MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

OAKDALE BRIDGE NORTH BOUND
OVER LITTLE ANDROSCOGGIN RIVER
US ROUTE 202 AND STATE ROUTES 100 AND 4
AUBURN, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of the Oakdale Bridge North Bound (NB) over the Little Androscoggin River in Auburn, Maine. The proposed bridge replacement will consist of two-span steel girder superstructure founded on H-pile supported integral abutments and a pipe pile bent pier. The following design recommendations are discussed in detail in Section 7.0 of this 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.

Pipe Pile Pier Bent - Piles for the pier bents may consist of concrete filled pipe piles driven to bedrock. Pipe piles can be driven open-ended or closed-ended. The pipe piles shall be designed at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the pipe piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. A modified strength limit state analysis should be performed that includes the ice pressures specified in MaineDOT BDG Section 3.9.

Extreme limit state design checks for piers shall include pile geotechnical and structural failure by buckling and uplift with respect to extreme event loading combinations related to seismic forces, ice loads, vessel collision and certain hydraulic events. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading. All piles should be designed to achieve a fixed condition for the design scour event.

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at the pier. The first pile driven at the pier should be dynamically tested with a minimum 24-hour restrike test to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

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 and Downdrag – The vertical alignment of the proposed bridge will be raised approximately 2.0 feet for construction of the proposed replacement bridge. Evaluation of the potential settlement due to the placement of the fill resulted in approximately 1.25 inches of settlement behind the proposed abutments. This settlement is estimated to occur over approximately 10 years and may require attention by a maintenance crew. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Settlements in excess of 0.4 inches in soils were driven piles are present will result in downdrag (negative skin friction) forces on piles. Downdrag forces should be used in pile design at the proposed abutments as contributing to the maximum factored axial load.

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 – Seismic analysis is required for multi-span bridges in Seismic Zone 2. The minimum analysis requirements for Seismic Effects are single mode elastic method/uniform load elastic method (SM/UL).

Construction Considerations – 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.

Wood was encountered in all of the borings within the native sand layer. It is likely that any wood encountered during pile driving activities will impact pile installation operations. These impacts include but are not limited to driving H-piles for abutment foundations, driving pipe piles for the pile bent pier 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.

All timber piling within the river shall be removed to a minimum of 1 foot below river bed. Payment shall be considered incidental to bridge removal.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of the Oakdale Bridge North Bound (NB) over the 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 Oakdale Bridge NB carries US Route 202 and State Routes 100 and 4 over the Little Androscoggin River and was constructed in 1931. The bridge consists of a three-span, concrete structure founded on timber pile supported abutments and timber pile supported concrete piers. The structure has a total length of approximately 166 feet on a 36 degree skew. The 2012 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and substructures are in fair condition (rating of 5) and the superstructure is in poor condition (rating of 4). The Bridge Sufficiency Rating is 38.2. 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 is in overall fair condition with isolated areas of moderately heavy deterioration primarily below leaking joints and above piers. Notes state that the abutments and wingwalls are generally solid with minor defects and the piers have assorted cracks and spalls at the bearing areas.

The MaineDOT Bridge Program and their structural design consultant, Vanasse Hangen Brustlin, Inc. (VHB), are proposing a replacement structure consisting of a two-span, curved, steel plate girder superstructure supported on H-pile supported integral abutments and a pipe pile bent pier. The overall length of the proposed replacement structure will be 210 feet. The proposed structure will have a skew of approximately 25 degrees. The proposed roadway profile will be raised approximately 1.75 feet at the abutments. Two-way traffic will be maintained during construction using one lane in each direction in the southbound corridor.

2.0 GEOLOGIC SETTING

The Oakdale Bridge NB in Auburn carries Washington Street (US Route 202 and State Routes 100 and 4) over the Little Androscoggin River as shown on Sheet 1 - Location Map found at the end of this report.

According to the Minot Quadrangle, Maine Surficial Geology map published by the Maine Geological Survey Open File No. 02-231 (2002) the surficial soils in the vicinity of the site consist of stream alluvium deposits. The stream alluvium deposits generally consist of sand, silt, gravel and muck in flood plains along present rivers and streams. In places, this deposit in interbedded with fresh water wetlands deposits, including along the Little Androscoggin River flood plains.

According to the Bedrock Geologic Map of Maine published by the Maine Geological Survey (1985) the bedrock in the vicinity of the site consists of interbedded pelite and limestone and/or dolostone of the Sangerville Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three (3) test borings. Test boring BB-ALAR-101 was conducted behind the south abutment. Test borings BB-ALAR-102 was conducted through the existing bridge deck, in the river near the proposed pier location. Test boring BB-ALAR-103 was conducted behind the north abutment. The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The borings were drilled between March 11 and 26, 2013 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D 4633-05 "Standard Test Method for Energy Measurement for Dynamic Penetrometers" in August 2012 and was found to deliver approximately 26 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 0.756 to the raw field N-values. This hammer efficiency factor (0.756) and both the raw field N-value and the corrected N-value (N_{60}) are shown on the boring logs.

Undisturbed tube samples were obtained in the soft soil deposits where possible. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. The subsurface conditions were logged in the field by a consultant engineer hired to assist on this project. 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) grain size analyses with hydrometer and water content, seven (7) Atterberg Limits tests, two (2) 1-D consolidation tests, and two (2) standard tube openings with laboratory vanes. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 4 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of sand fill; stream alluvium; glaciomarine silt, clayey silt and silty clay; and glacial till all underlain by bedrock. The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 – Interpretive Subsurface Profile both found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

5.1 Sand Fill

A layer of sand fill is present beneath the pavement at the abutment locations. Samples of the sand fill were:

- Brown, damp, gravelly, fine to coarse sand, trace silt;
- Brown, damp to moist, fine to coarse sand, some gravel, trace to little silt;
- Brown, damp to moist, fine to medium sand, some gravel, trace to little silt, and
- Brown, damp, fine to coarse sandy gravel, trace silt;

The thickness of the sand fill layer was approximately 13.0 feet in boring BB-ALAR-101 and approximately 12.0 feet in boring BB-ALAR-102. Corrected SPT N-values in the sand fill ranged from 1 to 47 blows per foot (bpf) indicating that the fill is very loose to dense in consistency.

5.2 Stream Alluvium

A deposit of steam alluvium was encountered beneath the fill at the approaches and in the riverbed. The thickness of the deposit ranged from approximately 6.0 feet in boring BB-ALAR-102 in the river to approximately 17.0 feet thick boring BB-ALAR-103. The deposit generally consisted of:

- Grey, silty fine sand, with decomposed wood;
- Grey, wet fine sand with roots and organics;
- Brown, moist, fine to coarse sand, some silt, trace gravel, trace clay, trace organics and wood;
- Greyish-brown, wet, fine to medium sand, little silt, trace clay; and

- Greyish-brown, wet, fine to coarse sand, trace silt, trace gravel, interbedded with wood.

Corrected SPT N-values in the stream alluvium ranged from weight of hammer (WOH) to 13 bpf indicating that the soil is very loose to medium dense in consistency. Water contents from two (2) samples obtained within the layer range from approximately 24% to 30%. Two (2) grain size analyses with hydrometer conducted on samples from the layer indicate that the soil is classified as an A-4 or A-2-4 by the AASHTO Classification System and a SC-SM by the Unified Soil Classification System (USCS).

5.3 Glaciomarine Silt, Clayey Silt and Silty Clay

A layer of glaciomarine silt, clayey silt and silty clay was encountered beneath the stream alluvium in all of the borings. The thickness of the layer ranged from approximately 31.5 feet in boring BB-ALAR-102 to approximately 39.4 feet in boring BB-ALAR-103. The following subsections describe the glaciomarine soils encountered in the borings:

Silt. The silt generally consisted of dark grey, wet, silt, some clay, trace fine sand, trace gravel. Corrected SPT N-values in the silt samples ranged from WOH to 3 bpf indicating that the soil is very soft to soft in consistency. Vane shear testing conducted within the silt showed measured undrained shear strengths ranging from approximately 223 to 321 pounds per square foot (psf) while the remolded shear strength ranged from approximately 22 to 76 psf. Based on the ratio of peak to remolded shear strength from the vane shear tests, the silt was determined to have sensitivity ranging from approximately 2.9 to 10.1 and is classified as medium sensitive to slightly quick. Water contents from two (2) silt samples ranged from approximately 21% to 30%. Two (2) grain size analysis conducted on silt samples indicate that the silt is classified as an A-6 or A-4 by the AASHTO Classification System and as a CL-ML or CL by the USCS.

Table 5-1, below, summarizes the results of the Atterberg Limits tests from samples of the silt:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ALAR-101 5D	29.6	35	20	15	0.64
BB-ALAR-102 2D	20.9	24	20	4	0.23

Table 5-1 – Summary of Atterberg Limits Testing Results for Silt Samples

Interpretation of these results indicates that since the calculated water content is between the liquid limit and the plastic limit the silt is over consolidated to heavily over consolidated.

Clayey Silt. The clayey silt generally consisted of:

- Dark grey, wet, clayey silt, trace to some fine sand and
- Dark grey, wet, clayey silt, trace to some fine to medium sand.

Vane shear testing conducted within the clayey silt showed measured undrained shear strengths ranging from approximately 192 to 783 pounds per square foot (psf) indicating that the soil is very soft to medium stiff in consistency. The remolded shear strength of the soil ranged from approximately 0 to 210 psf. Based on the ratio of peak to remolded shear strength from the vane shear tests, the clayey silt was determined to have sensitivity ranging from approximately 3.4 to 55.0 and is classified as medium sensitive to very quick. Water contents from three (3) clayey silt samples ranged from approximately 30% to 36%. Three (3) grain size analysis conducted on clayey silt samples indicate that the clayey silt is classified as an A-4 or A-6 by the AASHTO Classification System and as a CL by the USCS.

Table 5-2, below, summarizes the results of the Atterberg Limits tests from samples of the clayey silt:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ALAR-101 1U	32.9	30	22	8	1.36
BB-ALAR-103 7D	30.0	27	19	8	1.38
BB-ALAR-103 1U	35.9	35	21	14	1.06

Table 5-2 – Summary of Atterberg Limits Testing Results for Clayey Silt Samples

Interpretation of these results indicates that the clayey silt has liquidity indices in excess of 1 are and the soil is on the verge of being a viscous liquid when disturbed. Soils with liquidity indices in excess of 1 have a high liquefaction potential commonly referred to as “quick”. It can be inferred that overburden pressure and inter-particle cementation are providing stability for these soils.

Silty Clay. The silty clay generally consisted of:

- Grey, wet, silty clay and
- Grey, wet, silty clay, trace fine to medium sand.

Vane shear testing conducted within the silty clay showed measured undrained shear strengths ranging from approximately 357 to 1384 pounds per square foot (psf)) indicating that the soil is soft to stiff in consistency. The remolded shear strength of the soil ranged from approximately 82 to 313 psf. Based on the ratio of peak to remolded shear strength from the vane shear tests, the clayey silt was determined to have sensitivity ranging from approximately 3.8 to 11.5 and is classified as medium sensitive to slightly quick. Water contents from three (3) silty clay samples ranged from approximately 29% to 35%. Three (3) grain size analysis conducted on silty clay samples indicate that the silty clay is classified as an A-6 by the AASHTO Classification System and as a CL by the USCS.

Table 5-3, below, summarizes the results of the Atterberg Limits tests from samples of the silty clay:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ALAR-101 8D	34.9	36	22	14	0.92
BB-ALAR-102 4D	28.8	34	21	13	0.60

Table 5-3 – Summary of Atterberg Limits Testing Results for Silty Clay Samples

Interpretation of these results indicates that since the calculated water content is between the liquid limit and the plastic limit the silty clay is over consolidated.

5.4 Glacial Till

A layer of glacial till comprised of sand and gravel was encountered beneath the glaciomarine soils. The thickness of the layer ranged from approximately 3.4 feet in boring BB-ALAR-103 to 8.2 feet in boring BB-ALAR-101. The glacial till generally consisted of:

- Brownish grey and grey, wet, gravelly fine to coarse sand, trace to little silt, trace clay;
- Grey, wet, fine to coarse sand, some gravel, little silt, trace clay; and
- Grey, wet, fine to coarse sandy gravel, trace to little silt, trace clay.

Corrected SPT N-values in the glacial till layer ranged from 19 to 60 bpf indicating that the soil is medium dense to very dense in consistency. Water contents from four (4) samples obtained within the layer ranged from approximately 10% to 15%. Four (4) grain size analysis conducted on samples from the layer indicate that the soil is classified as an A-1-a or A-1-b by the AASHTO Classification System and as an SC-SM, GC-GM, or SW-SC by the Unified Soil Classification System.

5.5 Bedrock

Bedrock was encountered and cored in all of the borings. The Table 5-4 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB-ALAR-101	65.7 feet	139.8 feet	80 – 97%[1]
BB-ALAR-102	42.5 feet	138.6 feet	70 - 93%
BB-ALAR-103	71.8 feet	134.4 feet	90 – 92%

[1]Approximately 2 feet of weathered bedrock is present at the bedrock surface at this boring location.

Table 5-4 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as greenish-grey, medium to very coarse grained, hard, fresh to slightly weathered, porphyritic granite, with biotite rich zones in layers and closely to moderately spaced horizontal to low angle joints. The rock quality designation (RQD) of the bedrock was determined to range from 70 to 97 percent indicating a rock mass quality of fair excellent.

5.6 Groundwater

Groundwater was observed at a depth of less than approximately 10.0 feet below the existing ground surface in boring BB-ALAR-101. The water level, measured during drilling, is indicated on the boring log found in Appendix A. No groundwater was observed in borings BB-ALAR-102 and BB-ALAR-103. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water level indicated on the boring log does not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes and changes in water levels in the river.

6.0 FOUNDATION ALTERNATIVES

The following alternatives were considered for the bridge replacement:

- Rehabilitation of the existing structure;
- Replacement with a two-span, approximately 210-foot long structure with abutments founded on integral, driven H-pile supported abutments and a pile bent pier; and
- Replacement with a three-span, approximately 210-foot long structure with abutments founded on integral, driven H-pile supported abutments and two pile bent piers.

After consideration of all of the alternatives, the two-span, approximately 210-foot long structure with abutments founded on integral, driven H-pile supported abutments and a single pile bent pier structure was selected. This option provides a durable, low maintenance structure that can be constructed without significant waterway impacts or cofferdams. This report addresses only this selected structure and foundation types.

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 and a single pipe pile bent pier driven to bedrock which have been identified as the optimal substructures 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.[2] 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.

[2] Pile sizes HP 12x53, HP 14x73 and HP 14X89 are not allowed for steel integral bridges with a bridge length of 210 feet and a fixed head abutment per MaineDOT Bridge Design Guide (BDG) Table 5-3.

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

Location	Estimated Pile Cap Bottom Elevation	Approximate Depth to Bedrock From Ground Surface	Approximate Top of Rock Elevation	Estimated Pile Length (including 1 foot embedment into pile cap)
Abutment #1 BB-ALAR-101	195 feet	67.7 feet[3]	137.8 feet[3]	~59 feet
Abutment #2 BB-ALAR-103	195 feet	71.8 feet	134.4 feet	~62 feet

[3]Approximately 2 feet of weathered bedrock is present at the bedrock surface at this boring location.

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

These pile lengths do not take into account the additional up to 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
- Drivability resistance

The pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through 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 L-Pile® software.

<u>Structural Resistance.</u> The nominal axial 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 structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and an unbraced length (ℓ) of 1 foot (for scour) and an effective length factor (K) of 1.2. These factored axial structural compressive resistances are 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 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 (ϕ_c=0.50).” 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 can 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 for 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[4] ϕ_c=0.60	Controlling Geotechnical Resistance[5] ϕ_c=0.50	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	464[6]	387	303	303
HP 12x74	653	544	358	358
HP 14x73	641[6]	534	358	358
HP 14x89	782[6]	652	428	428
HP 14x117	1031	859	471	471

[4] Based on preliminary assumption of ℓ=1 foot and K=1.2
[5] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock*.
[6] Pile sizes HP 12x53, HP 14x73 and HP 14x89 are not allowed for bridges with a bridge length of 210 feet and a fixed head abutment per MaineDOT BDG Table 5-3.

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

Local experience supports the 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 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 displacements 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 overturning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to seismic loads, 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. Per LRFD Article 10.5.5.3, resistance factors at the extreme event shall be 1.0 except that for uplift resistance of piles, the resistance factor shall be taken as 0.8 or less. 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. The exception is the overall global stability of the foundation should be investigated at the Service I load combination and a resistance factor, ϕ, of 0.65 and uplift resistance of piles where ϕ, of 0.80 or less shall be used. 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 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 State Factored Axial Pile Resistance (kips)			
	Structural Resistance[7] ϕ=1.0	Geotechnical Resistance[8] ϕ=1.0	Drivability Resistance ϕ=1.0	Governing Resistance
HP 12x53	774[9]	774	466	466
HP 12x74	1088	1088	550	550
HP 14x73	1069[9]	1069	550	550
HP 14x89	1303[9]	1303	659	659
HP 14x117	1718	1718	725	725

[7]Based on preliminary assumption of ℓ=1 foot and K=1.2
[8] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock*.
[9] Pile sizes HP 12x53, HP 14x73 and HP 14x89 are not allowed for bridges with a bridge length of 210 feet and a fixed head abutment per MaineDOT BDG Table 5-3.

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

Local experience supports the estimated factored resistances from the drivability analyses. It is recommended that the maximum factored axial pile load used in design for the service and extreme limit states not exceed the governing resistance shown in the last column of Table 7-3 above.

7.1.3 Lateral Pile Resistance

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

Recommended geotechnical parameters for generation of p-y curves in lateral pile analyses are provided in Tables 7-4 and 7-5 below. In general, the model developed should emulate the soil at the site by using the soil layers (referenced in Tables 7-4 and 7-5 by elevations) and appropriate structural parameters and pile-head boundary conditions for the pile section being analyzed. It is recommended that the analyses be conducted assuming a fixed pile-head boundary condition.

Soil Layer	Elevation of Soil Layer at Abutment No. 1 (feet)	Elevation of Soil Layer at Abutment No. 2 (feet)	Water Table Condition	Effective Unit Weight lb/in^3 (lb/ft^3)
Sand Fill (above water table)	205.5 to 195.0	202.6 to 195.0	Above	0.0723 (125)
Sand Fill (below water table)	195.0 to 192.5	195.0 to 194.2	Below	0.0365 (63)
Stream Alluvium	192.5 to 181.5	194.2 to177.2	Below	0.0336 (58)
Silt	181.5 to 165.0	Not encountered	Below	0.0307 (53)
Clayey Silt	165.0 to 155.0	177.2 to 145.0	Below	0.0307 (53)
Silty Clay	155.0 to 148.0	145.0 to 137.8	Below	0.0307 (53)
Glacial Till	148.0 to 139.8	137.8 to 134.4	Below	0.0365 (63)

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

Soil Layer	k_s (lb/in^3)	Cohesion lb/in^2 (lb/ft^2)	E_{50} for clays	Friction Angle
Sand Fill (above water table)	25	-	-	32º
Sand Fill (below water table)	20	-	-	32º
Stream Alluvium	20	-	-	32°
Silt	30	2.083 (300)	0.020	-
Clayey Silt	100 (Abutment 1) 30 (Abutment 2)	5.556 (800)	0.010 (Abutment 1) 0.020 (Abutment 2)	-
Silty Clay	100	6.944 (1000)	0.010	-
Glacial Till	60	-	-	36°

Table 7-5 - 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 earth 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 foundation 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 0.005, 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 7-6 below:

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

Table 7-6 - 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 Pipe Pile Pier Bent

Pile bent piers were selected for intermediate structure support. Piles for the pier bents may consist of pipe piles driven to bedrock and filled with concrete. Pipe piles with diameters of 24, 26, 28 or 30 inches and wall thicknesses of 1/2 or 5/8 inch are recommended. Pipe piles should be fabricated in accordance with ASTM A252, Grade 3, with minimum yield strength of 45 ksi. For some pile diameters, Grade 3 Modified steel with yield strengths of 50 and 55 ksi are available. Consult with steel pile fabricators for availability if higher yield strength steel is needed. Piles shall be filled with Class A concrete. Piles should have straight butt-welded seams. Spiral seams are not recommended because the welded surfaces are vulnerable to thin fusion bonded epoxy coatings, ice abrasion and bumping during construction. Any welds between pile segments should be ground down and blended smooth with the pipe pile material. Pipe piles can be driven open-ended or closed-ended. Open ended piles should be equipped with a cutting shoe constructed from ASTM A148 grade 90/60 steel. Closed ended piles should be equipped with a conical point constructed from ASTM A148 grade 90/60 cast steel. Open-ended piles will require clean out of soils inside the pile to a depth specified by the structural engineer. Pipe pile pier bent piles should be end bearing and driven to the required nominal resistance on or within the bedrock.

Pipe piles shall be coated with a polyurea coating or a fusion bonded epoxy coating with a thickness of 18 to 20 mil and top coated in accordance with Special Provision 506. The polyurea coating or fusion bonded epoxy protective coating shall be applied to a minimum of 10 feet below river bed or 2 feet below the total scour depth. The portion of the pipe pile to be embedded in the concrete pile cap shall not be top coated. Cathodic protection by aluminum alloy anodes shall be used in addition to fusion bonded epoxy protective coating.

Pile lengths at the proposed pier may be roughly estimated based on Table 7-7 below:

Location/ Boring	Pile Orientation	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Estimated Pile Length (including 1 foot embedment into pile cap)
Pier 1 BB-ALAR-102	Plumb 4 in/ft Batter	±197.0 feet	42.5 feet	138.6 feet	~60 feet ~65 feet

Table 7-7 – Estimated Pile Lengths for Pipe Piles

This estimated pile length does not take into account the variability of the bedrock surface within the channel or the additional up to five (5) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

7.3.1 Strength Limit State

The design of pipe pile bents at the strength limit state considering the structural, geotechnical and drivability resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The pile group (pile bent) resistance after scour due to the design flood when subjected to the strength limit state load combinations shall provide adequate foundation resistance using the resistance factors given in this section. A modified strength limit state analysis should be performed that includes the ice pressures specified in MaineDOT BDG Section 3.9 – Ice Loads.

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 6.9.5.1 for composite members (pipe pile). The pipe piles have an unbraced length (ℓ) and require calculation of the λ-factor as specified in LRFD Article 6.9.5.1.

For the strength limit state, the factored axial compressive structural resistance of the pipe pile (P_r) shall be calculated using the resistance factors (ϕ_c) of 0.7 for pipe pile in good driving conditions as specified in LRFD Article 6.5.4.2. These factored axial structural compressive resistances are presented in Table 7-8 below. The proposed pier bent piles will have an unbraced pile length of approximately 21.5 feet.

Per LRFD Article 6.5.4.2, at the strength limit state, for pipe piles in compression and bending, the axial compressive resistance factor ϕ_c=0.8 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.9.2.2-1 or -2) with flexural resistance determined as specified in LRFD 6.12. The factored structural resistance for pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

Geotechnical Resistance. The nominal axial geotechnical compressive resistance in the strength limit state was calculated using 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 (ϕ_c=0.60)." These factored axial structural compressive resistances are presented in Table 7-8 below.

Drivability Resistance. The drivability of the eight (8) proposed pipe pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40.5 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. These factored axial structural compressive resistances are presented in Table 7-8 below.

A summary of the calculated factored axial compressive structural, geotechnical and drivability resistances for eight (8) pipe pile sections is presented in the Table 7-8 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pipe Pile		Strength Limit State Factored Resistance (kips)			
Diameter (inches)	Wall Thickness (inches)	Structural Resistance[10] (non-composite section) ϕ_c=0.70	Controlling Geotechnical Resistance[11] φ_{stat}=0.60	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
24	½	671	575	627	575
26	½	757	649	676	649
28	½	843	722	722	722
30	½	927	794	725	725
24	5/8	888	761	728	728
26	5/8	1003	860	737	727
28	5/8	1116	957	747	747
30	5/8	1229	1053	766	766

[10] Based on preliminary assumption of ℓ=21.5 feet and K=2.0

[11] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock.*

Table 7-8 - Factored Axial Resistances for Pipe Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. For the 24- and 26-inch diameter pipe piles with a ½-inch wall thickness, the factored axial geotechnical resistance is less than the factored axial structural resistance and the factored axial drivability resistance and thus controls the design. For the remaining six (6) pile sizes, the factored axial drivability resistance is less than the factored axial structural resistance and the factored axial geotechnical resistance and thus controls the design. It is recommended that the maximum factored axial pile load used in design for the strength limit state not exceed the governing resistances shown in the last column of Table 7-8 above.

7.3.2 Service Limit and Extreme Limit State Designs

The design of the piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles with a resistance factor of φ = 1.0 and overall stability of the pile group with a resistance factor of φ = 0.65. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

Extreme limit state design checks for the pier shall include pile geotechnical and structural failure by buckling and uplift with respect to extreme event loading combinations related to seismic loads, ice loads, vessel collision and certain hydraulic events including scour due to the 500-year flood and debris loading. Resistance factors, φ, for the extreme limit state shall be taken as 1.0 except that for uplift resistance of piles a resistance factor, ϕ, of 0.80 or less shall be used. The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

The axial structural resistance of eight (8) proposed pipe pile sections was investigated using a resistance factor of 1.0. The piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9. The axial geotechnical compressive resistance of eight (8) proposed pipe pile sections and was calculated 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 and a resistance factor of 1.0. The drivability of the eight (8) proposed pipe pile sections was considered. The maximum driving stresses in the pipe pile, assuming the use of 45 ksi steel, shall be less than 40.5 ksi. The resistance factor for a single pile in axial compression for the service and extreme limit states of 1.0 was used.

The calculated factored axial structural, geotechnical and drivability resistances for the eight (8) pipe pile sections are summarized in the Table 7-9 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pipe Pile		Service and Extreme Limit States Factored Resistance (kips)			
Diameter (inches)	Wall Thickness (inch)	Structural Resistance[12] (non-composite section) ϕ=1.0	Controlling Geotechnical Resistance[13] ϕ=1.0	Drivability Resistance ϕ=1.0	Governing Resistance
24	½	959	959	964	964
26	½	1082	1082	1040	1040
28	½	1204	1204	1110	1110
30	½	1324	1324	1115	1115
24	5/8	1268	1268	1120	1120
26	5/8	1433	1433	1134	1134
28	5/8	1595	1595	1150	1150
30	5/8	1755	1755	1178	1178

[12] Based on preliminary assumption of ℓ=21.5 feet and K=2.0
[13] Based on guidance in LRFD Article 10.7.3.2.3, *Piles Driven to Hard Rock.*

Table 7-9 - Factored Axial Resistances for Pipe Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The factored axial drivability resistance is less than the factored axial structural resistance and the factored axial geotechnical resistance. 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 7-9 above.

7.3.3 Estimated Effective Pile Lengths

Buckling stability of the piles shall be evaluated in accordance with the provisions in LRFD Articles 6.9, 6.12 and 6.15 using an effective pile length of the pile that accounts for the laterally unsupported length of the exposed pile extending through the air and/or water plus the embedment depth to pile fixity.

All piles should be designed to achieve a fixed condition for the design scour event. Preliminary depths to fixity for eight (8) proposed pipe pile sections were calculated, assuming only axial loading and without consideration of lateral loads, using the buckling methodology in LRFD Article 10.7.3.13.4. Table 7-10 below summarizes the estimated depths to fixity for the eight (8) proposed pile sections and the estimated design scour depth. The design scour depth provided by VHB was estimated to be between 4.8 and 5.5 feet. For the purposes of the geotechnical calculations the effective length of the pile was assumed to be the length of pile above the river bed (approximately 16 feet) plus the depth to fixity calculated for each proposed pile section. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Outside Pipe Pile Diameter/ Wall thickness	Preliminary Estimates of Depth to Fixity w/ no lateral loads applied (feet)	Estimated Exposed Pile Length Due to Design Scour (feet)	Estimated Unsupported Length, ℓ (length in air and water) (feet)	Estimated Effective Length For Buckling Analysis (depth to fixity + scour + unsupported length (feet)
24-in / ½ in	17.1	5.5	16.0	32.7
26-in / ½ in	18.4	5.5	16.0	33.4
28-in / ½ in	19.7	5.5	16.0	34.0
30-in / ½ in	20.9	5.5	16.0	34.7
24-in / ⅝ in	17.6	5.5	16.0	33.0
26-in / ⅝ in	19.0	5.5	16.0	33.7
28-in / ⅝ in	20.3	5.5	16.0	34.3
30-in / ⅝ in	21.5	5.5	16.0	35.0

Table 7-10 - Preliminary Estimates of Effective Pile Lengths for Pipe Piles Composite Section

Due to the depth of the overburden at the site, the pile sections will all achieve a fixed condition under normal conditions (no scour) and the design scour event when they are driven to end bearing on bedrock.

When the lateral and axial pile load groups are known, this data should be provided to the geotechnical engineer. An analysis of pile fixity can then be performed using L-Pile® or FB-MultiPier® software. If necessary, a more refined analysis of the pile bent can be performed by the structural engineer using MultiFrame 3D software.

7.3.4 Buckling and Combined Axial and Flexure

Pile group design shall consider loading effects due to combined axial and flexural loading, as outlined in LRFD Article 6.15. In designing piles for the bent group the effects of soil-structure interaction shall be considered in conformance with LRFD Article 10.7.3.12. The recommended design approach considers the non-linear response of soil with lateral displacement. Soil-structure interaction considering the non-linear response of soil can be modeled using L-Pile® or FB-MultiPier® software.

The factored structural resistances for pipe pile sections in combined axial compression and flexure and buckling analyses are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural engineer. For evaluating buckling and lateral stability in accordance with LRFD Article 10.7.3.13.4 use the effective pile lengths provided in Table 7-10.

