MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

LITTLE POND BRIDGE
US ROUTE 302
FRYEBURG, MAINE

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Table of Contents

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Little Pond Bridge over Little Pond Stream in Fryeburg, Maine. The proposed bridge replacement will consist of an approximately 43 foot long, single-span, precast, prestressed, concrete superstructure founded on H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-Piles – The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. The H-piles shall be design for all relevant strength, service and extreme limit state load groups. The structural resistance check should include checking axial, lateral, and flexural resistance. An L-Pile® analysis is recommended to evaluate the combined axial compression and flexure with factored axial loads, moments and pile head displacements applied. As the proposed integral H-piles will be modeled as fully fixed at the pile head, the resistance of the piles should be evaluated for structural compliance with the interaction equation.

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

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. In designing integral abutments for passive earth pressure, the Rankine earth pressure coefficient (K_p) of 3.25 is allowed if the displacement of the abutment is less than 2 percent of the abutment height. All abutment designs shall include a drainage system to intercept any water. The approach slab should be positively connected to the integral abutment. Additional lateral earth pressure due to construction surcharge or live load surcharge is required if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted.

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

Settlement - The roadway profile will be raised approximately 1.2 feet at Abutment No. 1 and approximately 0.9 feet at Abutment No. 2. Potential settlement due the placement of the proposed fill is estimated as less than 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Frost Protection - Integral abutments shall be embedded a minimum of 4.0 feet for frost protection. Foundations placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection.

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

Construction Considerations – Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. Using the excavated native soils as structural backfill should not be permitted. Materials excavated from the existing subbase and subgrade fill soils in approaches should not be used to re-base the new bridge approaches.

All timber piling shall be removed to 1 foot below river bed. Payment shall be considered incidental to bridge removal. There is a potential for the existing abutment piles or the old pavement encountered in boring BB-FLP-101 to interfere with the installation of the proposed piles. If the existing piles or buried obstructions such as old pavement are encountered during pile installation they shall be removed by the Contractor to the Resident's satisfaction.

1.0 INTRODUCTION

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

The existing Little Pond Bridge carries US Route 302 over Little Pond Stream and was constructed in 1935. The bridge consists of a single span concrete slab superstructure on concrete abutments and flared wingwalls. The existing bridge abutments and wingwalls are supported on rows of 60 foot long friction timber piles. The existing structure has a total length of approximately 23 feet. The 2010 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and superstructure are in serious condition (rating of 3) and the substructures are in satisfactory condition (rating of 6). The Bridge Sufficiency Rating is 21.4. The structure has a scour critical rating of “8 – Stable Above Footing” meaning that the foundations have been determined to be stable for the assessed or calculated scour condition. The scour is determined to be above the top of the footings. Inspection records note that the bridge deck is in serious condition with large spalls and exposed re-bar. There is evidence of abutment spalling at northeast corner of the structure and heavy scaling of southeast wingwall. A vertical crack was noted in the northeast breastwall. There is evidence of scour and major scaling of footings below flow line.

The MaineDOT Bridge Program is currently proposing to replace Little Pond Bridge in conjunction with a highway project which will reconstruct a portion US Route 302 in Fryeburg. The replacement structure will consist of a single-span, precast, prestressed, concrete voided slab superstructure founded on H-pile supported integral abutments constructed behind the location of the existing abutments. The project will include 450’ of approaches to match the future highway alignment. The span of the proposed replacement structure will be approximately 43 feet. The roadway centerline will be located slightly north of the existing centerline. The roadway profile will be raised approximately 1.2 ft at Abutment No. 1 and approximately 0.9 feet at Abutment No. 2. The proposed bridge will be constructed using staged construction techniques.

2.0 GEOLOGIC SETTING

Little Pond Bridge in Fryeburg carries US Route 302 over the Little Pond Stream 0.78 miles north of the Bridgton town line as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic map entitled Pleasant Mountain Quadrangle, Maine Open File No. 99-5 (1999) published by the Maine Geological Survey the surficial soils in the vicinity of the site consist of glacial lake deposits comprised of sand, gravel, and silt deposited in glacial Lake Pigwacket including fan, delta and lake-bottom sediments.

Specifically, the site is underlain by Pleasant Mountain stage deposits formed in an ice-dammed lake flanking the esker in the western part of the quadrangle.

