

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

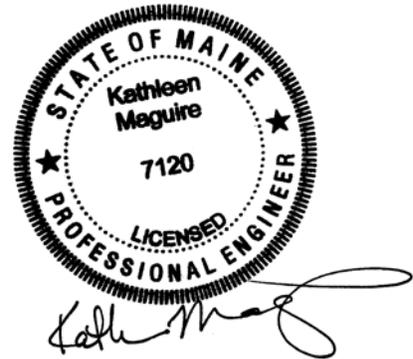
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**NUTTER'S BRIDGE
OVER LITTLEFIELD RIVER
ALFRED, MAINE**

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Nutter's Bridge on Back Road over Littlefield River in Alfred, Maine. The proposed replacement bridge will consist of a 43 foot single span prefabricated superstructure supported on short H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. However, it should be noted that the borings encountered layers of wood, cobbles and granite blocks at the abutment locations. Construction problems should therefore be anticipated with respect to piles being out of position or refusing on obstructions above the specified tip elevation. The use of short pile supported integral abutments is under consideration by the MaineDOT Bridge Program. Initial results indicate that although fixity is not achieved for piles less than 13 feet long, the piles do not experience stresses larger than those seen by longer integral abutment piles. Short piles supporting integral abutments should be designed in accordance with the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B and the MaineDOT Bridge Design Guide. The pile should be end bearing, driven to the required resistance on or within the bedrock. Using the assumption that 50 ksi steel will be used; the factored axial structural and geotechnical resistances of the piles exceed the factored axial drivability resistance and therefore the drivability resistance governs. The Contractor is required to perform a wave equation analysis and dynamic pile analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the maximum factored axial pile load divided by a resistance factor of 0.52. The maximum factored pile load should be shown on the plans. The piles should be oriented for weak axis bending. Driven piles should be fitted with a Rock Injector "H" Bearing Pile Point, manufactured by Titus Steel, Co. to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforce concrete failure. Strength limit state design shall also consider change in foundation conditions and pile group resistance after scour due to the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination. Extreme limit state design checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads.

Cast-in-place integral abutments sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure state. The Coulomb passive earth pressure coefficient, K_p , of 6.89 is recommended. Developing full passive requires displacements of

the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced Coulomb passive earth pressure coefficient, but not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p , of 3.25. A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, $\gamma_{EH} = 1.50$.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required 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.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. The approach slab should be positively connected to the abutment.

Bearing Resistance - In the event that any structure foundation is founded on spread footings bearing on native sand or bedrock the footings shall be proportioned to provide stability against bearing capacity failure. The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 27 ksf. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination.

Bearing resistance for foundations on fill soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 7 ksf for bases from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

Scour and Riprap - If using integral abutments at the site, pile lengths will be short and, therefore, scour protection will be critical. The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

Settlement - The horizontal and vertical alignment of the proposed bridge will be close to the existing bridge alignments. The bridge width will be less than state standards in order to match the existing corridor width. Post-construction settlements are anticipated to be negligible.

Frost Protection - Any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils and not those founded on bedrock. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

Seismic Design Considerations – Seismic analysis is not required for single span bridges regardless of seismic zone. However, superstructure connections and minimum support lengths should be designed in accordance with LRFD requirements.

Construction Considerations - Boulders, cobbles and wood were encountered within the existing fill at the site. There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions should be specified as incidental to related pile pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

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

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

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of Nutter's Bridge on Back Road over Littlefield River in Alfred, York County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the subsurface information obtained at the site, geotechnical design parameters, and foundation recommendations.

The existing bridge was constructed in 1950 and consists of a 25 foot long single-span structure with rolled steel stringers. The bridge abutments are comprised if a combination of stacked granite blocks and cast-in-place concrete. There is no available information about the existing abutment foundations. The 2009 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the north abutment has a large bulge in the center with large voids and the stones at the southeast corner of the abutment are shifted and appear unstable. Both abutments have cracked and broken granite blocks and undermined areas were observed at both abutments. Recommendations were made to monitor the granite block abutments for any movement. The maintenance inspection reports indicate that the bridge superstructure is in "poor" condition (rating of 4), the substructure is in "serious" condition (rating of 3) and the deck is in "fair" condition (rating of 5). The Bridge Sufficiency Rating is 30. The bridge has a scour critical rating of "U" meaning that the bridge has unknown foundations that have not been evaluated for scour. It is understood that the existing bridge will be completely removed and replaced.

The proposed bridge will consist of a 43 foot long single-span prefabricated concrete bridge supported on short H-pile supported integral abutments. This abutment type is considered experimental and is proposed based on the results to date of MaineDOT's short-pile integral abutment study. Constructability issues associated with driving the piles through layers of wood, granite blocks and cobbles should be anticipated. The horizontal and vertical alignment of the proposed bridge will be close to the existing bridge alignments. The bridge width will be less than state standards in order to match the existing corridor width. The existing bridge will be closed to traffic during construction.

2.0 GEOLOGIC SETTING

Nutter's Bridge on Back Road in Alfred crosses Littlefield River approximately 0.2 miles north of Route 111 as shown on Sheet 1 - Location Map found at the end of this report. Littlefield River flows in a southerly direction into Round Pond.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of ice contact glaciofluvial deposits and glaciomarine sediments. The ice contact glaciofluvial deposits are generally comprised of sand gravel and silt. These soils are generally deposited in areas where the topography is flat-topped kame terraces and deltas which are locally kettled and bounded by steep sides or hummocky terrain with numerous kames and kettles. These soils were generally deposited by melt water streams adjacent to stagnant glacial ice. The glaciomarine

sediments are generally comprised of sand, gravel, and minor amounts of silt. These soils are generally deposited in flat to moderately sloping areas that may be kettled where deposited over stagnant ice blocks. These soils were generally deposited where glacial meltwater streams and currents entered the sea. The project is located in the area of the inland marine limit of the late-glacial marine submergence, as mapped by Thompson and others (1983).

According to the Bedrock Geologic Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Carboniferous age muscovite-biotite granite. This unmetamorphosed plutonic rock is identified as the Lyman pluton.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Test boring BB-ALR-101 was drilled behind the location of Abutment No. 1 (south). Test borings BB-ALR-102 and BB-ALR-102A were drilled behind the location of Abutment No. 2 (north). The exploration locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The borings were drilled between October 29 and 30, 2009 using the Maine Department of Transportation (MaineDOT) drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on Sheet 3 - Boring Logs found end of this report.

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

The bedrock was cored in the borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of nine (9) standard grain size analyses including natural moisture content. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the abutments consisted of fill materials overlying native sand overlying bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

Fill Materials. Beneath the pavement, layers of fill materials were encountered. The fill layers encountered were:

- An upper fill layer comprised of brown, damp to moist, fine to coarse SAND, with some to trace gravel, and some to trace silt was encountered in all of the borings.
- A lower fill layer comprised of dark brown, wet, silty fine to coarse sand, with trace gravel and little organics was encountered in boring BB-ALR-101.
- A layer of wood fill approximately 2.0 feet thick was encountered in boring BB-ALR-101.
- A layer of granite blocks and cobbles approximately 5.4 feet thick was encountered in boring BB-ALR-102A.

The overall thickness of the fill layer ranged from approximately 13.5 feet in boring BB-ALR-101 to approximately 12.4 feet in boring BB-ALR-102A. Corrected SPT N-values in the upper fill layer ranged from 13 to 87 blows per foot (bpf) indicating that the soil is medium dense to very dense in consistency. Water contents from five (5) samples obtained within the upper fill layer range from approximately 3% to 10%. Five (5) grain size analyses conducted on samples from the upper fill layer indicate that the soil is classified as an A-1-b or A-2-4 by the AASHTO Classification System and a SM or SP-SM by the Unified Soil Classification System.

One corrected SPT N-value in the lower fill layer was 6 bpf indicating that the soil is loose in consistency. A water content from one (1) sample obtained within the lower fill layer was approximately 69%. One (1) grain size analyses conducted on a sample from the lower fill layer indicates that the soil is classified as an A-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

Native Sand. Beneath the fill materials a layer of native sand was encountered. This layer was found to be brown, wet, fine to coarse sand, with some gravel, little silt and some to trace organics. The thickness of the native sand layer ranged from approximately 7.8 feet in boring BB-ALR-101 to approximately 9.8 feet in boring BB-ALR-102. Corrected SPT N-values obtained in the native sand layer ranged from 8 to 73 bpf indicating that the soil is loose to very dense in consistency. Water contents from three (3) samples obtained within this layer range from approximately 12% to 23%. Three (3) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-1-b or A-2-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

Bedrock. Bedrock was encountered and cored in two (2) of the borings. Table 5-1 summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number/ Location	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
BB-ALR-101/ Abutment No. 1	21.3 feet	215.6 feet	81 - 100%
BB-ALR-102A/ Abutment No. 2	22.2 feet	214.1 feet	78 - 92%

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

The bedrock at the site can be identified as grey, white and greenish grey, fine-grained, fresh, muscovite-biotite granite with feldspar, quartz, and mica. The bedrock is a part of the Lyman pluton. The RQD of the bedrock ranged from 78 to 100% indicating a rock of good to excellent quality.

Groundwater. Groundwater was observed at a depths ranging from approximately 12.0 to 13.0 feet below the ground surface at the boring locations. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

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

- Cast-in-place concrete integral stub abutments supported on driven steel H-piles
- Precast concrete integral stub abutments supported on driven steel H-piles
- Cast-in-place, full height abutments founded on spread footings bearing on native sand
- Cast-in-place, full height abutments founded on spread footings bearing on bedrock

After consideration of all of the alternatives, short pile integral abutments located behind the existing abutments were selected because they require minimal future maintenance. However, from a constructability perspective a driven pile foundation is not ideal at the Nutter's Bridge site. The presence of a layer of wood at the proposed Abutment No. 1 location and a granite block and cobble layer at the proposed Abutment No. 2 location will impede driving piles within acceptable tolerances and to the required tip elevation. Construction delays in order to excavate obstructions or preauger or drive spud piles should be anticipated. A note on the Plans should alert the Contractor to these issues and specify that clearing obstructions will be incidental to related pile pay items. Pile order lengths should include additional pile to replace pile lengths damaged during driving. The presence of relatively shallow bedrock at the site indicates that integral abutment piles would typically be socketed in bedrock to achieve fixity. Preliminary results of a MaineDOT short-pile integral abutment study show that fixity may not be necessary.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for stub abutments founded on a single row of integral H-piles driven to bedrock which has been identified as the optimal substructure for the site. The use of short pile supported integral abutments is under consideration by the MaineDOT Bridge Program. Initial results indicate that although fixity is not achieved for piles less than 13 feet long, the piles do not experience stresses larger than those seen by longer integral abutment piles. The current study¹ indicates that the use of short pile supported integral abutments for bridges with spans not exceeding 115 feet is applicable.

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 design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with Rock Injector "H" Bearing Pile Point, manufactured by Titus Steel, Co. to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption. Special Provision 501 Foundation Piles – Rock Injector Pile Tip is provided in Appendix D – Special Provisions found at the end of this report.

¹ MaineDOT Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase I"

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 Free Length
Abutment #1 BB-ALR-101	228.2 feet	21.3 feet	215.6 feet	13 feet
Abutment #2 BB-ALR-102A	227.7 feet	22.2 feet	214.1 feet	14 feet

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

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

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.1.1 below. Short piles (less than 12 feet) should be designed in accordance with the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B and the MaineDOT Bridge Design Guide (BDG) and checked for combined axial and flexure using LPILE[®] software. .

