenish white, medium to coarse grained, GNEISS, with biotite and muscovite mica, quartz, feldspar, plagioclase and garnet, iron staining at top of bedrock, joints dipping at 20 to 50 degrees and 0 to 30 degrees, (Sangerville Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 28.7-29.7 ft (3:35) 29.7-30.7 ft (5:15) 30.7-31.7 ft (4:20) 31.7-32.7 ft (4:17) 32.7-33.7 ft (4:25) 95% Recovery	
30										
	R2	52.8/ 52.8	33.70 - 38.10	RQD = 89%					R2:Bedrock: Similar to above, joints dipping at 15 to 30 degrees, (Sangerville Formation). Rock Mass Quality = Good. R2:Core Times (min:sec) 33.7-34.7 ft (4:05) 34.7-35.7 ft (4:00) No water return 35.7-36.7 ft (4:10) No water return 36.7-37.7 ft (4:15) 37.7-38.1 ft (4:00) 100% Recovery Core Blocked	
35								191.00	38.10 Bottom of Exploration at 38.10 feet below ground surface.	
40										
45										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-ALAR-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
AC-BR-1928(400)X
BRIDGE NO. 3338
WIN 19284.00
BRIDGE PLANS

PROJ. MANAGER	________	BY	DATE
DESIGN-DETAILED	K.MAGUIRE	T.WHITE	MAR 2012
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

LITTLEFIELDS BRIDGE
LITTLE ANDROSCOGGIN 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):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River
Location: Auburn, Maine

Boring No.: BB-ALAR-101
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/6/11; 07:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+98.6, 7.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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	228.08		5" Pavement 0.42	
										Brown, dry, fine to coarse SAND, some silt, trace gravel, (description from soil on auger flights), (Fill). Cobble from 1.0-1.4 ft bgs.	
										Cobble from 3.0-3.6 ft bgs.	
5	1D	24/12	5.00 - 7.00	5/5/5/6	10	14				Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel, trace organics, (Fill).	G#261840 A-2-4, SM WC=14.3%
10	2D	24/14	10.00 - 12.00	8/9/10/9	19	27	46			Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#261841 A-2-4, SM WC=7.7%
							58				
							90				
							92	214.50		14.00	
							88				
15	3D	24/15	15.00 - 17.00	15/13/7/8	20	28	59			Grey, wet, medium dense, fine to coarse SAND, little gravel, trace silt. Changed to NW Casing at 15.0 ft bgs.	G#261842 A-1-b, SP-SM WC=15.1%
							70				
							33	210.50		18.00	
							208			**Bottom of Exploration at 18.00 feet below ground surface.** Casing was too crooked to work boring, abandon hole, moved to BB-ALAR-101A.	
20											
25											

Remarks:

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

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River Location: Auburn, Maine	Boring No.: BB-ALAR-101A WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/6/11-10/7/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+96.6, 7.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
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	228.08		5" Pavement —0.42—	
										Similar to BB-ALAR-101.	
5											
10							SPUN CASE AHEAD			Boulder from 9.5-10.5 ft bgs.	
	1D	21.6/14	14.00 - 15.80	3/4/3/50(3.6")	7	10				Brown, wet, medium dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles, (Fill). Changed to NW Casing at 15.0 ft bgs.	G#261843 A-1-b, SW-SM WC=11.5%
15											
	R1	19.2/19.2	18.80 - 20.40				NQ-2	209.70		—18.80— R1: BOULDER	
20											
	2D	15.6/14	20.50 - 21.80	11/39/50(3.6")	---			208.10		—20.40— Brown, wet, very dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#261844 A-3, SP-SM WC=16.1%
	R2	37.2/37.2	24.10 - 27.20				NQ-2	204.40		—24.10— R2: CONCRETE footing	
25											

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River Location: Auburn, Maine	Boring No.: BB-ALAR-101A WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/6/11-10/7/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+96.6, 7.0 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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											
	3D	24/16	27.50 - 29.50	14/15/15/14	30	42		201.30		27.20 Grey, wet, dense, fine to coarse Sandy GRAVEL, trace silt.	G#261845 A-1-a, SW-SM WC=11.3%
30											
								195.50		33.00	
35	4D	6/6	35.00 - 35.50	50	---		NQ-2			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt. Roller Coned ahead from 35.25-35.9 ft bgs.	
	R3	60/56	35.90 - 40.90	RQD = 80%				192.60		35.90 Top of Bedrock at Elev. 192.6 ft. R3:Bedrock: Banded black to greenish white, medium to coarse grained, GNEISS, with biotite and muscovite mica, quartz, feldspar, plagioclase and garnet, breaks along mica layers, cleavage sub-horizontal (0 to 20 degrees) with iron staining, (Sangerville Formation). Rock Mass Quality = Good. R3:Core Times (min:sec) 35.9-36.9 ft (2:45) 36.9-37.9 ft (1:45) 37.9-38.9 ft (2:20) 38.9-39.9 ft (2:25) 39.9-40.9 ft (3:00) 93% Recovery	
40	R4	60/60	40.90 - 45.90	RQD = 40%						R4:Bedrock: Silmiar to above with one joint dipping at 80 to 90 degrees and others at 50 degrees, (Sangerville Formation). Rock Mass Quality = Poor. R4:Core Times (min:sec) 40.9-41.9 ft (2:00) 41.9-42.9 ft (2:20) 42.9-43.9 ft (2:40) 43.9-44.9 ft (2:20) 44.9-45.9 ft (2:10) 100% Recovery	
45								182.60		45.90 **Bottom of Exploration at 45.90 feet below ground surface.**	
50											

Remarks:

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Boring No.: BB-ALAR-101A

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River
Location: Auburn, Maine

Boring No.: BB-ALAR-101B
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.5	**Auger ID/OD:**	5" Dia.
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	10/6/11; 14:00-14:30	**Drilling Method:**	Solid Stem	**Core Barrel:**	N/A
Boring Location:	108+94.6, 7.0 ft Rt.	**Casing ID/OD:**	N/A	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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			Cobble at 1.2 ft bgs.	
5											
								222.00		6.50	
										Bottom of Exploration at 6.50 feet below ground surface. BOULDER REFUSAL? Moved to BB-ALAR-101A.	
10											
15											
20											
25											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River
Location: Auburn, Maine