According to the Bedrock Geologic Map of Maine (1985) published by the Maine Geologic Survey, the bedrock in the vicinity of the site consists of igneous carboniferous muscovite-biotite granite commonly known as the Sebago pluton. This bedrock is anticipated to be hard and sound.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling two (2) test borings. Test boring BB-FPL-101 was conducted behind the west abutment and test boring BB-FLP-102 was conducted behind the east abutment. The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the soil stratigraphy across the site is shown on Sheet 3 – Interpretive Subsurface Profile found at the end of this report. The borings were drilled between April 14 and May 12, 2011 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheets 4 and 5 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in March of 2010 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value are shown on the boring logs. The bedrock was cored in the borings using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated.

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

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of twenty-four (24) standard grain size analyses with water content and fourteen (14) grain size analyses with hydrometer and water content. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheets 4 and 5 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

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

5.1 Fill Material

A layer of fill was encountered beneath the pavement in both of the borings. The fill consisted of:

- Brown, wet, fine to coarse sand, trace to little silt, trace to little gravel;
- Brown, damp, gravelly, fine to coarse sand, trace silt;
- Grey-brown, wet, fine to coarse sand, little silt, trace gravel, trace clay, little organics and old pavement fragments; and
- Grey, wet, fine to coarse sand, trace gravel, trace silt.

The thickness of the fill was approximately 17.0 feet in boring BB-FLP-101 and approximately 18.0 feet in boring BB-FLP-102. Corrected SPT N-values in the fill ranged from weight of hammer (WOH) to 8 blows per foot (bpf) indicating that the fill is very loose to loose in consistency. One corrected SPT N-value in the upper portion of the fill in boring BB-FLP-102 was 57 bpf. This value was influenced by the presence of gravel and is not indicative of the actual density of the fill layer. Water contents obtained from fill samples ranged from approximately 3% to 31%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-2-4, A-1-a or A-1-b by the AASHTO Classification System and an SC-SM, SW-SM or SM by the Unified Soil Classification System.

5.2 Upper Sand Layer

An upper sand layer was encountered beneath the fill in both of the borings. The upper sand consisted of:

- Grey-brown, wet, fine to coarse sand, some silt, trace gravel, trace clay, some organics;
- Grey, wet, fine to coarse sand, trace gravel, trace silt; and
- Olive-brown, wet, fine to medium sand, some silt, trace coarse sand, trace gravel, little organics.

The thickness of the upper sand layer was approximately 5.5 feet in boring BB-FLP-101 and approximately 10.0 feet in boring BB-FLP-102. Corrected SPT N-values in the upper sand ranged from WOH to 3 bpf indicating that the upper sand is very loose in consistency. Water contents from samples obtained within the upper sand range from approximately 51% to 52%. Grain size analyses conducted on samples of the upper sand indicate that the soil is classified as an A-2-4 by the AASHTO Classification System and an SC-SM or SM by the Unified Soil Classification System.

5.3 Sandy Silt/Silt

A layer of sandy silt and silt was encountered beneath the upper sand in both of the borings. The layer generally consisted of:

- Grey, wet, fine to medium sandy silt, trace coarse sand, trace clay; and
- Olive-brown and grey, wet, silt, trace to some fine to medium sand, trace to little clay, with black staining.

The thickness of the layer was approximately 17.5 feet in boring BB-FLP-101 and approximately 27.5 feet in boring BB-FLP-102. Corrected SPT N-values in the layer ranged from WOH to weight of rods (WOR) indicating that the soil is very soft in consistency. Water contents from samples obtained within the layer range from approximately 24% to 50%. Grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a ML or CL-ML by the Unified Soil Classification System.

5.4 Middle Sand

A middle layer of sand was encountered beneath the sandy silt and silt layer in both of the borings. The middle sand consisted of:

- Brown and gold, wet, fine to medium sand, trace to little silt, trace coarse sand, trace gravel;
- Brown, wet, fine to coarse sand, little silt, trace gravel; and
- Light brown, moist to wet, fine sand, trace to some silt, trace medium to coarse sand.

The thickness of the middle sand layer was approximately 25.0 feet in boring BB-FLP-101 and approximately 19.5 feet in boring BB-FLP-102. Corrected SPT N-values in the middle sand layer ranged from 4 to 49 bpf indicating that the layer is very loose to dense in consistency. Water contents from samples obtained within the layer range from approximately 18% to 24%. Grain size analyses conducted on the samples indicate that the soil is classified as an A-2-4 or A-4 by the AASHTO Classification System and an SM, SP or SP-SM by the Unified Soil Classification System.