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and scour at the design flow event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. The design and check floods for scour are defined in AASHTO LRFD Bridge Design Specifications 4th Edition (LRFD) Articles 2.6.4.4.2 and 3.7.5.

Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2. As the proposed piles for the project will be short and will not achieve fixity, the resistance for the pile will be determined for structural compliance with interaction equation.

7.1.1 Strength Limit State

The nominal 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. For preliminary analyses the H-piles were assumed fully embedded and the column slenderness factor, λ , was taken as 0.

The factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c , of 0.50 and a λ of 0. It is the responsibility of the structural designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced length and K-values from project specific L-Pile[®] analyses and recalculate structural resistances.

For the portion of the pile which is theoretically in pure compression, i.e. below the point of fixity, the factored structural axial resistances of five (5) H-pile sections were calculated using a resistance factor, ϕ_c , of 0.50. The factored structural axial resistance may be controlled by the combined axial and flexural resistance of the pile. This is the responsibility of the structural designer.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_{stat} , of 0.45.

The drivability of the five (5) proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is $\phi_{dyn} = 0.65$. Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. As it is likely that only two dynamic tests will be conducted at the site, this resistance factor has been reduced by 20% resulting in a $\phi_{dyn} = 0.52$.

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

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* $\phi_c = 0.50$ $\lambda = 0$	Geotechnical Resistance $\phi_{stat} = 0.45$	Drivability Resistance $\phi_{dyn} = 0.52$	Governing Resistance Based on Drivability Analyses
12 x 53	388	357	231	231
12 x 74	545	498	284	284
14 x 73	535	444	277	277
14 x 89	653	539	359	359
14 x 117	860	706	501	501

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

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

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

Since the abutment piles will be modeled with a fixed pile head and subjected to lateral and axial loads, bending moments and displacements, the piles should be analyzed for combined axial compression and flexure resistance per LRFD Articles 6.9.2.2 and 6.15. An L-Pile[®] analysis by the project geotechnical engineer is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, movements and pile head displacements applied. The resistance for the piles should be determined for compliance with the interaction equation. The upper portion of the pile is defined per LRFD Figure C6.15.2-1 as that portion of the pile above the point of second inflection in the movement vs. pile depth curve, or at the lowest point of zero inflection. Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor $\phi_c=0.7$ and the flexural resistance factor $\phi_f=1.0$ shall be applied to the combined axial and flexural resistance of the pile in the interaction equation. The resistance of the pile in the lower zone need only be checked against axial load.

7.1.2 Service and Extreme Limit States

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and displacements considering changes in foundation conditions due to scour at the design flood event. For the service limit state a resistance factor of 1.0 should be used for the calculation of structural, geotechnical and drivability axial pile resistances in accordance with LRFD Article 10.5.5.2. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor of $\phi=0.65$.

The extreme limit state design shall include a determination that there is adequate nominal foundation resistance remaining after scour due to the check flood to resist the unfactored extreme limit state load combination with a resistance factor of 1.0.

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

Pile Section	Service and Extreme Limit States Factored Axial Pile Resistance (kips)			
	Structural Resistance * $\phi = 1.0$ $\lambda = 0$	Geotechnical Resistance $\phi = 1.0$	Drivability Resistance $\phi = 1.0$	Governing Resistance Based on Drivability Analyses
12 x 53	775	793	445	445
12 x 74	1090	1106	546	546
14 x 73	1070	986	533	533
14 x 89	1305	1198	690	690
14 x 117	1720	1568	963	963

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

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

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

7.1.3 Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each 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 of 0.52. The factored pile load should be shown on the plans. If three to four piles are dynamically tested, the resistance factor may be increased by 20 percent to 0.65. Calculations for the pile resistance required by a drivability wave equation analysis are included the Appendix C- Calculations.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 13 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 stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider change in foundation conditions and pile group resistance after scour due to the design flood.

A resistance factor of $\phi = 1.0$ shall be used to assess abutment design at the service limit state including: settlement, excessive horizontal movement and movement resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, ϕ , of 0.65.

Extreme limit state design checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ , for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutments sections that are integral with the abutments shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure state. The Coulomb passive earth pressure coefficient, K_p , of 6.89 is recommended. Developing full passive requires displacements of the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced Coulomb passive earth pressure coefficient, but not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p , of 3.25. A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, $\gamma_{EH} = 1.50$.

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 7-4 below:

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

Table 7-4 - 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. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. The approach slab should be positively connected to the abutment.

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.

7.3 Bearing Resistance

In the event that any structure foundation is founded on spread footings bearing on fill, native sand or bedrock the footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.5. The stress distribution for spread footings on bedrock may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. The bearing resistance for any structure founded on competent, sound bedrock shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 27 ksf. This assumes a bearing resistance factor, ϕ_b , for spread footings on bedrock of 0.45, based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 20 ksf may be used and for preliminary footing sizing, and to control settlements when analyzing the service limit state load combination.

Bearing resistance for foundations on fill or native sand soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 5 ksf for wall system bases less than 8 feet wide and 7 ksf for bases from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 6 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

See Appendix C – Calculations, for supporting documentation.

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

7.4 Scour and Riprap

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

- Average diameter of particle at 50 percent passing, $D_{50} = 0.40$ mm
- Average diameter of particle at 95 percent passing, $D_{95} = 13.0$ mm
- Soil Classification AASHTO Soil Type A-2-4, A-4 or A-1-b

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

If using integral abutments at the site, pile lengths will be short and, therefore, scour protection will be critical. The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the bridge approach slopes and the slopes at abutments. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the MaineDOT Standard Specifications and a Class 1 nonwoven erosion control geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

7.5 Settlement

The horizontal and vertical alignment of the proposed bridge will be close to the existing bridge alignments. The bridge width will be less than state standards in order to match the existing corridor width. Post-construction settlements are anticipated to be negligible.

7.6 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT BDG Design Freezing Index map (MaineDOT BDG Figure 5-1) the site has a design freezing index of approximately 1200 F-degree days. A laboratory water content of 10% was used for granular soils above the water table. This correlates to a frost depth of 6.1 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis the site was assigned a design freezing index of 1123 F-degree days. A laboratory water content of 10% was used for granular soils above the water table. This results in a calculated frost depth of 5.0 feet.

It is recommended that any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

7.7 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, Nutter's Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed \$10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support lengths shall meet the requirements of LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

- Peak Ground Acceleration coefficient (PGA) = 0.098g
- Site Class D (site soils with an average N-value between 15 and 50 bpf)
- Acceleration coefficient (A_s) = 0.157
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.305g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.111g
- Seismic Zone 1 (based on S_{D1} less than 0.15g)

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

7.8 Construction Considerations

If using integral abutments at the site, pile lengths will be short and, therefore, scour protection will be critical. For scour protection, the integral abutments should be moved away from the channel. Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the Plans.

Boulders, cobbles and wood were encountered within the existing fill at the site. There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions shall be specified as incidental to related pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

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

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

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

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

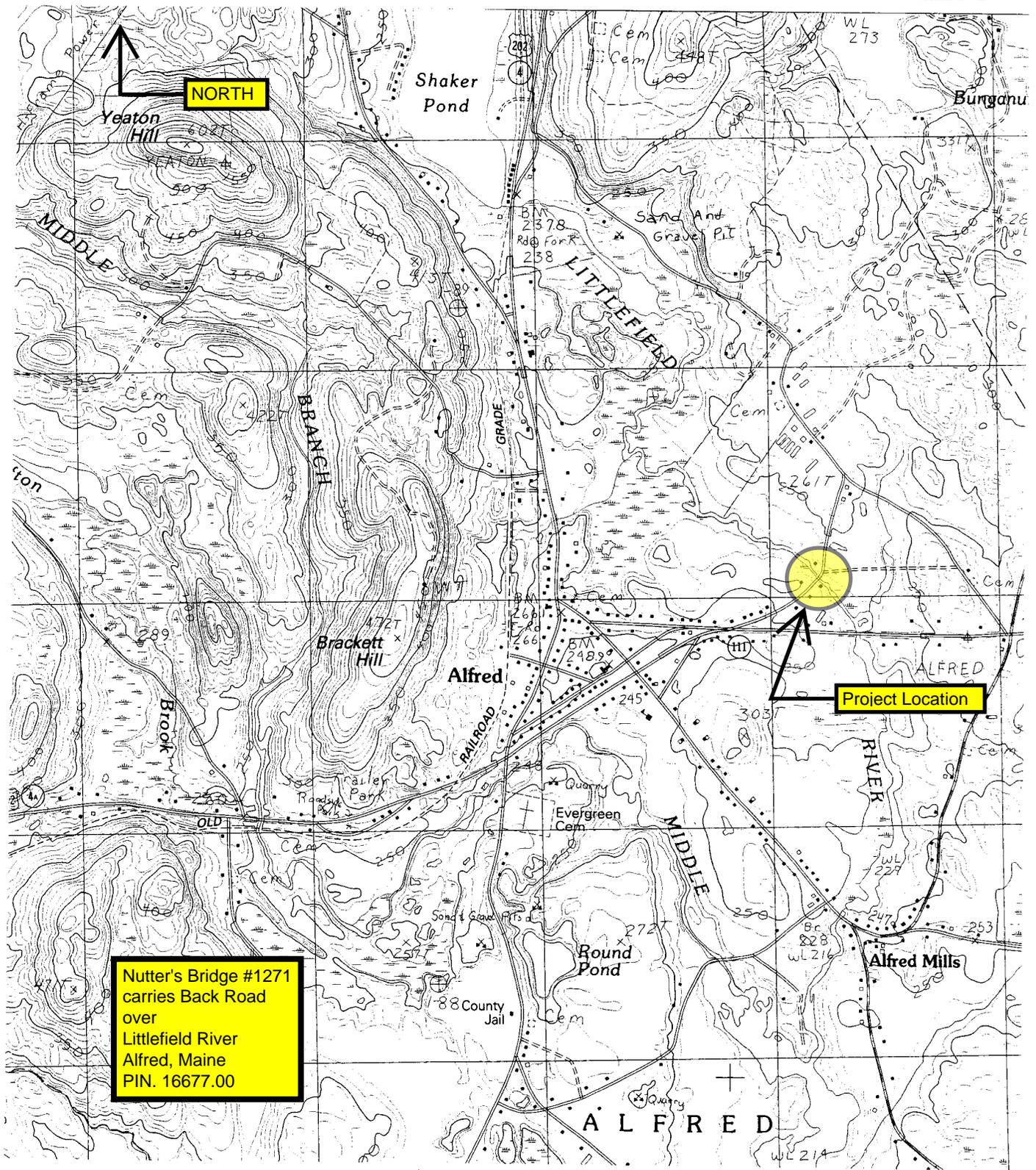
8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Nutter's Bridge in Alfred, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the

investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets

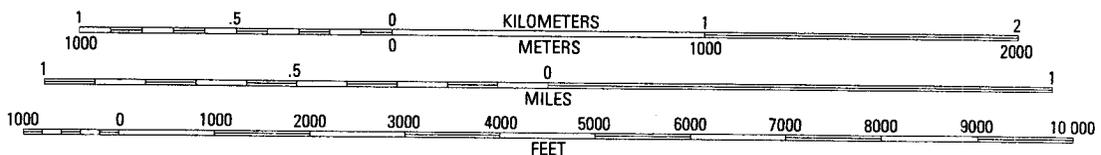


Nutter's Bridge #1271 carries Back Road over Littlefield River Alfred, Maine PIN. 16677.00