Boring No.: BB-ALAR-102
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	206.5	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/7/11, 10/11/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	109+63.1, 6.0 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	1D	24/14	0.00 - 2.00	WOH/4/9/50	13	18	SPUN CASE			Grey, wet, medium dense, gravelly, fine to coarse SAND, trace silt. Roller Coned ahead to 6.0 ft bgs.	G#261846 A-1-a, SW-SM WC=10.4%
										Changed to NW Casing at 2.0 ft bgs.	
										Boulder from 3.0-4.2 ft bgs.	
										Cobble from 4.5-5.0 ft bgs.	
5											
	2D	24/20	6.50 - 8.50	7/15/16/23	31	43				Grey, wet, dense, fine to coarse SAND, some gravel, little silt .	G#261847 A-1-b, SW-SM WC=11.0%
10	3D	16.8/13	10.00 - 11.40	30/30/50(4.8")	---					Grey, wet, very dense, fine to coarse SAND, some gravel, little silt.	
	R1	60/60	11.40 - 16.40	RQD = 67%			NQ-2	195.10		11.40 Top of Bedrock at Elev. 195.1 ft. R1:Bedrock: Greenish white, medium to coarse grained, GNEISS, with fine grained black specks, quartz, feldspar, muscovite mica and garnet, iron staining at joints, joints dipping at 60 degrees, (Sangerville Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 11.4-12.4 ft (1:34) 12.4-13.4 ft (2:20) 13.4-14.4 ft (2:30) 14.4-15.4 ft (2:30) 15.4-16.4 ft (4:20) 100% Recovery	
15											
	R2	60/60	16.40 - 21.40	RQD = 43%						R2:Bedrock: Similar to above, with joints at 60 to 90 degrees, (Sangerville Formation). Rock Mass Quality = Poor. R2:Core Times (min:sec) 16.4-17.4 ft (2:00) 17.4-18.4 ft (2:30) 18.4-19.4 ft (2:25) 19.4-20.4 ft (2:30) 20.4-21.4 ft (4:10) 100% Recovery	
20											
								185.10		21.40 **Bottom of Exploration at 21.40 feet below ground surface.**	
25											

Remarks:

11.0 " Concrete Deck.
26.8 ft from Bridge Deck to Ground.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River
Location: Auburn, Maine

Boring No.: BB-ALAR-103
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	229.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/5/11; 07:30-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	110+37.5, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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	228.60		6" Pavement — 0.50	
	1D	24/18	1.00 - 3.00	12/13/8/9	21	29				Brown, dry, medium dense, Gravelly fine to coarse SAND, trace silt.	G#261848 A-1-a, SW-SM WC=1.9%
5	2D	24/13	5.00 - 7.00	2/3/3/3	6	8				Brown, damp, loose, fine to coarse SAND, little silt, little gravel.	G#261849 A-2-4, SM WC=7.1%
							45				
10							10			Boulder from 9.7-10.7 ft bgs.	
	3D	4.8/4.8	11.70 - 12.10	50	---		12				
							16			Brown, damp, very dense, fine to coarse SAND, little silt, trace gravel. Boulder from 12.1-13.3 ft bgs. Roller Coned ahead to 14.5 ft bgs.	G#261850 A-3, SP-SM WC=15.1%
							38				
	4D	24/20	14.50 - 16.50	5/9/9/8	18	25	30			Cobble from 14.0-14.5 ft bgs. Brown, wet, medium dense, fine to coarse SAND, trace silt, trace gravel.	G#261851 A-3, SP-SM WC=18.4%
15							42				
							30				
							32				
							36	210.60		— 18.50	
							50				
20	5D	24/14	20.00 - 22.00	6/24/12/30	36	50	24			Grey-brown, wet, dense, Gravelly fine to coarse SAND, trace silt. Roller Coned ahead to 24.0 ft bgs.	G#261852 A-1-a, SW-SM WC=10.5%
							73				
							69				
							107				
25							116			Cobble from 24.5-25.0 ft bgs.	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River	**Location:**	Auburn, Maine	**Boring No.:**	BB-ALAR-103
				WIN:	19284.00
Driller:	MaineDOT	**Elevation (ft.)**	229.1	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/5/11; 07:30-13:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	110+37.5, 5.9 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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)
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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	12/12	25.50 - 26.50	40/60	---		96			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, (Till). Cobble from 26.5-27.0 ft bgs.	G#261853 A-1-b, SM WC=8.7%
							58				
							82			Roller Coned ahead to 28.7 ft bgs.	
	R1	60/57	28.70 - 33.70	RQD = 60%			100 NQ-2	200.40		28.70 Top of Bedrock at Elev. 200.4 ft. R1:Bedrock: Banded black to greenish white, medium to coarse grained, GNEISS, with biotite and muscovite mica, quartz, feldspar, plagioclase and garnet, iron staining at top of bedrock, joints dipping at 20 to 50 degrees and 0 to 30 degrees, (Sangerville Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 28.7-29.7 ft (3:35) 29.7-30.7 ft (5:15) 30.7-31.7 ft (4:20) 31.7-32.7 ft (4:17) 32.7-33.7 ft (4:25) 95% Recovery	
30											
	R2	52.8/52.8	33.70 - 38.10	RQD = 89%						R2:Bedrock: Similar to above, joints dipping at 15 to 30 degrees, (Sangerville Formation). Rock Mass Quality = Good. R2:Core Times (min:sec) 33.7-34.7 ft (4:05) 34.7-35.7 ft (4:00) No water return 35.7-36.7 ft (4:10) No water return 36.7-37.7 ft (4:15) 37.7-38.1 ft (4:00) 100% Recovery Core Blocked	
35											
								191.00		38.10	
40										**Bottom of Exploration at 38.10 feet below ground surface.**	
45											
50											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: CM-30-94
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.6	**Auger ID/OD:**	N/A
Operator:	Clyde Mann (Ret).	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1994	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	110+31.8, 11.7 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