7.3.5 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 the pier. The first pile driven 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 has refused on a cobble or boulder, 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 factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis, dynamic pile test measurements, CAPWAPs and as approved by the Resident. Driving stresses in the pipe pile determined in the drivability analysis shall be less than 40.5 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.

7.4 Scour and Riprap

A grain size analysis was performed on a soil sample taken at the approximate streambed elevation to generate a grain size curve for determining parameters to be used in scour analyses. The sample was assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.15 mm
- Average diameter of particle at 95 percent passing, D_{95} = 11.5 mm
- Soil Classification AASHTO Soil Type A-2-4

The grain size curve is 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 extreme limit state loads (load factor is 1.0). 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. 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 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04).

7.5 Settlement and Downdrag

The vertical alignment of the proposed bridge will be raised approximately 2.0 feet for construction of the proposed replacement bridge. The soils at the site are compressible and are susceptible to consolidation if the in-situ stresses are increased above the current levels (i.e., consolidation will occur if fill is placed or if structures are supported on compressible soils). Evaluation of the potential settlement due to the placement of up to 2.0 feet of fill resulted in approximately 1.25 inches of settlement. This settlement is estimated to occur over approximately 10 years and may require attention by a maintenance crew.

Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag (negative skin friction) forces on piles. The magnitude of downdrag has been estimated for the abutment piles based on the effective vertical stress and empirical β factors obtained from full scale tests. The calculated downdrag values are presented in Table 7-11 below:

Pile Section	Factored Downdrag Loads (DD) (kips)
HP 12 x 53	127
HP 12 x 74	130
HP 14 x 73	150
HP 14 x 89	152
HP 14 x 117	155

Table 7-11 – Factored Downdrag Loads (DD)

Calculations for the pile downdrag loads are included in Appendix C- Calculations found at the end of this report. Based on LRFD Table 3.4.1-2 and the use of an effective stress method to calculate downdrag, it is recommended that a load factor of γ_p=1.05 be applied to downdrag forces in both cohesive and cohesionless downdrag zones.

Downdrag forces can be handled or reduced by using one or more of the following techniques:

- Reduce soil settlement by preloading the soil
- Use lightweight fill materials
- Increase the capacity of the piles by increasing pile size and/or number
- Prevent direct contact between soil and pile by using a pile sleeve or pile membrane (e.g., Yellow Jacket™)
- Coating the pile with a friction reducer such as bitumen

Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible. See Appendix C - Calculations at the end of this report 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.

In the event that any foundation is placed on granular subgrade soils, it should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1224 F-degree days. In a granular soil with a water content of approximately 15%, this correlates to a frost depth of approximately 5.5 feet. 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 E (soil profile with average N-value for the upper 100 feet of the soils and rock profile of less than 15 bpf)
- Acceleration coefficient (A_s) = 0.221g
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.442g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.163g
- Seismic Zone 2 (based on S_{D1} greater than 0.15g and less than 0.30g)

According to Figure 2-2 of the MaineDOT BDG, the Oakdale Bridge NB on US Route 202/State Routes 4 and 100 is on the National Highway System (NHS) and is therefore considered to be functionally important. In conformance with LRFD Article 4.7.4.3, seismic analysis is required for multi-span bridges in Seismic Zone 2. The minimum analysis requirements for Seismic Effects are single mode elastic method/uniform load elastic method (SM/UL). Additional requirements for the determination of seismic design forces for foundations in Seismic Zone 2 are discussed in LRFD Article 3.10.9.3.

Per LRFD Article C3.10.9.1 abutments on multiple span bridges and retaining walls are subject to acceleration-augmented soil pressures as specified in LRFD Article 11.6.5. A seismic soil pressure shall be added to the static soil pressures and calculated using a dynamic earth pressure coefficient, K_{AE}, of 0.447 and a horizontal seismic acceleration coefficient, k_h, of 0.221g.

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

7.8 Construction Considerations

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.

Wood was encountered in all of the borings within the native sand layer. It is likely that any wood encountered during pile driving activities will impact pile installation operations. These impacts include but are not limited to driving H-piles for abutment foundations, driving pipe piles for the pile bent pier 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.

All timber piling within the river shall be removed to a minimum of 1 foot below river bed. Payment shall be considered incidental to bridge removal.

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

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

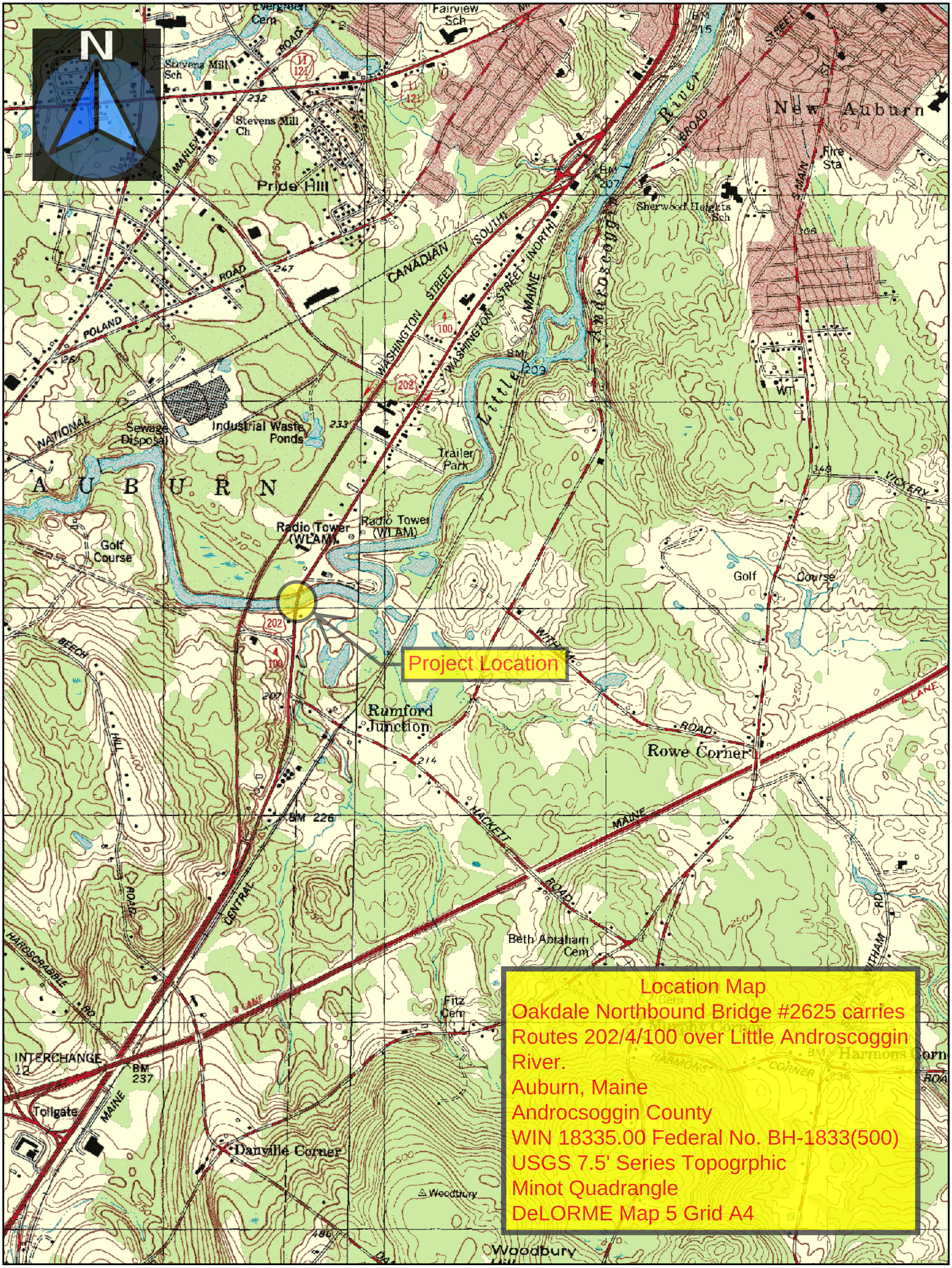
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements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

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Oakdale Bridge NB 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.

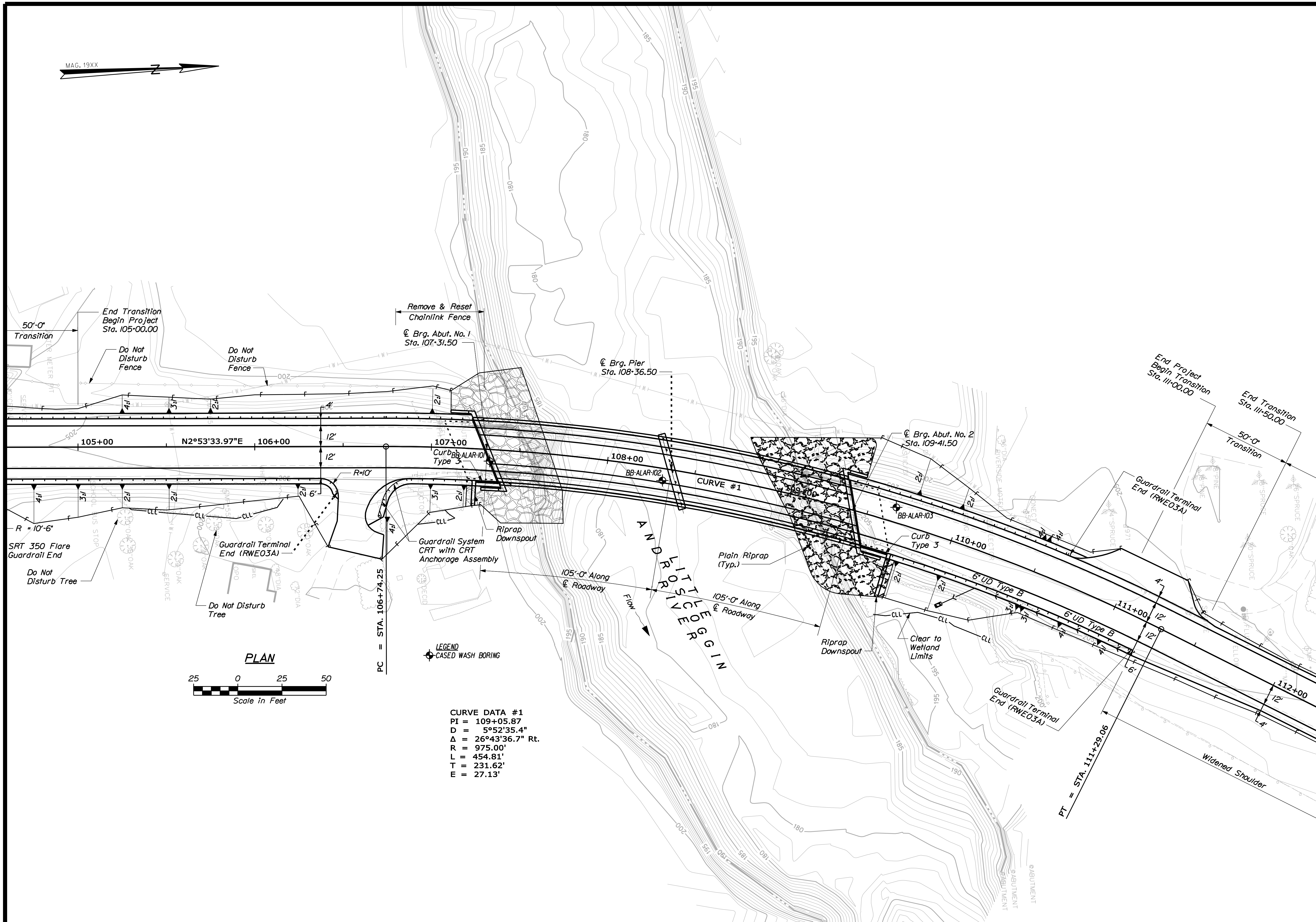
We also recommend that we be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



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.



CURVE DATA #1
PI = 109+05.87
D = 5°52'35.4"
Δ = 26°43'36.7" Rt.
R = 975.00'
L = 454.81'
T = 231.62'
E = 27.13'

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date: 7/25/2013

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1833(500)
WIN 18335.00
BRIDGE NO. 2625
BRIDGE PLANS

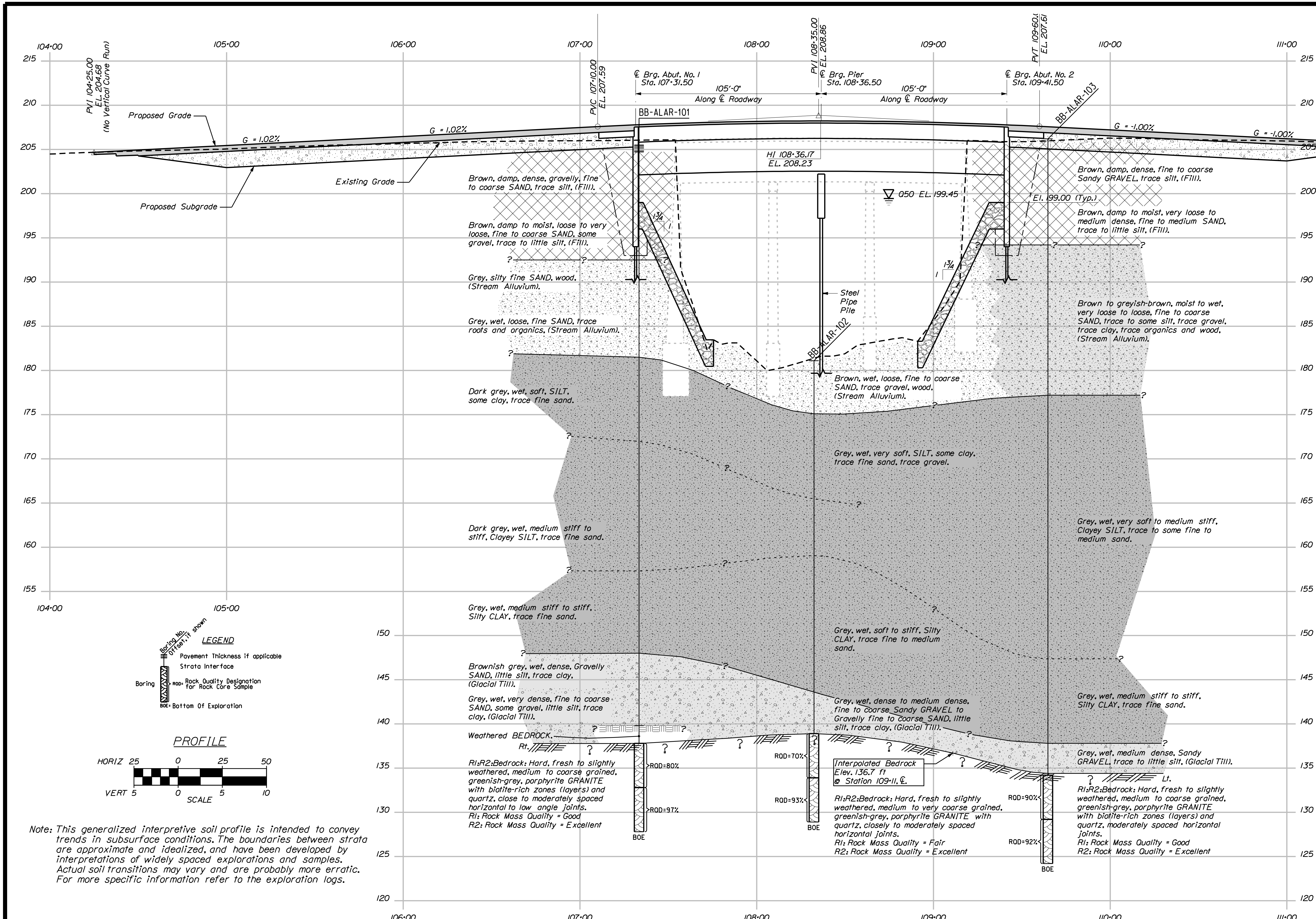
	BY	DATE
PROJ. MANAGER		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	T. WHITE	MAR 2013
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

OAKDALE NORTHBOUND BRIDGE
LITTLE ANDROSCOGGIN RIVER
AUBURN ANDROSCOGGIN COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1833(500)

WIN 18335.00

BRIDGE PLANS

BRIDGE NO. 2625

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED	T. WHITE	JUNE 2013
DESIGN2-DETAILED2	MAGUIRE	
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

OAKDALE NORTHBOUND BRIDGE
LITTLE ANDROSCOGGIN RIVER
AUBURN ANDROSCOGGIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:7/25/2013

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:7/25/2013

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

Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: BB-ALAR-101
WIN: 18335.00

Driller:	MaineDOT	Elevation (ft.)	205.5	Auger ID/OD:	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/11/13-3/13/13	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	107+33.4, 6.6 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	less than 10.0 ft.
Hammer Efficiency Factor: 0.756		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

Columns: 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

Depth	Sample No.	Pen./Rec.	Sample Depth	Blows	N-unc.	N_{60}	Casing Blows	Elevation	Visual Description and Remarks	Lab
0							SSA	204.71	9.5" Pavement (0.79)	
	1D	24/19	1.00 - 3.00	16/19/14/18	33	42			Brown, damp, dense, Gravelly fine to coarse SAND, trace silt, (Fill).	
5	2D	24/8	5.00 - 7.00	5/5/3/2	8	10			Brown, damp to moist, loose, fine to coarse SAND, some gravel, trace to little silt, (Fill).	
10	3D	24/3	10.00 - 12.00	3/1/1/2	2	3	16		Brown, moist, very loose, fine to medium SAND, little gravel, little silt, (Fill).	
15	4D	24/8	15.50 - 17.50	WOH/5/5/2	10	13	10	192.50	(13.00) WOOD, 19" long piece of wood through tip of spoon). Note: Grey, silty fine sand in decomposed wood in spoons no creosote odor noted.	
20	MD	24/0	20.00 - 22.00	5/3/2/5	5	6	10		Grab sample: Grey, wet, loose, fine SAND, trace roots/organics.	
25	5D / V1	24/24	25.00 - 27.00	push thru vane Su=321/67 psf	---			181.50	(24.00) Dark grey, wet, SILT, some clay, trace fine sand. #5D combined with 6D for laboratory testing. 55x110 mm vane raw torque readings: V1: 7.2/1.5 ft-lbs V2: 7.0/0.8 ft-lbs	G#266757 A-6, CL WC=29.6% LL=35 PL=20 PI=15
	V2		26.00 - 26.63	Su=313/36 psf						
30	6D	24/20	30.00 - 32.00	WOH/1/1/2	2	3	WASH AHEAD		Dark grey, wet, soft, SILT, some clay, with 1" seam grey, silty, fine sand at top of sample and numerous partings of fine sandy SILT throughout. #6D combined with 5D for laboratory testing.	
35	MU	24/0	35.00 - 37.00						Failed Tube attempt.	
40	1U	24/24	40.00 - 42.00	Piston Sampler					Dark grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V3: 12.0/2.5 ft-lbs V4: 16.0/4.7 ft-lbs	GC#266758 A-4, CL WC=32.9% LL=30 PL=22 PI=8
	V3		42.63 - 43.00	Su=536/112 psf						
	V4		43.63 - 44.00	Su=714/210 psf						
45	MU	24/0	45.00 - 47.00	Piston Sampler					Failed Tube attempt. Dark grey, Clayey SILT on exterior of tube.	
	7D / V5	24/18	47.00 - 49.00	Hydraulic Push Su=527/143 psf	---		26		Dark grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V5: 11.8/3.2 ft-lbs V6: 13.0/3.0 ft-lbs	
	V6		48.00 - 48.63	Su=580/134 psf						
50	2U	24/24	50.00 - 52.00	Piston Sampler			WASH AHEAD		Grey, wet, medium stiff to stiff, Silty CLAY.	
	V7		52.63 - 53.00	Su=759/201 psf					55x110 mm vane raw torque readings: V7: 17.0/4.5 ft-lbs V8: 26.5/6.5 ft-lbs	
	V8		53.63 - 54.00	Su=1183/290 psf						
55	8D / V9	24/24	55.00 - 57.00	Hydraulic Push Su=1295/290 psf	---				Grey, wet, stiff, Silty CLAY, trace fine sand. 55x110 mm vane raw torque readings: V9: 29.0/6.5 ft-lbs V10: 31.0/7.0 ft-lbs	G#266759 A-6, CL WC=34.9% LL=36 PL=22 PI=14
	V10		56.63 - 57.00	Su=1384/313 psf				148.00	(57.50)	
60	9D	24/14	60.00 - 62.00	13/12/17/17	29	37			Brownish grey, wet, dense, Gravelly SAND, little silt, trace clay, (Glacial Till). Changed to NW Casing at 60.0 ft bgs.	G#266760 A-1-b, SC-SM WC=10.1%
65	10D	19.2/10	64.00 - 65.60	9/13/35/20(1.2")	48	60	34		Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, trace clay, (Glacial Till).	G#266761 A-1-b, SC-SM WC=10.9%
							84	139.80	(65.70) Weathered BEDROCK. Washed Ahead to 67.0 ft bgs.	
	R1	60/56	67.70 - 72.70	RQD = 80%			985 NQ-2	137.80	985 blows for 0.7 ft. (67.70) Top of Bedrock at Elev. 137.8 ft. R1:Bedrock: Hard, fresh to slightly weathered, medium to coarse grained, greenish-grey, porphyritic GRANITE with biotite-rich zones (layers) and quartz, microcline, and mafic phenocrysts throughout. Closely to moderately spaced, horizontal to low angle joints; typically undulating, rough, fresh to discolored and open. Rock Mass Quality = Good R1:Core Times (min:sec) 67.7-68.7 ft (2:50) 68.7-69.7 ft (2:20) 69.7-70.7 ft (2:30) 70.7-71.7 ft (2:40) 71.7-72.7 ft (3:15) 93% Recovery R2:Bedrock: Similar to R1, except moderately spaced joints.	
70	R2	60/60	72.70 - 77.70	RQD = 97%						

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 2
Boring No.: BB-ALAR-101

Maine Department of Transportation — Soil/Rock Exploration Log (BB-ALAR-101, Page 2 of 2)

Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine. Boring No.: BB-ALAR-101. WIN: 18335.00. (Header as page 1.)

Depth	Elevation	Visual Description and Remarks
75	127.80	Rock Mass Quality = Excellent. R2:Core Times (min:sec) 72.7-73.7 ft (2:35) 73.7-74.7 ft (2:25) 74.7-75.7 ft (2:55) 75.7-76.7 ft (3:05) 76.7-77.7 ft (3:30) 100% Recovery (77.70) Bottom of Exploration at 77.70 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 2 of 2
Boring No.: BB-ALAR-101

Maine Department of Transportation — Soil/Rock Exploration Log (BB-ALAR-102)

Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine. Boring No.: BB-ALAR-102. WIN: 18335.00

Driller:	MaineDOT	Elevation (ft.)	181.1	Auger ID/OD:	N/A
Operator:	Enos/Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/14,15/13, 3/21/13	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	108+32.4, 6.8 ft Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Page 1 of 2 — Visual Description and Remarks:
- Brown, wet, loose, fine to coarse SAND, trace gravel with 2" thick layer of wood in bottom of sample. (1D, 24/16, 2.00 - 4.00, 3/3/3/5)
- (6.00, Elev. 175.10) Hydraulic Push. Grey, wet, very soft, SILT, some clay, trace fine sand, trace gravel. #2D combined with 3D for laboratory testing. (2D, 24/18, 6.00 - 8.00, WOR/WOR/WOR/WOH) G#266762 A-4, CL-ML WC=20.9% LL=24 PL=20 PI=4
- Grey, wet, very soft, SILT, some clay, trace fine sand, trace gravel. #3D combined with 2D for laboratory testing. 55x110 mm vane raw torque readings: V1: 5.0/0.5 ft-lbs V2: 5.0/1.7 ft-lbs (3D/V1, 24/10, 11.00 - 13.00, Hydraulic Push Su=223/22 psf; V2, 12.00 - 12.63, Su=223/76 psf)
- Failed Tube attempt. (MU, 24/0, 16.00 - 18.00, Piston Sampler)
- 55x110 mm vane raw torque readings: V3: 6.0/2.0 ft-lbs V4: 7.5/2.2 ft-lbs (V3, 18.63 - 19.00, Su=268/89 psf; V4, 19.63 - 20.00, Su=335/98 psf)
- Failed Tube attempt, bottom cap and sample slid out of tube into borehole. (MU, 24/0, 21.00 - 23.00, Piston Sampler)
- Grey, wet, soft to medium stiff, Silty CLAY, trace fine to medium sand. #4D combined with 5D for laboratory testing. 65x130 mm vane raw torque readings V5: 13.0/3.5 ft-lbs (4D/V5, 24/24, 23.00 - 25.00, Hydraulic Push Su=357/95 psf; V6, 24.00 - 24.37, Su=549/143 psf) G#266763 A-6, CL WC=28.8% LL=34 PL=21

Remarks:
1. Bridge Deck 10.5" thick.
2. Mudline (Ground Surface) 23.7 ft below top of Bridge Deck.
3. Split spoon and HW Casing Refusal at 42.5 ft bgs.

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 2
Boring No.: BB-ALAR-102

Maine Department of Transportation — Soil/Rock Exploration Log (BB-ALAR-102, Page 2 of 2)

Visual Description and Remarks:
- Increasing clay content noted at approximately 24.0 ft bgs. V6: 20.0/5.2 ft-lbs. Pushed bottom cap of 21.0 to 23.0 ft sample through sample, cap protruding from bottom tube when piston sampler retrieved. Representative sample jarred and labled 5D. Similar to above, medium stiff. #5D combined with 4D for laboratory testing. 65x130 mm vane raw torque readings: V7: 30.2/3.5 ft-lbs V8: 33.0/4.2 ft-lbs (MU/5D, 24/24, 26.00 - 28.00, Piston Sampler; V7, 28.57 - 29.00, Su=830/96 psf; V8, 29.57 - 30.00, Su=907/115 psf)
- Grey, wet, medium stiff to stiff, Silty CLAY, trace sand. 55x100 mm vane raw torque readings: V9: 22.5/4.0 ft-lbs V10: 22.0/3.0 ft-lbs (1U, 24/24, 31.00 - 33.00, Piston Sampler; V9, 33.63 - 34.00, Su=1005/179 psf; V10, 34.63 - 35.00, Su=982/134 psf)
- (37.50, Elev. 143.60) Failed vane attempt, unable to push past 37.5 ft bgs. Grey, wet, dense, fine to coarse sandy GRAVEL, little silt, trace clay, (Glacial Till). (6D/MV, 24/7, 37.50 - 39.50, 6/10/15/23, 25, 32) G#266764 A-1-a, GC-GM WC=14.6%
- Grey, wet, medium dense, Gravelly, fine to coarse SAND, trace silt, trace clay, with 1 piece of gravel in tip of spoon, (Glacial Till). (7D, 18/8, 41.00 - 42.50, 9/9/6/10(0"), 15, 19) G#266765 A-1-b, SM-SC WC=9.5%
- (42.50, Elev. 138.60) Top of Bedrock at Elev. 138.6 ft. R1:Bedrock: Hard, fresh to slightly weathered, greenish-grey, medium to very coarse-grained, porphyritic GRANITE with quartz, microcline, muscovite and few mafic phenocrysts throughout. Finer grained and abundant biotite from 42.5 to 43.4 ft and 45.4 to 45.7 ft. Closely to moderately spaced, horizontal joints; typically undulating, rough, fresh to discolored and open. Rock Quality Mass = Fair R1:Core Times (min:sec) 42.5-43.5 ft (7:00) 43.5-44.5 ft (8:00) 44.5-45.5 ft (7:15) 45.5-46.5 ft (6:40) 46.5-47.5 ft (6:50) 93% Recovery R2:Bedrock: Similar to R1, except abundant microcline and moderately-spaced joints. Rock Mass Quality = Excellent R2:Core Times (min:sec) 47.5-48.5 ft (6:10) 48.5-49.5 ft (6:15) 49.5-50.5 ft (6:45) 50.5-51.5 ft (7:00) 51.5-52.5 ft (6:30) 100% Recovery (R1, 60/56, 42.50 - 47.50, RQD = 70%; R2, 60/60, 47.50 - 52.50, RQD = 93%)
- (52.50, Elev. 128.60) Bottom of Exploration at 52.50 feet below ground surface.

Remarks:
1. Bridge Deck 10.5" thick.
2. Mudline (Ground Surface) 23.7 ft below top of Bridge Deck.
3. Split spoon and HW Casing Refusal at 42.5 ft bgs.