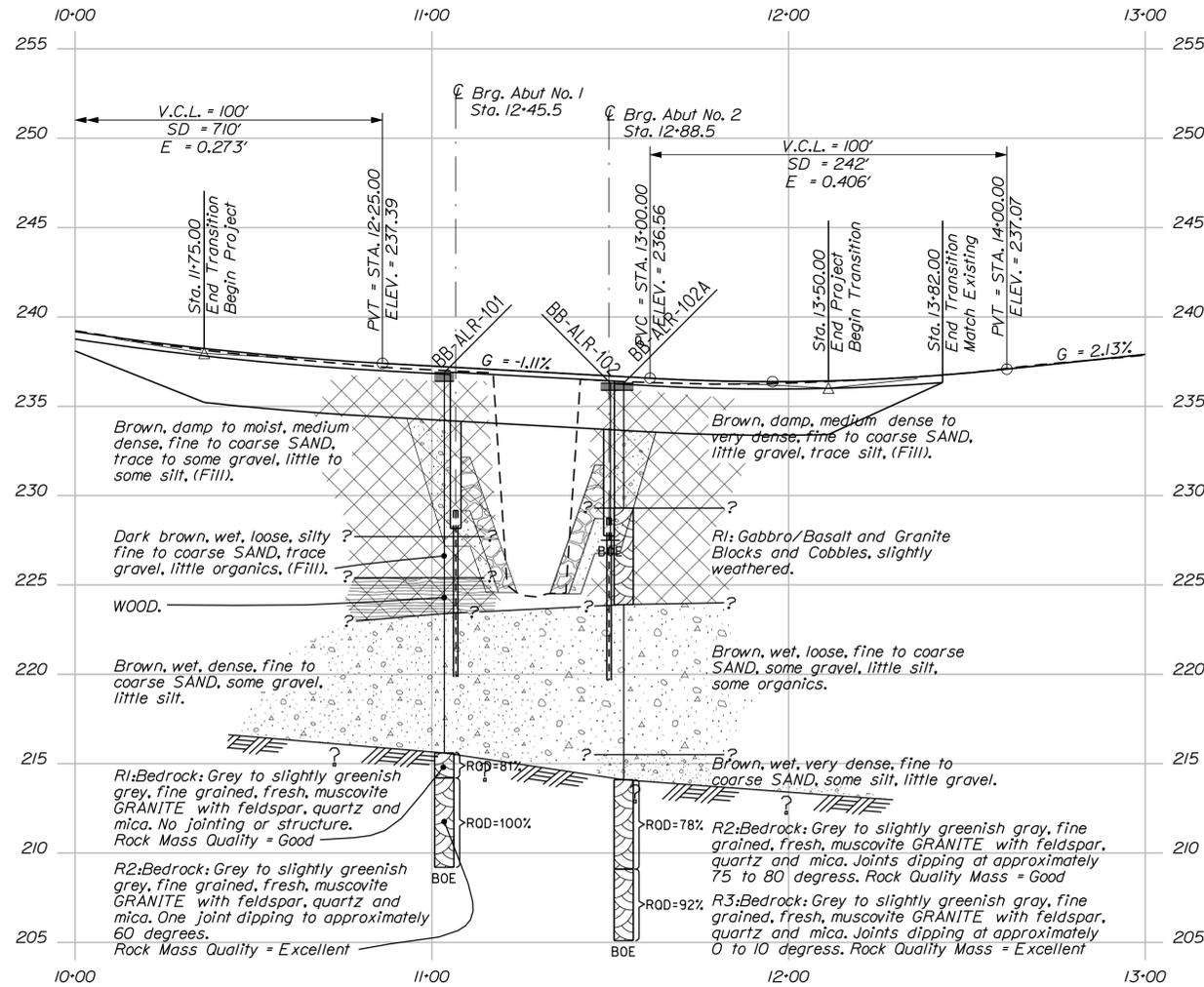
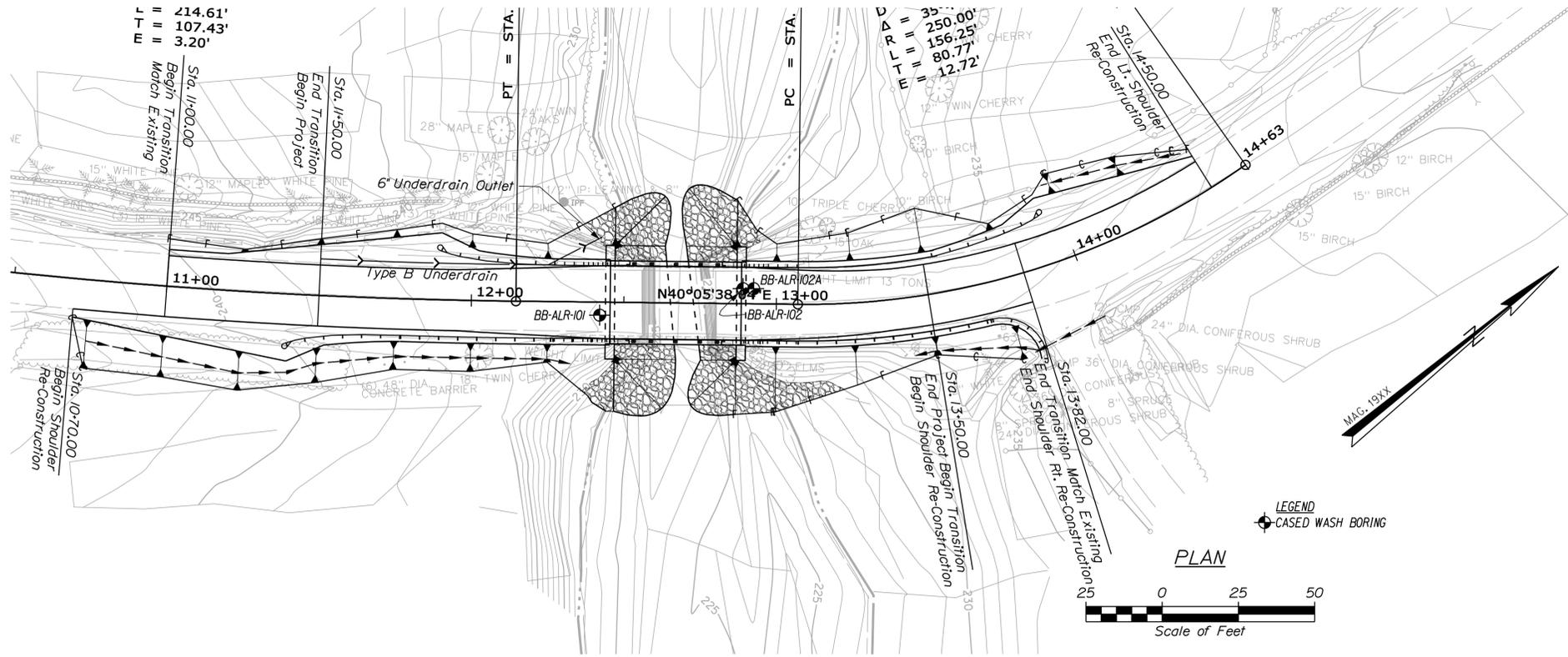
Project Location

ALFRED QUADRANGLE
 MAINE - YORK CO.
 7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000



CONTOUR INTERVAL 10 FEET



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.

PROJ. MANAGER	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
K. MAGUIRE	T. WHITE	NOV 2009			
CHECKED-REVIEWED					
DESIGNS DET AILED					
DESIGNS DET AILED					
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

NUTTER'S BRIDGE
LITTLEFIELD RIVER
YORK COUNTY
ALFRED
BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																													
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES																														
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p>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.</p> <p style="text-align: center;">Modified Burmister System</p> <table border="0"> <tr> <td style="text-align: center;"><u>Descriptive Term</u></td> <td style="text-align: center;"><u>Portion of Total</u></td> </tr> <tr> <td style="text-align: center;">trace</td> <td style="text-align: center;">0% - 10%</td> </tr> <tr> <td style="text-align: center;">little</td> <td style="text-align: center;">11% - 20%</td> </tr> <tr> <td style="text-align: center;">some</td> <td style="text-align: center;">21% - 35%</td> </tr> <tr> <td style="text-align: center;">adjective (e.g. sandy, clayey)</td> <td style="text-align: center;">36% - 50%</td> </tr> </table> <table border="0"> <tr> <td style="text-align: center;"><u>Density of Cohesionless Soils</u></td> <td style="text-align: center;"><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <tr> <td style="text-align: center;">Very loose</td> <td style="text-align: center;">0 - 4</td> </tr> <tr> <td style="text-align: center;">Loose</td> <td style="text-align: center;">5 - 10</td> </tr> <tr> <td style="text-align: center;">Medium Dense</td> <td style="text-align: center;">11 - 30</td> </tr> <tr> <td style="text-align: center;">Dense</td> <td style="text-align: center;">31 - 50</td> </tr> <tr> <td style="text-align: center;">Very Dense</td> <td style="text-align: center;">> 50</td> </tr> </table>	<u>Descriptive Term</u>	<u>Portion of Total</u>	trace	0% - 10%	little	11% - 20%	some	21% - 35%	adjective (e.g. sandy, clayey)	36% - 50%	<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50						
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		GP	Poorly-graded gravels, gravel sand mixtures, little or no 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	SW	Well-graded sands, gravelly sands, little or no fines																													
			SP	Poorly-graded sands, gravelly sand, little or no fines.																													
			SM	Silty sands, sand-silt mixtures																													
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	<p>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.</p> <table border="0"> <tr> <td style="text-align: center;"><u>Consistency of Cohesive soils</u></td> <td style="text-align: center;"><u>SPT N-Value blows per foot</u></td> <td style="text-align: center;"><u>Approximate Undrained Shear Strength (psf)</u></td> <td style="text-align: center;"><u>Field Guidelines</u></td> </tr> <tr> <td style="text-align: center;">Very Soft</td> <td style="text-align: center;">WOH, WOR, WOP, <2</td> <td style="text-align: center;">0 - 250</td> <td style="text-align: center;">Fist easily Penetrates</td> </tr> <tr> <td style="text-align: center;">Soft</td> <td style="text-align: center;">2 - 4</td> <td style="text-align: center;">250 - 500</td> <td style="text-align: center;">Thumb easily penetrates</td> </tr> <tr> <td style="text-align: center;">Medium Stiff</td> <td style="text-align: center;">5 - 8</td> <td style="text-align: center;">500 - 1000</td> <td style="text-align: center;">Thumb penetrates with moderate effort</td> </tr> <tr> <td style="text-align: center;">Stiff</td> <td style="text-align: center;">9 - 15</td> <td style="text-align: center;">1000 - 2000</td> <td style="text-align: center;">Indented by thumb with great effort</td> </tr> <tr> <td style="text-align: center;">Very Stiff</td> <td style="text-align: center;">16 - 30</td> <td style="text-align: center;">2000 - 4000</td> <td style="text-align: center;">Indented by thumb nail</td> </tr> <tr> <td style="text-align: center;">Hard</td> <td style="text-align: center;">>30</td> <td style="text-align: center;">over 4000</td> <td style="text-align: center;">Indented by thumb nail with difficulty</td> </tr> </table>	<u>Consistency of Cohesive soils</u>	<u>SPT N-Value blows per foot</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>	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 thumb nail	Hard	>30	over 4000	Indented by thumb nail with difficulty
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		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.																														
<p>Desired Soil Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Moisture (dry, damp, moist, wet, saturated)</p> <p>Density/Consistency (from above right hand side)</p> <p>Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)</p> <p>Gradation (well-graded, poorly-graded, uniform, etc.)</p> <p>Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)</p> <p>Structure (layering, fractures, cracks, etc.)</p> <p>Bonding (well, moderately, loosely, etc., if applicable)</p> <p>Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)</p> <p>Geologic Origin (till, marine clay, alluvium, etc.)</p> <p>Unified Soil Classification Designation</p> <p>Groundwater level</p>				<p>Rock Quality Designation (RQD):</p> <p>RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$</p> <p style="text-align: center;">*Minimum NQ rock core (1.88 in. OD of core)</p> <p style="text-align: center;">Correlation of RQD to Rock Mass Quality</p> <table border="0"> <tr> <td style="text-align: center;"><u>Rock Mass Quality</u></td> <td style="text-align: center;"><u>RQD</u></td> </tr> <tr> <td style="text-align: center;">Very Poor</td> <td style="text-align: center;"><25%</td> </tr> <tr> <td style="text-align: center;">Poor</td> <td style="text-align: center;">26% - 50%</td> </tr> <tr> <td style="text-align: center;">Fair</td> <td style="text-align: center;">51% - 75%</td> </tr> <tr> <td style="text-align: center;">Good</td> <td style="text-align: center;">76% - 90%</td> </tr> <tr> <td style="text-align: center;">Excellent</td> <td style="text-align: center;">91% - 100%</td> </tr> </table> <p>Desired Rock Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Texture (aphanitic, fine-grained, etc.)</p> <p>Lithology (igneous, sedimentary, metamorphic, etc.)</p> <p>Hardness (very hard, hard, mod. hard, etc.)</p> <p>Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)</p> <p>Geologic discontinuities/jointing:</p> <ul style="list-style-type: none"> -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.) <p>Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)</p> <p>RQD and correlation to rock mass quality (very poor, poor, etc.)</p> <p>ref: AASHTO Standard Specification for Highway Bridges</p> <p>17th Ed. Table 4.4.8.1.2A</p> <p>Recovery</p>		<u>Rock Mass Quality</u>	<u>RQD</u>	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%																
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<p>Maine Department of Transportation</p> <p>Geotechnical Section</p> <p>Key to Soil and Rock Descriptions and Terms</p> <p>Field Identification Information</p>				<p>Sample Container Labeling Requirements:</p> <table border="0"> <tr> <td>PIN</td> <td>Blow Counts</td> </tr> <tr> <td>Bridge Name / Town</td> <td>Sample Recovery</td> </tr> <tr> <td>Boring Number</td> <td>Date</td> </tr> <tr> <td>Sample Number</td> <td>Personnel Initials</td> </tr> <tr> <td>Sample Depth</td> <td></td> </tr> </table>		PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth																			
PIN	Blow Counts																																
Bridge Name / Town	Sample Recovery																																
Boring Number	Date																																
Sample Number	Personnel Initials																																
Sample Depth																																	