Definitions:
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R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN AHEAD				
5	1D		5.00 - 7.00		5				Loose, brown, silty fine to meduim SAND, some gravel.	
10	2D		10.00 - 12.00		>50		218.60		10.00	
15	3D		15.00 - 17.00		>50				Dense, brown, gravelly SAND with cobbles and boulders.	
20	4D		20.00 - 21.08		>50					
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River.	**Boring No.:**	CM-30-94		
Location:	Auburn, Maine	**WIN:**	19284.00		
Driller:	MaineDOT	**Elevation (ft.)**	228.6	**Auger ID/OD:**	N/A
Operator:	Clyde Mann (Ret).	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1994	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	110+31.8, 11.7 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks		Laboratory Testing Results/ AASHTO and Unified Class.
25	MD	1/1	25.00 - 25.08		>50	CORE	203.52		CONCRETE	25.08	
	R1	60.2/60	25.08 - 30.10								
30	R2		30.20 - 34.20			CORE	198.40		BEDROCK	30.20	
							194.40		**Bottom of Exploration at 34.20 feet below ground surface.**	34.20	
35											
40											
45											
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: CM-32-94
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.6	**Auger ID/OD:**	N/A
Operator:	Clyde Mann (Ret).	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1994	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+97.5, 12.3 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						SPUN AHEAD				
5	1D		5.00 - 7.00		17				Medium dense, brown, gravelly silty SAND, with cobbles and boulders.	
10										
	2D		11.50 - 13.50		25		217.10		11.50	
15	MD		15.00 - 17.00		10					
20	3D		20.00 - 22.00		4		208.60		20.00	
							208.30		4" Wood layer. 20.30	
									Loose to medium, brown, gravelly, medium to fine SAND with cobbles and boulders.	
25										

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Littlefields Bridge #3338 carries Hotel Road over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: CM-32-94
WIN: 19284.00

Driller:	MaineDOT	**Elevation (ft.)**	228.6	**Auger ID/OD:**	N/A
Operator:	Clyde Mann (Ret).	**Datum:**		**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	1994	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+97.5, 12.3 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	None Observed

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

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

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-value	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	4D		25.00 - 27.00		13					
30	R1	36/36	30.10 - 33.10			CORE	198.50		CONCRETE 30.10	
	R2	24/24	33.10 - 35.10			CORE	195.50		BEDROCK 33.10	
							194.50		34.10	
									Bottom of Exploration at 34.10 feet below ground surface.	
35										
40										
45										
50										

Remarks:

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Auburn Work Number: 19284.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification Unified	AASHTO	Frost
BB-ALAR-101, 1D	108+98.6	7.0 Rt.	5.0-7.0	261840	1	14.3			SM	A-2-4	II
BB-ALAR-101, 2D	108+98.6	7.0 Rt.	10.0-12.0	261841	1	7.7			SM	A-2-4	II
BB-ALAR-101, 3D	108+98.6	7.0 Rt.	15.0-17.0	261842	1	15.1			SP-SM	A-1-b	0
BB-ALAR-101A, 1D	108+96.6	7.0 Rt.	14.0-15.8	261843	1	11.5			SW-SM	A-1-b	0
BB-ALAR-101A, 2D	108+96.6	7.0 Rt.	20.5-21.8	261844	1	16.1			SP-SM	A-3	0
BB-ALAR-101A, 3D	108+96.6	7.0 Rt.	27.5-29.5	261845	1	11.3			SW-SM	A-1-a	0
BB-ALAR-102, 1D	109+63.1	6.0 Lt.	0.0-2.0	261846	2	10.4			SW-SM	A-1-a	0
BB-ALAR-102, 2D	109+63.1	6.0 Lt.	6.5-8.5	261847	2	11.0			SW-SM	A-1-b	0
BB-ALAR-103, 1D	110+37.5	5.9 Rt.	1.0-3.0	261848	3	1.9			SW-SM	A-1-a	0
BB-ALAR-103, 2D	110+37.5	5.9 Rt.	5.0-7.0	261849	3	7.1			SM	A-2-4	II
BB-ALAR-103, 3D	110+37.5	5.9 Rt.	11.7-12.1	261850	3	15.1			SP-SM	A-3	0
BB-ALAR-103, 4D	110+37.5	5.9 Rt.	14.5-16.5	261851	3	18.4			SP-SM	A-3	0
BB-ALAR-103, 5D	110+37.5	5.9 Rt.	20.0-22.0	261852	3	10.5			SW-SM	A-1-a	0
BB-ALAR-103, 6D	110+37.5	5.9 Rt.	25.5-26.5	261853	3	8.7			SM	A-1-b	II