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 2 of 2
Boring No.: BB-ALAR-102

Maine Department of Transportation — Soil/Rock Exploration Log (BB-ALAR-103)

Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine. Boring No.: BB-ALAR-103. WIN: 18335.00

Driller:	MaineDOT	Elevation (ft.)	206.2	Auger ID/OD:	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	3/22,25,26/13	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	109+64.8, 8.4 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.756		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Page 1 of 2 — Visual Description and Remarks:
- Brown, damp, dense, fine to coarse Sandy GRAVEL, trace silt, (Fill). (1D, 24/17, 2.00 - 4.00, 15/22/15/10, 37, 47)
- Brown, damp, medium dense, fine to medium SAND, trace silt, (Fill). (2D, 24/20, 5.00 - 7.00, 2/7/7/5, 14, 18)
- Brown, damp to moist, very loose, fine to medium SAND, trace to little silt; darker brown and siltier with trace organics in bottom 0.7 ft of sample, (Fill). (3D, 24/20, 10.00 - 12.00, WOH/WOH/1/1, 1, 1)
- (12.00, Elev. 194.20) Brown, moist, very loose, fine to coarse SAND, some silt, trace gravel, trace clay, trace organics. (4D, 24/7, 15.00 - 17.00, WOH/WOH/WOH/1) G#266766 A-4, SC-SM WC=23.5%
- Greyish-brown, wet, loose, fine to medium SAND, little silt, trace clay. (5D, 24/10, 19.00 - 21.00, 4/2/1/2, 3, 4) G#266767 A-2-4, SC-SM WC=30.2%
- Greyish-brown, loose, fine to coarse SAND, trace silt, trace gravel interbedded with wood. (6D, 24/8, 24.00 - 26.00, 2/2/2/2, 4, 5)

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 2
Boring No.: BB-ALAR-103

Maine Department of Transportation — Soil/Rock Exploration Log (BB-ALAR-103, Page 2 of 2)

Visual Description and Remarks:
- (29.00, Elev. 177.20) Grey, wet, very soft, Clayey SILT, trace fine to medium sand. #7D combined with 8D for laboratory testing. (7D, 24/18, 29.00 - 31.00, WOR/WOR/WOR/WOR) #G#266768 A-4, CL WC=30.0% LL=27 PL=19 PI=8
- Failed Tube attempt, (Grey, wet, very soft to soft, Clayey SILT, some fine sand on outside of tube). (MU, 24/0, 34.00 - 36.00, Piston Sampler)
- 65x130 mm vane raw torque readings: V1: 7.0/0.8 ft-lbs V2: 9.2/0.8 ft-lbs (V1, 36.57 - 37.00, Su=192/22 psf; V2, 37.57 - 38.00, Su=253/22 psf)
- Grey, wet, very soft to soft, Clayey SILT, trace fine to medium sand; soupy consistency. Predominately very fine sand in wash water from 39.0-44.0 ft bgs. #8D combined with 7D for laboratory testing. 65x130 mm vane raw torque readings V3: 8.8/0.2 ft-lbs V4: 10.8/0.0 ft-lbs (8D/V3, 24/12, 39.00 - 41.00, Hydraulic Push Su=242/6 psf; V4, 40.00 - 40.57, Su=297/0 psf)
- Grey, wet, soft, Clayey SILT, some fine to medium sand; soupy consistency. Predominately silt and fine sand in wash water from 44.0-49.0 ft bgs. 65x130 mm vane raw torque readings V5: 17.8/5.0 ft-lbs V6: 15.0/5.0 ft-lbs (9D/V5, 24/15, 44.00 - 46.00, Hydraulic Push; V6, 45.00 - 46.00, Su=412/14 psf)
- Grey, wet, soft to medium stiff, Clayey SILT, some fine to medium sand. 65x130 mm vane raw torque readings V7: 12.0/0.2 ft-lbs V8: 22.5/0.5 ft-lbs (10D/V7, 24/18, 49.00 - 51.00, Hydraulic Push Su=330/6 psf; V8, 50.00 - 51.00, Su=618/14 psf)
- Grey, wet, medium stiff, Clayey SILT, trace fine sand. 65x130 mm vane raw torque readings V9: 28.5/1.2 ft-lbs V10: 28.0/1.8 ft-lbs (1U, 24/24, 54.00 - 56.00, Piston Sampler; V9, 56.57 - 57.00, Su=783/33 psf; V10, 57.57 - 58.00, Su=769/49 psf) GC#266769 A-6, CL WC=35.9% LL=35 PL=21 PI=14
- Grey, wet, medium stiff, Silty CLAY, trace fine sand. 65x130 mm vane raw torque readings V11: 30.5/3.0 ft-lbs V12: 34.5/3.8 ft-lbs (11D/V11, 24/24, 59.00 - 61.00, Hydraulic Push Su=1027/89 psf; V12, 60.00 - 60.57, Su=948/104 psf)
- Grey, wet, medium stiff to stiff, Silty CLAY, trace fine sand. 55x110 mm vane raw torque readings V13: 29.0/2.0 ft-lbs V14: 22.2/5.0 ft-lbs (12D/V13, 24/24, 64.00 - 66.00, Hydraulic Push Su=1027/89 psf; V14, 65.00 - 65.63, Su=991/223 psf) G#266770 A-6, CL WC=35.4%
- (68.40, Elev. 137.80) Grey, wet, medium dense, Sandy GRAVEL, trace to little silt, (Glacial Till). Changed to NW Casing at 70.0 ft bgs. (13D, 24/6, 69.00 - 71.00, 45/11/11/17, 22, 28)
- 965 blows for 0.8 ft. NW Casing Refusal at 71.8 ft bgs: roller coned ahead to 72.0 ft and seat NW Casing.
- (71.80, Elev. 134.40) Top of Bedrock at Elev. 134.4 ft. R1:Bedrock: Hard, fresh to slightly weathered, medium to coarse grained, greenish-grey, porphyritic GRANITE with few biotite-rich zones (layers) and quartz, microcline and mafic phenocrysts throughout. Very coarse grained with less biotite and abundant muscovite phenocrysts from 73.7-76.2 ft. Moderately spaced horizontal joints; typically undulating, rough, fresh to discolored and open. Rock Mass Quality = Good R1:Core Times (min:sec) 72.0-73.0 ft (4:05) 73.0-74.0 ft (2:20) 74.0-75.0 ft (2:55) 75.0-76.0 ft (1:55) 76.0-77.0 ft (2:00) 98% Recovery R2:Bedrock: Similar to R1, except very coarse grained below 80.2 ft. Horizontal joints with one high angle joint. Rock Mass Quality = Excellent R2:Core Times (min:sec) 77.0-78.0 ft (2:00) 78.0-79.0 ft (2:05) 79.0-80.0 ft (2:15) 80.0-81.0 ft (2:40) 81.0-82.0 ft (2:55) 100% Recovery. Significant water loss throughout R2 core run. (R1, 60/59, 72.00 - 77.00, RQD = 90%; R2, 60/60, 77.00 - 82.00, RQD = 92%)
- (82.00, Elev. 124.20) Bottom of Exploration at 82.00 feet below ground surface.

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 2 of 2
Boring No.: BB-ALAR-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1833(500)
WIN 18335.00
BRIDGE NO. 2625
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED		
CHECKED-REVIEWED		
DESIGN2-DETAILED2	K.MAGUIRE	T.WHITE	JUL 2013
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

OAKDALE NORTHBOUND 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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Oakdale Northbound Bridge #2625 over Little Androscoggin River. **Location:** Auburn, Maine	**Boring No.:** BB-ALAR-101 **WIN:** 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	205.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/13-3/13/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	107+33.4, 6.6 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	less than 10.0 ft.
Hammer Efficiency Factor:	0.756	**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	204.71		9.5" Pavement — 0.79	
	1D	24/19	1.00 - 3.00	16/19/14/18	33	42				Brown, damp, dense, Gravelly fine to coarse SAND, trace silt, (Fill).	
5	2D	24/8	5.00 - 7.00	5/5/3/2	8	10				Brown, damp to moist, loose, fine to coarse SAND, some gravel, trace to little silt, (Fill).	
10	3D	24/3	10.00 - 12.00	3/1/1/2	2	3	16			Brown, moist, very loose, fine to medium SAND, little gravel, little silt, (Fill).	
							16				
							9				
							5	192.50		13.00	
							5				
15	4D	24/8	15.50 - 17.50	WOH/5/5/2	10	13	10			WOOD, (9" long piece of wood through tip of spoon). Note: Grey, silty fine sand in decomposed wood in spoon; no creosote odor noted.	
							10				
							12				
							18				
							21				
20	MD	24/0	20.00 - 22.00	5/3/2/5	5	6	10			Grab sample: Grey, wet, loose, fine SAND, trace roots/organics.	
							26				
							31				
							20				
							19	181.50		24.00	
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:	Oakdale Northbound Bridge #2625 over Little Androscoggin River.	**Location:**	Auburn, Maine	**Boring No.:**	BB-ALAR-101
				WIN:	18335.00
Driller:	MaineDOT	**Elevation (ft.)**	205.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/13-3/13/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	107+33.4, 6.6 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	less than 10.0 ft.
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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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.
25	5D	24/24	25.00 - 27.00	push thru vane	---		12			Dark grey, wet, soft, SILT, some clay, trace fine sand. *5D combined with 6D for laboratory testing. 55x110 mm vane raw torque readings: V1: 7.2/1.5 ft-lbs V2: 7.0/0.8 ft-lbs	*G#266757 A-6, CL WC=29.6% LL=35 PL=20 PI=15
	V1		25.63 - 26.00	Su=321/67 psf							
	V2		26.63 - 27.00	Su=313/36 psf			16				
							17				
							19				
							26				
30	6D	24/20	30.00 - 32.00	WOH/1/1/2	2	3	WASH AHEAD			Dark grey, wet, soft, SILT, some clay, with 1" seam grey, silty, fine sand at top of sample and numerous partings of fine sandy SILT throughout. *6D combined with 5D for laboratory testing.	
35	MU	24/0	35.00 - 37.00							Failed Tube attempt.	
40	1U	24/24	40.00 - 42.00	Piston Sampler						Dark grey, wet, medium stiff, Clayey SILT, trace fine sand.	GC#266758 A-4, CL WC=32.9% LL=30 PL=22 PI=8
	V3		42.63 - 43.00	Su=536/112 psf						55x110 mm vane raw torque readings: V3: 12.0/2.5 ft-lbs V4: 16.0/4.7 ft-lbs	
	V4		43.63 - 44.00	Su=714/210 psf							
45	MU	24/0	45.00 - 47.00	Piston Sampler						Failed Tube attempt. Dark grey, Clayey SILT on exterior of tube.	
							26				
	7D	24/18	47.00 - 49.00	Hydraulic Push	---		24			Dark grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V5: 11.8/3.2 ft-lbs V6: 13.0/3.0 ft-lbs	
	V5		47.63 - 48.00	Su=527/143 psf							
	V6		48.63 - 49.00	Su=580/134 psf			26				
							22				
50											

Remarks:

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* 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:** Oakdale Northbound Bridge #2625 over Little Androscoggin River. **Location:** Auburn, Maine	**Boring No.:** BB-ALAR-101 **WIN:** 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	205.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/13-3/13/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	107+33.4, 6.6 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	less than 10.0 ft.
Hammer Efficiency Factor:	0.756	**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.
50	2U	24/24	50.00 - 52.00	Piston Sampler			WASH AHEAD			Grey, wet, medium stiff to stiff, Silty CLAY.	
	V7		52.63 - 53.00	Su=759/201 psf						55x110 mm vane raw torque readings: V7: 17.0/4.5 ft-lbs V8: 26.5/6.5 ft-lbs	
	V8		53.63 - 54.00	Su=1183/290 psf							
55	8D V9	24/24	55.00 - 57.00 55.63 - 56.00	Hydraulic Push Su=1295/290 psf	---					Grey, wet, stiff, Silty CLAY, trace fine sand. 55x110 mm vane raw torque readings: V9: 29.0/6.5 ft-lbs V10: 31.0/7.0 ft-lbs	G#266759 A-6, CL WC=34.9% LL=36 PL=22 PI=14
	V10		56.63 - 57.00	Su=1384/313 psf							
								148.00		57.50	
60	9D	24/14	60.00 - 62.00	13/12/17/17	29	37				Brownish grey, wet, dense, Gravelly SAND, little silt, trace clay, (Glacial Till). Changed to NW Casing at 60.0 ft bgs.	G#266760 A-1-b, SC-SM WC=10.1%
							37				
							39				
	10D	19.2/10	64.00 - 65.60	9/13/35/20(1.2")	48	60	34			Grey, wet, very dense, fine to coarse SAND, some gravel, little silt, trace clay, (Glacial Till).	G#266761 A-1-b, SC-SM WC=10.9%
65							84	139.80		65.70 Weathered BEDROCK. Washed Ahead to 67.0 ft bgs..	
	R1	60/56	67.70 - 72.70	RQD = 80%			a85 NQ-2	137.80		a85 blows for 0.7 ft. 67.70 Top of Bedrock at Elev. 137.8 ft. R1:Bedrock: Hard, fresh to slightly weathered, medium to coarse grained, greenish-grey porphyrite GRANITE with biotite-rich zones (layers) and quartz, microcline, and mafic pherocrysts throughout. Closely to moderately spaced, horizontal to low angle joints; typically undulating, rough, fresh to discolored and open. Rock Mass Quality = Good R1:Core Times (min:sec) 67.7-68.7 ft (2:50) 68.7-69.7 ft (2:20) 69.7-70.7 ft (2:30) 70.7-71.7 ft (2:40) 71.7-72.7 ft (3:15) 93% Recovery	
70											
	R2	60/60	72.70 - 77.70	RQD = 97%						R2:Bedrock: Similar to R1, except moderately spaced joints. Rock Mass Quality = Excellent	
75											

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:** Oakdale Northbound Bridge #2625 over Little Androscoggin River. **Location:** Auburn, Maine	**Boring No.:** BB-ALAR-101 **WIN:** 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	205.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/11/13-3/13/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	107+33.4, 6.6 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	less than 10.0 ft.
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (psf)	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (psf)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer	WOH = weight of 140lb. hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	N_{60} = SPT N-uncorrected corrected for hammer efficiency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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.
75								127.80		R2:Core Times (min:sec) 72.7-73.7 ft (2:35) 73.7-74.7 ft (2:25) 74.7-75.7 ft (2:30) 75.7-76.7 ft (3:05) 76.7-77.7 ft (3:30) 100% Recovery 77.70 **Bottom of Exploration at 77.70 feet below ground surface.**	
80											
85											
90											
95											
100											

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: Oakdale Northbound Bridge #2625 over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: BB-ALAR-102
WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	181.1	**Auger ID/OD:**	N/A
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/14,15/13, 3/21/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+32.4, 6.8 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**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							PUSH				
	1D	24/16	2.00 - 4.00	3/3/3/5	6	8	12			Brown, wet, loose, fine to coarse SAND, trace gravel with 2" thick layer of wood in bottom of sample.	
							17				
							15				
5							19				
	2D	24/18	6.00 - 8.00	WOR/WOH/WOH/ WOH	---		aHYD PUSH	175.10		6.00 aHydraulic Push Grey, wet, very soft, SILT, some clay, trace fine sand, trace gravel. *2D combined with 3D for laboratory testing.	*G#266762 A-4, CL-ML WC=20.9% LL=24 PL=20 PI=4
10											
	3D	24/10	11.00 - 13.00	Hydraulic Push	---					Grey, wet, very soft, SILT, some clay, trace fine sand, trace gravel. *3D combined with 2D for laboratory testing. 55x110 mm vane raw torque readings: V1: 5.0/0.5 ft-lbs V2: 5.0/1.7 ft-lbs	
	V1		11.63 - 12.00	Su=223/22 psf							
	V2		12.63 - 13.00	Su=223/76 psf							
15											
	MU	24/0	16.00 - 18.00	Piston Sampler						Failed Tube attempt.	
	V3		18.63 - 19.00	Su=268/89 psf						55x110 mm vane raw torque readings: V3: 6.0/2.0 ft-lbs V4: 7.5/2.2 ft-lbs	
	V4		19.63 - 20.00	Su=335/98 psf							
20											
	MU	24/0	21.00 - 23.00	Piston Sampler						Failed Tube attempt, bottom cap and sample slid out of tube into borehole.	
	4D	24/24	23.00 - 25.00	Hydraulic Push	---					Grey, wet, soft to medium stiff, Silty CLAY, trace fine to medium sand. *4D combined with 5D for laboratory testing. 65x130 mm vane raw torque readings:	*G#266763 A-6, CL WC=28.8% LL=34
	V5		23.57 - 24.00	Su=357/96 psf							
25	V6		24.57 - 25.00	Su=549/143 psf							

Remarks:

1. Bridge Deck 10½" thick.
2. Mudline (Ground Surface) 23.7 ft below top of Bridge Deck.
3. Split spoon and HW Casing Refusal at 42.5 ft bgs.

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: Oakdale Northbound Bridge #2625 over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: BB-ALAR-102
WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	181.1	**Auger ID/OD:**	N/A
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/14,15/13, 3/21/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+32.4, 6.8 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**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										V5: 13.0/3.5 ft-lbs Increasing clay content noted at approxmately 24.0 ft bgs. V6: 20.0/5.2 ft-lbs	PL=21 PI=13
	MU/5D	24/24	26.00 - 28.00	Piston Sampler						Pushed bottom cap of 21.0 to 23.0 ft sample through sample, cap protruding from bottom tube when piston sampler retrieved. Representative sample jarred and labled 5D. Similar to above, medium stiff. *5D combined with 4D for laboratory testing. 65x130 mm vane raw torque readings:	
	V7		28.57 - 29.00	Su=830/96 psf						V7: 30.2/3.5 ft-lbs	
	V8		29.57 - 30.00	Su=907/115 psf						V8: 33.0/4.2 ft-lbs	
30											
	1U	24/24	31.00 - 33.00	Piston Sampler						Grey, wet, medium stiff to stiff, Silty CLAY, trace sand.	
	V9		33.63 - 34.00	Su=1005/179 psf						55x100 mm vane raw torque readings: V9: 22.5/4.0 ft-lbs	
	V10		34.63 - 35.00	Su=982/134 psf						V10: 22.0/3.0 ft-lbs	
35											
	6D/MV	24/7	37.50 - 39.50	6/10/15/23	25	32		143.60		37.50 Failed vane attempt, unable to push past 37.5 ft bgs. Grey, wet, dense, fine to coarse sandy GRAVEL, little silt, trace clay, (Glacial Till).	G#266764 A-1-b, GC-GM WC=14.6%
							52				
							72				
40							67				
	7D	18/8	41.00 - 42.50	9/9/6/10(0")	15	19	49			Grey, wet, medium dense, Gravelly, fine to coarse SAND, trace silt, trace clay, with 1 piece of gravel in tip of spoon, (Glacial Till).	G#266765 A-1-a, SW-SC WC=9.5%
	R1	60/56	42.50 - 47.50	RQD = 70%			NQ-2	138.60		42.50 Top of Bedrock at Elev. 138.6 ft. R1:Bedrock: Hard, fresh to slightly weathered, greenish-grey, medium to very coarse-grained, porphyrite GRANITE with quartz, microcline, muscovite and few mafic pherocrysts throughout. Finer grained and abundant biotite from 42.5 to 43.4 ft and 45.4 to 45.7 ft. Closely to moderately spaced, horizontal joints; typically undulating, rough, fresh to discolored and open. Rock Quality Mass = Fair R1:Core Times (min:sec) 42.5-43.5 ft (7:00)	
45										43.5-44.5 ft (9:00) 44.5-45.5 ft (7:15)	
	R2	60/60	47.50 - 52.50	RQD = 93%						45.5-46.5 ft (6:40) 46.5-47.5 ft (6:50) 93% Recovery R2:Bedrock: Similar to R1, except abundant microcline and	
50											

Remarks:

1. Bridge Deck 10½" thick.
2. Mudline (Ground Surface) 23.7 ft below top of Bridge Deck.
3. Split spoon and HW Casing Refusal at 42.5 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project:	Oakdale Northbound Bridge #2625 over Little Androscoggin River.	**Location:**	Auburn, Maine	**Boring No.:**	BB-ALAR-102
				WIN:	18335.00
Driller:	MaineDOT	**Elevation (ft.)**	181.1	**Auger ID/OD:**	N/A
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/14,15/13, 3/21/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	108+32.4, 6.8 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**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.
50										moderately-spaced joints. Rock Mass Quality = Excellent R2:Core Times (min:sec) 47.5-48.5 ft (5:10) 48.5-49.5 ft (6:15)	
								128.60		49.5-50.5 ft (6:45) 50.5-51.5 ft (7:00) 51.5-52.5 ft (6:30) 100% Recovery	
										52.50 **Bottom of Exploration at 52.50 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

1. Bridge Deck 10½" thick.
2. Mudline (Ground Surface) 23.7 ft below top of Bridge Deck.
3. Split spoon and HW Casing Refusal at 42.5 ft bgs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River.
Location: Auburn, Maine

Boring No.: BB-ALAR-103
WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	206.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/22,25,26/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	109+64.8, 8.4 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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)
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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				
	1D	24/17	2.00 - 4.00	15/22/15/10	37	47				Brown, damp, dense, fine to coarse Sandy GRAVEL, trace silt, (Fill).	
5	2D	24/20	5.00 - 7.00	2/7/7/5	14	18				Brown, damp, medium dense, fine to medium SAND, trace silt, (Fill).	
10	3D	24/20	10.00 - 12.00	WOH/WOH/1/1	1	1				Brown, damp to moist, very loose, fine to medium SAND, trace to little silt; darker brown and siltier with trace organics in bottom 0.7 ft of sample, (Fill).	
								194.20		12.00	
15	4D	24/7	15.00 - 17.00	WOH/WOH/WOH/1	---		12			Brown, moist, very loose, fine to coarse SAND, some silt, trace gravel, trace clay, trace organics.	G#266766 A-4, SC-SM WC=23.5%
							10				
							14				
							32				
	5D	24/10	19.00 - 21.00	4/2/1/2	3	4	36			Greyish-brown, wet, very loose, fine to medium SAND, little silt, trace clay.	G#266767 A-2-4, SC-SM WC=30.2%
20							39				
							47				
							56				
							73				
25	6D	24/8	24.00 - 26.00	2/2/2/2	4	5	53			Greyish-brown, loose, fine to coarse SAND, trace silt, trace gravel interbedded with wood.	

Remarks:

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine	Boring No.: BB-ALAR-103 WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	206.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/22,25,26/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	109+64.8, 8.4 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
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S_u = Insitu Field Vane 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							54				
							55				
							52				
							49	177.20		29.00	
	7D	24/18	29.00 - 31.00	WOR/WOR/WOR/ WOR	---		33			Grey, wet, very soft, Clayey SILT, trace fine to medium sand. *7D combined with 8D for laboratory testing.	*G#266768 A-4, CL WC=30.0% LL=27 PL=19 PI=8
30							30				
							30				
							26				
							24				
	MU	24/0	34.00 - 36.00	Piston Sampler			25			Failed Tube attempt. (Grey, wet, very soft to soft, Clayey SILT, some fine sand on outside of tube).	
35							23				
	V1		36.57 - 37.00	Su=192/22 psf			20			65x130 mm vane raw torque readings: V1: 7.0/0.8 ft-lbs	
	V2		37.57 - 38.00	Su=253/22 psf			15			V2: 9.2/0.8 ft-lbs	
							15				
	8D	24/12	39.00 - 41.00	Hydraulic Push	---		11			Grey, wet, very soft to soft, Clayey SILT, trace fine to medium sand; soupy consistency. Predominately very fine sand in wash water from 39.0-44.0 ft bgs. *8D combined with 7D for laboratory testing.	
40	V3		39.57 - 40.00	Su=242/6 psf						65x130 mm vane raw torque readings: V3: 8.8/0.2 ft-lbs	
	V4		40.57 - 41.00	Su=297/0 psf			33			V4: 10.8/0.0 ft-lbs	
							37				
							24				
							26				
	9D	24/15	44.00 - 46.00	Hydraulic Push	---		18			Grey, wet, soft, Clayey SILT, some fine to medium sand; soupy consistency. Predominately silt and fine sand in wash water from 44.0-49.0 ft bgs.	
45	V5		44.57 - 45.00	Su=489/0 psf						65x130 mm vane raw torque readings: V5: 17.8/0.0 ft-lbs	
	V6		45.57 - 46.00	Su=412/14 psf			20			V6: 15.0/0.5 ft-lbs	
							15				
							17				
							19				
	10D	24/18	49.00 - 51.00	Hydraulic Push	---		25			Grey, wet, soft to medium stiff, Clayey SILT, some fine to medium sand.	
50											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine	Boring No.: BB-ALAR-103 WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	206.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/22,25,26/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	109+64.8, 8.4 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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)
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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.
50	V7 V8		49.57 - 50.00 50.57 - 51.00	Su=330/6 psf Su=618/14 psf			23			65x130 mm vane raw torque readings: V7: 12.0/0.2 ft-lbs V8: 22.5/0.5 ft-lbs	
							20				
							20				
							20				
	1U	24/24	54.00 - 56.00	Piston Sampler			33			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	GC#266769 A-6, CL WC=35.9% LL=35 PL=21 PI=14
55							32				
	V9		56.57 - 57.00	Su=783/33 psf			31			65x130 mm vane raw torque readings: V9: 28.5/1.2 ft-lbs	
	V10		57.57 - 58.00	Su=769/49 psf			29			V10: 28.0/1.8 ft-lbs	
							32				
	11D	24/24	59.00 - 61.00	Hydraulic Push	---		34			Grey, wet, medium stiff, Silty CLAY, trace fine sand. 65x130 mm vane raw torque readings:	
60	V11 V12		59.57 - 60.00 60.57 - 61.00	Su=838/82 psf Su=948/104 psf			28			V11: 30.5/3.0 ft-lbs V12: 34.5/3.8 ft-lbs	
							28				
							28				
							36				
	12D	24/24	64.00 - 66.00	Hydraulic Push	---		WASH AHEAD			Grey, wet, medium stiff to stiff, Silty CLAY, trace fine sand. 55x110 mm vane raw torque readings:	G#266770 A-6, CL WC=35.4%
65	V13 V14		64.63 - 65.00 65.63 - 66.00	Su=1027/89 psf Su=991/223 psf						V13: 23.0/2.0 ft-lbs V14: 22.2/5.0 ft-lbs	
								137.80		68.40	
	13D	24/6	69.00 - 71.00	45/11/11/17	22	28	39			Grey, wet, medium dense, Sandy GRAVEL, trace to little silt, (Glacial Till) Changed to NW Casing at 70.0 ft bgs.	
70							43				
							a65	134.40		a65 blows for 0.8 ft. NW Casing Refusal at 71.8 ft bgs; roller coned ahead to 72.0 ft and seat NW Casing.	
	R1	60/59	72.00 - 77.00	RQD = 90%			NQ-2			71.80 Top of Bedrock at Elev. 134.4 ft. R1:Bedrock: Hard, fresh to slightly weathered, medium to coarse grained, greenish-grey, porphyritic GRANITE with few biotite-rich zones (layers) and quartz, microcline and mafic pherocrysts throughout. Very coarse grained with less biotite and abundant muscovite	
75											

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: Oakdale Northbound Bridge #2625 over Little Androscoggin River. Location: Auburn, Maine	Boring No.: BB-ALAR-103 WIN: 18335.00

Driller:	MaineDOT	**Elevation (ft.)**	206.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Enos/Wilder/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	3/22,25,26/13	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	109+64.8, 8.4 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.756	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

D = Split Spoon Sample	R = Rock Core Sample	S_u = Insitu Field Vane Shear Strength (psf)	$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
MD = Unsuccessful Split Spoon Sample attempt	SSA = Solid Stem Auger	T_v = Pocket Torvane Shear Strength (psf)	WC = water content, percent
U = Thin Wall Tube Sample	HSA = Hollow Stem Auger	q_p = Unconfined Compressive Strength (ksf)	LL = Liquid Limit
MU = Unsuccessful Thin Wall Tube Sample attempt	RC = Roller Cone	N-uncorrected = Raw field SPT N-value	PL = Plastic Limit
V = Insitu Vane Shear Test, PP = Pocket Penetrometer	WOH = weight of 140lb. hammer	Hammer Efficiency Factor = Annual Calibration Value	PI = Plasticity Index
MV = Unsuccessful Insitu Vane Shear Test attempt	WOR/C = weight of rods or casing	N_{60} = SPT N-uncorrected corrected for hammer efficiency	G = Grain Size Analysis
	WO1P = Weight of one person	N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected	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.
75										phenocrysts from 73.7-76.2 ft. Moderately spaced horizontal joints; typically undulating, rough, fresh to discolored and open. Rock Mass Quality = Good R1:Core Times (min:sec) 72.0-73.0 ft (4:05) 73.0-74.0 ft (2:20) 74.0-75.0 ft (2:05) 75.0-76.0 ft (1:55) 76.0-77.0 ft (2:00) 98% Recovery	
	R2	60/60	77.00 - 82.00	RQD = 92%						R2:Bedrock: Similar to R1, except very coarse grained below 80.2 ft, Horizontal joints with one high angle joint. Rock Mass Quality = Excellent R2:Core Times (min:sec) 77.0-78.0 ft (2:00) 78.0-79.0 ft (2:20) 79.0-80.0 ft (2:15) 80.0-81.0 ft (2:40) 81.0-82.0 ft (2:55) 100% Recovery Significant water loss throughout R2 core run.	
80											
								124.20		82.00	
										Bottom of Exploration at 82.00 feet below ground surface.	
85											
90											
95											
100											

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: 18335.00

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, 5D	107+33.4	6.6 Rt.	25.0-27.0	*266757	1	29.6	35	15	CL	A-6	III
BB-ALAR-101, 6D	107+33.4	6.6 Rt.	30.0-32.0	*266757	---						
BB-ALAR-101, 1U	107+33.4	6.6 Rt.	40.0-42.0	266758	1	32.9	30	8	CL	A-4	IV
BB-ALAR-101, 8D	107+33.4	6.6 Rt.	55.0-57.0	266759	1	34.9	36	14	CL	A-6	III
BB-ALAR-101, 9D	107+33.4	6.6 Rt.	60.0-62.0	266760	1	10.1			SC-SM	A-1-b	II
BB-ALAR-101, 10D	107+33.4	6.6 Rt.	64.0-65.6	266761	1	10.9			SC-SM	A-1-b	II
BB-ALAR-102, 2D	108+32.4	6.8 Rt.	6.0-8.0	*266762	2	20.9	24	4	CL-ML	A-4	IV
BB-ALAR-102, 3D	108+32.4	6.8 Rt.	11.0-13.0	*266762	---						
BB-ALAR-102, 4D	108+32.4	6.8 Rt.	23.0-25.0	*266763	2	28.8	34	13	CL	A-6	III
BB-ALAR-102, 5D	108+32.4	6.8 Rt.	26.0-28.0	*266763	---						
BB-ALAR-102, 6D	108+32.4	6.8 Rt.	37.5-39.5	266764	2	14.6			GC-GM	A-1-b	II
BB-ALAR-102, 7D	108+32.4	6.8 Rt.	41.0-42.5	266765	2	9.5			SW-SC	A-1-a	0
BB-ALAR-103, 4D	109+64.8	8.4 Lt.	15.0-17.0	266766	3	23.5			SC-SM	A-4	III
BB-ALAR-103, 5D	109+64.8	8.4 Lt.	19.0-21.0	266767	3	30.2			SC-SM	A-2-4	II
BB-ALAR-103, 7D	109+64.8	8.4 Lt.	29.0-31.0	*266768	3	30.0	27	8	CL	A-4	IV
BB-ALAR-103, 8D	109+64.8	8.4 Lt.	39.0-41.0	*266768	---						
BB-ALAR-103, 1U	109+64.8	8.4 Lt.	54.0-56.0	266769	3	35.9	35	14	CL	A-6	III
BB-ALAR-103, 12D	109+64.8	8.4 Lt.	64.0-66.0	266770	3	35.4			CL	A-6	III

*Combined samples with same Reference Numbers.