Driller: MaineDOT	Elevation (ft.): 236.9	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 11+03.5, 5.1 Rt.	Casing ID/OD: NW	Water Level*: 12.0' bgs.

Hammer Efficiency Factor: 0.84 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)
MU = Unsuccessful Thin Wall Tube Sample attempt RC = Roller Cone N-uncorrected = Raw field SPT N-value
V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value
MV = Unsuccessful Insitu Vane Shear Test attempt WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency
 WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	236.40	Pavement		
	1D/A	24/18	1.50 - 3.50	3/4/5/5	9	13				(ID) 1.5-2.5' bgs. Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill). (1D/A) 2.5-3.5' bgs. Brown, damp, medium dense, fine to coarse SAND, some silt, trace gravel, (Fill).	G#236853 A-1-b, SM WC=3.7% G#236854 A-2-4, SM WC=9.5%	
5	2D	24/15	5.00 - 7.00	8/14/6/6	20	28				Brown, moist, medium dense, fine to coarse SAND, some gravel, some silt (Fill).	G#236855 A-2-4, SM WC=9.8%	
10	3D	24/16	10.00 - 12.00	2/2/2/8	4	6	12		227.40	Dark brown, wet, loose, Silty fine to coarse SAND, trace gravel, little organics, muck (Fill).	G#236856 A-4, SM WC=68.7%	
							10		225.40	Wood layer from 11.5-13.5' bgs, (Fill).		
							80		223.40			
15	4D	24/16	15.00 - 17.00	12/14/14/14	28	39	55			Brown, wet, dense, fine to coarse SAND, some gravel, little silt.	G#236857 A-2-4, SM WC=11.6%	
							68					
							75					
							72					
							76					
20	5D	16.8/6	20.00 - 21.40	13/14/40(4.8")	---					Similar to above, except very dense.		
	R1	16.8/13	21.30 - 22.70	RQD = 81%				NQ-2	215.60	Top of Bedrock at Elev. 215.6'. Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. No jointing or structure. Rock Mass Quality = Good. R1:Core Times (min:sec) 21.3-22.3' (10:14) 22.3-22.7' (12:05) 81% Recovery		
	R2	60/60	22.70 - 27.70	RQD = 100%								

Remarks:
700-800# down pressure on Core Barrel.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Nutter's Bridge #1271 carries Back Rd. over Littlefield River Location: Alfred, Maine	Boring No.: BB-ALR-101 PIN: 16677.00
--	---	---

Driller: MaineDOT	Elevation (ft.): 236.9	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 11+03.5, 5.1 Rt.	Casing ID/OD: NW	Water Level*: 12.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25										Changed Core Bit Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. One joint dipping to approximately 60 degrees. Rock Mass Quality = Excellent. R2: Core Times (min:sec) 22.7-23.7' (3:58) 23.7-24.7' (4:55) 24.7-25.7' (4:20) 25.7-26.7' (4:40) 26.7-27.7' (5:24) 100% Recovery Bottom of Exploration at 27.70 feet below ground surface.		
26												
27												
28												
29												
30												
31												
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												

Remarks:
700-800# down pressure on Core Barrel.

Driller: MaineDOT	Elevation (ft.): 236.3	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 08:30-14:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 11+53.8, 4.9 Lt.	Casing ID/OD: NW & HW	Water Level*: 13.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer WOR/C = weight of rods or casing N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

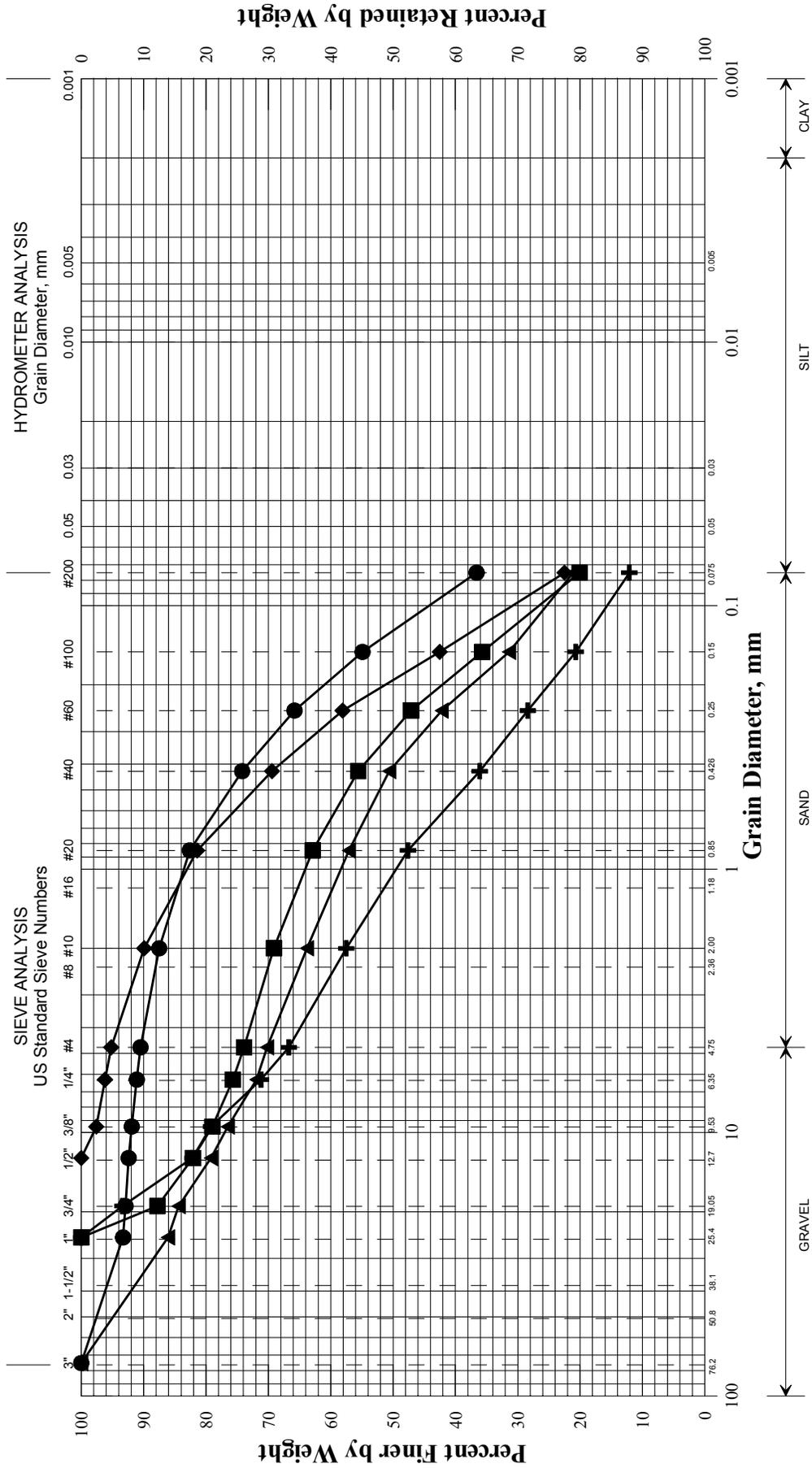
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
25										Rock Mass Quality = Good. R2:Core Times (min:sec) 22.2-23.2' (4:44) 23.2-24.2' (4:28) 24.2-25.2' (5:36) 25.2-26.2' (4:19) 26.2-27.2' (5:02) 100% Recovery Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. Joints dipping at approximately 0 to 10 degrees. Rock Mass Quality = Excellent. R3:Core Times (min:sec) 27.2-28.2' (5:37) 28.2-29.2' (7:36) 29.2-30.2' (7:20) 30.2-31.2' (10:16) 96% Recovery 31.20' Bottom of Exploration at 31.20 feet below ground surface.		
	R3	48/46	27.20 - 31.20	RQD = 92%								
30								205.10				
35												
40												
45												
50												

Remarks:
700-800# down pressure on Core Barrel.