5.5 Silt/Sandy Silt/Silty Sand/Sand

A layer of interbedded silt, sandy silt, silty sand and sand was encountered beneath the middle sand layer. The layer generally consisted of:

- Brown, wet, fine sandy silt;
- Brown, light brown and grey , wet, silty fine sand;
- Brown and grey, wet, silt, trace to some fine sand, trace to some clay; and
- Brown, wet, fine to medium sand trace silt.

The thickness of the layer was approximately 29.0 feet in boring BB-FLP-101 and approximately 44.0 feet in boring BB-FLP-102. Corrected SPT N-values in the sand layers ranged from 15 to 53 bpf indicating that the sand layers are medium dense to very dense in consistency. Corrected SPT N-values in the silt layers ranged from 10 to 59 bpf indicating that the silt layers are stiff to hard in consistency. Water contents from samples obtained within the layer ranged from approximately 22% to 27%. Grain size analysis conducted on samples from the layer indicate that the soil is classified as an A-4, A-2-4 or A-3 by the AASHTO Classification System and as an ML, SM, SP or CL-ML by the Unified Soil Classification System.

5.6 Lower Sand/Sandy Silt

A lower layer of sand was encountered beneath the interbedded silt, sandy silt, silty sand and sand layer in both of the borings. The lower sand and sandy silt consisted of:

- Brown, wet, fine sand, some silt;
- Brown, grey and light brown, wet, fine to coarse sand, trace to little gravel, trace silt; and
- Grey wet, fine to medium sandy silt, trace coarse sand.

The thickness of the layer was approximately 48.0 feet in boring BB-FLP-101 and approximately 34.0 feet in boring BB-FLP-102. Corrected SPT N-values in the sand layers ranged from WOR to 81 bpf indicating that the sand layers are very loose to very dense in consistency. One corrected SPT N-value in the sandy silt layer was 28 bpf indicating that the sandy silt layer is very stiff in consistency. Water contents from samples obtained within the layer range from approximately 11% to 22%. Grain size analyses conducted on the samples indicate that the soil is classified as an A-1-b, A-3 or A-4 by the AASHTO Classification System and an SP, SP-SM or ML by the Unified Soil Classification System.

5.7 Till

A layer of till was encountered just above the bedrock surface but was not sampled. The thickness of the layer was approximately 4.7 feet in boring BB-FLP-101 and approximately 1.8 feet in boring BB-FLP-102.

5.8 Bedrock

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

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB-FLP-101	157.7 feet	242.5 feet	n/a
BB-FLP-102	154.8 feet	244.7 feet	90%

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

The bedrock is identified as grey to salmon colored, medium grained, granite with mica and iron staining, joints dipping at approximately 60 degrees. The rock quality designation (RQD) of the bedrock was determined to be 90 percent indicating a rock mass quality of good.

5.9 Groundwater

Groundwater was observed at a depth of approximately 5.5 to 7.0 feet below the existing ground surface in the borings. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

The following foundation alternatives were considered for the bridge replacement:

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

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

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

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

7.1 Integral Abutment H-Piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with pile tips to protect the tips and improve penetration.

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

Location	Estimated Pile Cap Bottom Elevation	Approximate Depth to Bedrock From Ground Surface	Approximate Top of Rock Elevation	Estimated Pile Length
Abutment #1 BB-FLP-101	392.8 feet	157.7 feet	242.5 feet	151 feet
Abutment #2 BB-FLP-102	392.3 feet	154.8 feet	244.7 feet	148 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 H-piles shall be designed for 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. The H-piles shall also be checked for fixity and 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 displacements considering changes in foundation conditions due to scour at the design flood 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 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.

7.1.1 Strength Limit State Design

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. It is the responsibility of the structural engineer to recalculate the nominal structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e". Preliminary estimates of the factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and an unbraced length (ℓ) of 48 inches and an effective length factor (K) of 1.2.

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 proposed H-pile sections were calculated using a resistance factor, φ_{stat}, of 0.45.

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

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

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ_c=0.60	Geotechnical Resistance φ_{stat}=0.45	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	451	349	332	332
HP 12x74	636	487	432	432
HP 14x73	629	434	422	422
HP 14x89	768	527	469	469
HP 14x117	1013	690	586	586

* based on preliminary assumption of ℓ=48" and K=1.2

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

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

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

7.1.2 Service and Extreme Limit State Design

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. 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.