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.

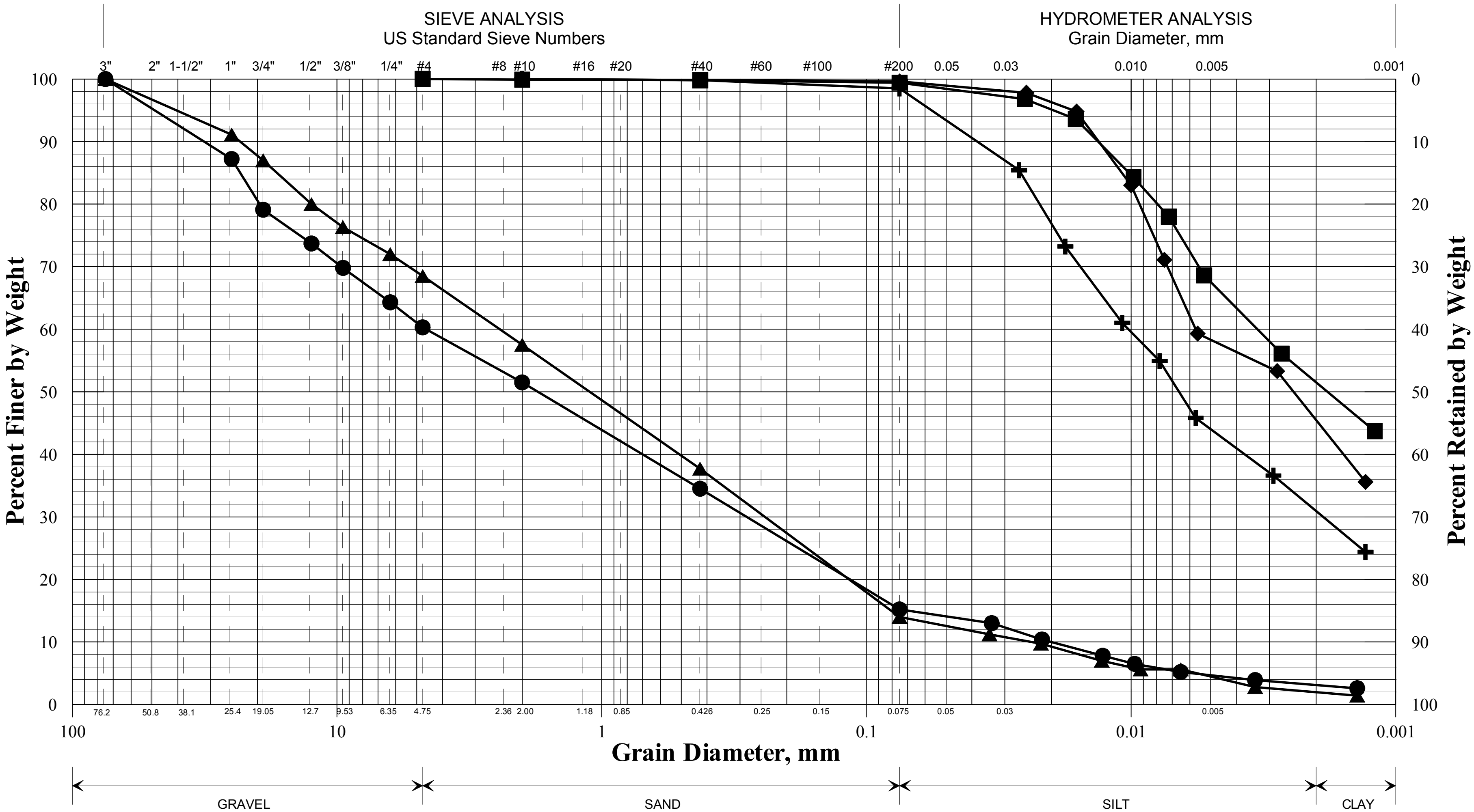
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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 NP = Non Plastic

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



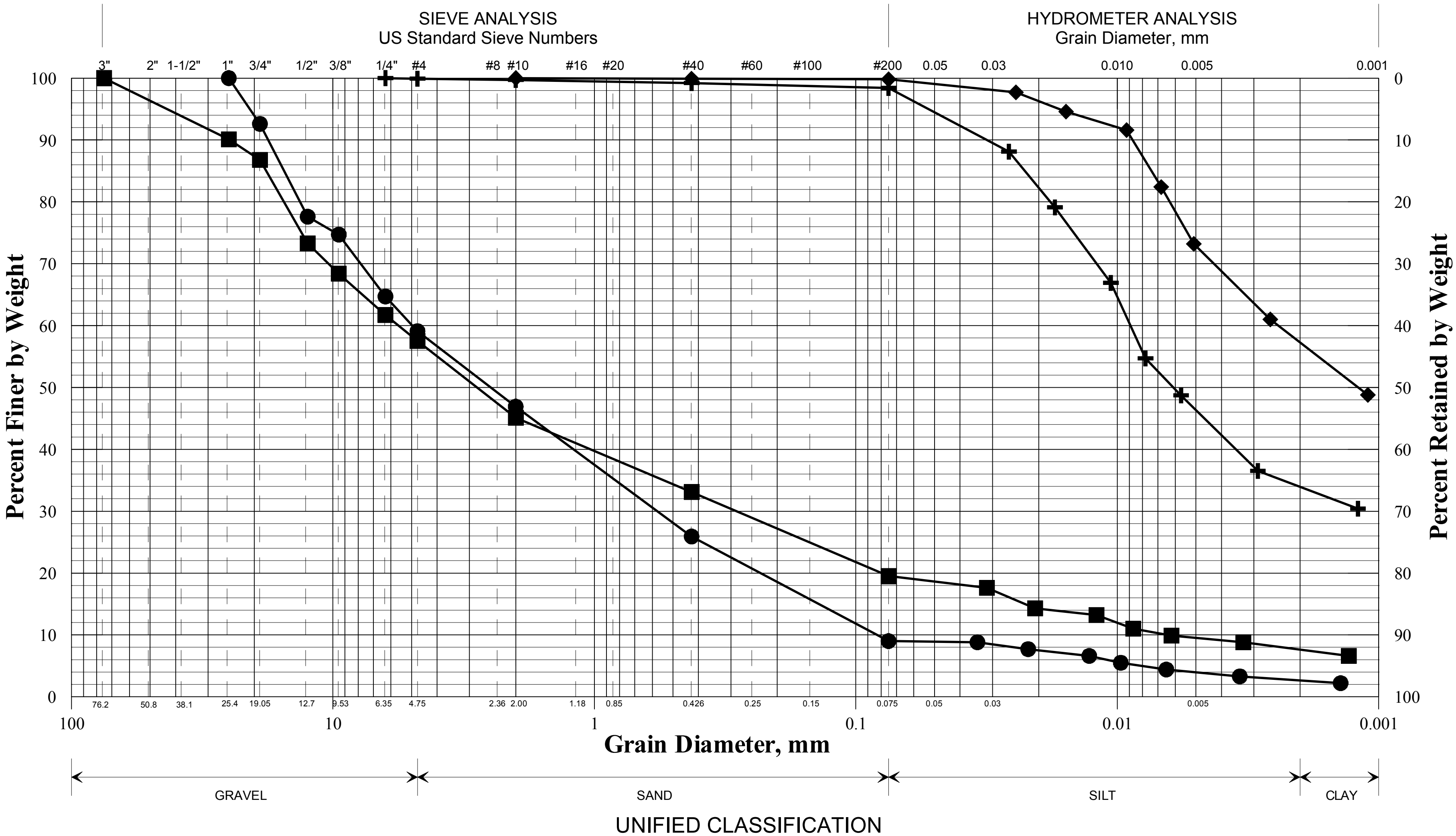
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ALAR-101/5D & 6D	107+33.4	6.6 RT	25-27/30-32	SILT, some clay, trace sand.	29.6	35	20	15
◆	BB-ALAR-101/1U	107+33.4	6.6 RT	40.0-42.0	Clayey SILT, trace sand.	32.9	30	22	8
■	BB-ALAR-101/8D	107+33.4	6.6 RT	55.0-57.0	Silty CLAY, trace sand.	34.9	36	22	14
●	BB-ALAR-101/9D	107+33.4	6.6 RT	60.0-62.0	Gravelly SAND, little silt, trace clay.	10.1			
▲	BB-ALAR-101/10D	107+33.4	6.6 RT	64.0-65.6	SAND, some gravel, little silt, trace clay.	10.9			
✖									

WIN
018335.00
Town
Auburn
Reported by/Date
WHITE, TERRY A 4/30/2013

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

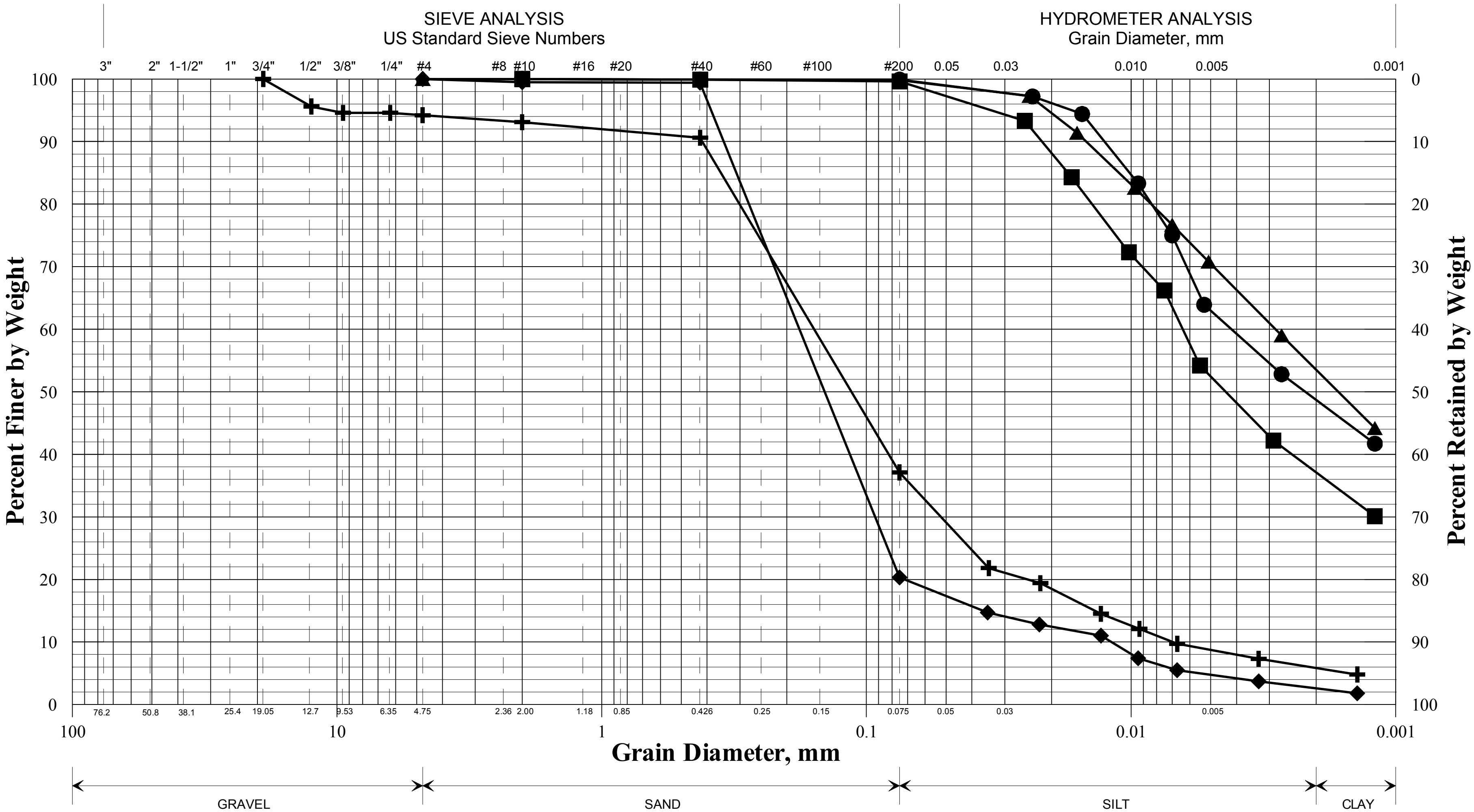


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ALAR-102/2D & 3D	108+32.4	6.8 RT	6-8/11-13	SILT, some clay, trace sand, trace gravel.	20.9	24	20	4
◆	BB-ALAR-102/4D & 5D	108+32.4	6.8 RT	23-25/26-28	Silty CLAY, trace sand.	28.8	34	21	13
■	BB-ALAR-102/6D	108+32.4	6.8 RT	37.5-39.5	Sandy GRAVEL, little silt, trace clay.	14.6			
●	BB-ALAR-102/7D	108+32.4	6.8 RT	41.0-42.5	Gravelly SAND, trace silt, trace clay.	9.5			
▲									
✖									

WIN
018335.00
Town
Auburn
Reported by/Date
WHITE, TERRY A 4/25/2013

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



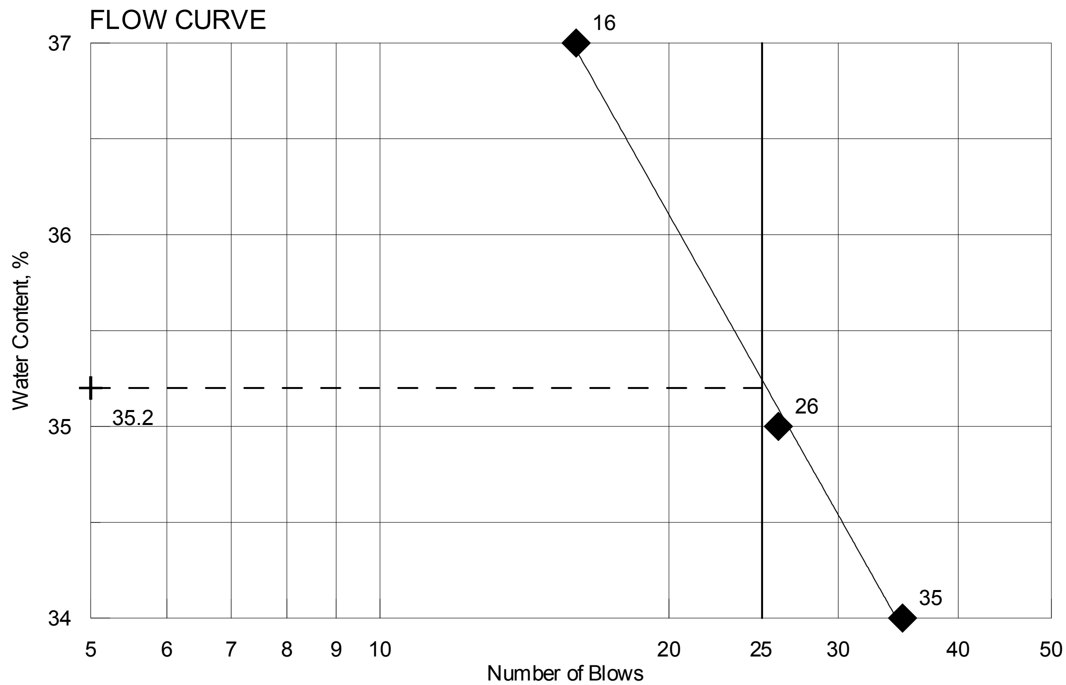
UNIFIED CLASSIFICATION

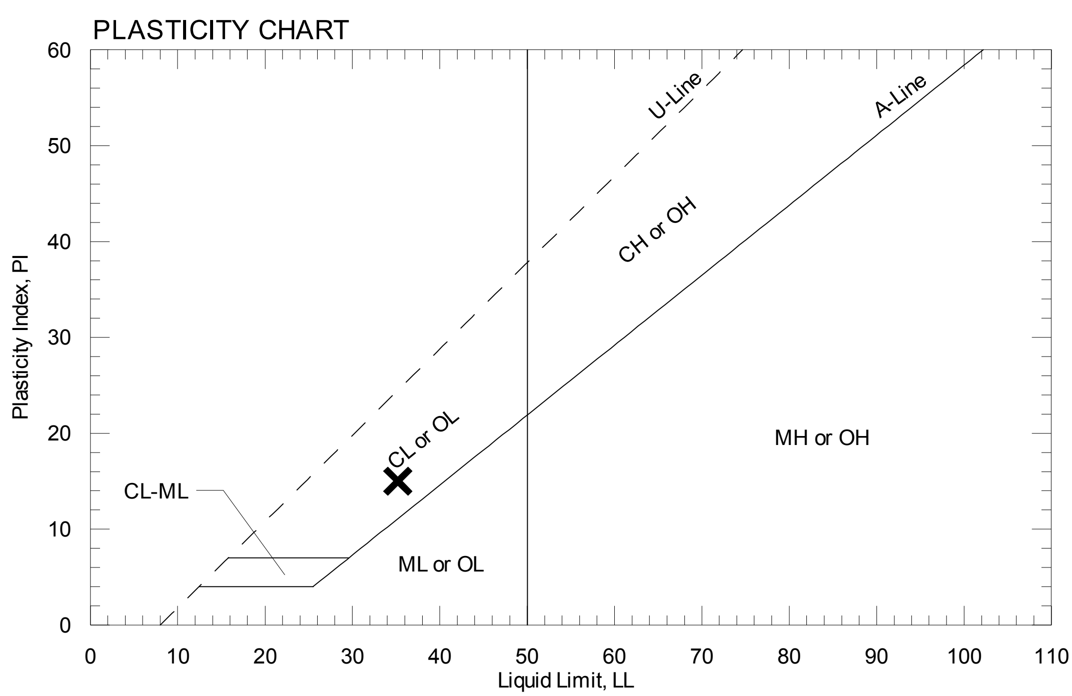
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ALAR-103/4D	109+64.8	8.4 LT	15.0-17.0	SAND, some silt, trace gravel, trace clay.	23.5			
◆	BB-ALAR-103/5D	109+64.8	8.4 LT	19.0-21.0	SAND, little silt, trace clay.	30.2			
■	BB-ALAR-103/7D & 8D	109+64.8	8.4 LT	29-31/39-41	Clayey SILT, trace sand.	30.0	27	19	8
●	BB-ALAR-103/1U	109+64.8	8.4 LT	54.0-56.0	Clayey SILT, trace sand.	35.9	35	21	14
▲	BB-ALAR-103/12D	109+64.8	8.4 LT	64.0-66.0	Silty CLAY, trace sand.	35.4			
✖									

WIN
018335.00
Town
Auburn
Reported by/Date
WHITE, TERRY A 4/30/2013

SHEET 3

TOWN	Auburn	Reference No.	266757
WIN	018335.00	Water Content, %	29.6
Sampled	3/11/2013	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-ALAR-101/5D & 6D	Plastic Limit (T 90), %	20
Station	107+33.4	Plasticity Index (T 90), %	15
Depth	25-27/30-32	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
266758	BB-ALAR-101/1U	GEOTECHNICAL (UNDISTURBED)	3/11/2013	4/11/2013

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **107+33.4** Offset, ft: **6.6** **RT** Dbfg, ft: **40.0-42.0**

WIN/Town **018335.00 - AUBURN** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.8
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.6
[0.0249 mm]	97.8
[0.0161 mm]	94.8
[0.0100 mm]	83.0
[0.0075 mm]	71.1
[0.0056 mm]	59.3
[0.0028 mm]	53.3
[0.0013 mm]	35.6

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, % **34.0**

	Initial	Final		Void Ratio	% Strain
Water Content, %	34.6	23.9	Pmin	0.35 tsf	
Dry Density, lbs/ft³	86.2	101.1	Pp	1.5 tsf	
Void Ratio	0.91	0.63	Pmax	3.1 tsf	
Saturation, %	100.3	100	Cc/C'c	0.2138	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	30
Plastic Limit (T 90), %	22
Plasticity Index (T 90), %	8
Specific Gravity, Corrected to 20°C (T 100)	2.64
Loss on Ignition (T 267) — Loss, % / H2O, %	
Water Content (T 265), %	32.9

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-0.5	0.2	0.02	0.23	0.02	33.3	Alternating layers of light to dark gray silty clay, silt line at 1 1/4".
0.63-1.0	0.22	0.01	0.24	0.02	33.9	Alternating layers of light to dark gray silty clay, silt line at 9".
1.0-1.5	0.22	0.01	0.24	0.03	32.7	Alternating layers of light to dark gray silty clay
1.5-2.0	0.24	0.02	0.23	0.02	32.5	Alternating layers of light to dark gray silty clay.

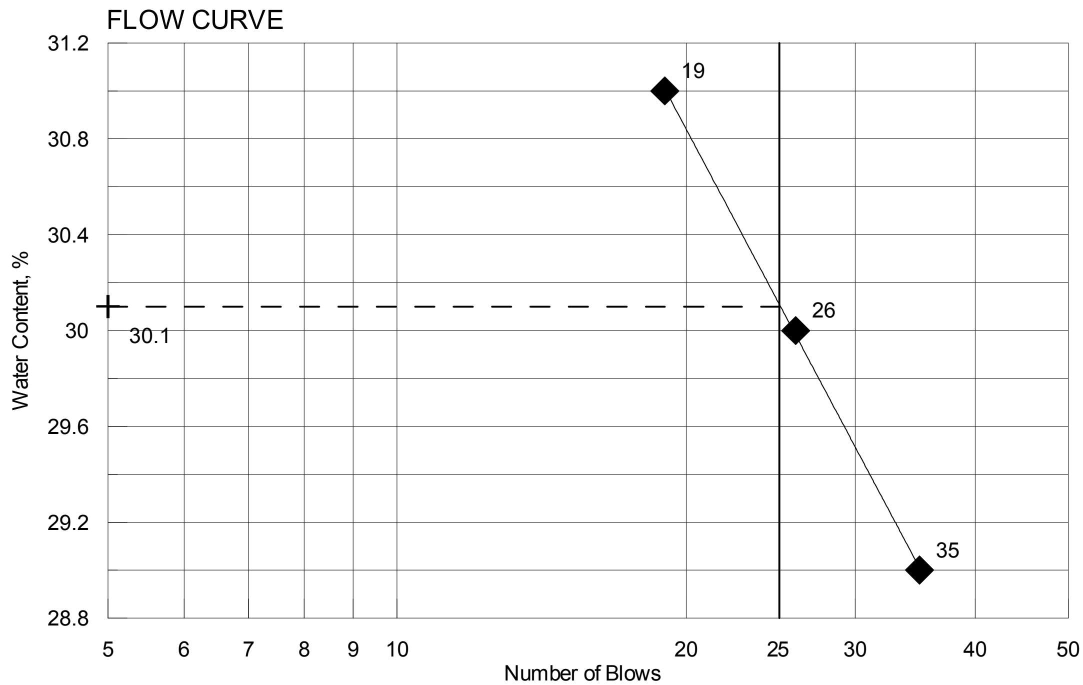
Comments:

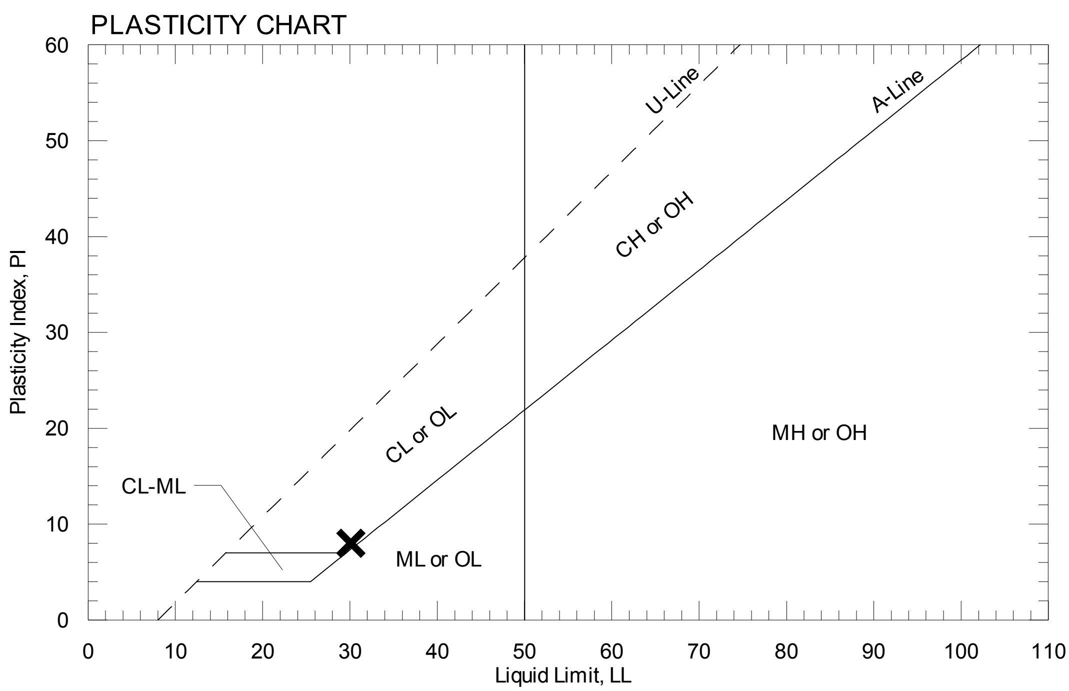
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **4/25/2013**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	266758
WIN	018335.00	Water Content, %	32.9
Sampled	3/11/2013	Liquid Limit @ 25 blows (T 89), %	30
Boring No./Sample No.	BB-ALAR-101/1U	Plastic Limit (T 90), %	22
Station	107+33.4	Plasticity Index (T 90), %	8
Depth	40.0-42.0	Tested By	BBURR





CONSOLIDATION TEST DATA

Project:	Location: AUBURN	Project No.: 18335.00
Boring No.: BB-ALAR-101	Tested By: G LIDSTONE	Checked By:
Sample No.: 1U	Test Date: 4/17/13	Depth: 40-42FT
Test No.: 266758	Sample Type: Shelby Tube	Elevation: ---

Soil Description: SILT
Remarks:

Specific Gravity: 2.64	Liquid Limit: 30	Initial Height: 1.03 in
Initial Void Ratio: 0.91	Plastic Limit: 22	Specimen Diameter: 2.48 in
Final Void Ratio: 0.63	Plasticity Index: 8	

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	121	RING	RING	150
Wt. Container + Wet Soil, gm	216.28	414.41	402.25	210.66
Wt. Container + Dry Soil, gm	178.23	375.24	375.24	183.68
Wt. Container, gm	66.45	262.06	262.06	70.65
Wt. Dry Soil, gm	111.78	113.18	113.18	113.03
Water Content, %	34.04	34.61	23.87	23.87
Void Ratio	---	0.91	0.63	---
Degree of Saturation, %	---	100.26	100.07	---
Dry Unit Weight, pcf	---	86.224	101.13	---

CONSOLIDATION TEST DATA

Project: OAKDALE NB
Boring No.: BB-ALAR-101
Sample No.: 1U
Test No.: 266758

Location: AUBURN
Tested By: G LIDSTONE
Test Date: 4/17/13
Sample Type: Shelby Tube

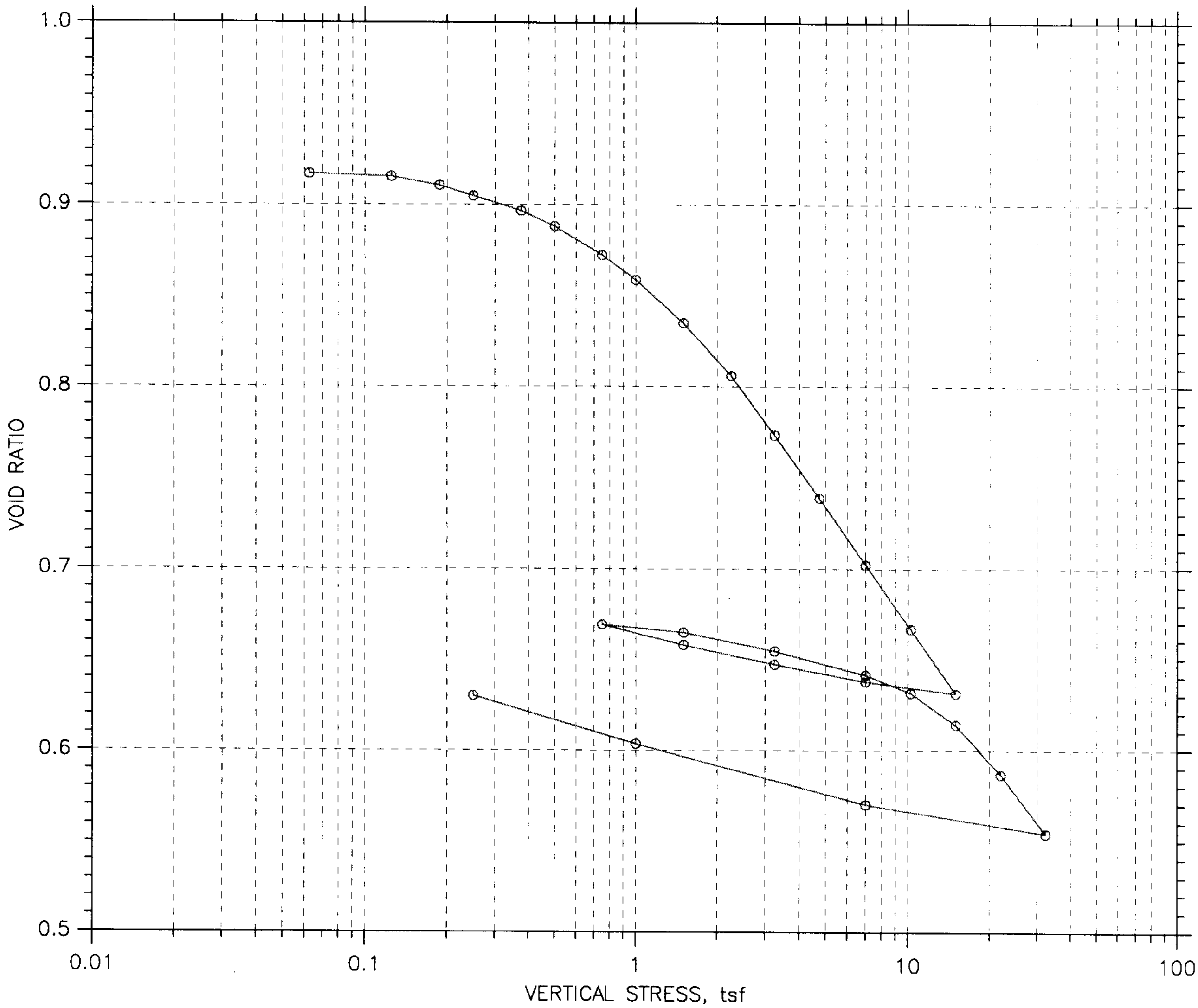
Project No.: 18335.00
Checked By:
Depth: 40-42FT
Elevation: ---