Appendix B

Laboratory Data

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

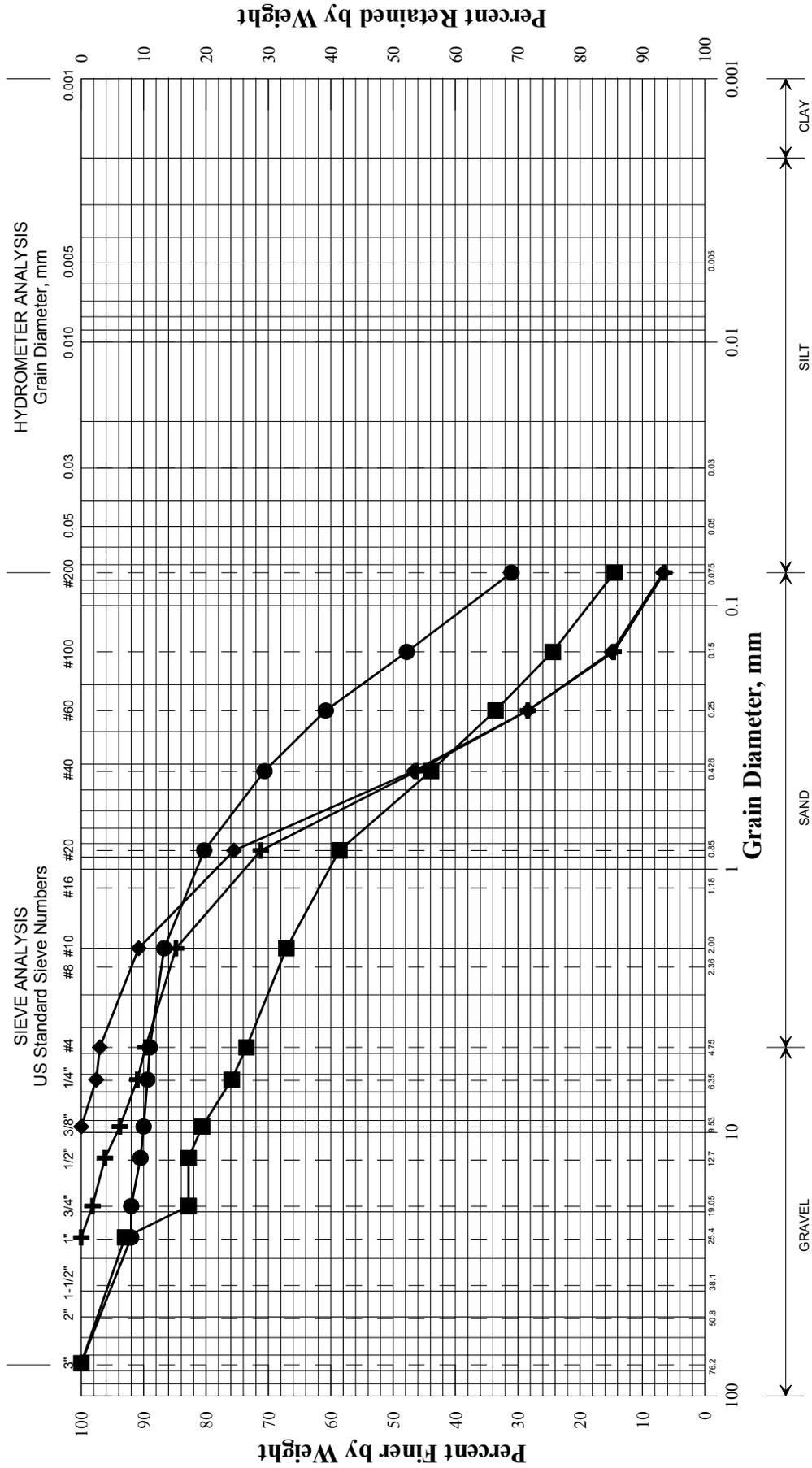


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	11+03.5	5.1 RT	1.5-2.5	SAND, some gravel, little silt.	3.7			
◆	11+03.5	5.1 RT	2.5-3.5	SAND, some silt, trace gravel.	9.5			
■	11+03.5	5.1 RT	5.0-7.0	SAND, some gravel, some silt.	68.7			
●	11+03.5	5.1 RT	10.0-12.0	Silty SAND, trace gravel.	11.6			
▲	11+03.5	5.1 RT	15.0-17.0	SAND, some gravel, little silt.				
×								

016677.00	PIN
Alfred	Town
WHITE, TERRY A	Reported by/Date
	2/1/2010

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
+	11+50.3	4.8 LT	1.0-3.0	SAND, little gravel, trace silt.	3.7			
◆	11+50.3	4.8 LT	5.0-6.5	SAND, trace silt, trace gravel.	3.3			
■	11+53.8	4.9 LT	12.4-14.4	SAND, some gravel, little silt.	22.8			
●	11+53.8	4.9 LT	20.5-22.0	SAND, some silt, little gravel.	13.4			
▲								
×								

016677.00	PIN
Alfred	Town
WHITE, TERRY A	Reported by/Date
	2/1/2010

Appendix C

Calculations

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

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

Look at the following piles:

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

Note: All matrices set up in this order

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

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

Where λ = normalized column slenderness factor

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

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

$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$

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

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "severe".

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

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 388 \\ 545 \\ 535 \\ 653 \\ 860 \end{pmatrix} \cdot \text{kip}$ **HP 12 x 53**
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117 Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

Where λ = normalized column slenderness factor

$$\lambda = (Kl/r_s \pi)^2 \cdot F_y / E \quad \text{eq. 6.9.4.1-3}$$

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

$$P_n := 0.66^{\lambda} \cdot F_y \cdot A_s \quad P_n = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

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

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

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

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

Service/Extreme Limit States

Geotechnical Resistance

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

Bedrock Type:

Granite RQD ranges from 78 to 100%

Use RQD = 90% and $\phi = 34$ to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design
 Specifications 4th Edition 2007

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 25000 \cdot \text{psi}$

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

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

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

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

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

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

$K_{sp} = \begin{pmatrix} 0.6821 \\ 0.6766 \\ 0.6143 \\ 0.6119 \\ 0.6078 \end{pmatrix}$ K_{sp} includes a factor of safety of 3

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

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

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

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

Nominal Geotechnical Tip Resistance, R_p :

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

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

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

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

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

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

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

Service/Extreme
Limit States

DRIVABILITY ANALYSIS Ref: LRFD Article 10.7.8

For steel piles in compression or tension

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

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

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

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

Compute Resistance that can be achieved in a drivability analysis:

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

Table 10.5.5.2.3-1 pg 10-38 gives resistance factor for dynamic test, ϕ_{dyn} : $\phi_{dyn} := 0.65$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability. There will probably only be 4 to 5 piles total at each abutment. Only 1 or 2 piles will be tested - one per abutment will be requested. Therefore, reduce the ϕ by 20%

$$\phi_{dyn.reduced} := 0.65 \cdot 0.8$$

$$\phi_{dyn.reduced} = 0.52$$

Pile Size = 12 x 53

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

State of Maine Dept. Of Transportation				28-Jan-2010		
Alfred Nutters Bridge Drivability 12x53				GRLWEAP (TM) Version 2003		
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft	
440.0	44.68	2.10	9.1	6.65	11.14	
441.0	44.71	2.09	9.2	6.66	11.13	
442.0	44.83	2.11	9.2	6.67	11.17	
443.0	44.91	2.11	9.2	6.68	11.20	
444.0	44.94	2.12	9.3	6.69	11.20	
445.0	45.06	2.13	9.3	6.69	11.23	
446.0	45.10	2.15	9.3	6.70	11.23	
447.0	45.19	2.17	9.3	6.71	11.26	
448.0	45.23	2.17	9.4	6.72	11.26	
449.0	45.23	2.19	9.5	6.72	11.26	

Limited driving stress to 45 ksi

DELMAG D 19-42

Strength Limit State: $\phi_{dyn.reduced} = 0.52$

Efficiency 0.800

$R_{dr_12x53_factored} := 445 \cdot kip \cdot \phi_{dyn.reduced}$

Helmet 3.20 kips
 Hammer Cushion 109975 kips/in

$R_{dr_12x53_factored} = 231 \cdot kip$

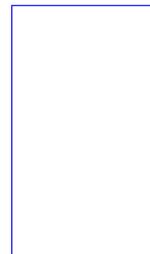
Skin Quake 0.100 in
 Toe Quake 0.040 in
 Skin Damping 0.050 sec/ft
 Toe Damping 0.150 sec/ft

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

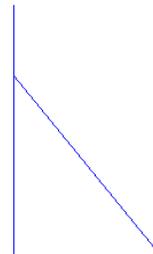
Pile Length 18.00 ft
 Pile Penetration 13.00 ft
 Pile Top Area 15.50 in²

$R_{dr_12x53_servext} := 445 \cdot kip$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
 (Proportional)

Pile Size = 12 x 74

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

State of Maine Dept. Of Transportation				28-Jan-2010	
Alfred Nutters Bridge Drivability 12x74				GRLWEAP (TM) Version 2003	
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
540.0	44.57	2.14	6.6	9.21	17.41
541.0	44.69	2.16	6.6	9.22	17.43
542.0	44.74	2.17	6.6	9.22	17.44
543.0	44.87	2.17	6.6	9.23	17.46
544.0	44.91	2.19	6.6	9.23	17.47
545.0	44.94	2.21	6.6	9.24	17.48
546.0	45.02	2.22	6.7	9.24	17.50
547.0	45.08	2.23	6.7	9.25	17.51
548.0	45.19	2.24	6.7	9.26	17.52
549.0	45.13	2.26	6.7	9.27	17.52

DELMAG D 19-42

Limited to driving stress to 45 ksi

Strength Limit State: $\phi_{dyn.reduced} = 0.52$

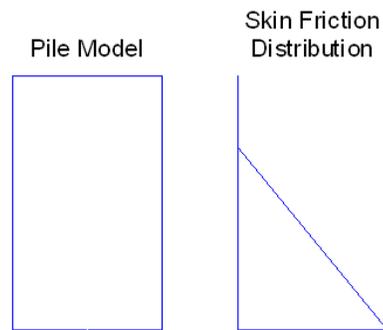
$$R_{dr_12x74_factored} := 546 \cdot \text{kip} \cdot \phi_{dyn.reduced}$$

$$R_{dr_12x74_factored} = 284 \cdot \text{kip}$$

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

$$R_{dr_12x74_servext} := 546 \cdot \text{kip}$$

Efficiency	0.800
Helmet Hammer Cushion	3.20 kips 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	18.00 ft
Pile Penetration	13.00 ft
Pile Top Area	21.80 in ²



Res. Shaft = 10 %
 (Proportional)

Pile Size = 14 x 73

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

State of Maine Dept. Of Transportation				28-Jan-2010		
Alfred Nutters Bridge Drivability 14x73				GRLWEAP (TM) Version 2003		
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft	
530.0	44.69	2.11	6.4	9.18	17.40	
531.0	44.81	2.11	6.4	9.19	17.42	
532.0	44.91	2.13	6.5	9.19	17.43	
533.0	45.01	2.14	6.5	9.20	17.45	
534.0	45.04	2.15	6.5	9.20	17.46	
535.0	45.08	2.17	6.5	9.21	17.48	
536.0	45.19	2.18	6.5	9.21	17.50	
537.0	45.24	2.20	6.5	9.22	17.51	
538.0	45.39	2.20	6.5	9.23	17.52	
539.0	45.26	2.22	6.6	9.23	17.48	