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

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

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ=1.0	Geotechnical Resistance φ=1.0	Drivability Resistance φ=1.0	Governing Resistance
HP 12x53	752	775	510	510
HP 12x74	1059	1081	665	665
HP 14x73	1049	964	649	649
HP 14x89	1280	1171	722	722
HP 14x117	1688	1533	901	901

*based on preliminary assumption of ℓ=48" and K=2.0

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

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

7.1.3 Driven Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test with CAPWAP analysis at each integral 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 0.65. The factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident and verified by dynamic pile test measurements and CAPWAP analysis. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Stub Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider changes in foundation conditions and pile group resistance after scour due to the design flood.

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

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

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction angle of 20 degrees. Cast-in-place integral abutment 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, γ_{EH} = 1.50 for the integral abutment backwall design.

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. The approach slab should be positively connected to the integral 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 Scour and Riprap

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

- Average diameter of particle at 50 percent passing, D_{50} = 0.13 mm
- Average diameter of particle at 95 percent passing, D_{95} = 0.58 mm
- Soil Classification AASHTO Soil Type A-2-4

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

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

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

Bridge approach slopes and slopes at wingwalls shall be armored with 3 feet of plain riprap. Stone riprap shall conform to item number 703.26 of MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class “1” Erosion Control Geotextile per Standard Details 610(02) through 610(04).

7.4 Settlement

The roadway profile will be raised approximately 1.2 feet at Abutment No. 1 and approximately 0.9 feet at Abutment No. 2. Potential settlement due the placement of the proposed fill is estimated as less than 1 inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

7.5 Frost Protection

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

In the event that any foundation is placed on granular subgrade soils, it should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1600 F-degree days. In a granular soil with a water content of approximately 10%, this correlates to a frost depth of approximately 6.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.0 feet below finished exterior grade for frost protection. See Appendix C - Calculations at the end of this report for supporting documentation.

7.6 Seismic Design Considerations

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) = 0.102g
- Site Class E (soil profile with average N-value for the upper 100 feet of soil profile less than 15 blows per foot)
- Acceleration coefficient (A_s) = 0.254
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.495g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.173g
- Seismic Zone 2, based on: $0.15g < S_{D1} < 0.30g$ (LRFD Table 3.10.6-1)

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

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

7.7 Construction Considerations

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

All timber piling shall be removed to 1 foot below river bed. Payment shall be considered incidental to bridge removal. There is a potential for the existing abutment piles or the old pavement encountered in boring BB-FLP-101 to interfere with the installation of the proposed piles. If the existing piles or buried obstructions such as old pavement are encountered during pile installation they shall be removed by the Contractor to the Resident's satisfaction. This condition should be noted on the plans and the work should be considered incidental to pile installation.