Soil Description: Clayey SILT
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	-0.002938	0.918	-0.28	0.1	0.0	1.05e-004	0.00e+000	1.05e-004
2	0.125	-0.002053	0.916	-0.20	4.7	0.0	1.29e-006	0.00e+000	1.29e-006
3	0.188	0.0005123	0.911	0.05	2.8	2.9	2.16e-006	2.10e-006	2.13e-006
4	0.25	0.003648	0.906	0.35	16.4	0.0	3.70e-007	0.00e+000	3.70e-007
5	0.375	0.008048	0.897	0.78	4.7	4.3	1.27e-006	1.41e-006	1.34e-006
6	0.5	0.01267	0.889	1.23	8.0	6.2	7.42e-007	9.63e-007	8.38e-007
7	0.75	0.02111	0.873	2.05	4.5	3.7	1.30e-006	1.57e-006	1.42e-006
8	1	0.02844	0.860	2.76	6.9	0.0	8.37e-007	0.00e+000	8.37e-007
9	1.5	0.04127	0.836	4.00	4.7	3.9	1.22e-006	1.46e-006	1.33e-006
10	2.25	0.05682	0.807	5.51	4.6	3.9	1.20e-006	1.40e-006	1.29e-006
11	3.25	0.07455	0.774	7.23	4.5	4.1	1.18e-006	1.29e-006	1.23e-006
12	4.75	0.09319	0.740	9.03	3.4	3.5	1.50e-006	1.48e-006	1.49e-006
13	7	0.1128	0.703	10.94	2.4	3.7	2.09e-006	1.33e-006	1.63e-006
14	10.3	0.132	0.668	12.79	2.0	2.7	2.33e-006	1.74e-006	1.99e-006
15	15	0.1511	0.632	14.65	1.6	1.9	2.82e-006	2.40e-006	2.59e-006
16	7	0.1477	0.639	14.32	0.0	0.0	3.26e-004	0.00e+000	3.26e-004
17	3.25	0.1425	0.648	13.82	0.2	0.0	1.90e-005	0.00e+000	1.90e-005
18	1.5	0.1368	0.659	13.27	0.9	0.0	4.87e-006	0.00e+000	4.87e-006
19	0.75	0.1307	0.670	12.67	2.4	2.4	1.95e-006	1.96e-006	1.95e-006
20	1.5	0.1331	0.666	12.90	0.5	0.5	9.69e-006	9.41e-006	9.54e-006
21	3.25	0.1386	0.655	13.44	0.7	0.0	6.14e-006	0.00e+000	6.14e-006
22	7	0.1457	0.642	14.13	0.5	0.3	9.72e-006	1.67e-005	1.23e-005
23	10.3	0.151	0.632	14.64	0.5	0.3	9.33e-006	1.37e-005	1.11e-005
24	15	0.1603	0.615	15.54	0.9	0.6	4.95e-006	7.46e-006	5.95e-006
25	22	0.1752	0.588	16.98	1.1	1.4	3.80e-006	2.96e-006	3.33e-006
26	32.3	0.1926	0.555	18.67	0.9	1.0	4.36e-006	4.26e-006	4.31e-006
27	7	0.184	0.571	17.84	0.0	0.0	2.08e-004	0.00e+000	2.08e-004
28	1	0.1661	0.604	16.10	1.2	0.0	3.45e-006	0.00e+000	3.45e-006
29	0.25	0.152	0.631	14.73	7.6	8.5	5.70e-007	5.09e-007	5.38e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT



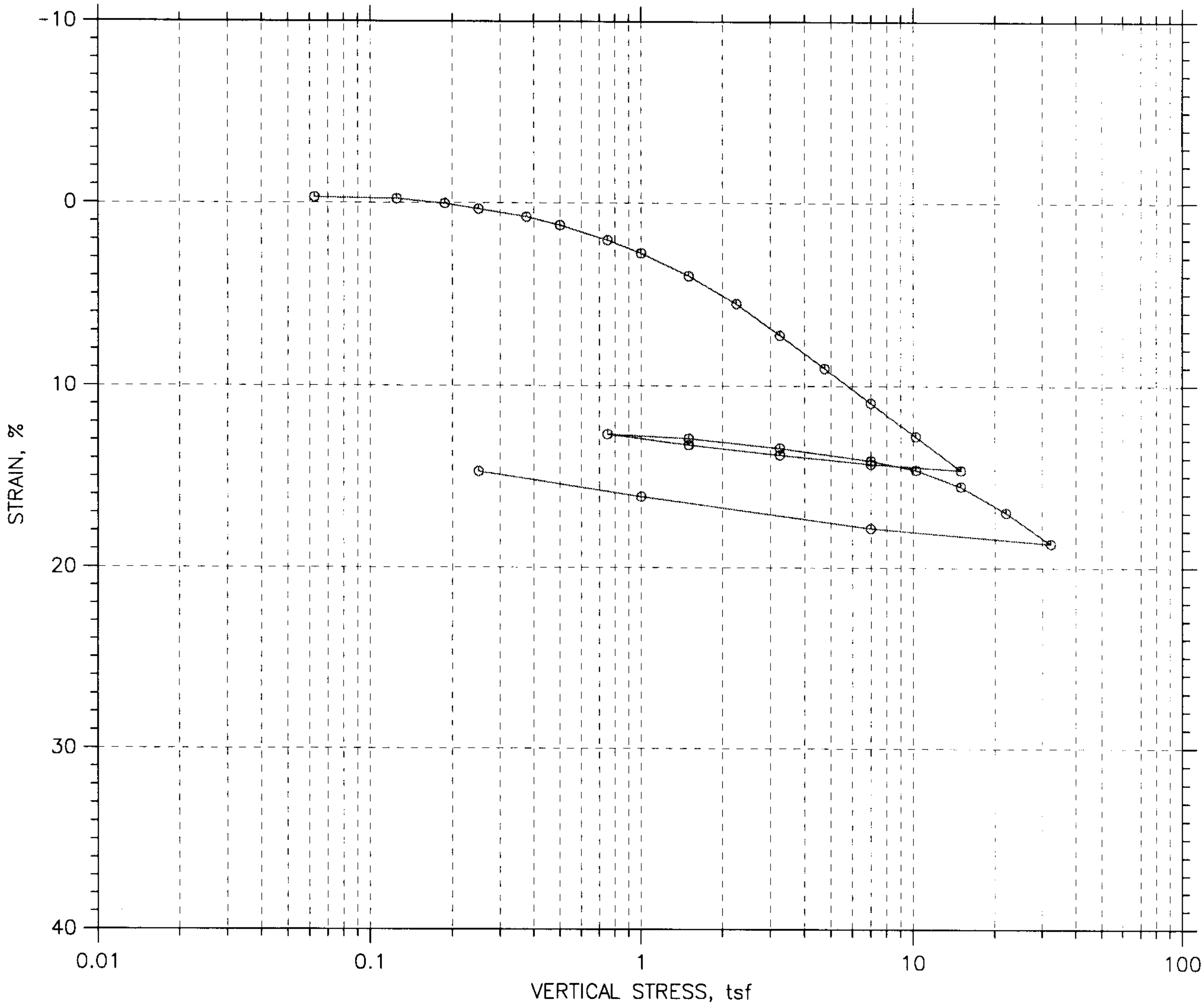
					Before Test	After Test
Overburden Pressure, tsf:				Water Content, %	34.61	23.87
Preconsolidation Pressure, tsf:				Dry Unit Weight, pcf	86.224	101.13
Compression Index:				Saturation, %	100.26	100.07
Diameter: 2.485 in		Height: 1.031 in		Void Ratio	0.91	0.63
LL: 30	PL: 22	PI: 8	GS: 2.64			

Project:	Location: AUBURN	Project No.: 18335.00
Boring No.: BB-ALAR-101	Tested By: G LIDSTONE	Checked By:
Sample No.: 1U	Test Date: 4/17/13	Depth: 40-42FT
Test No.: 266758	Sample Type: Shelby Tube	Elevation: ---
Description: SILT		
Remarks:		

Thu, 25-APR-2013 09:33:30

CONSOLIDATION TEST DATA

SUMMARY REPORT

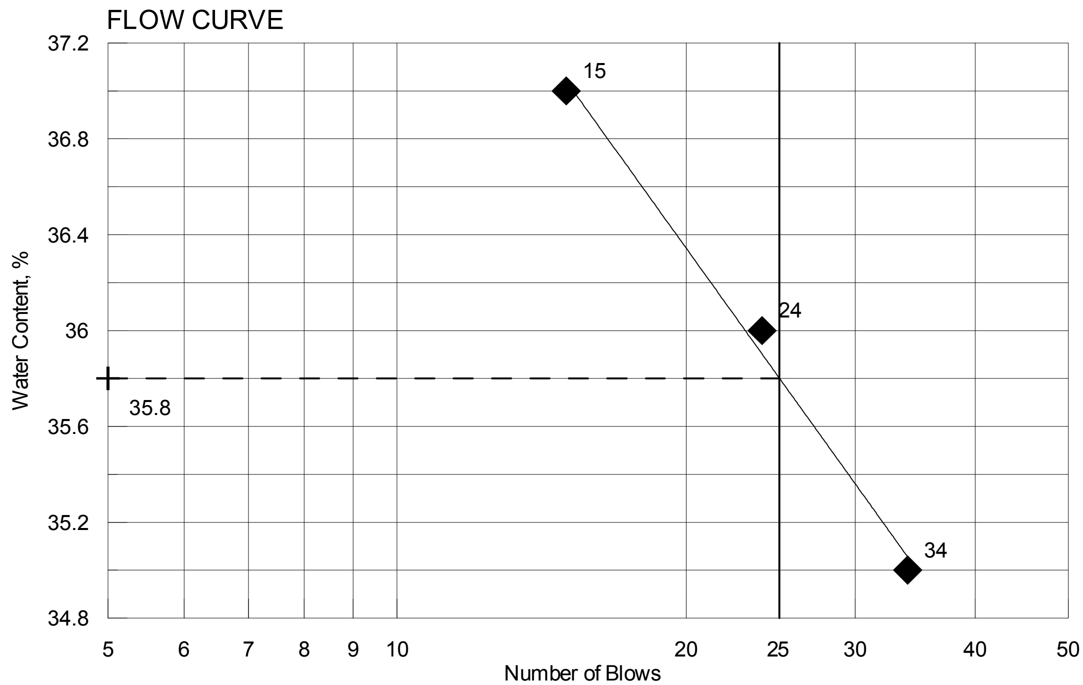


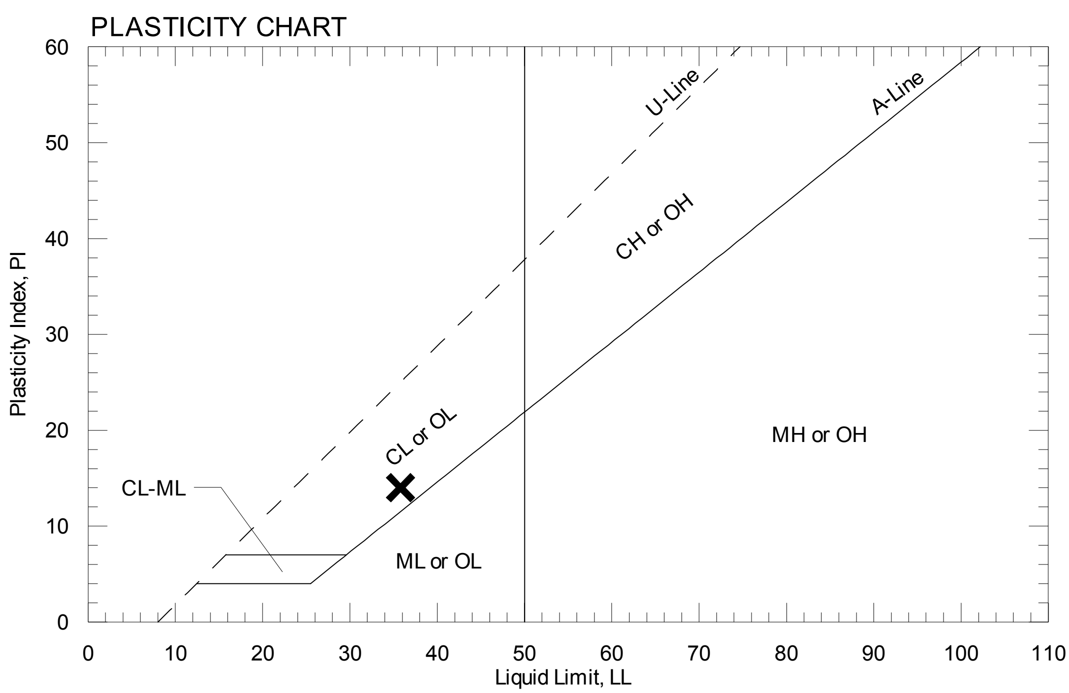
					Before Test	After Test
Overburden Pressure, tsf:				Water Content, %	34.61	23.87
Preconsolidation Pressure, tsf:				Dry Unit Weight, pcf	86.224	101.13
Compression Index:				Saturation, %	100.26	100.07
Diameter: 2.485 in		Height: 1.031 in		Void Ratio	0.91	0.63
LL: 30	PL: 22	PI: 8	GS: 2.64			

Project:	Location: AUBURN	Project No.: 18335.00
Boring No.: BB-ALAR-101	Tested By: G LIDSTONE	Checked By:
Sample No.: 1U	Test Date: 4/17/13	Depth: 40-42FT
Test No.: 266758	Sample Type: Shelby Tube	Elevation: ---
Description: SILT		
Remarks:		

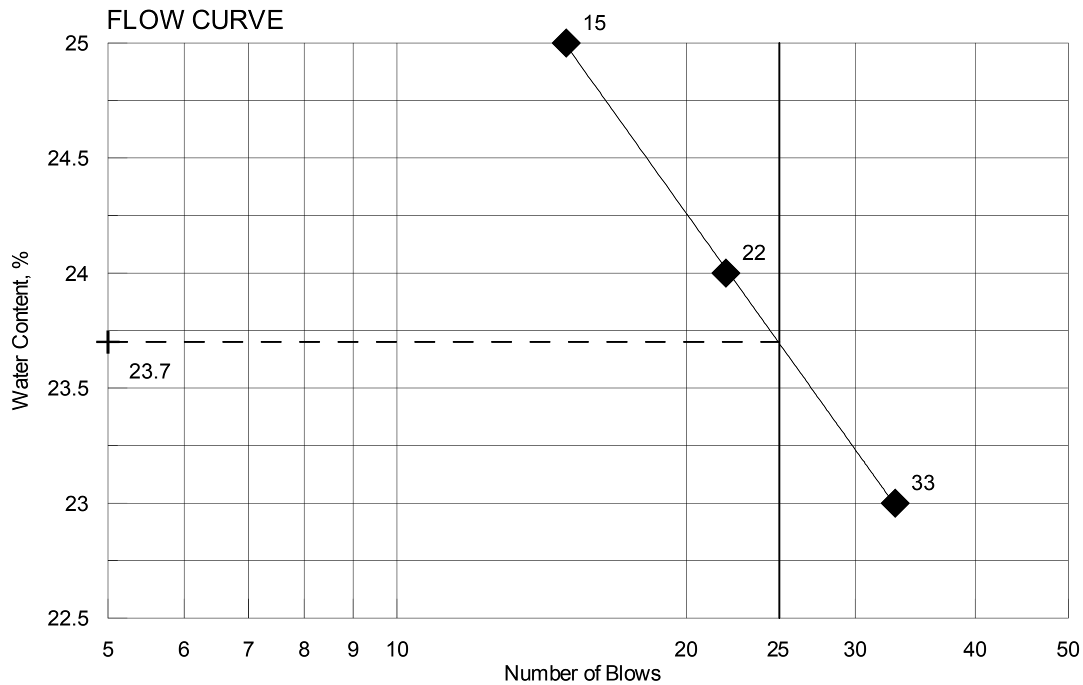
Thu, 25-APR-2013 10:29:25

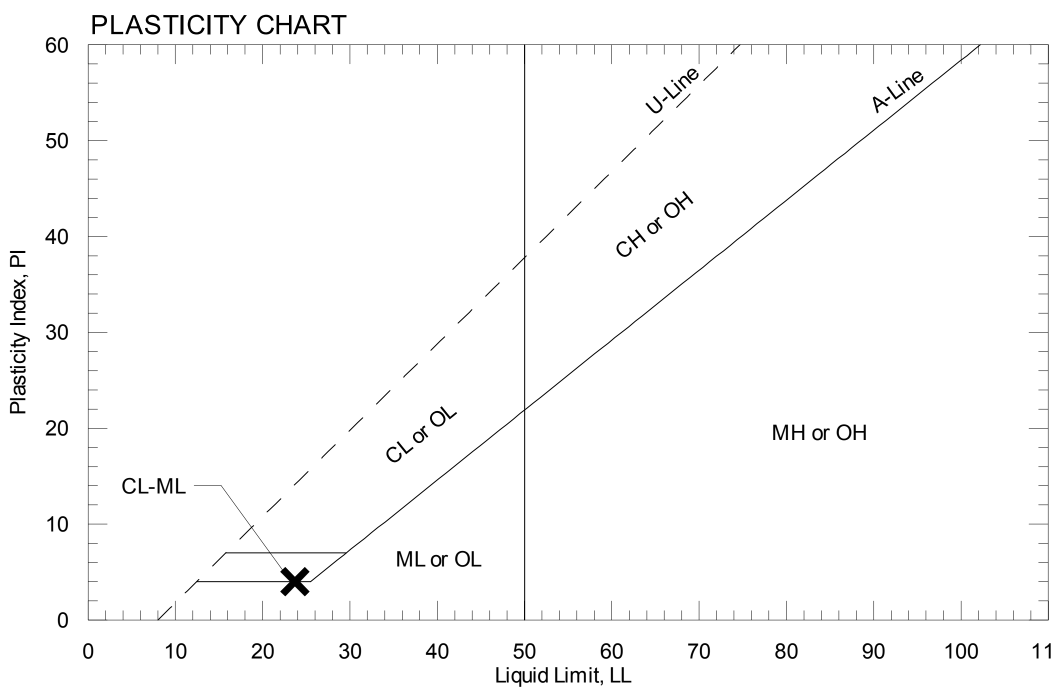
TOWN	Auburn	Reference No.	266759
WIN	018335.00	Water Content, %	34.9
Sampled	3/13/2013	Liquid Limit @ 25 blows (T 89), %	36
Boring No./Sample No.	BB-ALAR-101/8D	Plastic Limit (T 90), %	22
Station	107+33.4	Plasticity Index (T 90), %	14
Depth	55.0-57.0	Tested By	BBURR



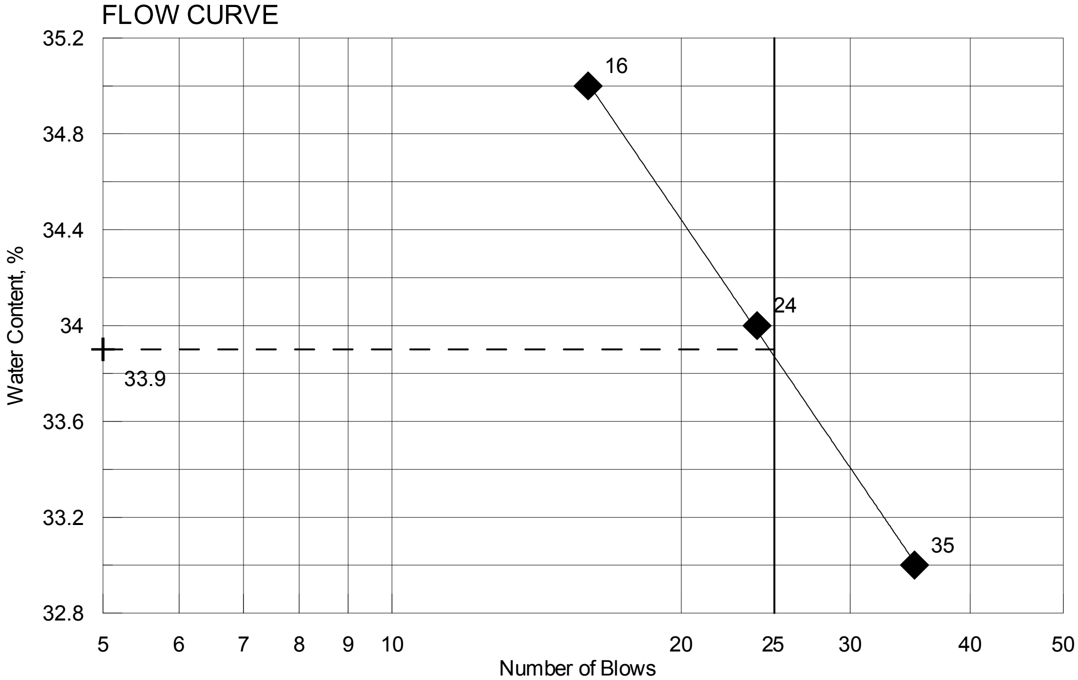


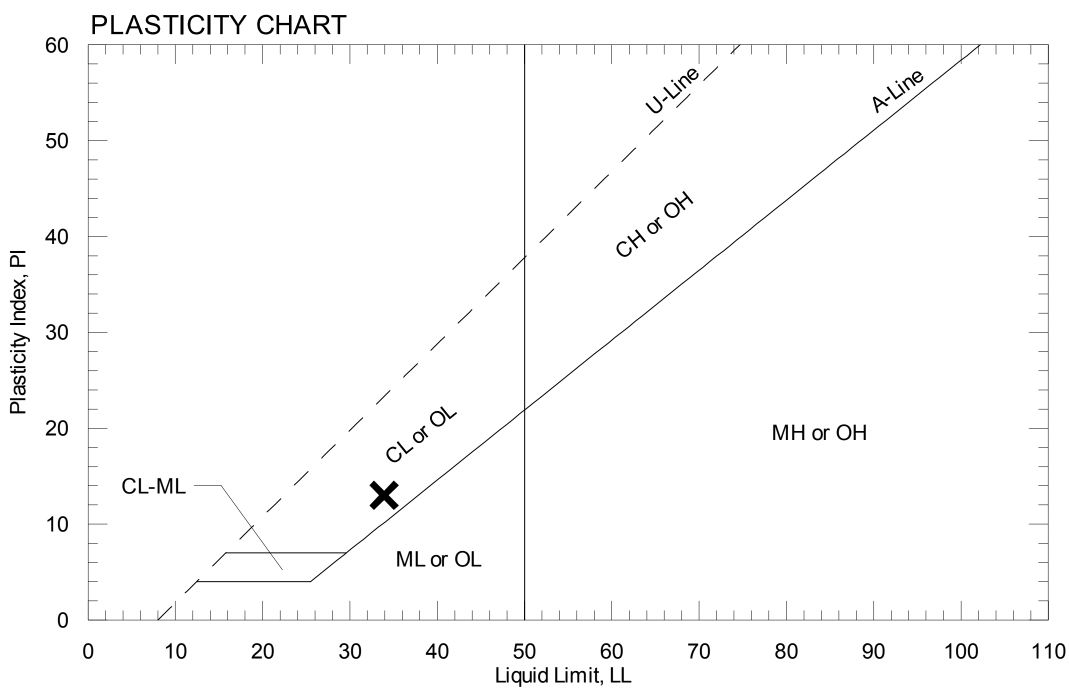
TOWN	Auburn	Reference No.	266762
WIN	018335.00	Water Content, %	20.9
Sampled	3/11/2013	Liquid Limit @ 25 blows (T 89), %	24
Boring No./Sample No.	BB-ALAR-102/2D & 3D	Plastic Limit (T 90), %	20
Station	108+32.4	Plasticity Index (T 90), %	4
Depth	6-8/11-13	Tested By	BBURR



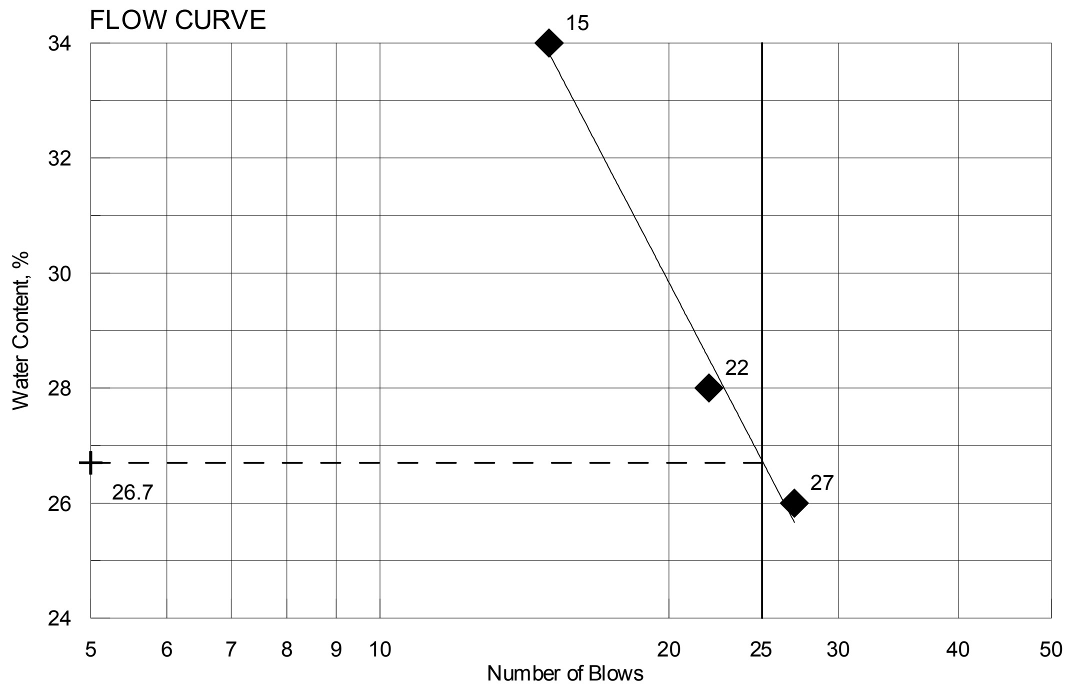


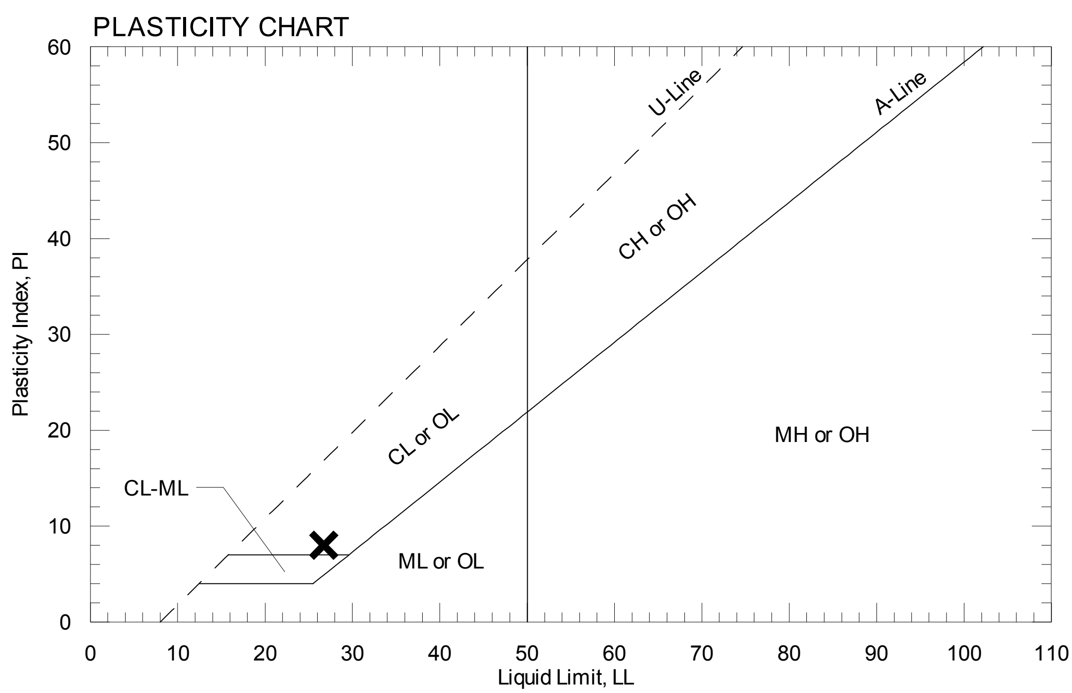
TOWN	Auburn	Reference No.	266763
WIN	018335.00	Water Content, %	28.8
Sampled	3/14/2013	Liquid Limit @ 25 blows (T 89), %	34
Boring No./Sample No.	BB-ALAR-102/4D & 5D	Plastic Limit (T 90), %	21
Station	108+32.4	Plasticity Index (T 90), %	13
Depth	23-25/26-28	Tested By	BBURR





TOWN	Auburn	Reference No.	266768
WIN	018335.00	Water Content, %	30
Sampled	3/22/2013	Liquid Limit @ 25 blows (T 89), %	27
Boring No./Sample No.	BB-ALAR-103/7D & 8D	Plastic Limit (T 90), %	19
Station	109+64.8	Plasticity Index (T 90), %	8
Depth	29-31/39-41	Tested By	BBURR





MaineDOT

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
266769	BB-ALAR-103/1U	GEOTECHNICAL (UNDISTURBED)	3/25/2013	4/11/2013

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **109+64.8** Offset, ft: **8.4 LT** Dbfg, ft: **54.0-56.0**

WIN/Town **018335.00 - AUBURN** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	100.0
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	99.9
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	99.9
[0.0236 mm]	97.2
[0.0153 mm]	94.4
[0.0094 mm]	83.3
[0.0070 mm]	75.0
[0.0053 mm]	63.9
[0.0027 mm]	52.8
[0.0012 mm]	41.7

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, % **35.1**

	Initial	Final		Void Ratio	% Strain
Water Content, %	39.7	26.4	Pmin	0.63 tsf	
Dry Density, lbs/ft³	84.5	99.9	Pp	1.4 tsf	
Void Ratio	1.05	0.73	Pmax	2.1 tsf	
Saturation, %	104.9	100	Cc/C'c	0.4102	

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	35
Plastic Limit (T 90), %	21
Plasticity Index (T 90), %	14
Specific Gravity, Corrected to 20°C (T 100)	2.77
Loss on Ignition (T 267) — Loss, %	
Loss on Ignition (T 267) — H2O, %	
Water Content (T 265), %	35.9

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths
0-0.5	0.29	0	0.23	0	36.4	Alternating layers of light to dark gray silty clay, silt line at 4 1/4".
0.63-1.0	0.23	0.01	0.21	0	40.5	Alternating layers of light to dark gray silty clay.
1.0-1.5	0.19	0	0.22	0	40.1	Alternating layers of light to dark gray silty clay, silt line at 14 1/2".
1.5-2.0	0.19	0	0.21	0	40.2	Alternating layers of light to dark gray silty clay.