DELMAG D 19-42

Limit to driving stress to 45 ksi

Strength Limit State: $\phi_{dyn.reduced} = 0.52$

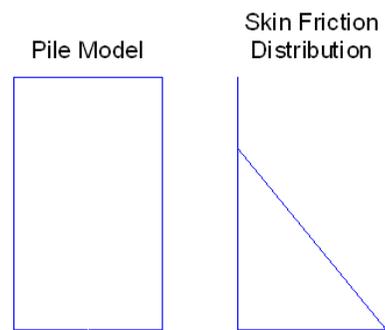
$$R_{dr_14x73_factored} := 533 \cdot \text{kip} \cdot \phi_{dyn.reduced}$$

$$R_{dr_14x73_factored} = 277 \cdot \text{kip}$$

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

$$R_{dr_14x73_servext} := 533 \cdot \text{kip}$$

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.050 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	18.00 ft
Pile Penetration	13.00 ft
Pile Top Area	21.40 in ²



Res. Shaft = 10 %
 (Proportional)

Pile Size = 14 x 89

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

State of Maine Dept. Of Transportation				28-Jan-2010		
Alfred Nutters Bridge Drivability 14x89				GRLWEAP (TM) Version 2003		
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft	
685.0	44.76	3.00	8.9	9.82	17.94	
686.0	44.79	3.00	8.9	9.83	17.94	
687.0	44.82	3.00	8.9	9.83	17.94	
688.0	44.96	3.03	8.9	9.84	17.99	
689.0	44.98	3.03	8.9	9.85	18.00	
690.0	45.03	3.04	8.9	9.85	18.00	
691.0	45.05	3.04	9.0	9.85	18.00	
692.0	45.06	3.03	9.0	9.86	17.99	
693.0	45.12	3.04	9.0	9.86	17.99	
694.0	45.14	3.05	9.0	9.87	18.00	

DELMAG D 19-42

Limit blow count to 15 bows per inch

Strength Limit State: $\phi_{dyn.reduced} = 0.52$

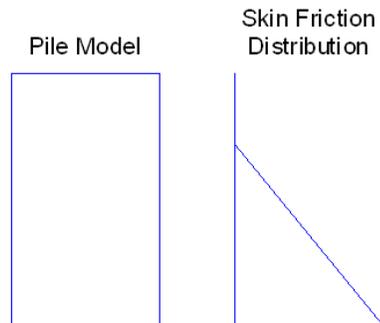
$R_{dr_14x89_factored} := 690 \cdot kip \cdot \phi_{dyn.reduced}$

$R_{dr_14x89_factored} = 359 \cdot kip$

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

$R_{dr_14x89_servext} := 690 \cdot kip$

Efficiency	0.800
Helmet Hammer Cushion	3.20 kips 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	18.00 ft
Pile Penetration	13.00 ft
Pile Top Area	26.10 in ²



Res. Shaft = 10 %
 (Proportional)

Pile Size = 14 x 117

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

State of Maine Dept. Of Transportation			28-Jan-2010			
Alfred Nutters Bridge 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	
960.0	44.54	5.00	14.8	10.81	18.68	
961.0	44.54	4.99	14.9	10.81	18.66	
962.0	44.55	5.00	14.9	10.81	18.66	
963.0	44.60	5.00	15.0	10.81	18.65	
964.0	44.63	5.03	14.9	10.81	18.69	
965.0	44.63	5.04	15.0	10.81	18.67	
966.0	44.65	5.03	15.1	10.81	18.65	
967.0	44.66	5.03	15.1	10.81	18.64	
968.0	44.66	5.03	15.2	10.81	18.62	
969.0	44.67	5.03	15.2	10.81	18.61	

Limit to blow count to 15 blows per inch

Strength Limit State: $\phi_{\text{dyn.reduced}} = 0.52$

$R_{\text{dr}_14 \times 117_{\text{factored}}} := 963 \cdot \text{kip} \cdot \phi_{\text{dyn.reduced}}$

$R_{\text{dr}_14 \times 117_{\text{factored}}} = 501 \cdot \text{kip}$

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

$R_{\text{dr}_14 \times 117_{\text{servext}}} := 963 \cdot \text{kip}$

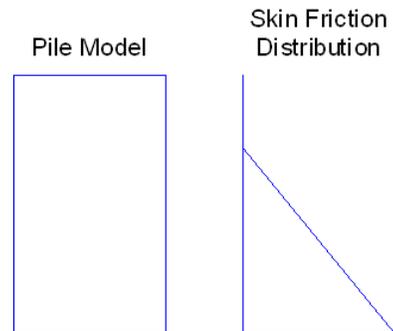
DELMAG D 19-42

Stroke 10.81 feet
 Efficiency 0.800

Helmet 3.20 kips
 Hammer Cushion 109975 kips/in

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

Pile Length 18.00 ft
 Pile Penetration 13.00 ft
 Pile Top Area 34.40 in²



Res. Shaft = 10 %
 (Proportional)

Earth Pressure:

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

For cases where interface friction is considered use Coulomb Theory

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

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

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

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

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

$$K_p = 6.89$$

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

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

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

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

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

Bearing Resistance - Bedrock:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on bedrock

Presumptive Bearing Resistance for Service Limit State ONLY

Bedrock at the site is Granite which is "good" to "excellent" in quality.
RQD = 78 to 100%

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

Due to RQD look at "medium hard rock"

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

Consistency In Place: Medium hard rock

Bearing Resistance: Ordinary Range (ksf) 16 - 24

Recommended Value of Use (ksf): 20 ksf

Based on RQD values ranging from 38% to 72%

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

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

Part 2 - Strength Limit State

Nominal and Factored Bearing Resistance - spread footing on bedrock

Nominal Bearing Resistance for Strength Limit State

Bedrock at the site is Granite which is "good" to "excellent" in quality.
RQD = 78 to 100%

Reference: AASHTO LRFD Bridge Design Specifications Third Edition Article 10.6.3.2:
For footings on competent rock, reliance on simple and direct analyses based
on uniaxial compressive rock strengths and RQD may be applicable. Where engineering
judgment does not verify the presence of competent rock, the competency of the rock mass should
be verified using the procedures for RMR rating in Article 10.4.6.4.

Due to competency of bedrock (RQD 78 to 100%), RMR method is not required.

Reference: Foundation Analysis and Design by JE Bowles Fifth Edition

Section 4-16 pg 277 Bearing Capacity of Rock

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

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

Bearing Capacity factors by Stagg and Zienkiewicz 1968

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

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

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

Terzaghi Shape factors from Table 4-1 pg 220 For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

Assume $\gamma_r := 168 \cdot \text{pcf}$ for the rock Bowles, Table 4-11 pg 278 ($\gamma = 26.4 \text{ kN/m}^3 = 168 \text{ pcf}$)

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

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

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

$$q_{ult} = \begin{pmatrix} 100 \\ 134 \\ 167 \\ 201 \end{pmatrix} \cdot \text{ksf}$$

Reduce ultimate bearing based on lowest RQD = 78%

$$q_{reduced} := q_{ult} \cdot (0.78)^2$$

$$q_{reduced} = \begin{pmatrix} 61 \\ 81 \\ 102 \\ 122 \end{pmatrix} \cdot \text{ksf}$$

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

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

$$q_{factored} = \begin{pmatrix} 27 \\ 37 \\ 46 \\ 55 \end{pmatrix} \cdot \text{ksf} \quad B := \begin{pmatrix} 6 \\ 8 \\ 10 \\ 12 \end{pmatrix} \cdot \text{ft}$$

At the Strength Limit State: **Recommend a limiting factored bearing resistance of 27 ksf**

Bearing Resistance - Native Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

Reference: AASHTO LRFD Bridge Design Specifications 4th Edition
Table C10.6.2.6.1-1 Presumptive Bearing Resistances for Spread Footings at the
Service Limit State Modified after US Department of Navy (1982)

Type of Bearing Material: Coarse to medium sand, with little gravel (SW, SP)

Based on corrected N-values ranging from 8 to 20 - Soils are loose to dense

Consistency In Place: Medium dense

Bearing Resistance: Ordinary Range (ksf) 4 to 8

Recommended Value of Use: 6 ksf

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

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

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

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

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

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on native soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

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

Depth to Groundwater table: $D_w := 17 \cdot \text{ft}$ Based on boring logs

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

Look at several footing widths

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

Terzaghi Shape factors from Table 4-1

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

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

For $\phi=32$ deg

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

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

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

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

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

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

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

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

Based on these footing widths

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

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

At Strength Limit State:

Recommend a limiting factored bearing resistance of 5 ksf for walls less than 8 feet wide.
 Recommend a limiting factored bearing resistance of 7 ksf for walls between 8.5 and 12 feet wide.

Frost Protection:

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

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

From the lab testing: soils are coarse grained with a water content = ~10%

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

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

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 Sanford

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

Air Design Freezing Index	=	1123 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	898 F-days
Mean Annual Temperature	=	46.8 deg F
Design Length of Freezing Season	=	116 days

Layer #:	Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-	Coarse	60.2	10.0	125.0	28	34	2.0	1.6	1,800

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 = 5.02 ft = 60.2 in.

$$\text{Frost_depth}_{\text{modberg}} := 60.2 \cdot \text{in}$$

$$\text{Frost_depth}_{\text{modberg}} = 5.0167 \text{ ft}$$

Use Frost Depth = 5.0 feet for design

Seismic:

16677.00 Alfred Nutter's Bridge		
Date and Time: 2/1/2010 2:42:41 PM		
Conterminous 48 States		
2007 AASHTO Bridge Design Guidelines		
AASHTO Spectrum for 7% PE in 75 years		
State - Maine		
Zip Code - 04002		
Zip Code Latitude = 43.493500		
Zip Code Longitude = -070.697500		
Site Class B		
Data are based on a 0.05 deg grid spacing.		
Period (sec)	Sa (g)	
0.0	0.098	PGA - Site Class B
0.2	0.191	Ss - Site Class B
1.0	0.046	S1 - Site Class B
Conterminous 48 States		
2007 AASHTO Bridge Design Guidelines		
Spectral Response Accelerations SDs and SD1		
State - Maine		
Zip Code - 04002		
Zip Code Latitude = 43.493500		
Zip Code Longitude = -070.697500		
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1		
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40		
Data are based on a 0.05 deg grid spacing.		
Period (sec)	Sa (g)	
0.0	0.157	As - Site Class D
0.2	0.305	SDs - Site Class D
1.0	0.111	SD1 - Site Class D

Seismic Design Parameters for 2007 AASHTO Seismic Design Guidelines

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

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

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

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

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

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock Injector Pile Tips)

Subsection 501.10 Prefabricated Pile Tips of the Standard Specifications is amended as follows:

Pipe tips for use on all piles shall be Rock Injector “H” Bearing Pile Points (Point # HPP-R-14) manufactured by Titus Steel Co. or approved equal. Material specifications, attachment of pile tips and seating of piles shall be in accordance with Manufacturer’s recommendations and in accordance with the Standard Specifications.

Payment will be made under:

<u>Pay Item</u>	<u>Description</u>	<u>Pay Unit</u>
501.903	Pile Tips – Rock Injector Point	Each

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

Add the following paragraph to Section 610.02:

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

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

Add the following paragraph to Section 610.032.a.