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

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

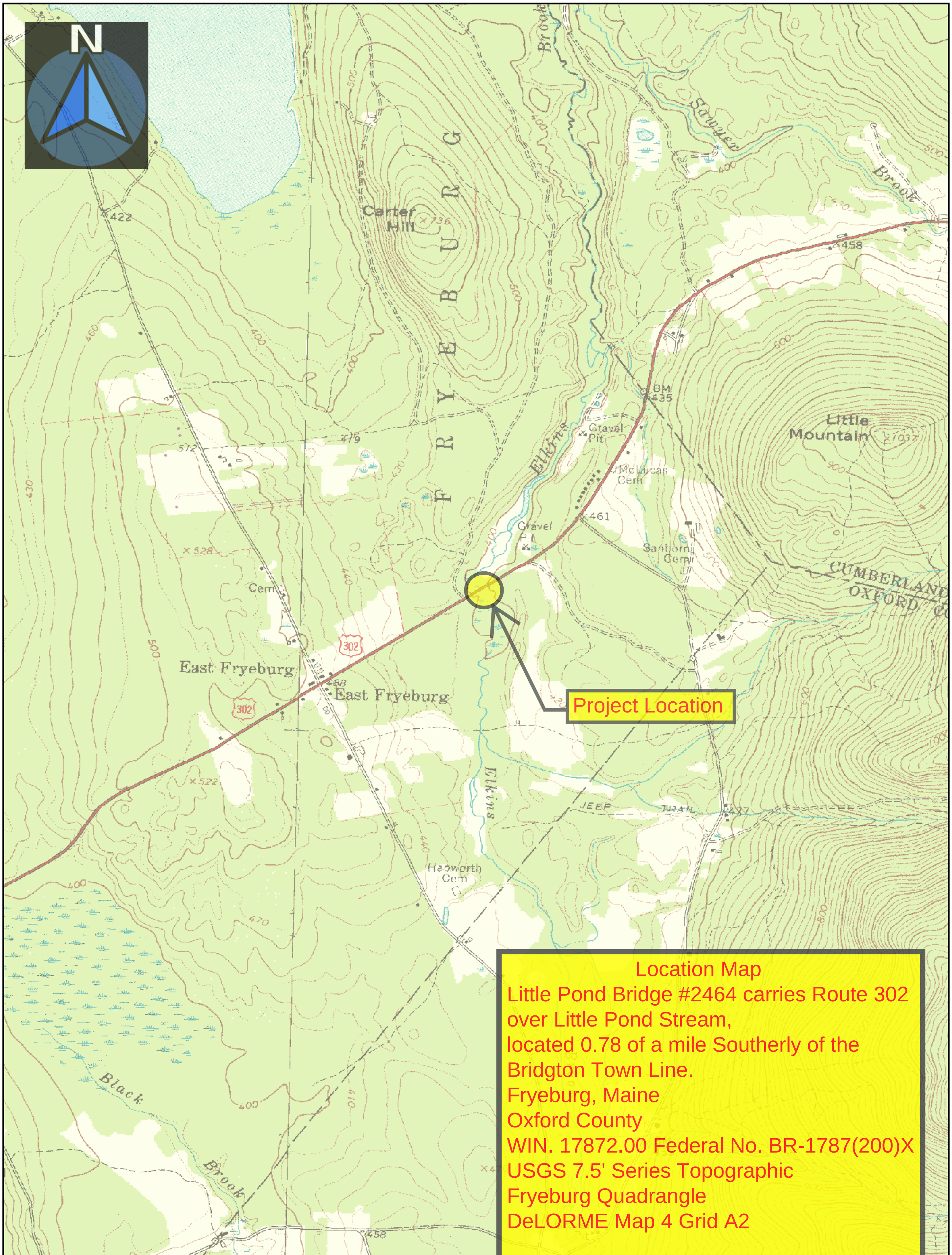
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements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

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Little Pond Bridge in Fryeburg in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