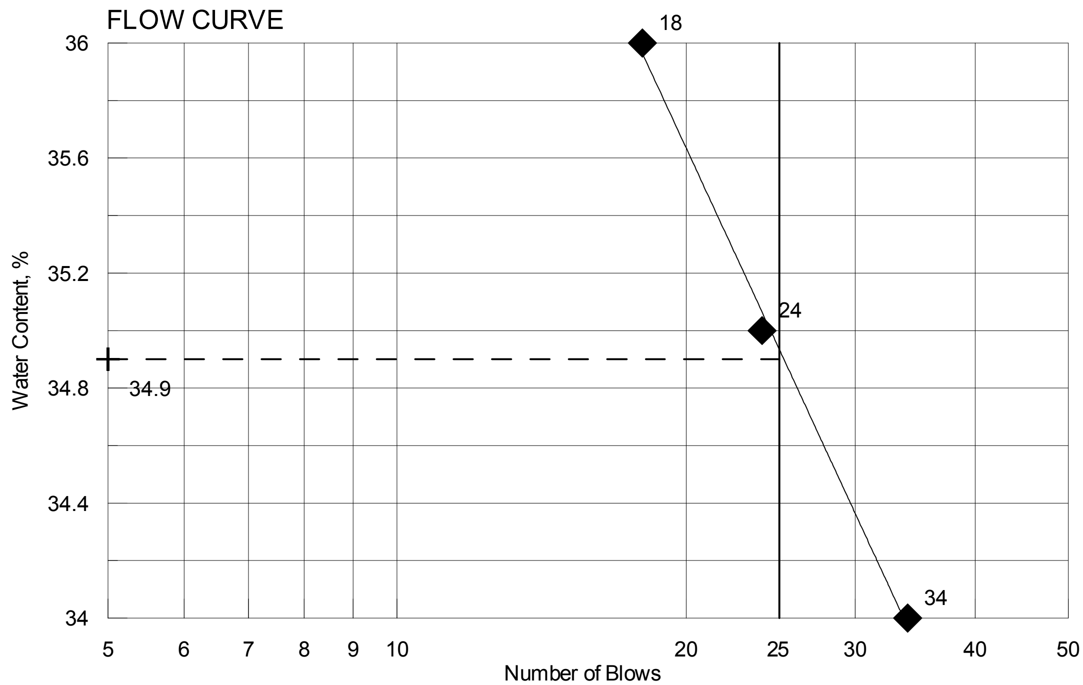
Comments:

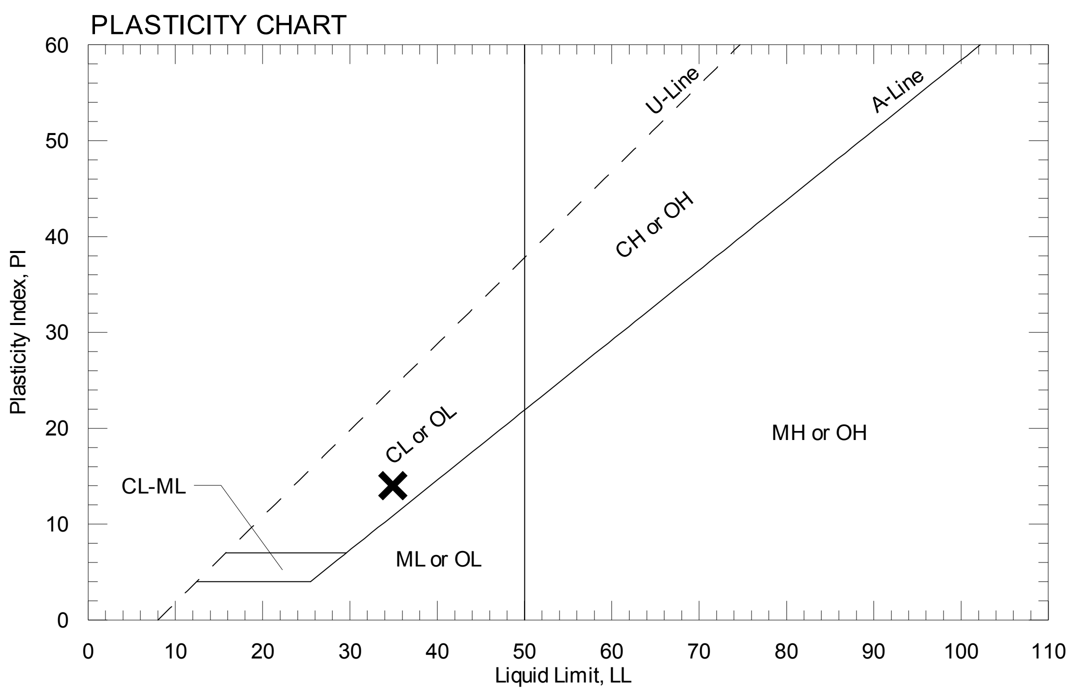
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **4/29/2013**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	266769
WIN	018335.00	Water Content, %	35.9
Sampled	3/25/2013	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-ALAR-103/1U	Plastic Limit (T 90), %	21
Station	109+64.8	Plasticity Index (T 90), %	14
Depth	54.0-56.0	Tested By	BBURR





CONSOLIDATION TEST DATA

Project:
Boring No.: BB-ALAR-103
Sample No.: 1U
Test No.: 266769

Location: AUBURN
Tested By: G LIDSTONE
Test Date: 4/18/13
Sample Type: Shelby Tube

Project No.: 18335.00
Checked By:
Depth: 54-56FT
Elevation: ---

Soil Description: SILT
Remarks:

Specific Gravity: 2.77
Initial Void Ratio: 1.05
Final Void Ratio: 0.73

Liquid Limit: 35
Plastic Limit: 21
Plasticity Index: 14

Initial Height: 0.98 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	28	RING	RING	84
Wt. Container + Wet Soil, gm	158.31	410.08	396.01	202.17
Wt. Container + Dry Soil, gm	129.38	368.07	368.07	174.28
Wt. Container, gm	46.89	262.12	262.12	68.52
Wt. Dry Soil, gm	82.49	105.95	105.95	105.76
Water Content, %	35.07	39.65	26.37	26.37
Void Ratio	---	1.05	0.73	---
Degree of Saturation, %	---	104.93	99.93	---
Dry Unit Weight, pcf	---	84.489	99.9	---

CONSOLIDATION TEST DATA

Project: OAKDALE NB
Boring No.: BB-ALAR-103
Sample No.: 1U
Test No.: 266769

Location: AUBURN
Tested By: G LIDSTONE
Test Date: 4/18/13
Sample Type: Shelby Tube

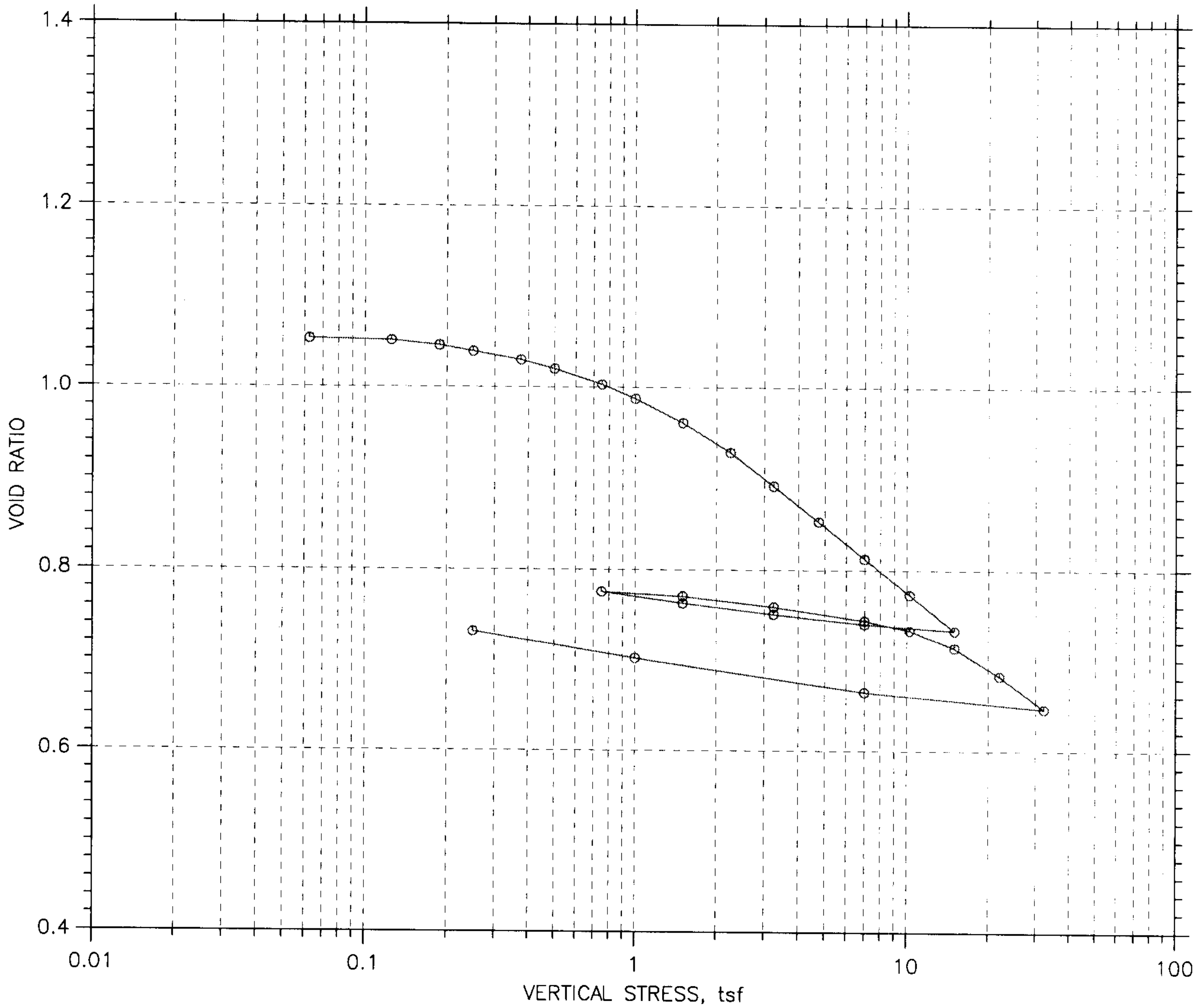
Project No.: 18335.00
Checked By:
Depth: 54-56FT
Elevation: ---

Soil Description: Clayey SILT
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	-0.002938	1.170	-0.28	0.1	0.0	4.87e-005	0.00e+000	4.87e-005
2	0.125	-0.0005825	1.165	-0.06	2.3	0.0	2.75e-006	0.00e+000	2.75e-006
3	0.188	0.002895	1.158	0.28	6.9	1.8	8.95e-007	3.49e-006	1.42e-006
4	0.25	0.00598	1.151	0.57	4.9	4.3	1.25e-006	1.42e-006	1.33e-006
5	0.375	0.009772	1.144	0.94	2.3	2.9	2.61e-006	2.07e-006	2.31e-006
6	0.5	0.01322	1.136	1.27	3.6	3.0	1.69e-006	1.99e-006	1.83e-006
7	0.75	0.01888	1.125	1.81	2.1	2.1	2.84e-006	2.85e-006	2.84e-006
8	1	0.02444	1.113	2.35	8.8	0.0	6.75e-007	0.00e+000	6.75e-007
9	1.5	0.03331	1.095	3.20	2.1	1.6	2.77e-006	3.63e-006	3.14e-006
10	2.25	0.04653	1.067	4.47	2.3	2.2	2.51e-006	2.64e-006	2.58e-006
11	3.25	0.08986	0.977	8.63	0.7	0.1	7.60e-006	7.19e-005	1.37e-005
12	4.75	0.1264	0.901	12.14	7.0	7.4	7.13e-007	6.70e-007	6.90e-007
13	7	0.1586	0.834	15.23	4.6	4.4	9.98e-007	1.05e-006	1.02e-006
14	10.3	0.1839	0.782	17.66	3.4	3.6	1.26e-006	1.19e-006	1.22e-006
15	15	0.2095	0.729	20.11	3.5	3.5	1.18e-006	1.15e-006	1.16e-006
16	7	0.2049	0.738	19.68	0.1	0.0	3.78e-005	0.00e+000	3.78e-005
17	3.25	0.1992	0.750	19.13	0.5	0.0	8.90e-006	0.00e+000	8.90e-006
18	1.5	0.1928	0.763	18.51	1.3	0.0	3.09e-006	0.00e+000	3.09e-006
19	0.75	0.1853	0.779	17.79	3.6	3.1	1.14e-006	1.32e-006	1.22e-006
20	1.5	0.1866	0.776	17.92	0.5	0.0	7.99e-006	0.00e+000	7.99e-006
21	3.25	0.193	0.763	18.53	0.8	0.0	4.99e-006	0.00e+000	4.99e-006
22	7	0.2032	0.742	19.51	0.8	0.0	5.36e-006	0.00e+000	5.36e-006
23	10.3	0.2108	0.726	20.24	1.1	0.0	3.46e-006	0.00e+000	3.46e-006
24	15	0.2222	0.702	21.34	1.4	1.2	2.81e-006	3.34e-006	3.05e-006
25	22	0.2379	0.670	22.85	1.4	1.7	2.66e-006	2.26e-006	2.45e-006
26	32.3	0.2572	0.629	24.70	1.2	1.6	3.04e-006	2.22e-006	2.56e-006
27	7	0.2479	0.649	23.80	0.0	0.0	1.51e-004	0.00e+000	1.51e-004
28	1	0.2265	0.693	21.75	2.0	0.0	1.86e-006	0.00e+000	1.86e-006
29	0.25	0.2088	0.730	20.05	7.5	0.0	5.13e-007	0.00e+000	5.13e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT



					Before Test	After Test
Overburden Pressure, tsf:				Water Content, %	39.65	26.37
Preconsolidation Pressure, tsf:				Dry Unit Weight, pcf	84.489	99.9
Compression Index:				Saturation, %	104.93	99.93
Diameter: 2.485 in		Height: 0.985 in		Void Ratio	1.05	0.73
LL: 35	PL: 21	PI: 14	GS: 2.77			

Project:	Location: AUBURN	Project No.: 18335.00
Boring No.: BB-ALAR-103	Tested By: G LIDSTONE	Checked By:
Sample No.: 1U	Test Date: 4/18/13	Depth: 54-56FT
Test No.: 266769	Sample Type: Shelby Tube	Elevation: ---
Description: SILT		
Remarks:		

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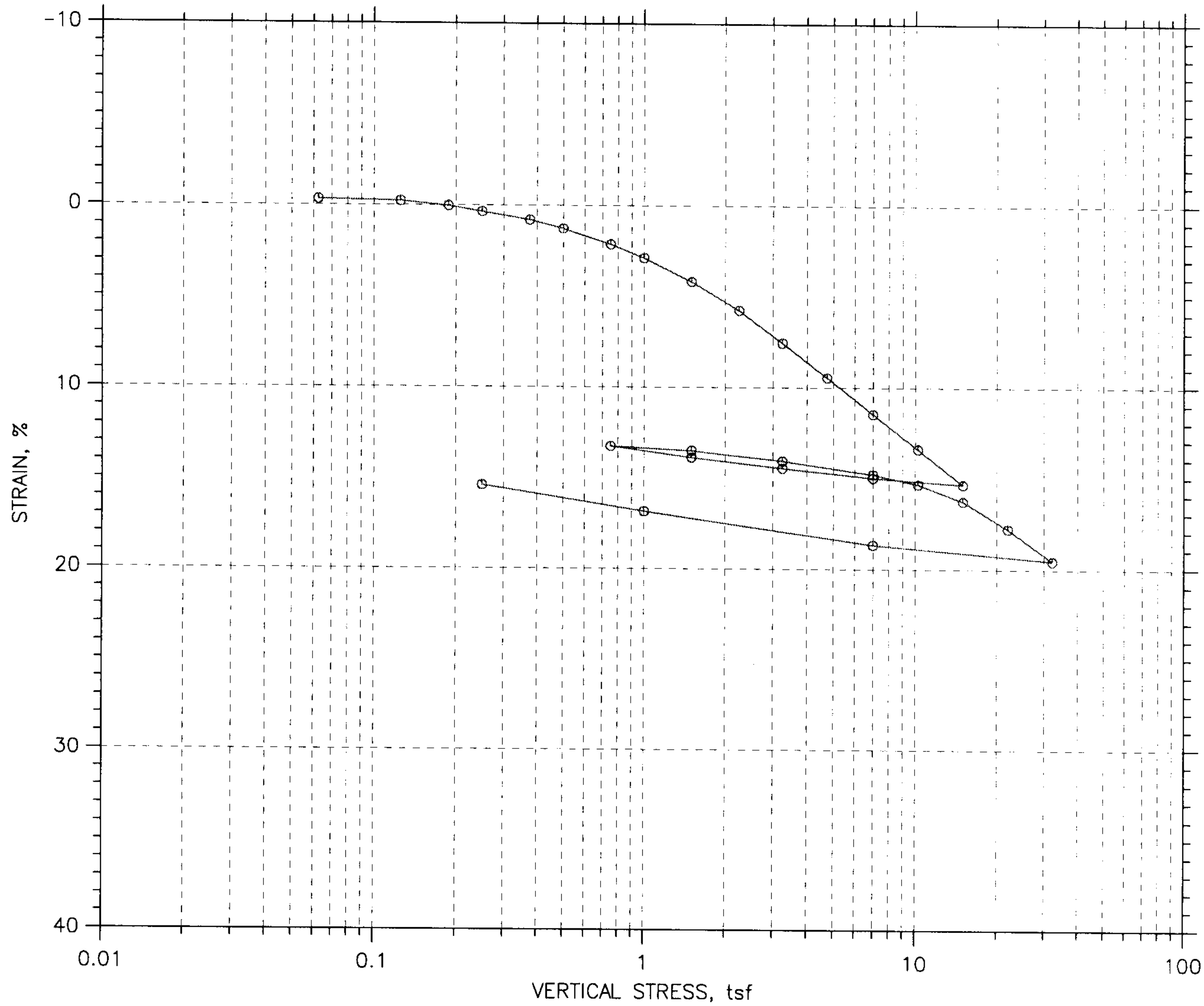
CONSOLIDATION TEST DATA

SUMMARY REPORT



					Before Test	After Test
Overburden Pressure, tsf:				Water Content, %	39.65	26.37
Preconsolidation Pressure, tsf:				Dry Unit Weight, pcf	84.489	99.9
Compression Index:				Saturation, %	104.93	99.93
Diameter: 2.485 in		Height: 0.985 in		Void Ratio	1.05	0.73
LL: 35	PL: 21	PI: 14	GS: 2.77			

Project:	Location: AUBURN	Project No.: 18335.00
Boring No.: BB-ALAR-103	Tested By: G LIDSTONE	Checked By:
Sample No.: 1U	Test Date: 4/18/13	Depth: 54-56FT
Test No.: 266769	Sample Type: Shelby Tube	Elevation: ---
Description: SILT		
Remarks:		

Mon, 29-APR-2013 08:42:49

Appendix C

Calculations

LIQUIDITY INDEX (LI):

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

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

Sample	Soil	WC	LL	PL	PI	LI	Plasticity	
BB-ALAR-101, 5D	Silt	29.6	35	20	15	0.64	medium plasticity	overconsolidated
BB-ALAR-101, 1U	Clayey Silt	32.9	30	22	8	1.36	low plasticity	viscous liquid when remolded
BB-ALAR-101, 8D	Silty Clay	34.9	36	22	14	0.92	medium plasticity	overconsolidated
BB-ALAR-102, 2D	Silt	20.9	24	20	4	0.23	slightly plastic	some to heavily overconsolidated
BB-ALAR-102, 4D	Silty Clay	28.8	34	21	13	0.60	medium plasticity	overconsolidated
BB-ALAR-103, 7D	Clayey Silt	30.0	27	19	8	1.38	low plasticity	viscous liquid when remolded
BB-ALAR-103, 1U	Clayey Silt	35.9	35	21	14	1.06	medium plasticity	viscous liquid when remolded

CONSOLIDATION TEST RESULTS

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

BB-ALAR-101 Sample 1U

Determine in-situ over burden stress:

Sample depth = 41.0 ft below ground surface

Groundwater table at 10.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.91$

Clay is overlain by:
13.0 ft of sand fill at 125 pcf
11.0 ft of sand at 125 pcf
17.0 ft of silt and clay at 115 pcf

$$\sigma'_{vo} := 10 \cdot ft \cdot 125 \cdot pcf + 3.0 \cdot ft \cdot (125 - 62.4) \cdot pcf + 11 \cdot ft \cdot (125 - 62.4) \cdot pcf + 17 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3021 \cdot psf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.5 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 0.9932$ normally consolidated

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 0.774$ $p_2 := 15 \cdot tsf$ $e_2 := 0.632$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.2138$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{7.23}{100}$ $\varepsilon_2 := \frac{14.65}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1117$$ or: $$C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1119$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.670$ $p_2 := 3.25 \cdot tsf$ $e_2 := 0.655$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0236$$

BB-ALAR-103 Sample 1U

Determine in-situ over burden stress:

Sample depth = 55.0 ft below ground surface

Groundwater table at 12.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.05$

Clay is overlain by:
12.0 ft of sand fill at 125 pcf
17.0 ft of silt at 125 pcf
26.0 ft of silt and clay at 115 pcf

$$\sigma'_{vo} := 12.0 \cdot ft \cdot 125 \cdot pcf + 17 \cdot ft \cdot (125 - 62.4) \cdot pcf + 26 \cdot ft \cdot (115 - 62.4) \cdot pcf \qquad \sigma'_{vo} = 3932 \cdot psf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.4 \cdot tsf$

Determine OCR: $OCR := \dfrac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 0.7121$ normally consolidated

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 1.067$ $p_2 := 15 \cdot tsf$ $e_2 := 0.729$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.4102$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \dfrac{4.47}{100}$ $\varepsilon_2 := \dfrac{20.11}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1898$$ or: $$C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.2001$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.779$ $p_2 := 3.25 \cdot tsf$ $e_2 := 0.763$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0251$$

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

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 $l_{unbraced} := 12 \cdot in$ Assume 1 foot unbraced - scour (unlikely)

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

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

$$P_e = \begin{pmatrix} 174999 \\ 256564 \\ 359780 \\ 448914 \\ 611956 \end{pmatrix} \cdot kip$$

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

$$\frac{P_e}{P_o} = \begin{pmatrix} 226 \\ 235 \\ 336 \\ 344 \\ 356 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

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

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "good" based on borings.

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: eq. 6.9.2.1-1

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

$$P_r = \begin{pmatrix} 464 \\ 653 \\ 641 \\ 782 \\ 1031 \end{pmatrix} \cdot kip$$

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

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

$$P_r = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

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 Geotechnical Method

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

Bedrock Type:

Granite RQD 90%

Use RQD = 90% and ϕ = 34 to 40 deg (LRFD Table C10.4.6.4-1)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

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 in bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

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

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 20000 \cdot psi$

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

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 1920 \\ 1905 \\ 1729 \\ 1723 \\ 1711 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

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

$$R_p := \overrightarrow{(3q_a \cdot A_s)} \qquad R_p = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$$

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 in 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

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 279 \\ 389 \\ 347 \\ 421 \\ 552 \end{pmatrix} \cdot kip$$

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 $\phi = 1.0$ LRFD 10.5.5.1 and 10.5.5.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 620 \\ 865 \\ 771 \\ 937 \\ 1226 \end{pmatrix} \cdot kip$$

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 Piles Driven to Hard Rock 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 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 conditions."