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

Add the following paragraph to Section 610.032.b:

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

Add the following to Section 610.032:

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

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

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

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

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

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

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces

Subrounded: Particles have nearly plane sides but have well-rounded corners and edges

Rounded: Particles have smoothly curved sides and no edges

Soils Report 2010-04

Addendum #1

To: File
cc: TEDOCS
Author: Kate Maguire, PE
Subject: Soils Report No. 2010-04
Updated Foundation Type and Stationing
Document Type: 24
Date: April 29, 2010
Bridge No.: 1271
Route: N/A
PIN: 16677.00
Town: Alfred

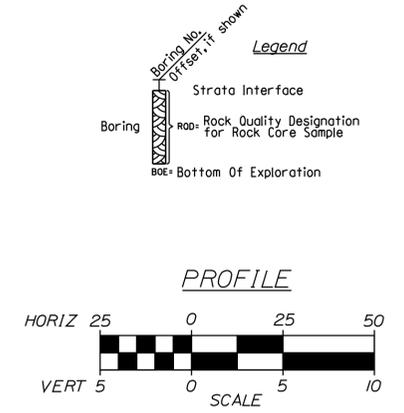
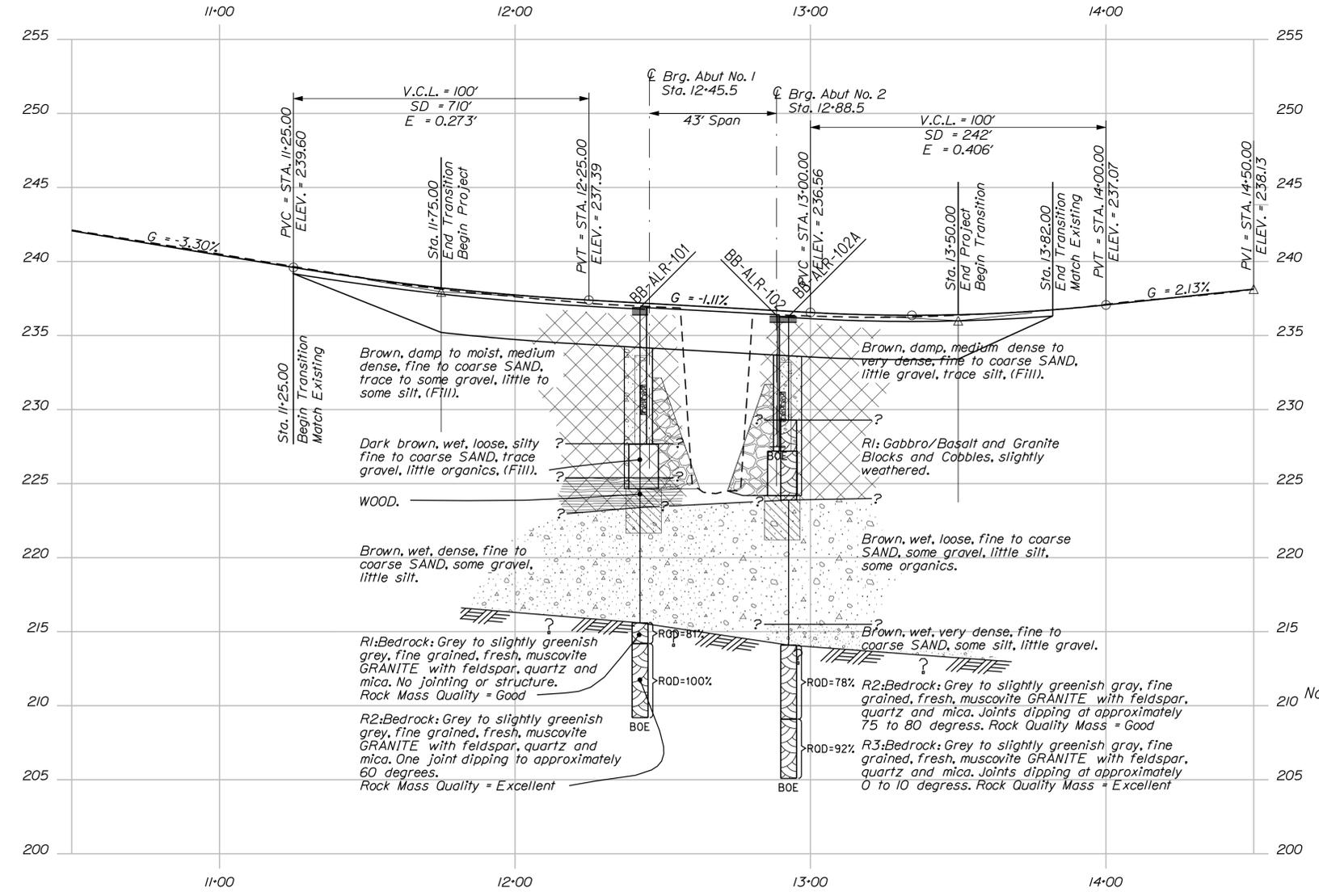
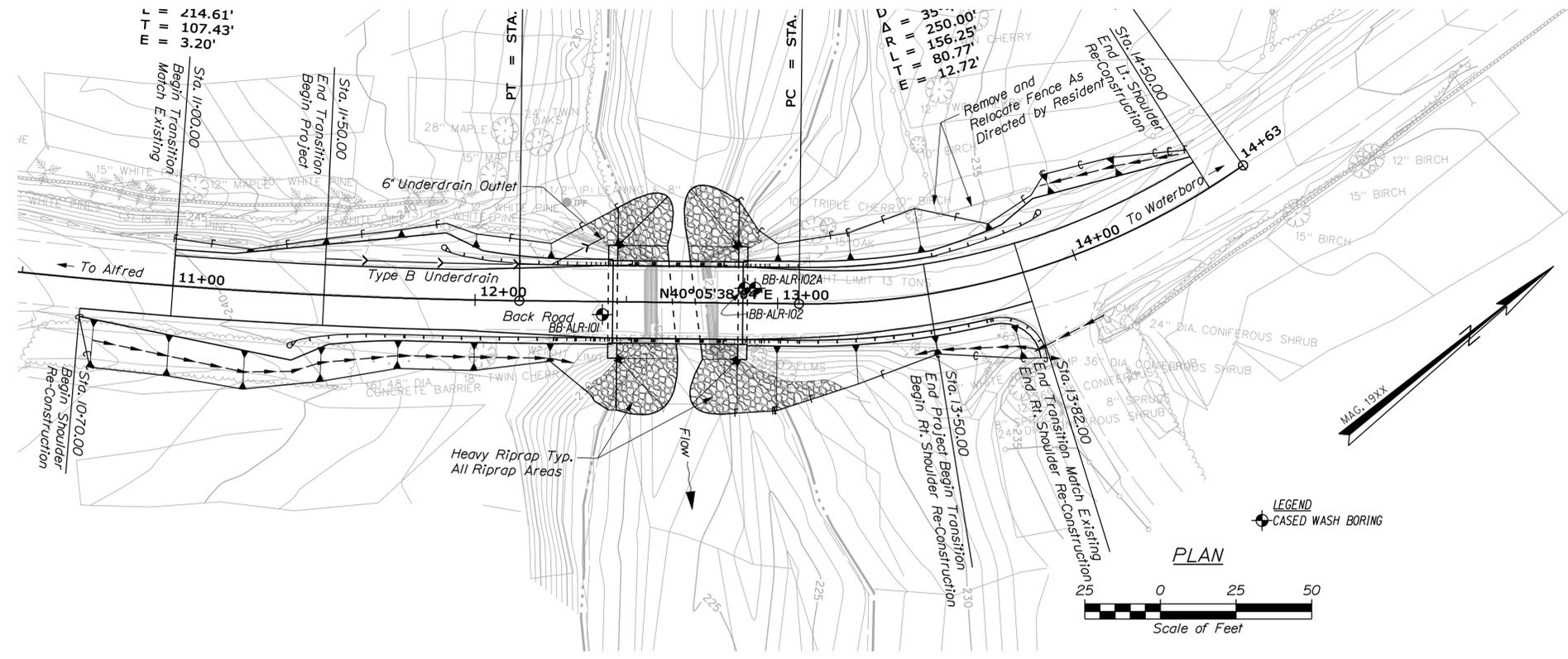
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The following changes are made to the Geotechnical Design Report for the Replacement of Nutter's Bridge over Littlefield River Alfred, Maine, Soils Report No. 2010-04:

At the time the original soils report was published (March 2, 2010) it was the intent of the Bridge Program to replace Nutter's Bridge with a pile supported integral structure. During final design, the structural designer made the decision to use spread footings founded on soil. It is the recommendation of the project geotechnical engineer that the spread footings be founded a minimum of 2.0 feet below the design scour depth and armored with 3.0 feet of riprap (MaineDOT BDG Section 2.3.11). It is understood that no scour analysis was performed for the design of the footings. A scour analysis and appropriate design of the spread footings is recommended. In lieu of a scour analysis, the spread footing should be founded on the bedrock surface.

The following pages are attached as updates to the original pages contained in Soils Report No. 2010-04 and present the updated stationing used in the Contract Documents:

1. Sheet 2 of 3 – Shows updated stationing in for profile and updated foundation type.
2. Appendix A Boring Logs – The stationing indicated in the title block of the boring logs is updated.
3. Appendix B Laboratory Data – The stationing indicated on the Laboratory Testing Summary Sheet and the Grain size curve sheets is updated.



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

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
AC-BR-1667(700)X		BRIDGE NO. 1271	
PIN 16677.00		BRIDGE PLANS	
PROJ. MANAGER	DATE	CHECKED	SIGNATURE
DESIGNED	NOV 2009	DESIGNED	
DESIGNED		DESIGNED	
REVISIONS 1		REVISIONS 1	
REVISIONS 2		REVISIONS 2	
REVISIONS 3		REVISIONS 3	
REVISIONS 4		REVISIONS 4	
FIELD CHANGES		DATE	
NUTTER'S BRIDGE			
LITTLEFIELD RIVER			
YORK COUNTY			
ALFRED			
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE			
SHEET NUMBER			
2			
OF 3			

Driller: MaineDOT	Elevation (ft.): 236.9	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 12+42.1, 4.3 Rt.	Casing ID/OD: NW	Water Level*: 12.0' bgs.

Hammer Efficiency Factor: 0.84 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 N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0									SSA	236.40	Pavement	
	1D/A	24/18	1.50 - 3.50	3/4/5/5	9	13					(ID) 1.5-2.5' bgs. Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill). (1D/A) 2.5-3.5' bgs. Brown, damp, medium dense, fine to coarse SAND, some silt, trace gravel, (Fill).	G#236853 A-1-b, SM WC=3.7% G#236854 A-2-4, SM WC=9.5%
5	2D	24/15	5.00 - 7.00	8/14/6/6	20	28					Brown, moist, medium dense, fine to coarse SAND, some gravel, some silt (Fill).	G#236855 A-2-4, SM WC=9.8%
10	3D	24/16	10.00 - 12.00	2/2/2/8	4	6	12			227.40	Dark brown, wet, loose, Silty fine to coarse SAND, trace gravel, little organics, muck (Fill).	G#236856 A-4, SM WC=68.7%
							10			225.40	Wood layer from 11.5-13.5' bgs, (Fill).	
							80			223.40		
15	4D	24/16	15.00 - 17.00	12/14/14/14	28	39	55				Brown, wet, dense, fine to coarse SAND, some gravel, little silt.	G#236857 A-2-4, SM WC=11.6%
							68					
							75					
							72					
							76					
20	5D	16.8/6	20.00 - 21.40	13/14/40(4.8")	---						Similar to above, except very dense.	
	R1	16.8/13	21.30 - 22.70	RQD = 81%			NQ-2			215.60	Top of Bedrock at Elev. 215.6'. Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. No jointing or structure. Rock Mass Quality = Good. R1:Core Times (min:sec) 21.3-22.3' (10:14) 22.3-22.7' (12:05) 81% Recovery	
	R2	60/60	22.70 - 27.70	RQD = 100%								

Remarks:
700-800# down pressure on Core Barrel.