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

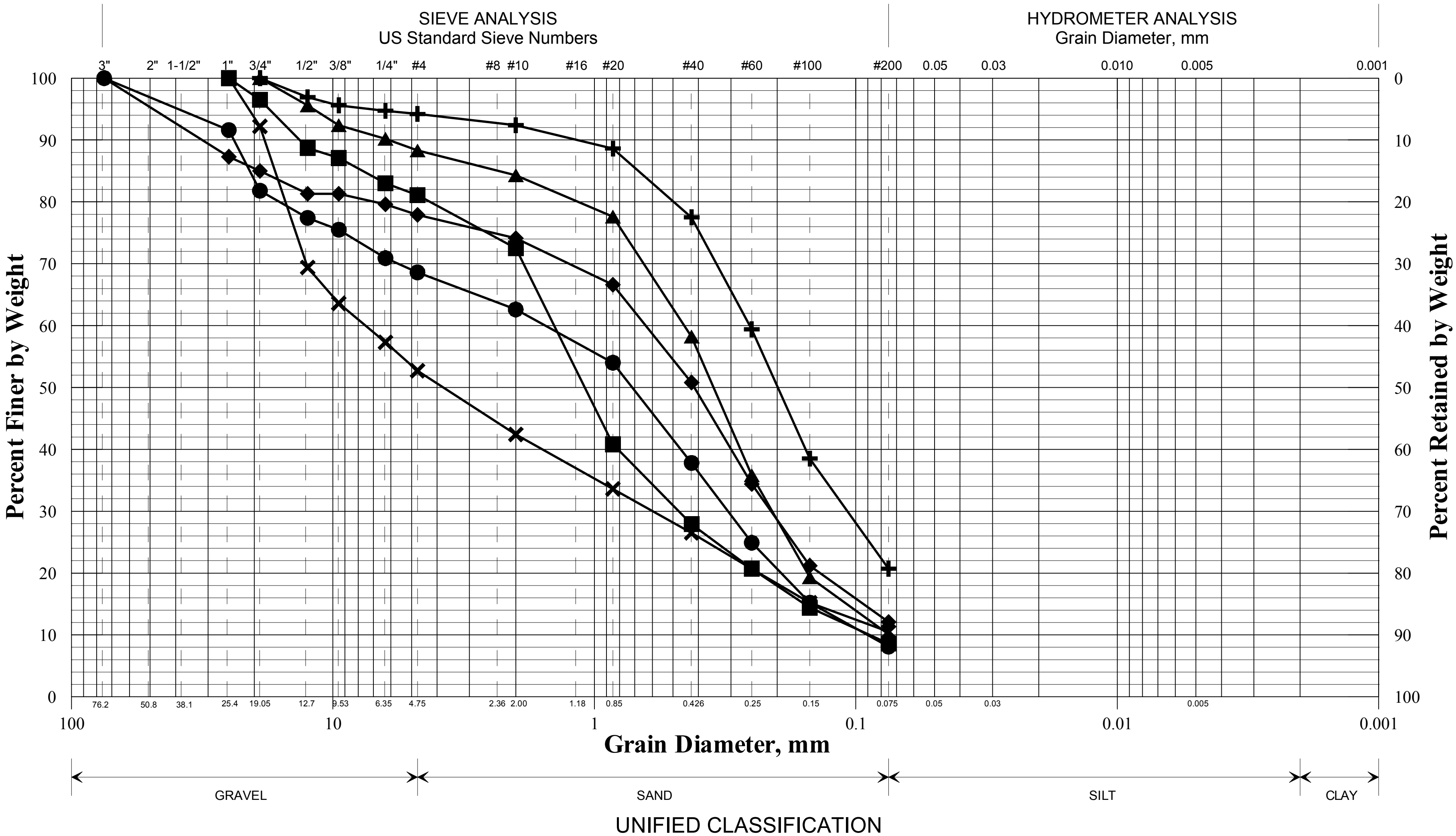
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ALAR-101/1D	108+98.6	7.0 RT	5.0-7.0	SAND, some silt, trace gravel.	14.3			
◆	BB-ALAR-101/2D	108+98.6	7.0 RT	10.0-12.0	SAND, some gravel, little silt.	7.7			
■	BB-ALAR-101/3D	108+98.6	7.0 RT	15.0-17.0	SAND, little gravel, trace silt.	15.1			
●	BB-ALAR-101A/1D	108+96.6	7.0 RT	14.0-15.8	SAND, some gravel, trace silt.	11.5			
▲	BB-ALAR-101A/2D	108+96.6	7.0 RT	20.5-21.8	SAND, little gravel, little silt.	16.1			
✖	BB-ALAR-101A/3D	108+96.6	7.0 RT	27.5-29.5	Sandy GRAVEL, trace silt.	11.3			

WIN
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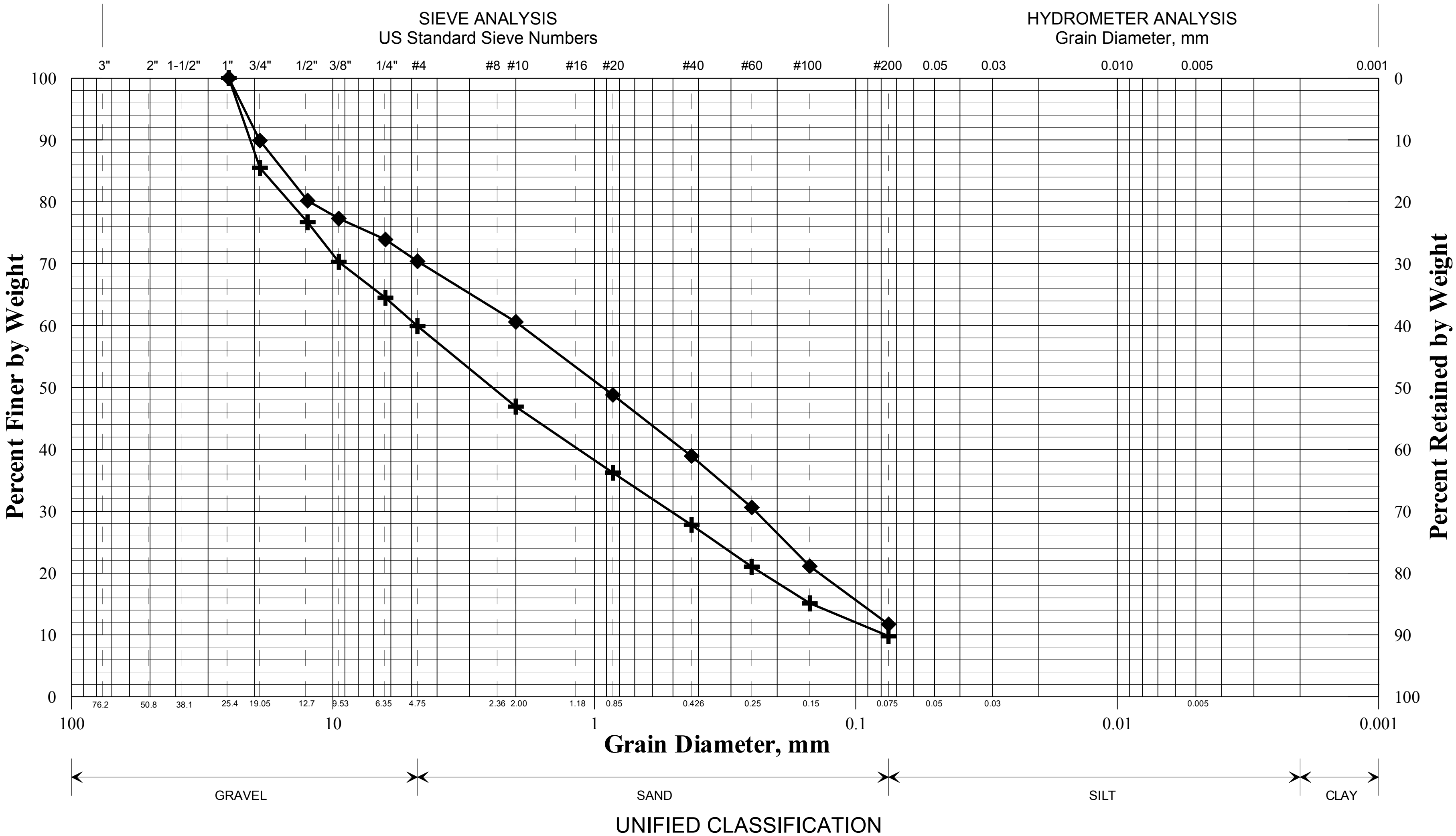
State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ALAR-102/1D	109+63.1	6.0 LT	0.0-2.0	Gravelly SAND, trace silt.	10.4			
◆	BB-ALAR-102/2D	109+63.1	6.0 LT	6.5-8.5	SAND, some gravel, little silt.	11.0			
■									
●									
▲									
✖									