Nominal Structural Resistance:
previously calculated

$$P_n = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

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

Determine *Factored* Axial Geotechnical Resistance at the **Strength Limit State**

Apply resistance factor for severe driving from LRFD Article 6.5.4.2

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

Factored Axial Geotechnical Resistance
Strength Limit State

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

$$P_{strength} = \begin{pmatrix} 387 \\ 544 \\ 534 \\ 652 \\ 859 \end{pmatrix} \cdot kip$$

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

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

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

$$\phi = 1.0$$

Factored Axial Geotechnical Resistance -
Service and Extreme Limit States

$$P_{serv\_ext} := \phi \cdot P_n$$

$$P_{serv\_ext} = \begin{pmatrix} 774 \\ 1088 \\ 1069 \\ 1303 \\ 1718 \end{pmatrix} \cdot kip$$

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 a Delmag D 19-42 hammer on third fuel setting

State of Maine Dept. Of Transportation — 06-May-2013
18335 Auburn Oakdale Drivability HP12x53 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
464.0	44.44	5.61	11.8	8.46	19.89
466.0	44.54	5.62	12.0	8.48	19.94
468.0	44.62	5.60	12.2	8.49	19.96
470.0	44.70	5.60	12.3	8.50	19.98
472.0	44.79	5.59	12.5	8.51	20.00
474.0	44.87	5.62	12.7	8.52	20.07
476.0	44.94	5.61	12.9	8.53	20.09
478.0	45.03	5.59	13.1	8.55	20.10
480.0	45.10	5.59	13.3	8.56	20.12
482.0	45.18	5.62	13.4	8.57	20.18

Limit driving stress to 45 ksi -
blow count limited to 12 bpi as >12 bpi exceeds 45 ksi

$R_{dr\_12x53} := 466 \cdot kip$

Strength Limit State:

$R_{dr\_12x53\_strength} := R_{dr\_12x53} \cdot \phi_{dyn}$

$R_{dr\_12x53\_strength} = 303 \cdot kip$

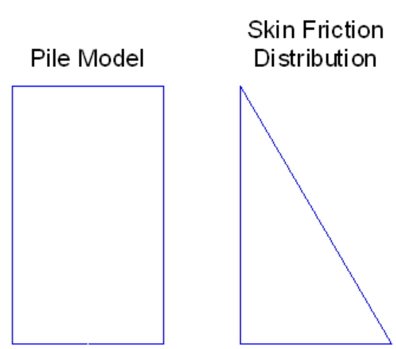
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_12x53\_servext} := R_{dr\_12x53} \cdot \phi$

$R_{dr\_12x53\_servext} = 466 \cdot kip$

DELMAG D 19-42

Efficiency	0.800	
Helmet	2.70	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	65.00	ft
Pile Penetration	65.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 10 %
(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 — 18-Jul-2013
18335 Auburn Oakdale Drivability HP12x74 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
512.0	42.03	3.74	5.0	6.58	27.95
514.0	42.14	3.76	5.1	6.60	27.99
520.0	42.46	3.83	5.2	6.63	28.14
540.0	43.55	3.99	5.7	6.76	28.65
550.0	44.06	4.07	6.0	6.82	28.85
560.0	44.60	4.17	6.2	6.88	29.10
565.0	44.83	4.21	6.4	6.90	29.16
570.0	45.09	4.26	6.5	6.93	29.30
575.0	45.34	4.31	6.7	6.96	29.42
580.0	45.57	4.35	6.8	6.99	29.49

Limit driving stress to 45 ksi -
blow count limited to 6 bpi as >6 bpi exceeds 45 ksi

$R_{dr\_12x74} := 550 \cdot kip$

Strength Limit State:

$R_{dr\_12x74\_strength} := R_{dr\_12x74} \cdot \phi_{dyn}$

$R_{dr\_12x74\_strength} = 358 \cdot kip$

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

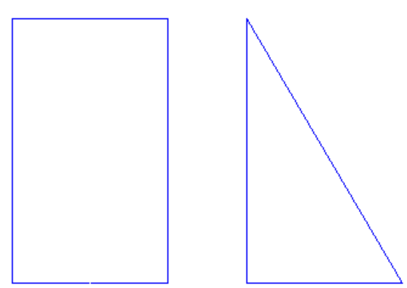
$R_{dr\_12x74\_servext} := R_{dr\_12x74} \cdot \phi$

$R_{dr\_12x74\_servext} = 550 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	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	65.00	ft
Pile Penetration	65.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

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

State of Maine Dept. Of Transportation — 18-Jul-2013
18335 Auburn Oakdale Dravability HP14x73 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
510.0	42.37	3.81	5.0	6.59	28.15
515.0	42.64	3.87	5.1	6.62	28.29
520.0	42.92	3.93	5.2	6.65	28.36
530.0	43.48	4.04	5.5	6.71	28.64
535.0	43.73	4.09	5.6	6.74	28.72
540.0	44.01	4.13	5.7	6.77	28.85
545.0	44.23	4.16	5.9	6.80	28.93
550.0	44.55	4.20	6.0	6.84	29.12
560.0	45.00	4.26	6.3	6.89	29.27
570.0	45.54	4.35	6.6	6.95	29.55

Limit driving stress to 45 ksi -
blow count limited to 6 bpi as >6 bpi exceeds 45 ksi

$R_{dr\_14x73} := 550 \cdot kip$

Strength Limit State:

$R_{dr\_14x73\_strength} := R_{dr\_14x73} \cdot \phi_{dyn}$

$R_{dr\_14x73\_strength} = 358 \cdot kip$

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

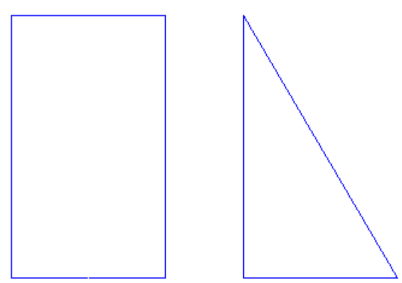
$R_{dr\_14x73\_servext} := R_{dr\_14x73} \cdot \phi$

$R_{dr\_14x73\_servext} = 550 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	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	65.00	ft
Pile Penetration	65.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(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 18-Jul-2013
18335 Auburn Oakdale Drivability HP14x89 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
654.0	44.16	3.69	8.8	7.15	29.01
655.0	44.19	3.69	8.9	7.16	29.02
656.0	44.23	3.69	8.9	7.16	29.03
657.0	44.27	3.69	9.0	7.17	29.05
658.0	44.31	3.69	9.0	7.17	29.07
659.0	44.34	3.69	9.0	7.17	29.08
660.0	44.39	3.69	9.1	7.18	29.10
661.0	44.38	3.69	9.1	7.18	29.12
662.0	44.43	3.69	9.2	7.19	29.13
663.0	44.44	3.69	9.2	7.19	29.14

Limit driving stress to 45 ksi -
blow count limited to 9 bpi as >9 bpi exceeds 45 ksi

$R_{dr\_14x89} := 659 \cdot kip$

Strength Limit State:

$R_{dr\_14x89\_strength} := R_{dr\_14x89} \cdot \phi_{dyn}$

$R_{dr\_14x89\_strength} = 428 \cdot kip$

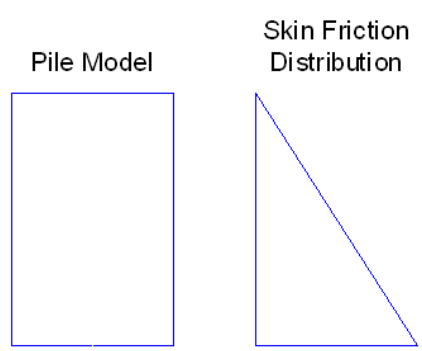
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \phi$

$R_{dr\_14x89\_servext} = 659 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52998	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	65.00	ft
Pile Penetration	65.00	ft
Pile Top Area	26.10	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag 46-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation — 18-Jul-2013
18335 Auburn Oakdale Drivability 14x117 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
718.0	43.14	2.84	5.9	7.52	36.44
719.0	43.25	2.86	5.9	7.52	36.55
720.0	43.28	2.86	6.0	7.53	36.55
721.0	43.31	2.87	6.0	7.54	36.65
722.0	43.35	2.87	6.0	7.53	36.58
723.0	43.35	2.88	6.0	7.54	36.67
724.0	43.41	2.88	6.0	7.55	36.67
725.0	43.48	2.88	6.0	7.56	36.66
726.0	43.49	2.89	6.1	7.56	36.68
727.0	43.56	2.90	6.1	7.56	36.79

Limit driving stress to 45 ksi -
blow count limited to 6 bpi as >6 bpi exceeds 45 ksi

$R_{dr\_14x117} := 725 \cdot kip$

Strength Limit State:

$R_{dr\_14x117\_strength} := R_{dr\_14x117} \cdot \phi_{dyn}$

$R_{dr\_14x117\_strength} = 471 \cdot kip$

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

$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \phi$

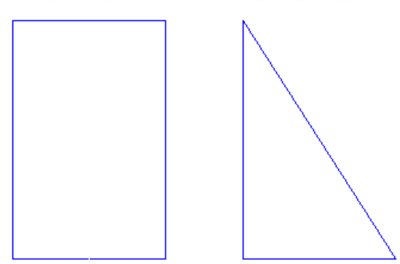
$R_{dr\_14x117\_servext} = 725 \cdot kip$

DELMAG D 46-32

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	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	65.00	ft
Pile Penetration	65.00	ft
Pile Top Area	34.40	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(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 $\beta := 0 \cdot deg$

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

$K_p = 6.89$

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.

Pipe Pile Supported Pier

Calculate Depth to Fixity for pipe piles (composite section):

Soil conditions at boring BB-ALAR-102:

6 ft of sand, 31.5 ft of silt and clay, and 5 ft of glacial till over bedrock

Consider Pile sizes:

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$dia_{steelcor} := dia_{steel} - 2 \cdot cor \qquad dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in \qquad wall_{cor} := wall_t - cor \qquad wall_{cor} = \begin{pmatrix} 0.375 \\ 0.5 \end{pmatrix} \cdot in$$

$$dia_{conccore\_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore\_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore\_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES with 1/8" corrosion loss

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steelcor}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES with 1/8" corrosion loss

Transformed pile properties of 1/2 inch wall pile:

unit weight of concrete: $wc := 0.15$ in kips per cubic foot

compressive strength of concrete: $f_c := 4.35$ in ksi Class A concrete

Modulus of elasticity of concrete: $E_c := 33000 \cdot wc^{1.5} \cdot \sqrt{f_c} \cdot 1000 \cdot psi$ $E_c = 3998 \cdot ksi$

Steel modulus: $E_{steel} := 29000 \cdot ksi$

$$n := \frac{E_{steel}}{E_c} \qquad n = 7.25$$

MaineDOT Structural Engineers routinely use:

$n := 7.6$

Moment of inertia of concrete core:

$$I_{c\_0.5} := \frac{\pi \cdot dia_{conccore\_0.5}{}^4}{64} \qquad I_{c\_0.5} = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.5} := \frac{\overrightarrow{\pi \cdot \left(dia_{steelcor}{}^4 - dia_{conccore\_0.5}{}^4\right)}}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.091 \\ 0.116 \\ 0.146 \\ 0.18 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.5} := \overrightarrow{\left(\frac{I_{c\_0.5}}{n} + I_{s\_0.5}\right)} \qquad I_{t\_0.5} = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.5} := \pi \cdot \frac{dia_{conccore\_0.5}{}^2}{4} \qquad A_{conc\_0.5} = \begin{pmatrix} 415.48 \\ 490.87 \\ 572.56 \\ 660.52 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.5} := A_{0.5} + \frac{A_{conc\_0.5}}{n} \qquad A_{t\_0.5} = \begin{pmatrix} 0.571 \\ 0.656 \\ 0.747 \\ 0.844 \end{pmatrix} \cdot ft^2$$

LRFD Eq. C10.7.3.13.4-1 for depth to fixity in feet: $1.4*(E_pI_w/E_s)^{0.25}$ (in clays)

E_p Young's modulus of pile (ksi)

I_w moment of inertia of pile (ft^{4})

E_s= soil modulus for clays = $0.456S_u$ (ksi)

S_u = undrained shear strength of clays (ksf)

$$E_{steel} = 2.9 \times 10^7 psi \qquad I_{t\_0.5} = \begin{pmatrix} 0.178 \\ 0.238 \\ 0.311 \\ 0.4 \end{pmatrix} ft^4$$

Shear strength of silt and clay: Us an average value of 500 psf from vane shear testing $S_u := 500 \cdot psf$

Determine soil modulus: $E_s := 0.465 \cdot S_u \qquad E_s = 0.2325 \cdot ksf$

Depth of Fixity:

$$D_{fix\_0.5} := 1.4 \cdot \left(\frac{\frac{E_{steel}}{144} \cdot I_{t\_0.5}}{E_s} \right)^{0.25}$$

$$D_{fix\_0.5} = \begin{pmatrix} 17.1 \\ 18.4 \\ 19.7 \\ 20.9 \end{pmatrix} ft$$

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

Depth to fixity for 1/2" wall pipe piles (composite section)

<u>Transformed pile properties of 5/8 inch wall pile:</u>

$$n = 7.6$$

Diameter of concrete core:

$$dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

Diameter of steel pipe

$$dia_{steelcor} = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$$

Moment of inertia of concrete core:

$$I_{c\_0.625} := \frac{\pi \cdot dia_{conccore\_0.625}{}^4}{64} \qquad I_{c\_0.625} = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.625} := \overrightarrow{\frac{\pi \cdot \left(dia_{steelcor}{}^{4} - dia_{conccore\_0.625}{}^{4}\right)}{64}} \qquad I_{s\_0.625} = \begin{pmatrix} 0.119 \\ 0.152 \\ 0.192 \\ 0.237 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.625} := \frac{I_{c\_0.625}}{n} + I_{s\_0.625} \qquad I_{t\_0.625} = \begin{pmatrix} 0.202 \\ 0.269 \\ 0.351 \\ 0.45 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.625} := \pi \cdot \frac{dia_{conccore\_0.625}{}^{2}}{4} \qquad A_{conc\_0.625} = \begin{pmatrix} 406.49 \\ 481.11 \\ 562 \\ 649.18 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.625} := A_{0.625} + \frac{A_{conc\_0.625}}{n} \qquad A_{t\_0.625} = \begin{pmatrix} 0.625 \\ 0.715 \\ 0.811 \\ 0.912 \end{pmatrix} \cdot ft^2$$

LRFD Eq. C10.7.3.13.4-1 for depth to fixity in feet: $1.4*(E_pI_w/E_s)^{0.25}$ (in clays)

E_p Young's modulus of pile (ksi)

I_w moment of inertia of pile ($ft^{4)}$

E_s= soil modulus for clays = $0.456S_u$ (ksi)

S_u = undrained shear strength of clays (ksf)

$$E_{steel} = 2.9 \times 10^{7}\, psi \qquad I_{t\_0.625} = \begin{pmatrix} 0.2025 \\ 0.2694 \\ 0.3512 \\ 0.4498 \end{pmatrix} ft^4$$

Shear strength of silt and clay: Us an average value of 500 psf from vane shear testing $S_u := 500 \cdot psf$

Determine soil modulus: $E_s := 0.465 \cdot S_u$ $\qquad E_s = 0.2325 \cdot ksf$

Depth of Fixity:

$$D_{fix\_0.625} := 1.4 \cdot \left(\frac{\frac{E_{steel}}{144} \cdot I_{t\_0.625}}{E_s}\right)^{0.25}$$

$$D_{fix\_0.625} = \begin{pmatrix} 17.6 \\ 19 \\ 20.3 \\ 21.5 \end{pmatrix} ft$$

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Depth to fixity for 5/8" wall pipe piles (composite section)

Nominal Axial Structural Resistance of pipe piles

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012 with 2013 Interims

Pier - Pipe Pile driven to bedrock

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check combined axial compression and flexure with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2.
Use LRFD Equation 6.9.5.1-1 or 6.9.5.1-2 to compute the nominal compressive structural resistance for pipe pile sections.

λ in Equation 6.9.5.1-1 and -2 has to be computed for the pipe piles since they have an unbraced length.

Yield strength of steel shell: $F_y := 45 \cdot ksi$

Compressive strength of concrete core: $f_c := 4350 \cdot psi$ Class A concrete

Yield strength of longitudinal reinforcement: $F_{yr} := 60 \cdot ksi$

Assume unsupported length is from bottom of pile cap including 5.5 feet of scour

Compute λ per 6.9.5.1-3 for composite members:

Effective length factor per LRFD Article 4.6.2.5:

Use case (e) in Table C4.6.2.5-1 $K := 2.0$

Exposed length of pile = Pile length through air + pile length trough water + calculated scour depth

Pile length through air + Pile length through water : $L_{air\_and\_water} := 16 \cdot ft$

Scour depth calculated to be approximately 5.5 feet $L_{ex} := 5.5 \cdot ft$ Scour data provided by VBH

Unbraced length of column:

$L_{UB} := L_{air\_and\_water} + L_{ex}$ $L_{UB} = 21.5\,ft$ for all piles

Longitudinal reinforcement:

Assume longitudinal reinforcement of 12 - #8 bars (1-inch) bars equally spaced for all pile sections.

$$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4} \qquad A_r = 9.42 \cdot in^2$$

Composite Column Constant per Table 6.9.5.1-1

for filled tube sections: $C1 := 1.0$ $C2 := 0.85$ $C3 := 0.40$

Variable Fe:

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.5}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.5}}{A_{0.5}} \qquad F_{e\_0.5} = \begin{pmatrix} 121.32 \\ 124.63 \\ 128.18 \\ 131.91 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.625}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.625}}{A_{0.625}} \qquad F_{e\_0.625} = \begin{pmatrix} 101.64 \\ 104.11 \\ 106.76 \\ 109.55 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Radius of gyration of both sets of steel sections:

$$r_{s\_0.5} := \overrightarrow{\sqrt{\frac{I_{s\_0.5}}{A_{0.5}}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.6888 \\ 0.7477 \\ 0.8066 \\ 0.8655 \end{pmatrix} ft$$

for 1/2" walls

$$r_{s\_0.625} := \overrightarrow{\sqrt{\frac{I_{s\_0.625}}{A_{0.625}}}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.6852 \\ 0.7441 \\ 0.803 \\ 0.8619 \end{pmatrix} ft$$

for 5/8" walls

E_e term:

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.5}}{A_{0.5}}}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 52028 \\ 54063 \\ 56097 \\ 58132 \end{pmatrix} \cdot ksi$$

for 1/2" walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \overrightarrow{\frac{A_{conc\_0.625}}{A_{0.625}}}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 45988 \\ 47514 \\ 49040 \\ 50566 \end{pmatrix} \cdot ksi$$

for 5/8" walls

Lambda (λ) term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right]} \qquad \lambda_{0.5} = \begin{pmatrix} 0.9208 \\ 0.7725 \\ 0.6579 \\ 0.5675 \end{pmatrix}$$

for 1/2" walls

$$\lambda_{0.625} := \overrightarrow{\left[\left(\frac{K \cdot L_{UB}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right]} \qquad \lambda_{0.625} = \begin{pmatrix} 0.882 \\ 0.7414 \\ 0.6325 \\ 0.5463 \end{pmatrix}$$

for 5/8" walls

Nominal Axial Structural Resistance of Composite member with 1/2-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.5} := \overrightarrow{\left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_{0.5}\right)}$$

$$P_{n\_0.5} = \begin{pmatrix} 2279 \\ 2703 \\ 3145 \\ 3606 \end{pmatrix} \cdot kip$$ for 1/2" walls

Nominal Axial Structural Resistance of Composite member with 5/8-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.625} := \overrightarrow{\left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_{0.625}\right)}$$

$$P_{n\_0.625} = \begin{pmatrix} 2573 \\ 3034 \\ 3514 \\ 4011 \end{pmatrix} \cdot kip$$ for 5/8" walls

Determine Axial Structural Resistance for Non-Composite Member (just steel shell)

Pipe pile Steel area:

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls
with corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls
with corrosion loss

yield strength: $F_y := 45 \cdot ksi$

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 $F_y = 45 \cdot ksi$

$$P_{o0.5} := Q \cdot F_y \cdot A_{0.5}$$

$$P_{o0.5} = \begin{pmatrix} 1239 \\ 1345 \\ 1451 \\ 1557 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{o0.625} := Q \cdot F_y \cdot A_{0.625}$$

$$P_{o0.625} = \begin{pmatrix} 1643 \\ 1785 \\ 1926 \\ 2068 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

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 $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 Design value of K when ideal conditions are approximated:
head: rotation fixed, translation free
tip: rotation free, translation fixed

l = unbraced length $L_{UB} = 21.5\,ft$ for all piles

r_s = radius of gyration $r_{s\_0.5} = \begin{pmatrix} 8.265 \\ 8.972 \\ 9.679 \\ 10.386 \end{pmatrix} \cdot in$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$r_{s\_0.625} = \begin{pmatrix} 8.222 \\ 8.929 \\ 9.636 \\ 10.343 \end{pmatrix} \cdot in$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

LRFD Eq. 6.9.4.1.2-1

$$P_{e\_0.5} := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot L_{UB}}{r_{s\_0.5}}\right)^2} \cdot A_{0.5}\right]}$$

$$P_{e\_0.5} = \begin{pmatrix} 2022 \\ 2587 \\ 3248 \\ 4013 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{e\_0.625} := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot L_{UB}}{r_{s\_0.625}}\right)^2} \cdot A_{0.625}\right]}$$

$$P_{e\_0.625} = \begin{pmatrix} 2654 \\ 3399 \\ 4272 \\ 5284 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

LRFD Article 6.9.4.1.1

$$\frac{P_{e\_0.5}}{P_{o0.5}} = \begin{pmatrix} 1.632 \\ 1.9231 \\ 2.2381 \\ 2.577 \end{pmatrix} \qquad \frac{P_{e\_0.625}}{P_{o0.625}} = \begin{pmatrix} 1.6149 \\ 1.9045 \\ 2.2181 \\ 2.5555 \end{pmatrix}$$

Nominal Axial Structural Resistance of Non-composite member with 1/2-inch wall

If Pe/Po> or = 0.44 then: LRFD Equation 6.9.4.1.1-1

$$P_{nc\_0.5} := \overrightarrow{\left[\left[0.658^{\left(\frac{P_{o0.5}}{P_{e\_0.5}}\right)}\right] \cdot P_{o0.5}\right]}$$

$$P_{nc\_0.5} = \begin{pmatrix} 959 \\ 1082 \\ 1204 \\ 1324 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

Nominal Axial Structural Resistance of Non-composite member with 5/8-inch wall

$$P_{nc\_0.625} := \overrightarrow{\left[\left[0.658^{\left(\frac{P_{o0.625}}{P_{e\_0.625}}\right)}\right] \cdot P_{o0.625}\right]}$$

$$P_{nc\_0.625} = \begin{pmatrix} 1268 \\ 1433 \\ 1595 \\ 1755 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

<u>Factored Axial Structural Resistance of a single Pipe Pile:</u>

Strength limit state resistance factor for pipe piles
in compression, good driving conditions - LRFD 6.5.4.2

$$\phi_c := 0.7$$

Factored Structural Resistance (Pr):
(Composite Section)

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.5} = \begin{pmatrix} 1595 \\ 1892 \\ 2202 \\ 2524 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.625} = \begin{pmatrix} 1801 \\ 2124 \\ 2459 \\ 2808 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Factored Structural Resistance (Pr) for the lower portion of open-ended piles or breached close-ended piles is a function of only the steel shell.
(Non-Composite Section)

$$P_{r\_0.5tip} := \phi_c \cdot P_{nc\_0.5}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 671 \\ 757 \\ 843 \\ 927 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{r\_0.625tip} := \phi_c \cdot P_{nc\_0.625}$$

$$P_{r\_0.625tip} = \begin{pmatrix} 888 \\ 1003 \\ 1116 \\ 1229 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Service and Extreme Limit States Axial Structural Resistance

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

$$\phi := 1.0$$

Factored Compressive Resistance for Service and Extreme Limit States:

$$P_{\_0.5tipf} := \phi \cdot P_{nc\_0.5} \qquad P_{\_0.5tipf} = \begin{pmatrix} 959 \\ 1082 \\ 1204 \\ 1324 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{\_0.625tipf} := \phi \cdot P_{nc\_0.625} \qquad P_{\_0.625tipf} = \begin{pmatrix} 1268 \\ 1433 \\ 1595 \\ 1755 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

COMPUTE GEOTECHNICAL RESISTANCE OF PIPE PILES

Pipe pile capacity based on steel shell end bearing on bedrock - driven through sand, clay and silt.

Bedrock Type:

Granite RQD 90%

Use RQD = 90% and ϕ = 34 to 40 deg (LRFD Table C10.4.6.4-1)

Pipe piles evaluated:
24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Uniaxial Compressive Strength of Sandstone from AASHTO Standard Spec for Highway Bridges 17th Ed. Table 4.4.8.1.2B pg 64
Granite 2100 - 49000 psi Use 20000 psi

$$Q_{uc} := 20000 \cdot psi \qquad \phi_1 := 32 \cdot deg$$

Diameter of piles:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Pipe pile wall thickness:

$$wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

Corrosion loss per MaineDOT BDG:

$$cor := \frac{1}{8} in$$

$$A_{0.5} = \begin{pmatrix} 27.54 \\ 29.89 \\ 32.25 \\ 34.61 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES
with 1/8" corrosion loss

$$A_{0.625} = \begin{pmatrix} 36.52 \\ 39.66 \\ 42.8 \\ 45.95 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES
with 1/8" corrosion loss

LRFD Code specifies Canadian Geotechnical Society Method 1985 for resistance determination of end bearing piles on bedrock. (LRFD Table 10.5.5.2.3-1)
Use Canadian Foundation Manual 4th Edition 2006 Section 18.6.3.3.

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:

$b := dia_{steelcor}$ $b = \begin{pmatrix} 23.75 \\ 25.75 \\ 27.75 \\ 29.75 \end{pmatrix} \cdot in$

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

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: $B_s := 0 \cdot ft$

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

$q_{aA} := Q_{uc} \cdot K_{sp} \cdot d_f$ $q_{aA} = \begin{pmatrix} 1380 \\ 1337 \\ 1300 \\ 1268 \end{pmatrix} \cdot ksf$

Nominal Geotechnical Tip Resistance, R_p:

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

$R_{pA0.5} := \overrightarrow{(3q_{aA} \cdot A_{0.5})}$ $R_{pA0.5} = \begin{pmatrix} 792 \\ 833 \\ 874 \\ 914 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$R_{pA0.625} := \overrightarrow{(3q_{aA} \cdot A_{0.625})}$ $R_{pA0.625} = \begin{pmatrix} 1050 \\ 1105 \\ 1159 \\ 1214 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

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

$R_{f0.5} := \phi_{stat} \cdot R_{pA0.5}$ $R_{f0.5} = \begin{pmatrix} 356 \\ 375 \\ 393 \\ 411 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Strength Limit State
for 1/2" walls

$R_{f0.625} := \phi_{stat} \cdot R_{pA0.625}$ $R_{f0.625} = \begin{pmatrix} 473 \\ 497 \\ 522 \\ 546 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Strength Limit State
for 5/8" walls

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

$R_{fse0.5} := \phi \cdot R_{pA0.5}$ $R_{fse0.5} = \begin{pmatrix} 792 \\ 833 \\ 874 \\ 914 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Service/Extreme
Limit States
for 1/2" walls

$R_{fse0.625} := \phi \cdot R_{pA0.625}$ $R_{fse0.625} = \begin{pmatrix} 1050 \\ 1105 \\ 1159 \\ 1214 \end{pmatrix} \cdot kip$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

Service/Extreme
Limit States
for 5/8" walls

Axial Geotechnical Resistance Piles Driven to Hard Rock 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 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 conditions."

Nominal Structural Tip Resistance, R_p previously calculated:

$$P_{nc\_0.5} = \begin{pmatrix} 959 \\ 1082 \\ 1204 \\ 1324 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{nc\_0.625} = \begin{pmatrix} 1268 \\ 1433 \\ 1595 \\ 1755 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

Determine *Factored* Axial Geotechnical Resistance at the **Strength Limit State**

Apply resistance factor for severe driving, pipe piles from LRFD Article 6.5.4.2 $\phi_{csevere} := 0.6$

Factored Axial Geotechnical Resistance
Strength Limit State

$$P_{strength\_A0.5} := \phi_{csevere} \cdot P_{nc\_0.5}$$

$$P_{strength\_A0.5} = \begin{pmatrix} 575 \\ 649 \\ 722 \\ 794 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{strength\_A0.625} := \phi_{csevere} \cdot P_{nc\_0.625}$$

$$P_{strength\_A0.625} = \begin{pmatrix} 761 \\ 860 \\ 957 \\ 1053 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

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

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

Factored Axial Geotechnical Resistance -
Service and Extreme Limit States

$$P_{serv\_ext\_A0.5} := \phi \cdot P_{nc\_0.5}$$

$$P_{serv\_ext\_A0.5} = \begin{pmatrix} 959 \\ 1082 \\ 1204 \\ 1324 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 1/2" walls

$$P_{serv\_ext\_A0.625} := \phi \cdot P_{nc\_0.625}$$

$$P_{serv\_ext\_A0.625} = \begin{pmatrix} 1268 \\ 1433 \\ 1595 \\ 1755 \end{pmatrix} \cdot kip$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

for 5/8" walls

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)

$f_y := 45 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 40.5 \cdot ksi$ driving stresses in pile cannot exceed 40 ksi

Compute Resistance that can be achieved in a drivability analysis:

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

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Calculate area of steel pipe pile without corrosion loss: There will be no corrosion at time of pile installation.

$$dia_{steel} = \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

$$wall_t = \begin{pmatrix} 0.5 \\ 0.625 \end{pmatrix} \cdot in$$

$$dia_{conccore\_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore\_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore\_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 36.91 \\ 40.06 \\ 43.2 \\ 46.34 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

STEEL AREA
FOR 1/2" PILES
with no corrosion loss

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 45.9 \\ 49.82 \\ 53.75 \\ 57.68 \end{pmatrix} \cdot in^2$$

24 in diameter
26 in diameter
28 in diameter
30 in diameter

STEEL AREA
FOR 5/8" PILES
with no corrosion loss

Pile Size = 24"D x 1/2"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335.00 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
962.0	39.96	4.73	11.0	7.59	29.69
964.0	40.02	4.74	11.0	7.60	29.74
970.0	40.16	4.76	11.2	7.62	29.80
973.0	40.22	4.76	11.3	7.63	29.83
976.0	40.29	4.77	11.4	7.64	29.85
979.0	40.35	4.78	11.5	7.65	29.88
982.0	40.43	4.80	11.6	7.66	29.98
985.0	40.50	4.81	11.7	7.67	30.01
990.0	40.59	4.82	11.8	7.69	30.08
995.0	40.71	4.84	12.0	7.70	30.15

Limit blow count to 11 bpi - driving stress exceeds 40.5 ksi at blow count above 11 bpi

$R_{dr\_24x0.5} := 964 \cdot kip$

Strength Limit State:

$R_{dr\_24x0.5\_strength} := R_{dr\_24x0.5} \cdot \phi_{dyn}$

$R_{dr\_24x0.5\_strength} = 627 \cdot kip$

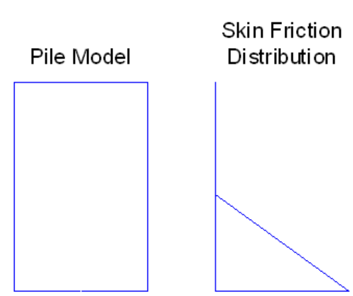
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_24x0.5\_servext} := R_{dr\_24x0.5} \cdot \phi$

$R_{dr\_24x0.5\_servext} = 964 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	36.91	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 26"D x 1/2"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1000.0	39.06	4.93	11.7	7.61	29.13
1010.0	39.27	5.04	12.0	7.63	29.23
1020.0	39.44	5.14	12.3	7.67	29.31
1030.0	39.68	5.24	12.6	7.70	29.48
1040.0	39.90	5.27	13.0	7.73	29.57
1050.0	40.06	5.25	13.3	7.76	29.67
1060.0	40.25	5.31	13.7	7.79	29.74
1070.0	40.50	5.38	14.0	7.82	29.91
1080.0	40.64	5.37	14.4	7.85	29.99
1090.0	40.87	5.26	14.8	7.88	30.08

Limit blow count to 13 bpi - driving stress exceeds 40.5 ksi at blow count above 13 bpi

$R_{dr\_26x0.5} := 1040 \cdot kip$

Strength Limit State:

$R_{dr\_26x0.5\_strength} := R_{dr\_26x0.5} \cdot \phi_{dyn}$

$R_{dr\_26x0.5\_strength} = 676 \cdot kip$

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

$R_{dr\_26x0.5\_servext} := R_{dr\_26x0.5} \cdot \phi$

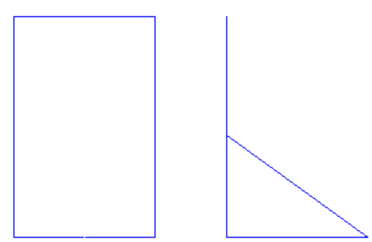
$R_{dr\_26x0.5\_servext} = 1040 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	40.06	in2