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Nutter's Bridge #1271 carries Back Rd. over Littlefield River Location: Alfred, Maine	Boring No.: BB-ALR-101 PIN: 16677.00
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Driller: MaineDOT	Elevation (ft.): 236.9	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 07:30-11:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 12+42.1, 4.3 Rt.	Casing ID/OD: NW	Water Level*: 12.0' bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
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 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25										Changed Core Bit Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. One joint dipping to approximately 60 degrees. Rock Mass Quality = Excellent. R2: Core Times (min:sec) 22.7-23.7' (3:58) 23.7-24.7' (4:55) 24.7-25.7' (4:20) 25.7-26.7' (4:40) 26.7-27.7' (5:24) 100% Recovery Bottom of Exploration at 27.70 feet below ground surface.		
26												
27												
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Remarks:
700-800# down pressure on Core Barrel.

Driller: MaineDOT	Elevation (ft.): 236.3	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Wright	Datum: NAVD 88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 10/30/09; 08:30-14:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 12+92.5, 5.0 Lt.	Casing ID/OD: NW & HW	Water Level*: 13.0' bgs.

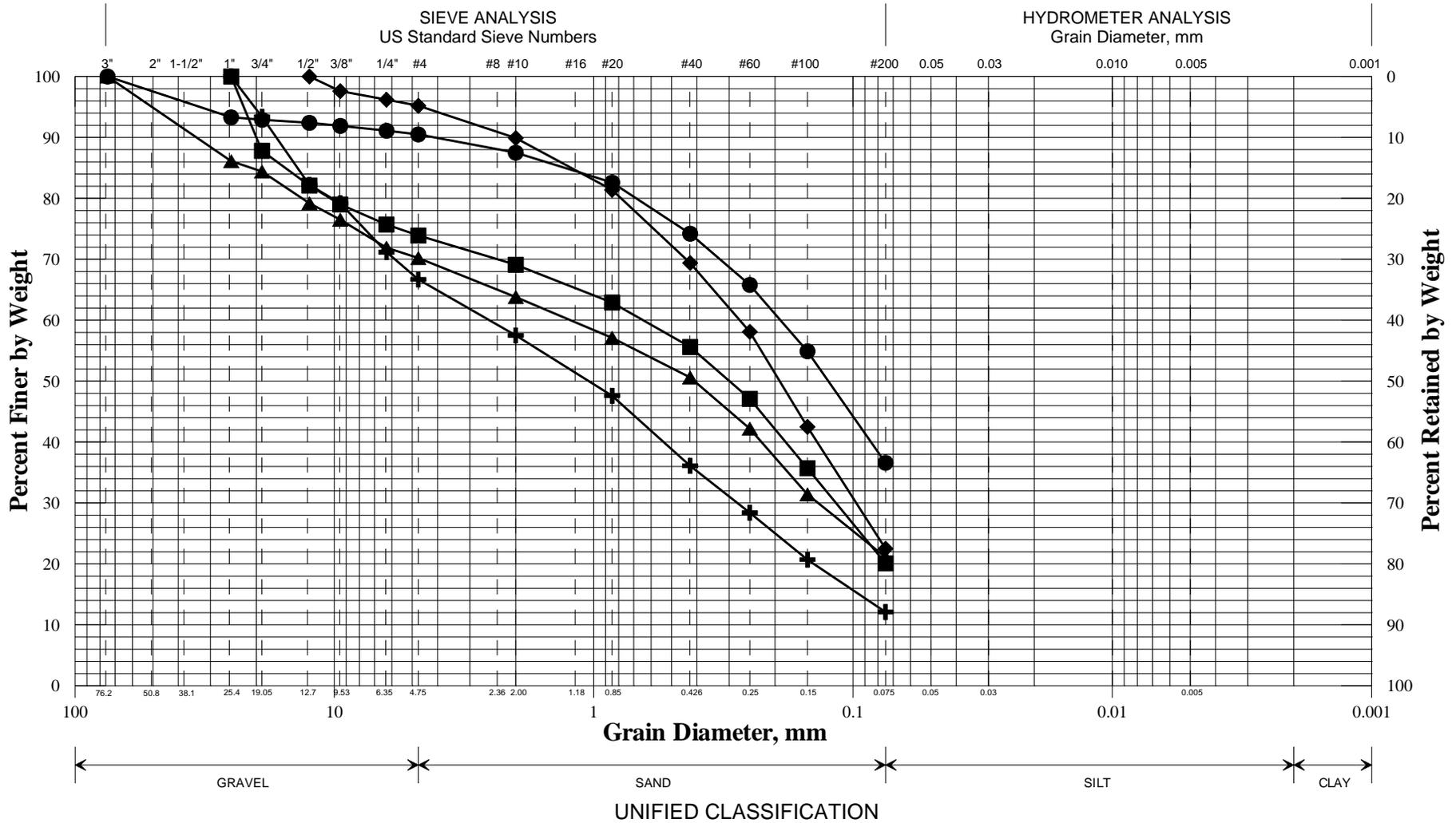
Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
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 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%))	N-uncorrected	N ₆₀	Casing Blows					
25										Rock Mass Quality = Good. R2:Core Times (min:sec) 22.2-23.2' (4:44) 23.2-24.2' (4:28) 24.2-25.2' (5:36) 25.2-26.2' (4:19) 26.2-27.2' (5:02) 100% Recovery Bedrock: Grey to slightly greenish grey, fine grained, fresh, muscovite GRANITE with feldspar, quartz and mica. Joints dipping at approximately 0 to 10 degrees. Rock Mass Quality = Excellent. R3:Core Times (min:sec) 27.2-28.2' (5:37) 28.2-29.2' (7:36) 29.2-30.2' (7:20) 30.2-31.2' (10:16) 96% Recovery 31.20' Bottom of Exploration at 31.20 feet below ground surface.		
	R3	48/46	27.20 - 31.20	RQD = 92%								
30								205.10				
35												
40												
45												
50												

Remarks:
700-800# down pressure on Core Barrel.

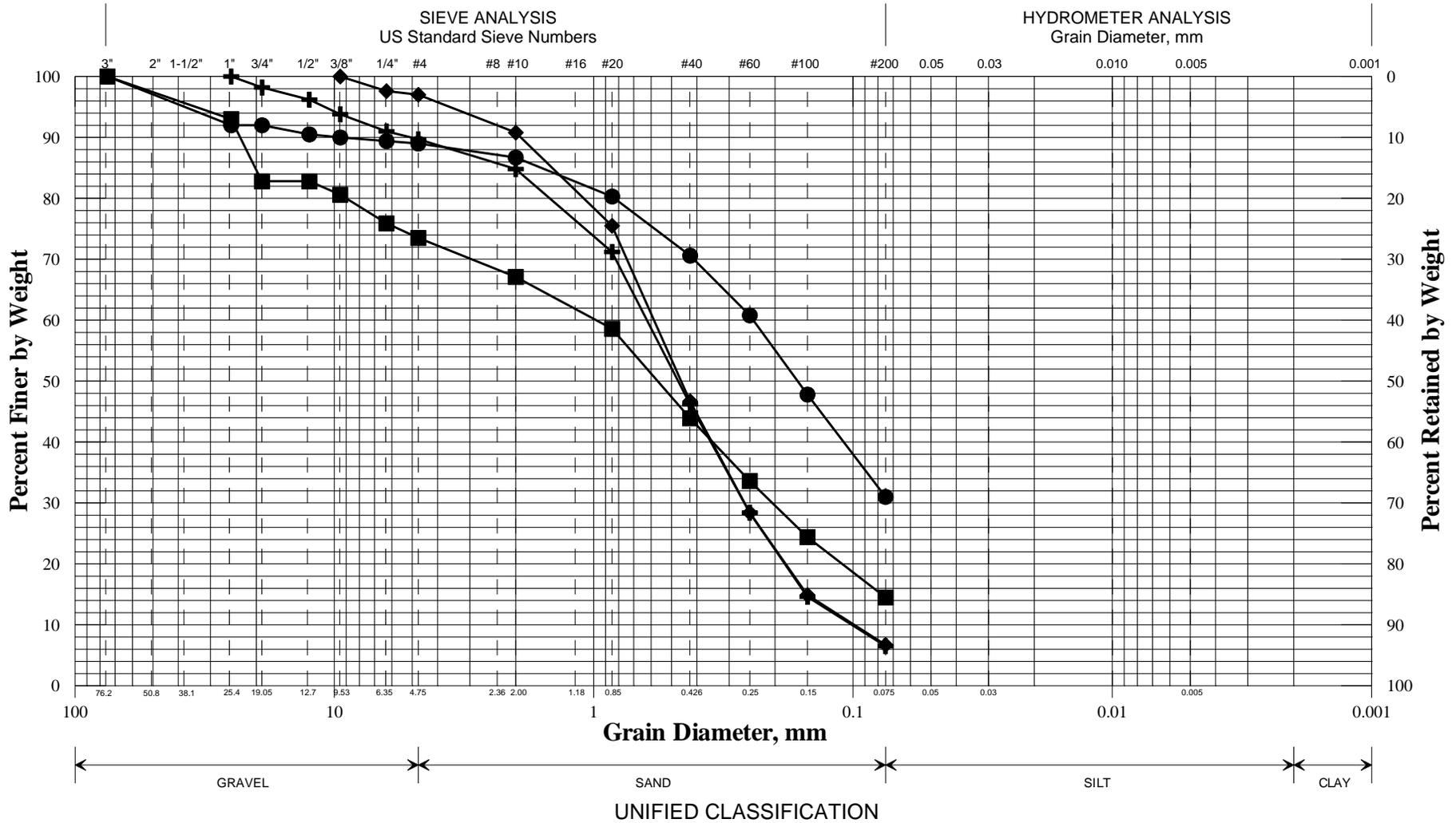
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-ALR-101/1D	12+42.1	4.3 RT	1.5-2.5	SAND, some gravel, little silt.	3.7			
◆	BB-ALR-101/1DA	12+42.1	4.3 RT	2.5-3.5	SAND, some silt, trace gravel.	9.5			
■	BB-ALR-101/2D	12+42.1	4.3 RT	5.0-7.0	SAND, some gravel, some silt.	9.8			
●	BB-ALR-101/3D	12+42.1	4.3 RT	10.0-12.0	Silty SAND, trace gravel.	68.7			
▲	BB-ALR-101/4D	12+42.1	4.3 RT	15.0-17.0	SAND, some gravel, little silt.	11.6			
x									

PIN
016677.00
Town
Alfred
Reported by/Date
WHITE, TERRY A 2/1/2010

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-ALR-102/1D	12+89	4.9 LT	1.0-3.0	SAND, little gravel, trace silt.	3.7			
◆	BB-ALR-102/2D	12+89	4.9 LT	5.0-6.5	SAND, trace silt, trace gravel.	3.3			
■	BB-ALR-102A/1D	12+92.5	5.0 LT	12.4-14.4	SAND, some gravel, little silt.	22.8			
●	BB-ALR-102A/2D	12+92.5	5.0 LT	20.5-22.0	SAND, some silt, little gravel.	13.4			
▲									
×									

PIN	
016677.00	
Town	
Alfred	
Reported by/Date	
WHITE, TERRY A	2/1/2010