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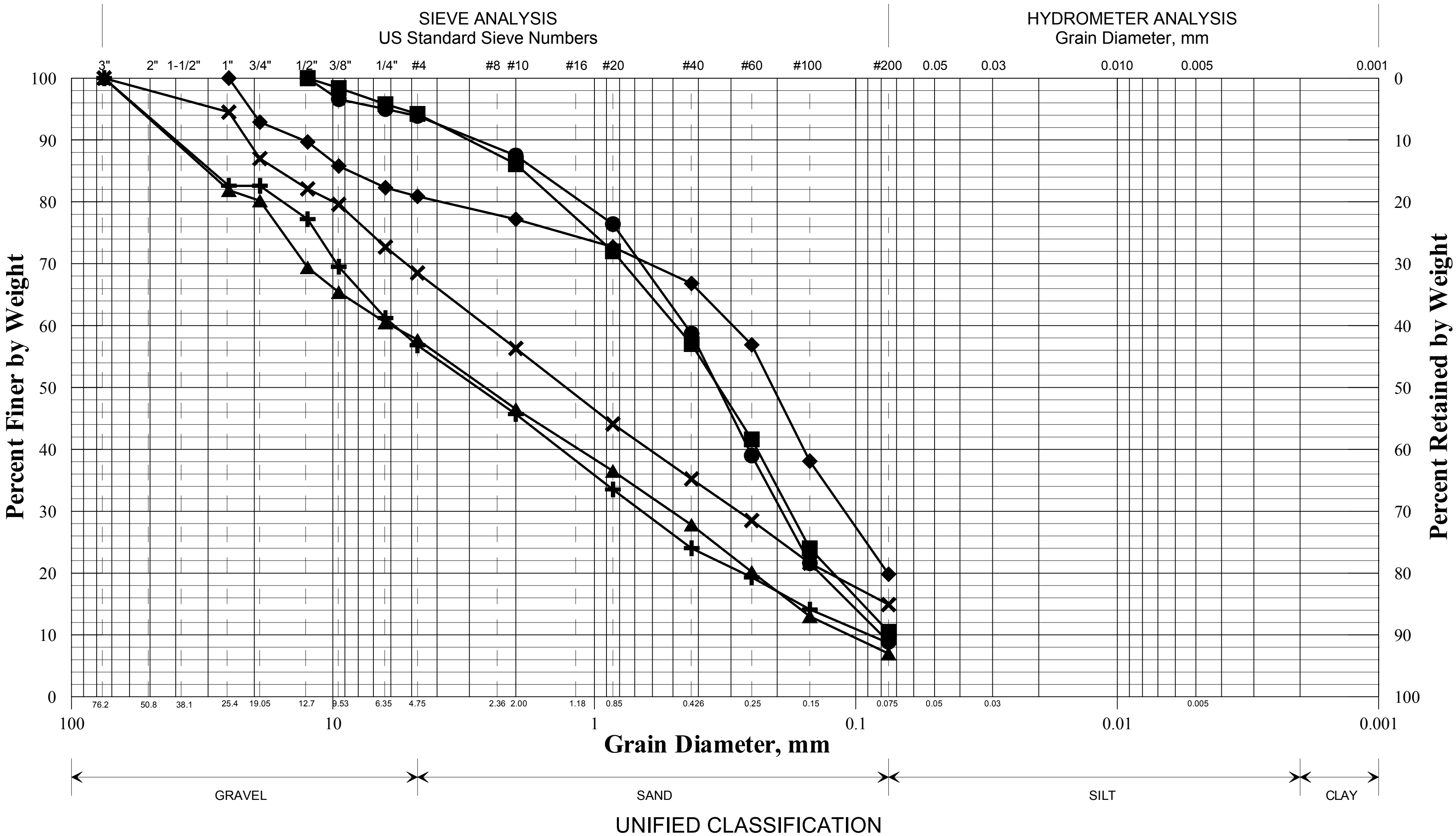
019284.00
Town
Auburn
Reported by/Date
WHITE, TERRY A 10/28/2011

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-ALAR-103/1D	110+37.5	5.9 RT	1.0-3.0	Gravelly SAND, trace silt.	1.9			
◆	BB-ALAR-103/2D	110+37.5	5.9 RT	5.0-7.0	SAND, little silt, little gravel.	7.1			
■	BB-ALAR-103/3D	110+37.5	5.9 RT	11.7-12.1	SAND, little silt, trace gravel.	15.1			
●	BB-ALAR-103/4D	110+37.5	5.9 RT	14.5-16.5	SAND, trace silt, trace gravel.	18.4			
▲	BB-ALAR-103/5D	110+37.5	5.9 RT	20.0-22.0	Gravelly SAND, trace silt.	10.5			
✖	BB-ALAR-103/6D	110+37.5	5.9 RT	25.5-26.5	SAND, some gravel, little silt.	8.7			

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019284.00
Town
Auburn
Reported by/Date
WHITE, TERRY A 10/28/2011

Appendix C

Calculations

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

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: $F_y := 50 \cdot ksi$

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

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

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

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

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

E = steel modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 1.2$ LRFD Table C4.6.2.5-1 Design value: ideal conditions, rotation fixed, translation free at head; rotation fixed, translation fixed at tip

l = unbraced length $l_{unbraced} := 1 \cdot in$ Old abutments left in place - no scour (0 makes the equation blow up)

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 25199912 \\ 36945151 \\ 51808364 \\ 64643546 \\ 88121644 \end{pmatrix} \cdot kip$$

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

LRFD Article 6.9.4.1.1

LRFD Equation 6.9.4.1.1-1

$$\frac{P_e}{P_o} = \begin{pmatrix} 32516 \\ 33895 \\ 48419 \\ 49535 \\ 51234 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

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

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

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 "severe".