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 28"D x 1/2"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1100.0	39.40	4.64	14.6	7.78	29.34
1105.0	39.47	4.64	14.8	7.80	29.39
1110.0	39.56	4.63	15.0	7.82	29.50
1115.0	39.65	4.64	15.2	7.83	29.55
1120.0	39.73	4.65	15.4	7.84	29.58
1125.0	39.81	4.66	15.7	7.86	29.64
1130.0	39.92	4.68	15.9	7.87	29.69
1135.0	40.02	4.72	16.1	7.88	29.80
1140.0	40.08	4.73	16.3	7.90	29.84
1145.0	40.19	4.77	16.5	7.91	29.89

Limit blow count to 15 bpi - per MaineDOT Standard Spec

$$R_{dr\_28x0.5} := 1110 \cdot kip$$

Strength Limit State:

$$R_{dr\_28x0.5\_strength} := R_{dr\_28x0.5} \cdot \phi_{dyn}$$

$$R_{dr\_28x0.5\_strength} = 722 \cdot kip$$

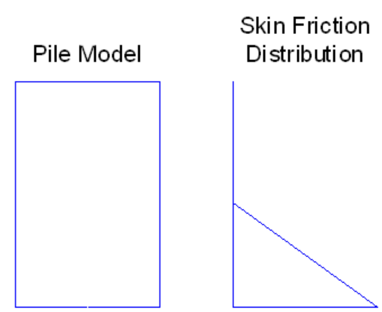
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_28x0.5\_servext} := R_{dr\_28x0.5} \cdot \phi$$

$$R_{dr\_28x0.5\_servext} = 1110 \cdot kip$$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	43.20	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 30"D x 1/2"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale 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
1100.0	37.70	3.99	14.4	7.63	28.58
1105.0	37.77	4.03	14.7	7.64	28.60
1110.0	37.84	4.05	14.9	7.65	28.64
1115.0	37.90	4.08	15.0	7.67	28.73
1120.0	38.04	4.11	15.3	7.68	28.77
1125.0	38.14	4.13	15.4	7.69	28.86
1130.0	38.22	4.16	15.7	7.70	28.90
1135.0	38.29	4.18	15.9	7.71	28.92
1140.0	38.36	4.21	16.1	7.72	29.03
1145.0	38.43	4.23	16.3	7.74	29.05

Limit blow count to 15 bpi - per MaineDOT Standard Specifications

$R_{dr\_30x0.5} := 1115 \cdot kip$

Strength Limit State:

$R_{dr\_30x0.5\_strength} := R_{dr\_30x0.5} \cdot \phi_{dyn}$

$R_{dr\_30x0.5\_strength} = 725 \cdot kip$

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

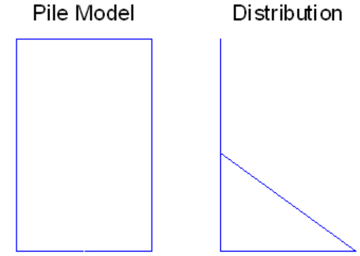
$R_{dr\_30x0.5\_servext} := R_{dr\_30x0.5} \cdot \phi$

$R_{dr\_30x0.5\_servext} = 1115 \cdot kip$

APE D 36-32

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	46.34	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 24"D x 5/8"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1100.0	38.05	4.00	14.2	7.69	28.88
1110.0	38.19	4.05	14.6	7.71	29.01
1120.0	38.41	4.11	15.0	7.74	29.15
1130.0	38.58	4.17	15.4	7.76	29.21
1140.0	38.74	4.22	15.8	7.79	29.34
1150.0	38.90	4.28	16.2	7.82	29.47
1160.0	39.11	4.33	16.7	7.84	29.61
1170.0	39.26	4.38	17.1	7.87	29.74
1180.0	39.38	4.42	17.5	7.89	29.87
1190.0	39.52	4.46	18.1	7.91	29.93

Limit blow count to 15 bpi - per MaineDOT Standard Specifications

$R_{dr\_24x0.625} := 1120 \cdot kip$

Strength Limit State:

$R_{dr\_24x0.625\_strength} := R_{dr\_24x0.625} \cdot \phi_{dyn}$

$R_{dr\_24x0.625\_strength} = 728 \cdot kip$

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

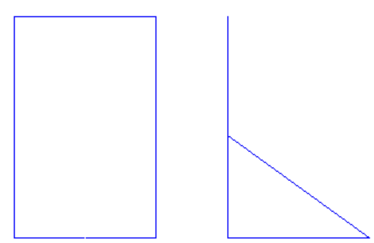
$R_{dr\_24x0.625\_servext} := R_{dr\_24x0.625} \cdot \phi$

$R_{dr\_24x0.625\_servext} = 1120 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	45.90	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 26"D x 5/8"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1130.0	36.75	3.87	14.9	7.62	28.56
1131.0	36.70	3.88	14.9	7.63	28.60
1132.0	36.78	3.88	15.0	7.63	28.57
1133.0	36.76	3.89	15.0	7.63	28.54
1134.0	36.81	3.89	15.0	7.63	28.59
1135.0	36.81	3.90	15.1	7.63	28.57
1136.0	36.85	3.90	15.1	7.64	28.61
1137.0	36.84	3.91	15.2	7.64	28.58
1138.0	36.79	3.91	15.2	7.64	28.63
1139.0	36.89	3.90	15.2	7.64	28.67

Limit blow count to 15 bpi - per MaineDOT Standard Specifications

$R_{dr\_26x0.625} := 1134 \cdot kip$

Strength Limit State:

$R_{dr\_26x0.625\_strength} := R_{dr\_26x0.625} \cdot \phi_{dyn}$

$R_{dr\_26x0.625\_strength} = 737 \cdot kip$

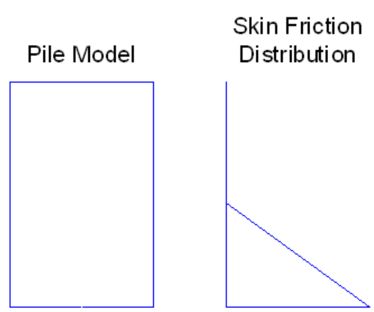
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_26x0.625\_servext} := R_{dr\_26x0.625} \cdot \phi$

$R_{dr\_26x0.625\_servext} = 1134 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	49.82	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 28"D x 5/8"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation — 22-Jul-2013
18335 Auburn Oakdale NB — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1100.0	34.50	3.47	13.5	7.44	27.58
1110.0	34.68	3.47	13.7	7.46	27.72
1120.0	34.81	3.51	14.0	7.49	27.77
1130.0	34.97	3.54	14.3	7.50	27.92
1140.0	35.15	3.58	14.6	7.52	27.98
1150.0	35.30	3.61	15.0	7.54	28.05
1160.0	35.49	3.64	15.3	7.56	28.19
1170.0	35.66	3.71	15.6	7.58	28.25
1180.0	35.82	3.80	16.0	7.60	28.32
1190.0	35.98	3.89	16.3	7.62	28.46

Limit blow count to 15 bpi - per MaineDOT Standard Specifications

$R_{dr\_28x0.625} := 1150 \cdot kip$

Strength Limit State:

$R_{dr\_28x0.625\_strength} := R_{dr\_28x0.625} \cdot \phi_{dyn}$

$R_{dr\_28x0.625\_strength} = 747 \cdot kip$

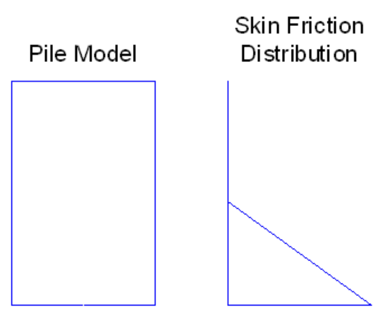
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_28x0.625\_servext} := R_{dr\_28x0.625} \cdot \phi$

$R_{dr\_28x0.625\_servext} = 1150 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	53.75	in2



Res. Shaft = 10 %
(Proportional)

Pile Size = 30"D x 5/8"W

Assume Contractor will use an APE D36-26 hammer on lowest fuel setting

Pile Bent Pier: Unbraced length = length in air + length in water + scour depth 5.5 ft = 21.5 ft.

State of Maine Dept. Of Transportation 22-Jul-2013
18335 Auburn Oakdale NB GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1170.0	34.04	3.54	14.8	7.46	27.56
1172.0	34.05	3.56	14.8	7.46	27.57
1174.0	34.05	3.57	14.9	7.47	27.58
1176.0	34.10	3.59	15.0	7.47	27.58
1178.0	34.17	3.60	15.0	7.48	27.66
1180.0	34.23	3.61	15.1	7.48	27.68
1182.0	34.28	3.63	15.1	7.48	27.68
1184.0	34.27	3.64	15.2	7.49	27.69
1186.0	34.33	3.65	15.3	7.49	27.70
1188.0	34.34	3.66	15.4	7.50	27.70

Limit blow count to 15 bpi - per MaineDOT Standard Specifications

$R_{dr\_30x0.625} := 1178 \cdot kip$

Strength Limit State:

$R_{dr\_30x0.625\_strength} := R_{dr\_30x0.625} \cdot \phi_{dyn}$

$R_{dr\_30x0.625\_strength} = 766 \cdot kip$

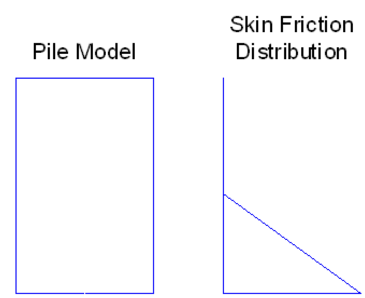
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_30x0.625\_servext} := R_{dr\_30x0.625} \cdot \phi$

$R_{dr\_30x0.625\_servext} = 1178 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	5.00	kips
Hammer Cushion	121030	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	40.00	ft
Pile Penetration	18.50	ft
Pile Top Area	57.68	in2



Res. Shaft = 10 %
(Proportional)

Settlement Analyses:

Reference: FHWA Soils and Foundations Reference Manual - Volume 1 (FHWA NHI-06-088) Hough pg 7-16

The roadway grade at centerline may be raised by as much as 2.0 feet .
Look at a simplified soil profile based on BB-ALAR-103:

Finished Grade

Proposed Fill - Look at 2.0 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Existing Grade

Existing Fill - fine to coarse sand

$H_1 := 12 \cdot ft$ $\gamma_{fill\_sand} := 125 \cdot pcf$ $N_{fill\_sand} := 18$

Groundwater at top of native sand
$\gamma_w := 62.4pcf$

Native Sand - loose fine to coarse sand

$H_2 := 17.0 \cdot ft$ $\gamma_{native\_sand} := 125 \cdot pcf$ $N_{native\_sand} := 4$

Clayey Silt and Silty Clay
Total Layer height: H = 40.0 ft - divide into 4 layers

$H_3 := 40.0 \cdot ft$

$H_{3claysilt1} := 10.0 \cdot ft$ $\gamma_{claysilt} := 115 \cdot pcf$ $C_{c\_claysilt1} := 0.2138$ $C_{r\_claysilt1} := 0.0236$ $e_{oclaysilt1} := 0.91$

$H_{3claysilt2} := 10.0 \cdot ft$ $C_{c\_claysilt2} := 0.2138$ $C_{r\_claysilt2} := 0.0236$ $e_{oclaysilt2} := 0.91$

$H_{3claysilt3} := 10.0 \cdot ft$ $C_{c\_claysilt3} := 0.4102$ $C_{r\_claysilt3} := 0.0251$ $e_{oclaysilt3} := 1.05$

$H_{3claysilt4} := 10.0 \cdot ft$ $C_{c\_claysilt4} := 0.4102$ $C_{r\_claysilt4} := 0.0251$ $e_{oclaysilt4} := 1.05$

Glacial Till - Sand - fine sand, medium dense

$H_4 := 3.0 \cdot ft$ $\gamma_{till} := 125 \cdot pcf$ $N_{till} := 28$

BEDROCK

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name:	Oakdale NB	Client:	Auburn
Project Number:	18335.00	Project Manager	Benoit
Date:	5/15/13	Computed by:	km

Embank. slope a = 21.00(ft)
Embank. width b = 42.00(ft)
p load/unit area = 250.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 22.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	250.00
2.00	246.58
4.00	239.63
6.00	231.84
8.00	223.57
10.00	214.94
12.00	206.11
14.00	197.22
16.00	188.42
18.00	179.83
20.00	171.54
22.00	163.63
24.00	156.12
26.00	149.04
28.00	142.38
30.00	136.14
32.00	130.31
34.00	124.86
36.00	119.76
38.00	115.00
40.00	110.55
42.00	106.39
44.00	102.49
46.00	98.84
48.00	95.42
50.00	92.20
52.00	89.18
54.00	86.33
56.00	83.64
58.00	81.11
60.00	78.71
62.00	76.45
64.00	74.30
66.00	72.27
68.00	70.33
70.00	68.50
72.00	66.75

at 6.0 ft $\Delta\sigma_{zfill\_sand} := 231.84 \cdot psf$

at 20.5 ft $\Delta\sigma_{znative\_sand} := 169.56 \cdot psf$

at 34.0 ft $\Delta\sigma_{zclaysilt1} := 124.86 \cdot psf$

at 44.0 ft $\Delta\sigma_{zclaysilt2} := 102.49 \cdot psf$

at 54.0 ft $\Delta\sigma_{zclaysilt3} := 86.33 \cdot psf$

at 64.0 ft $\Delta\sigma_{zclaysilt4} := 74.30 \cdot psf$

at 70.5 ft $\Delta\sigma_{ztill} := 68.06 \cdot psf$

Existing Fill/Sand

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

Calculate vertical stress: $\sigma_{fill\_sand\_o} := \frac{H_1}{2} \cdot (\gamma_{fill\_sand})$ $\sigma_{fill\_sand\_o} = 750 \cdot psf$ at mid-point

Corrected SPT N_{60}-value (bpf)

$N_{fill\_sand} = 18$ $C_{N\_fill\_sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{fill\_sand\_o}}\right)$ Eq. 10.4.6.2.4 LRFD

$C_{N\_fill\_sand} = 1.3298$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_fill\_sand} \cdot N_{fill\_sand}$ $N1_{60} = 24$
From Eq 3-3 pg 3-36

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

Bearing Capacity Index: $C1_{fill} := 77$

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

$\Delta\sigma_{zfill\_sand} = 231.84 \cdot psf$

Native Sand

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

Calculate vertical stress: $\sigma_{native\_sand\_o} := H_1 \cdot (\gamma_{fill\_sand}) + \frac{H_2}{2} \cdot (\gamma_{native\_sand} - \gamma_w)$ $\sigma_{native\_sand\_o} = 2032.1 \cdot psf$ at mid-point

Corrected SPT N_{60}-value (bpf)

$N_{native\_sand} = 4$ $C_{N\_native\_sand} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{native\_sand\_o}}\right)$ Eq. 10.4.6.2.4 LRFD

$C_{N\_native\_sand} = 0.9965$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_native\_sand} \cdot N_{native\_sand}$ $N1_{60} = 4$
From Eq 3-3 pg 3-36

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

Bearing Capacity Index: $C1_{native} := 36$

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

$\Delta\sigma_{znative\_sand} = 169.56 \cdot psf$

Clayey Silt / Silty Clay - 4 layers

Clayey Silt Layer 1:

Average values from lab data: $e_{oclaysilt1} = 0.91$ $C_{c\_claysilt1} = 0.2138$

$$\sigma_{claysilt1o} := \frac{H_{3claysilt1}}{2} \cdot (\gamma_{claysilt} - \gamma_w) + H_2 \cdot (\gamma_{native\_sand} - \gamma_w) + H_1 \cdot (\gamma_{fill\_sand})$$ $\sigma_{claysilt1o} = 2827.2 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclaysilt1} = 124.86 \cdot psf$

Clayey Silt Layer 2:

Average values from lab data: $e_{oclaysilt2} = 0.91$ $C_{c\_claysilt2} = 0.2138$

$$\sigma_{claysilt2o} := \frac{H_{3claysilt2}}{2} \cdot (\gamma_{claysilt} - \gamma_w) + H_{3claysilt1} \cdot (\gamma_{claysilt} - \gamma_w) + H_2 \cdot (\gamma_{native\_sand} - \gamma_w) + H_1 \cdot (\gamma_{fill\_sand})$$

$\sigma_{claysilt2o} = 3353.2 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclaysilt2} = 102.49 \cdot psf$

Clayey Silt Layer 3:

Average values from lab data: $e_{oclaysilt3} = 1.05$ $C_{c\_claysilt3} = 0.4102$

$$\sigma_{claysilt3o} := \frac{H_{3claysilt3}}{2} \cdot (\gamma_{claysilt} - \gamma_w) + (H_{3claysilt2} + H_{3claysilt1}) \cdot (\gamma_{claysilt} - \gamma_w) + H_2 \cdot (\gamma_{native\_sand} - \gamma_w) + H_1 \cdot (\gamma_{fill\_sand})$$

$\sigma_{claysilt3o} = 3879.2 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclaysilt3} = 86.33 \cdot psf$

Clayey Silt Layer 4:

Average values from lab data: $e_{oclaysilt4} = 1.05$ $C_{r\_claysilt4} = 0.0251$

$$\sigma_{claysilt4o} := \frac{H_{3claysilt4}}{2} \cdot (\gamma_{claysilt} - \gamma_w) + (30 \cdot ft) \cdot (\gamma_{claysilt} - \gamma_w) + H_2 \cdot (\gamma_{native\_sand} - \gamma_w) + H_1 \cdot (\gamma_{fill\_sand})$$

$\sigma_{claysilt4o} = 4405.2 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclaysilt4} = \blacksquare \cdot psf$

Glacial Till

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

Calculate vertical stress:

$$\sigma_{till\_o} := \frac{H_4}{2}(\gamma_{till} - \gamma_w) + H_3 \cdot (\gamma_{claysilt} - \gamma_w) + H_2 \cdot (\gamma_{native\_sand} - \gamma_w) + H_1 \cdot (\gamma_{fill\_sand}) \qquad \sigma_{till\_o} = 4762.1 \cdot psf$$

at mid-point

Corrected SPT N_{60}-value (bpf)

$$N_{till} = 28 \qquad C_{N\_till} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{till\_o}}\right) \qquad C_{N\_till} = 0.7117$$

Eq. 10.4.6.2.4 LRFD

Corrected N-value normalized for overburden $N1_{60}$:
From Eq 3-3 pg 3-36

$$N1_{60} := C_{N\_till} \cdot N_{till} \qquad N1_{60} = 20$$

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

Bearing Capacity Index: $C1_{till} := 68$

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

$$\Delta\sigma_{ztill} = 68.06 \cdot psf$$

Calculate Settlement:

Fill/Sand: $$\Delta H_1 := H_1 \cdot \frac{1}{C1_{fill}} \cdot \log\left(\frac{\sigma_{fill\_sand\_o} + \Delta\sigma_{zfill\_sand}}{\sigma_{fill\_sand\_o}}\right) \qquad \Delta H_1 = 0.2188 \cdot in$$

Native Sand: $$\Delta H_2 := H_2 \cdot \frac{1}{C1_{native}} \cdot \log\left(\frac{\sigma_{native\_sand\_o} + \Delta\sigma_{znative\_sand}}{\sigma_{native\_sand\_o}}\right) \qquad \Delta H_2 = 0.1972 \cdot in$$

Clayey Silt Layer 1: $$\Delta H_{3cs1} := H_{3claysilt1} \cdot \left(\frac{C_{c\_claysilt1}}{1 + e_{oclaysilt1}}\right) \cdot \log\left(\frac{\sigma_{claysilt1o} + \Delta\sigma_{zclaysilt1}}{\sigma_{claysilt1o}}\right) \qquad \Delta H_{3cs1} = 0.2521 \cdot in$$

Clayey Silt Layer 2: $$\Delta H_{3cs2} := H_{3claysilt2} \cdot \left(\frac{C_{c\_claysilt2}}{1 + e_{oclaysilt2}}\right) \cdot \log\left(\frac{\sigma_{claysilt2o} + \Delta\sigma_{zclaysilt2}}{\sigma_{claysilt2o}}\right) \qquad \Delta H_{3cs2} = 0.1756 \cdot in$$

Clayey Silt Layer 3: $$\Delta H_{3cs3} := H_{3claysilt3} \cdot \left(\frac{C_{c\_claysilt3}}{1 + e_{oclaysilt3}}\right) \cdot \log\left(\frac{\sigma_{claysilt3o} + \Delta\sigma_{zclaysilt3}}{\sigma_{claysilt3o}}\right) \qquad \Delta H_{3cs3} = 0.2295 \cdot in$$

Clayey Silt Layer 4: $$\Delta H_{3cs4} := H_{3claysilt4} \cdot \left(\frac{C_{r\_claysilt4}}{1 + e_{oclaysilt4}}\right) \cdot \log\left(\frac{\sigma_{claysilt4o} + \Delta\sigma_{zclaysilt4}}{\sigma_{claysilt4o}}\right) \qquad \Delta H_{3cs4} = 0.0107 \cdot in$$

Glacial Till - Sand: $$\Delta H_4 := H_4 \cdot \frac{1}{C1_{till}} \cdot \log\left(\frac{\sigma_{till\_o} + \Delta\sigma_{ztill}}{\sigma_{till\_o}}\right) \qquad \Delta H_4 = 0.0033 \cdot in$$

Total Settlement = $$\Delta H_T := \Delta H_1 + \Delta H_2 + \Delta H_{3cs1} + \Delta H_{3cs2} + \Delta H_{3cs3} + \Delta H_{3cs4} + \Delta H_4 \qquad \Delta H_T = 1.0872 \cdot in$$

Consolidation Settlement = $$\Delta H_{claysilt} := \Delta H_{3cs1} + \Delta H_{3cs2} + \Delta H_{3cs3} + \Delta H_{3cs4} \qquad \Delta H_{claysilt} = 0.6679 \cdot in$$

Per LRFD Article 3.11.8 - If the settlement of the soil layer is 0.4 in or greater relative to the pile or shaft, downdrag can be assumed to fully develop.

Since the calculated settlement exceeds 0.4 inches: DOWNDRAG SHOULD BE CONSIDERED

Time Rate of Settlement:

Determine the time for 90% consolidation for primary settlement
Reference: FHWA Soils and Foundation Reference Manual - Volume 1 page 7-30

Thickness of the silt/clay layer = $H_{siltclay} := 40.0 \cdot ft$

Assume double drainage due to presence of sand layers above and below the clay layer.

$H_{scv} := 20 \cdot ft$

Time factor from Table on page 7-32
At 90% primary consolidation
$T_v := 0.848$

Coefficient of consolidation from lab data: $C_v := 1.02 \cdot 10^{-6} \cdot \frac{ft^2}{sec}$ $C_v = 0.0881 \cdot \frac{ft^2}{day}$

Time rate of settlement to achieve 90% Primary Consolidation

$$t_{90} := \frac{T_v \cdot H_{scv}^2}{C_v} \qquad t_{90} = 3848.947 \cdot day \qquad year := 365 \cdot day$$

$$t_{90} = 10.5451 \cdot year$$

Determination of Downdrag:

Reference: Construction Behavior Report: Effects of Bitumen Coating on the Axial and Lateral Loadings of Abutment Piles Subject to Downdrag, Technical Report 95-4 December 1998 by Leif A. Dixon and Thomas C. Sandford

Use beta method to determine downdrag

Granular soil (Dixon & Sandford) pg 152 $\beta_{gr} := 0.11$

Silt/Clay (use 1/2 the NAVFAC Values of 0.2 to 0.25) $\beta_{clay} := 0.10$

Assumed values:

Unit weight of existing sand fill $\gamma_{sand} := 125 \cdot pcf$

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

Unit weight of silt/clay $\gamma_{siltclay} := 115 \cdot pcf$

Effective unit weight of silt/clay $\gamma'_{siltclay} := \gamma_{siltclay} - \gamma_w$ $\gamma'_{siltclay} = 52.6 \cdot pcf$

Stress from overburden material. Overburden consists of approximately 29 feet of sand on 40 feet of silt fill over glacial till. Water table is at the top of the silt layer.

Change in overburden Stress due to fill = $\sigma_{v\_ob} := 2.0 \cdot ft \cdot \gamma_{sand}$ $\sigma_{v\_ob} = 250 \cdot psf$

at 14.5 ft $\Delta\sigma_{14.5} := 195.02 \cdot psf$

at 49.0 ft $\Delta\sigma_{49.0} := 93.81 \cdot psf$

values from STRESS output on pg 40 of these calculations

Effective vertical stress in middle of each layer

Total thickness of each stratum

$D_{sand} := 29 \cdot ft$ $D_{siltclay} := 40 \cdot ft$

$$\sigma'_{v\_sand} := \Delta\sigma_{14.5} + \frac{D_{sand}}{2} \cdot \gamma_{sand} \qquad \sigma'_{v\_sand} = 2007.5 \cdot psf$$

$$\sigma'_{v\_silt} := \Delta\sigma_{49.0} + D_{sand} \cdot \gamma_{sand} + \frac{D_{siltclay}}{2} \cdot \gamma_{siltclay} \qquad \sigma'_{v\_silt} = 6018.8 \cdot psf$$

Pile parameters:

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

Box perimeter: $P := 2 \cdot (d + b)$

$$P = \begin{pmatrix} 47.65 \\ 48.69 \\ 56.39 \\ 57.05 \\ 58.19 \end{pmatrix} \cdot in$$

Magnitude of maximum downdrag, considered over entire clay thickness

$$Q_{dd\_nom} := \left(D_{sand} \cdot \sigma'_{v\_sand} \cdot \beta_{gr} + D_{siltclay} \cdot \sigma'_{v\_silt} \cdot \beta_{clay}\right) \cdot P$$

Nominal Downdrag: $Q_{dd\_nom} = \begin{pmatrix} 121 \\ 124 \\ 143 \\ 145 \\ 148 \end{pmatrix} \cdot kip$

For these piles:

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

Based on LRFD Table 3.4.1-2, a downdrag load factor of 1.05 is recommended

$$\gamma_{p\_DD} := 1.05$$

Calculate Factored Downdrag load:

Factored Downdrag: $Q_{DD} := Q_{dd\_nom} \cdot \gamma_{p\_DD}$

$$Q_{DD} = \begin{pmatrix} 127 \\ 130 \\ 150 \\ 152 \\ 155 \end{pmatrix} \cdot kip$$

For these piles:

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

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 = 72.4 inches

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

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.

****

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

Use Modberg Frost Depth = 5.5 feet for design

Seismic:

BB-ALAR-101					BB-ALAR-102					BB-ALAR-103				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
2	42	sand	3	0.071	3	8	sand	4	0.500	3	47	gravel	4	0.085
6	10	sand	5	0.500	7	1	silt	5	5.000	6	18	sand	4	0.222
11	3	sand	5	1.667	12	1	silt	6	6.000	11	1	sand	5	5.000
16.5	13	silty sand	5	0.385	17	2	silty clay	6	3.000	16	1	sand	5	5.000
21	6	sand	5	0.833	24	3	silty clay	5	1.667	20	4	sand	5	1.250
26	3	silt	5	1.667	27	5	silty clay	5	1.000	25	5	sand	5	1.000
31	3	silt	10	3.333	32	10	silty clay	5	0.500	30	1	clayey silt	5	5.000
41	3	clayey silt	8	2.667	38.5	32	gravel	4	0.125	35	1	clayey silt	5	5.000
46	6	clayey silt	3	0.500	41	19	sand	2.5	0.132	40	2	clayey silt	5	2.500
48	6	clayey silt	3	0.500	42.5	100	**bedrock**	57.5	0.575	45	3	clayey silt	5	1.667
51	6	silty clay	3	0.500						50	3	clayey silt	5	1.667
56	10	silty clay	4	0.400						55	6	clayey silt	5	0.833
61	37	sand	4	0.108						60	7	silty clay	5	0.714
64.5	60	sand	2	0.033						65	8	silty clay	5	0.625
65	100	**bedrock**	35	0.350						70	28	gravel	4	0.143
										72	100	**bedock**	28	0.280
SUM			**100**	**13.514**	**SUM**			**100**	**18.498**	**SUM**			**100**	**30.986**
			di/di/N	**7.400**				**di/di/N**	**5.406**				**di/di/N**	**3.227**
Note: Weight of rod (WOR) and weight of hammer (WOH) values are taken as N=1.												SUM	Nav.	5.34427
												Nav < 15 bpf; Site Class E		

18335 Auburn Oakdale Bridge NB
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 E - Fpga = 2.50, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.221	As - Site Class E
0.2	0.442	SDs - Site Class E
1.0	0.163	SD1 - Site Class E

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.

Calculate Seismic Active Earth Pressure Coefficient, K_{AE}

friction angle of soil $\phi_s := 32 \cdot deg$

horizontal seismic acceleration coefficient (dim.), k_h $k_h := 0.221$ A_s from previous page

vertical seismic acceleration coefficient (dim.), k_v $k_v := 0$ typically set to 0

$$\theta_{MO} := \mathrm{atan}\left[\frac{k_h}{(1 - k_v)}\right] \qquad \theta_{MO} = 0.2175$$

Slope of wall (abutment) to the vertical, β $\beta_s := 0 \cdot deg$ abutment is vertical

wall backface interface friction angle, δ
δ=0.67*friction angle of soil $\delta_s := 23 \cdot deg$

backfill slope angle, i (degrees) $i_s := 0$ roadway is flat

Calculate seismic active earth pressure coefficient:

$$K_{AE} := \frac{\cos(\phi_s - \theta_{MO} - \beta_s)^2}{\cos(\theta_{MO}) \cdot \cos(\beta_s)^2 \cdot \cos(\delta_s + \beta_s + \theta_{MO})} \cdot \left(1 + \sqrt{\frac{\sin(\phi_s + \delta_s) \cdot \sin(\phi_s - \theta_{MO} - i_s)}{\cos(\delta_s + \beta_s + \theta_{MO}) \cdot \cos(i_s - \beta_s)}}\right)^{-2}$$

$$K_{AE} = 0.4474$$