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

Sheets

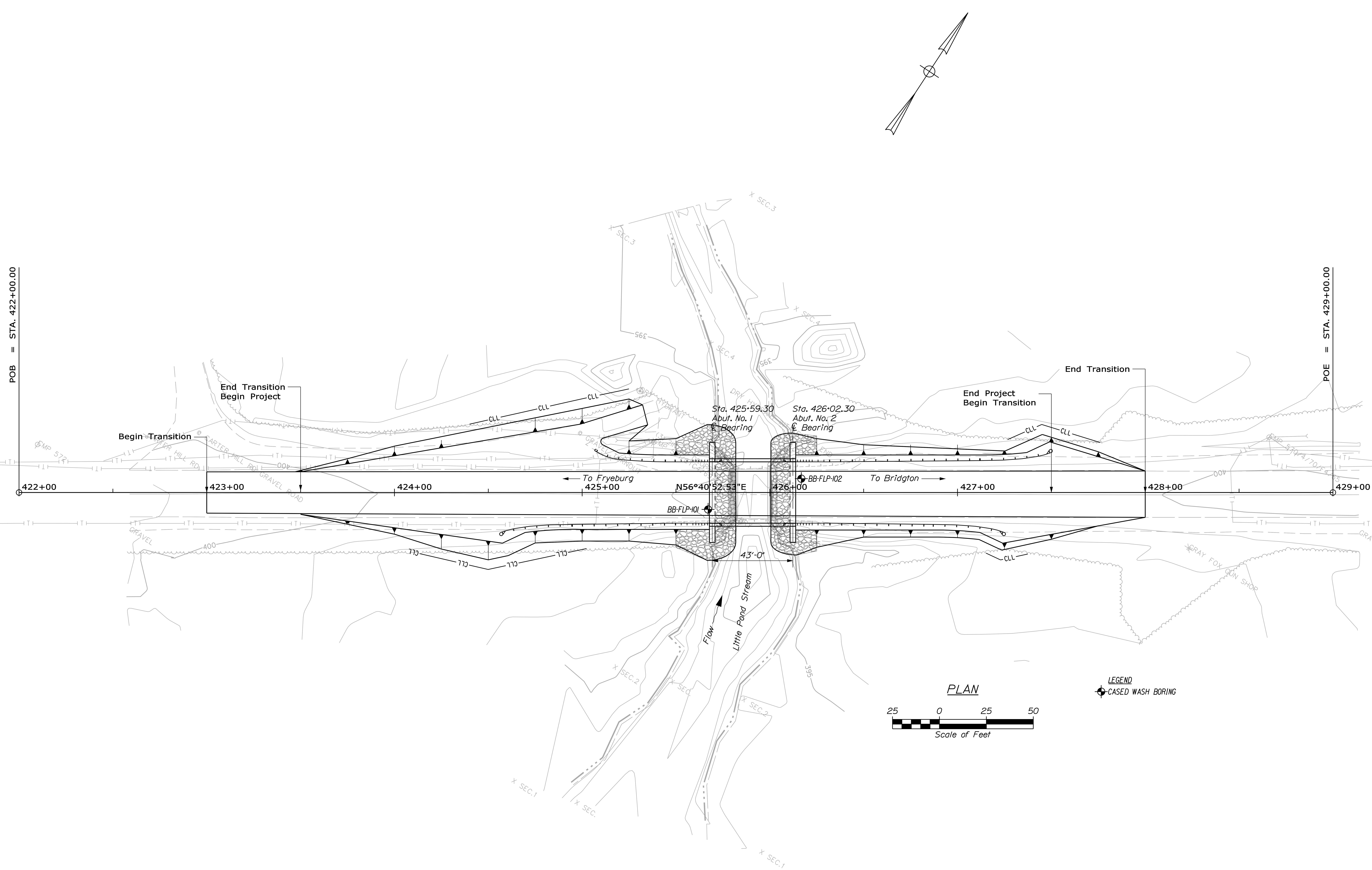


Location Map
Little Pond Bridge #2464 carries Route 302
over Little Pond Stream,
located 0.78 of a mile Southerly of the
Bridgton Town Line.
Fryeburg, Maine
Oxford County
WIN. 17872.00 Federal No. BR-1787(200)X
USGS 7.5' Series Topographic
Fryeburg Quadrangle
DeLORME Map 4 Grid A2

Map Scale 1:24000

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

Filename: ...\00\geotech\msta\006_BLP1.dgn   Division: GEOTECH   Username: terry.white   Date:10/27/2011



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1787(200)X

BRIDGE PLANS

WIN
17872.00

BRIDGE NO. 2464

	MAP	BY	DATE	SIGNATURE
PROJ. MANAGER				
DESIGN-DETAILED	K.MAGUIRE	T.WHITE	JUN 2011	
CHECKED-REVIEWED				
DESIGN2-DETAILED2				
DESIGN3-DETAILED3				P.E. NUMBER
REVISIONS 1				
REVISIONS 2				
REVISIONS 3				
REVISIONS 4				DATE
FIELD CHANGES				