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

From Article 6.5.4.2 $\phi_{csevere} := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$P_r := \phi_{csevere} \cdot P_n$

$$P_r = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot kip$$

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

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.5.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$P_r := \phi \cdot P_n$

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

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

Service/Extreme Limit States

Geotechnical Resistance - by Canadian Geotech Method pre LRFD Table 10.5.5.2.3-1

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

Bedrock Type:

Gneiss RQD range 40% to 89%

Use RQD = 60% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

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 gneiss compressive strength ranges from 3500 to 45000 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: $\delta := \frac{1}{64} \cdot in$ 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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$$K_{sp} = \begin{pmatrix} 0.6667 \\ 0.6614 \\ 0.6005 \\ 0.5981 \\ 0.5941 \end{pmatrix}$$

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

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2400 \\ 2381 \\ 2162 \\ 2153 \\ 2139 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

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

$$R_p := \overrightarrow{(3q_a \cdot A_s)} \qquad R_p = \begin{pmatrix} 775 \\ 1081 \\ 964 \\ 1171 \\ 1533 \end{pmatrix} \cdot kip$$

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

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 349 \\ 487 \\ 434 \\ 527 \\ 690 \end{pmatrix} \cdot kip$$

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 10.5.5.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 775 \\ 1081 \\ 964 \\ 1171 \\ 1533 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit States

Axial Geotechnical Resistance per LRFD Article 10.7.3.2.3

LRFD Article 10.7.3.2.3 states: "The nominal resistance of piles driven to point bearing on hard rock where pile penetratic into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exce the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving conditions."

Determine *Nominal* Axial Geotechnical Resistance per LRFD Article 10.7.3.2.3

Nominal Structural Resistance:
From page 2

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

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

Apply resistance factor for severe driving from LRFD Article 6.5.4.2

$$\phi_{csevere} := 0.5$$

Nominal Axial Geotechnical Resistance

$$P_{nomgeotech} := \phi_{csevere} \cdot P_n$$

$$P_{nomgeotech} = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot \text{kip}$$

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

Nominal Axial Geotechnical Bearing Resistance shall not exceed $P_{nomgeotech}$.

Deternine *Factored* Axial Geotechnical Resistance at the **Strength Limit State**:

Apply resistance factor for driving criteria established by dynamic testing LRFD Table 10.5.5.2.3-1

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

Factored Axial Geotechnical Resistance - Strength Limit State

$$P_{fac\_strength} := \phi_{dyn} \cdot P_{nomgeotech}$$

$$P_{fac\_strength} = \begin{pmatrix} 252 \\ 354 \\ 348 \\ 424 \\ 559 \end{pmatrix} \cdot \text{kip}$$

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

Deternine *Factored* Axial Geotechnical Resistance at the **Service and Extreme Limit States**:

Apply resistance factor for driving criteria established by dynamic testing LRFD Table 10.5.5.2.3-1

$$\phi = 1.0$$

Factored Axial Geotechnical Resistance - Service and Extreme Limit States

$$P_{fac\_serv\_ext} := \phi \cdot P_{nomgeotech}$$

$$P_{fac\_serv\_ext} = \begin{pmatrix} 387 \\ 545 \\ 535 \\ 652 \\ 860 \end{pmatrix} \cdot \text{kip}$$

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

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x 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 and 6.5.4.2 resistance during pile driving

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

$\phi_{dyn} := 0.65$

Pile Size = 12 x 53

Assume Contractor will use an MKT DE 42 hammer

State of Maine Dept. Of Transportation — 13-Feb-2012
Auburn Littlefields Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
425.0	44.60	2.11	8.3	8.88	13.61
426.0	44.68	2.16	8.3	8.89	13.60
427.0	44.66	2.15	8.4	8.89	13.60
428.0	44.73	2.17	8.4	8.90	13.58
429.0	45.00	2.18	8.3	8.99	13.75
430.0	45.03	2.22	8.3	8.98	13.76
431.0	45.06	2.23	8.4	8.99	13.76
432.0	45.03	2.25	8.5	8.97	13.73
433.0	45.09	2.25	8.5	8.98	13.72
434.0	45.12	2.26	8.6	8.98	13.72

Limited driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_12x53\_factored} := 429 \cdot kip \cdot \phi_{dyn}$$

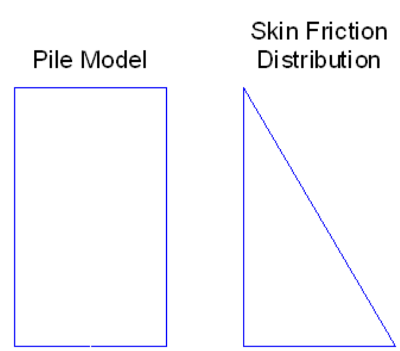
$$R_{dr\_12x53\_factored} = 279 \cdot kip$$

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

$$R_{dr\_12x53\_servext} := 429 \cdot kip$$

MKT DE 42/35

Efficiency	0.700	
Helmet	1.20	kips
Hammer Cushion	14175	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	28.00	ft
Pile Penetration	28.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 20 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag D 36-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation — 13-Feb-2012
19284 Auburn Littlefields Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
620.0	44.76	2.08	6.0	6.62	24.48
621.0	44.82	2.09	6.0	6.63	24.48
622.0	44.89	2.10	6.0	6.64	24.56
623.0	44.95	2.11	6.0	6.64	24.58
624.0	45.02	2.11	6.0	6.64	24.60
625.0	45.06	2.12	6.0	6.65	24.61
626.0	45.10	2.12	6.0	6.65	24.63
627.0	45.14	2.13	6.1	6.66	24.64
628.0	45.17	2.14	6.1	6.66	24.66
629.0	45.22	2.14	6.1	6.66	24.61

Limited driving stress to 45 ksi

Strength Limit State:

$R_{dr\_12x74\_factored} := 624 \cdot kip \cdot \phi_{dyn}$

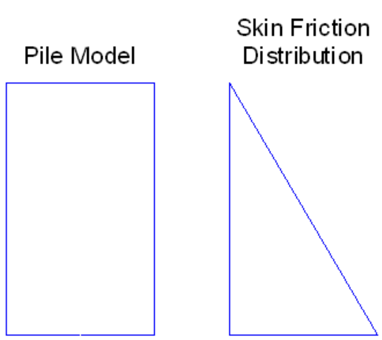
$R_{dr\_12x74\_factored} = 406 \cdot kip$