LITTLE POND BRIDGE
LITTLE POND STREAM
FRYEBURG OXFORD COUNTY

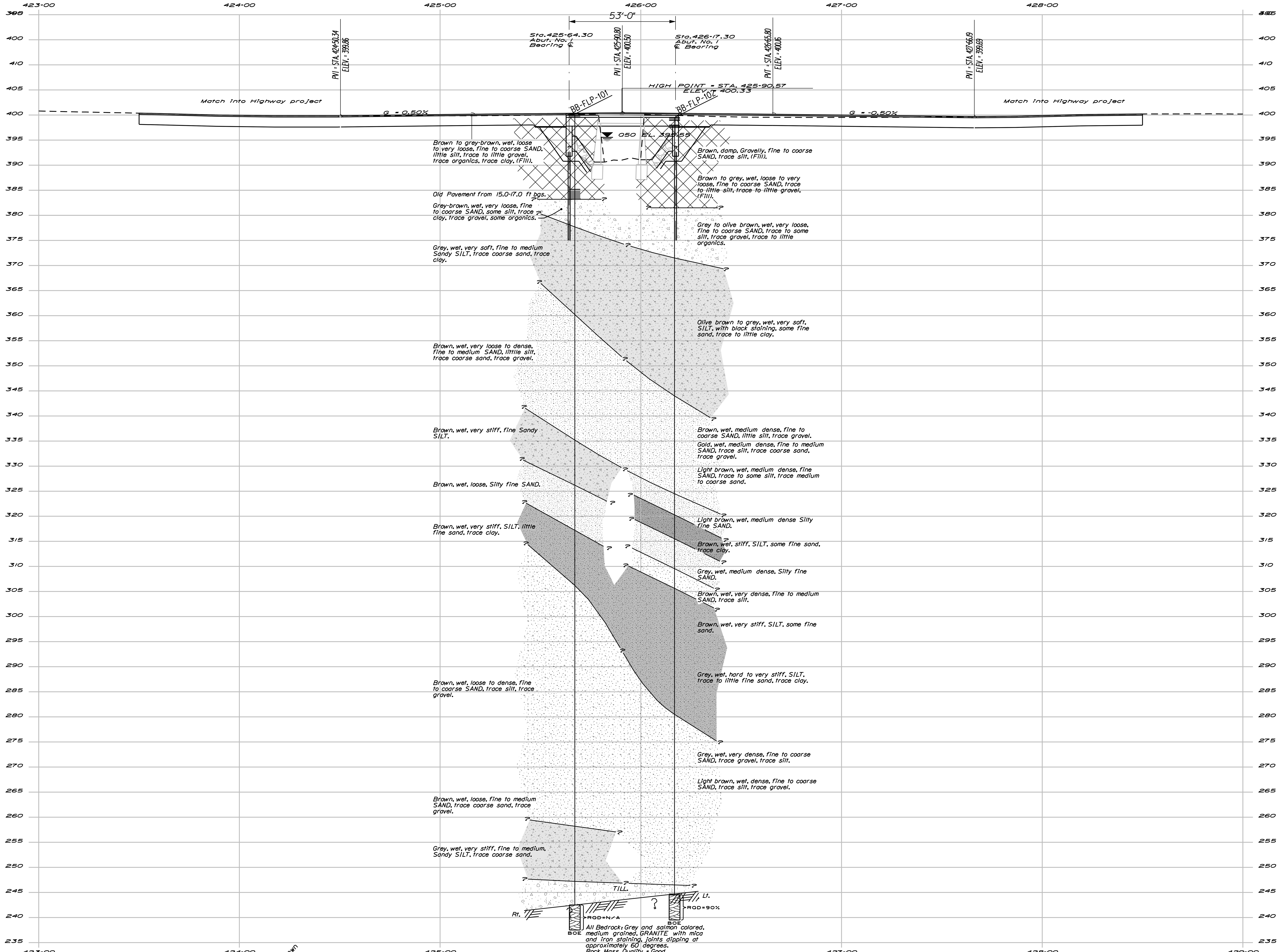
BORING LOCATION PLAN

SHEET NUMBER

2

OF 5

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:10/27/2011



PROFILE

HORIZ 25 0 25 50

VERT 10 0 10 20

SCALE

LEGEND

Boring No., Offset, if shown

Pavement Thickness if applicable

Strata Interface

Boring — RQD= Rock Quality Designation for Rock Core Sample

BOE= Bottom Of Exploration

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1787(200)X

BRIDGE PLANS

WIN 17872.00

BRIDGE NO. 2464

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

SIGNATURE

P.E. NUMBER

DATE

LITTLE POND BRIDGE
LITTLE POND STREAM
FRYEBURG OXFORD COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

3

OF 5

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

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	Elevation (ft.)	400.2	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/10/11-5/12/11	Drilling Method:	Cased Wash Boring	Core Barrel:	BX
Boring Location:	425+67.1, 9.0 ft Rt.	Casing ID/OD:	NW	Water Level*:	7.0 ft bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	399.53		8" Pavement — 0.67	
5	1D	24/7	5.00 - 7.00	3/2/4/3	6	8				Brown, wet, loose, fine to coarse SAND, little silt, little gravel, (Fill).	
10	2D	24/4	10.00 - 12.00	WOH/WOH/WOH/WOH	---		WOH			Brown, wet, very loose, fine to coarse SAND, little gravel, little silt, (Fill).	
							2				
							2				
							5				
							8				
15	3D	24/17	15.00 - 17.00	2/1/WOH/WOH	1	1	3			Grey-brown, wet, very loose, fine to coarse SAND, little silt, trace gravel, trace clay, little organics, old pavement, (Fill).	G#244976 A-2-4, SC-SM WC=31.4%
							3				
							7	383.20		17.00	
							7				
							7				
20	4D	24/16	20.00 - 22.00	1/2/WOH/WOH	2	3	9			Grey-brown, wet, very loose, fine to coarse SAND, some silt, trace clay, trace gravel, some organics.	G#244977 A-2-4, SC-SM WC=52.0%
							9				
							17	377.70		22.50	
							22				
							20				
25	5D	24/20	25.00 - 27.00	WOH/WOH/WOH/WOH	---		20			Grey, wet, very soft, fine to medium Sandy SILT, trace coarse sand, trace clay.	G#244978 A-4, ML WC=27.7%
							19				
							18				
							16				
							16				
30							18				
							15				
							18				
							16				
							18				
35	6D	24/18	35.00 - 37.00	WOH/WOH/WOH/WOH	---		34			Grey, wet, very soft, fine to medium Sandy SILT, trace coarse sand, trace clay.	G#244979 A-4, ML WC=24.2%
							28				
							30				
							28				
							28				
40							47	360.20		40.00	
							44				
							42				
							35				
							37				
45	7D	24/18	45.00 - 47.00	WOH/WOH/3/5	3	4	78			Brown, wet, very loose, fine to medium SAND, little silt, trace coarse sand.	G#244980 A-2-4, SM WC=17.8%
							59				
							47				
							53				
							72				
50							146				
							177				
							230				
							189				
							216				