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

$R_{dr\_12x74\_servext} := 624 \cdot kip$

DELMAG D 36-32

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	28.00	ft
Pile Penetration	28.00	ft
Pile Top Area	21.80	in2



Res. Shaft = 20 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag 36-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation — 13-Feb-2012
19284 Auburn Littlefields Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
605.0	44.87	2.07	5.7	6.58	24.45
606.0	44.92	2.08	5.8	6.59	24.46
607.0	44.97	2.08	5.8	6.59	24.48
608.0	45.00	2.09	5.8	6.59	24.50
609.0	45.06	2.09	5.8	6.60	24.52
610.0	45.13	2.10	5.8	6.60	24.54
611.0	45.19	2.10	5.8	6.60	24.56
612.0	45.25	2.10	5.8	6.61	24.57
613.0	45.27	2.11	5.9	6.61	24.59
614.0	45.33	2.11	5.9	6.62	24.60

Limited driving stress to 45 ksi

Strength Limit State:

$$R_{dr\_14x73\_factored} := 608 \cdot kip \cdot \phi_{dyn}$$

$$R_{dr\_14x73\_factored} = 395 \cdot kip$$

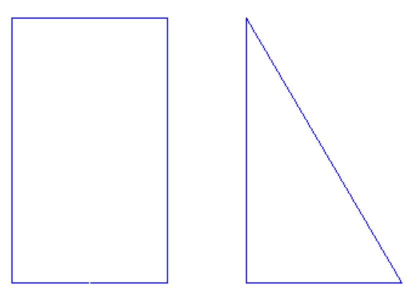
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_14x73\_servext} := 608 \cdot kip$$

DELMAG D 36-32

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	28.00	ft
Pile Penetration	28.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag 36-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation 13-Feb-2012
19284 Auburn Littlefields Drivability GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
807.0	44.81	3.90	9.5	7.17	25.29
808.0	44.85	3.91	9.6	7.17	25.27
809.0	44.94	3.93	9.6	7.18	25.35
810.0	44.94	3.93	9.6	7.18	25.36
811.0	45.01	3.94	9.7	7.19	25.34
812.0	45.04	3.95	9.7	7.19	25.36
813.0	45.05	3.95	9.7	7.19	25.33
814.0	45.10	3.96	9.8	7.19	25.35
815.0	45.12	3.98	9.8	7.20	25.42
816.0	45.15	3.98	9.8	7.20	25.42

DELMAG D 36-32

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	28.00	ft
Pile Penetration	28.00	ft
Pile Top Area	26.10	in2

Limited driving stress to 45 ksi

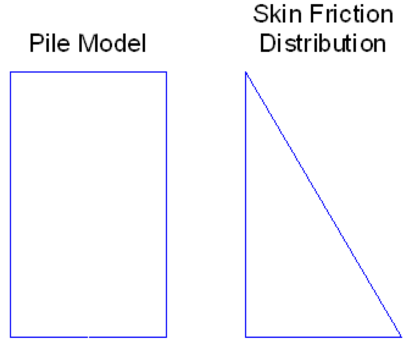
Strength Limit State:

$R_{dr\_14x89\_factored} := 811 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x89\_factored} = 527 \cdot kip$

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

$R_{dr\_14x89\_servext} := 811 \cdot kip$



Res. Shaft = 20 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag 36-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation — 13-Feb-2012
19284 Auburn Littlefields Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
998.0	42.18	2.10	14.7	7.53	24.68
999.0	42.29	2.14	14.7	7.54	24.73
1000.0	42.27	2.11	14.8	7.54	24.71
1001.0	42.27	2.10	14.9	7.54	24.70
1002.0	42.26	2.06	15.0	7.54	24.66
1003.0	42.35	2.11	14.9	7.54	24.73
1004.0	42.32	2.05	15.0	7.54	24.71
1005.0	42.39	2.06	15.0	7.55	24.75
1006.0	42.38	2.09	15.1	7.55	24.74
1007.0	42.39	2.03	15.2	7.55	24.72

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x117\_factored} := 1002 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x117\_factored} = 651 \cdot kip$

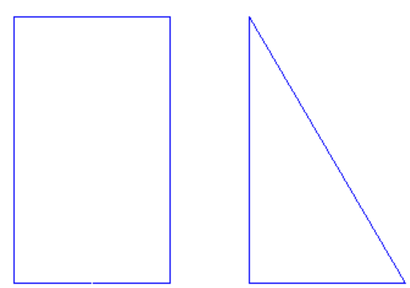
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x117\_servext} := 1002 \cdot kip$

DELMAG D 36-32

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	28.00	ft
Pile Penetration	28.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

For cases where interface friction is considered (for gravity structures) use Coulomb Theory

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

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

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

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

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

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

$K_p = 6.89$

Rankine Theory - Passive Earth Pressure 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 5th 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 18 to >50 - Soils are medium dense to very 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 at the service limit state.

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on native soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. Footings will be embedded 5.5 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: $D_w := 10 \cdot ft$ Based on boring logs

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

Look at several footing widths/
stem lengths

$$B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

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)

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

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

$$q_{nominal} = \begin{pmatrix} 6.1 \\ 6.7 \\ 7.4 \\ 8.1 \\ 8.8 \end{pmatrix} \cdot tsf$$

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

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

$$q_{factored} = \begin{pmatrix} 2.7 \\ 3 \\ 3.3 \\ 3.7 \\ 4 \end{pmatrix} \cdot tsf$$

Based on these footing widths

$$q_{factored} = \begin{pmatrix} 5.5 \\ 6.1 \\ 6.7 \\ 7.3 \\ 7.9 \end{pmatrix} \cdot ksf \qquad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \\ 14 \end{pmatrix} \cdot ft$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 6 ksf for wall footings or bases less than 8 feet wide.
Recommend a limiting factored bearing resistance of 7 ksf for wall footings or bases between 8.5 and 14 feet wide.

Settlement Analysis:

Reference: FHWA Soils and Foundations Reference Manual - Volume 1 (FHWA NHI-06-088) Hough pg 7-16 and
AASHTO LRFD Bridge Design Specifications 5th Edition 2010

The roadway grade at Abutment No. 1may be raised by as much as 3.5 feet.
Look at a simplified soil profile based on BB-ALAR-101/101A

Finished Grade

Proposed Fill - Look at 3.5 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Existing Grade

Existing Fill - fine to coarse sand $H_{1fill} := 27.0 \cdot ft$ $\gamma_{fill} := 125 \cdot pcf$ $N_{fill} := 20$

Groundwater at 10.0 ft bgs

$\gamma_w := 62.4pcf$

Sand - fine to coarse sand $H_{2sand} := 9 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{sand} := 42$

Bedrock - Gneiss

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name:	Littlefields	Client:	Auburn
Project Number:	19284.00	Project Manager:	Benoit
Date:	02/14/11	Computed by:	KM

Embank. slope a = 12.00(ft)
Embank. width b = 24.00(ft)
p load/unit area = 437.50(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 12.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	437.50
1.00	425.87
2.00	414.09
3.00	402.04
4.00	389.66
5.00	376.98
6.00	364.07
7.00	351.03
8.00	338.00
9.00	325.12
10.00	312.49
11.00	300.22
12.00	288.38
13.00	277.02
14.00	266.18
15.00	255.86
16.00	246.07
17.00	236.81
18.00	228.05
19.00	219.77
20.00	211.96
21.00	204.58
22.00	197.62
23.00	191.05
24.00	184.84
25.00	178.97
26.00	173.42
27.00	168.17
28.00	163.19
29.00	158.48
30.00	154.00
31.00	149.75
32.00	145.71
33.00	141.87

at 13.5 ft $\Delta\sigma_{z1fill} := 271.6 \cdot psf$

at 31.5 ft $\Delta\sigma_{z2sand} := 147.73 \cdot psf$

Existing Fill

$tsf := psf \cdot 1000$

Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point: $\sigma_{1fill\_o} := 10 \cdot ft \cdot (\gamma_{fill}) + 3.5 \cdot ft \cdot (\gamma_{fill} - \gamma_w)$ $\sigma_{1fill\_o} = 1.4691 \cdot tsf$

Corrected SPT N_{60}-value (bpf) $N_{fill} = 20$

At P_o = 1.5 tsf $C_{N\_1fill} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{1fill\_o}}\right)$ $C_{N\_1fill} = 1.105$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_1fill} \cdot N_{fill}$ $N1_{60} = 22$
From LRFD Eq 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C_{1fill} := 73$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1fill} = 271.6 \cdot psf$

Sand Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{2sand\_o} := \left[\frac{H_{2sand}}{2} \cdot (\gamma_{sand} - \gamma_w)\right] + 10.0 \cdot ft \cdot (\gamma_{fill}) + 17.0 \cdot ft \cdot (\gamma_{fill} - \gamma_w) \qquad \sigma_{2sand\_o} = 2.5959 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{sand} = 42$

At P_o = 2.6 tsf $C_{N\_2sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{2sand\_o}}\right)$ $C_{N\_2sand} = 0.9146$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_2sand} \cdot N_{sand}$ $N1_{60} = 38$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C_{2sand} := 109$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2sand} = 147.73 \cdot psf$

Calculate Settlement:

Existing Fill: $$\Delta H_{1fill} := H_{1fill} \cdot \frac{1}{C_{1fill}} \cdot \log\left(\frac{\sigma_{1fill\_o} + \Delta\sigma_{z1fill}}{\sigma_{1fill\_o}}\right) \qquad \Delta H_{1fill} = 0.327 \cdot in$$

Native Sand: $$\Delta H_{2sand} := H_{2sand} \cdot \frac{1}{C_{2sand}} \cdot \log\left(\frac{\sigma_{2sand\_o} + \Delta\sigma_{z2sand}}{\sigma_{2sand\_o}}\right) \qquad \Delta H_{2sand} = 0.0238 \cdot in$$

$\Delta H_T := \Delta H_{1fill} + \Delta H_{2sand}$ $\Delta H_T = 0.35 \cdot in$

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 assume a water content = ~15%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1400 frost penetration = 79.2 inches

$\text{Frost\_depth} := 72.4\text{in}$ $\quad$ $\text{Frost\_depth} = 6 \cdot \text{ft}$

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.

$\text{Frost\_depth}_{\text{modberg}} := 66.6 \cdot \text{in}$ $\quad$ $\text{Frost\_depth}_{\text{modberg}} = 5.55\,\text{ft}$

Use Modberg Frost Depth = 5.5 feet for design

Seismic:

Seismic Site Classification
Ref: LRFD Table C3.10.3.1-1
Method B: Average N for the top 100 feet of soil

BB-ALAR-101A					BB-ALAR-103				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
6	14	sand	7	0.5	2	29	sand	3	0.103448
11	27	sand	5	0.185185	6	8	sand	5	0.625
14.9	10	sand	4	0.4	11.9	50	sand	5	0.1
16	28	sand	7	0.25	15.5	25	sand	5	0.2
21.2	50	sand	8	0.16	21	50	sand	5	0.1
28.5	42	sand	4	0.095238	26	50	sand	5	0.1
35	100	bedrock	65	0.65	28.7	100	bedrock	72	0.72
SUM			**100**	**2.240423**				**100**	**1.948448**

di/di/N **44.63442**

di/di/N **51.32289**

SUM	Nav	47.97866

15<Nav<50 bpf; Site Class D

19284 Auburn Littlefields Bridge

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 Provision

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 inches wide, by 0.5 inch 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: backfill and bedding material shall only contain particles that will pass the 3-inch square mesh sieve and 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, Service 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)
   3. 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.
   4. 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.

   Live load surcharge on Prefabricated Concrete Modular Gravity walls shall be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from LRFD Table 3.11.6.4-2 with consideration for the distance from the wall pressure surface to the edge of traffic. 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 $\gamma P_{H1,}$ 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. Safety Against Structural Failure. Prefabricated units shall be designed for all strength and reinforcement requirements in accordance with LRFD Section 5 and LRFD Article 11.11.5.

E. 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.

F. The maximum calculated factored bearing pressure under the Prefabricated Concrete Modular Gravity block wall shall be clearly indicated on the design drawings.

G. 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.

H. 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.

I. Design Life. Design life shall be in accordance with AASHTO requirements or 75 years; the more stringent requirements apply.

J. 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 or 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 foot 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 foot 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