55	8D	24/19	55.00 - 57.00	14/18/17/16	35	49	102			Brown, wet, dense, fine to medium SAND, little silt, trace coarse sand, trace gravel. 1.0 ft running sand. Kept casing full of water from 55.0 ft to bottom of boring.	G#244981 A-2-4, SM WC=22.0%
							116				
							132				
							138				
							138				
60							184				
							186				
							224				
							221				
							229				
65	9D	24/13	65.00 - 67.00	9/6/7/8	13	18	116	335.20		65.00 Brown, wet, very stiff, fine Sandy SILT.	G#244982 A-4, ML WC=22.1%
							108				
							111				
							132				
							178				
70							176				
							189				
							216				
							221				
							227	326.20		74.00	
75											

Remarks:

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

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

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine
Boring No.: BB-FLP-101
WIN: 17872.00
Driller: MaineDOT | Elevation (ft.) 400.2 | Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5/10/11-5/12/11 | Drilling Method: Cased Wash Boring | Core Barrel: BX
Boring Location: 425+67.1, 9.0 ft Rt. | Casing ID/OD: NW | Water Level*: 7.0 ft bgs.
Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test, PP = Pocket Penetrometer; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR/C = weight of rods or casing; WO1P = Weight of one person; Su = Insitu Field Vane Shear Strength (psf); Tv = Pocket Torvane Shear Strength (psf); qp = Unconfined Compressive Strength (ksf); N-uncorrected = Raw field SPT N-value; Hammer Efficiency Factor = Annual Calibration Value; N60 = SPT N-uncorrected corrected for hammer efficiency; N60 = (Hammer Efficiency Factor/60%)*N-uncorrected; Su(lab) = Lab Vane Shear Strength (psf); WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	10D	24/17	75.00 - 77.00	WOH/3/4/7	7	10	101			Brown, wet, loose, Silty fine SAND.	G#244983 A-4, SM WC=21.8%
							109				
							162				
							162				
							216				
80							217				
							221				
							324				
							225	317.20		83.00	
							230				
85	11D	24/14	85.00 - 87.00	19/10/7/9	17	24	216			Brown, wet, very stiff, SILT, little fine sand, trace clay.	G#244984 A-4, ML WC=22.0%
							162				
							270				
							216				
							189				
90							218				
							267				
							236				
							218				
							225	306.20		94.00	
95	12D	24/12	95.00 - 97.00	18/12/11/11	23	32	196			Brown, wet, dense, fine SAND, some silt.	
							184				
							216				
							219				
							240				
100							243				
							243				
							258	298.20		102.00	
							238				
							270				
105	13D		105.00 - 107.00	WOH/WOR/WOR/WOR	---		310			Brown, wet, loose, fine to coarse SAND, trace gravel, trace silt. 6.0 ft running sand, sample from running sand.	G#244985 A-1-b, SP WC=20.3%
							327				
							328				
							330				
							281				
110							329				
							346				
							324				
							297				
							292				
115	14D	24/16	115.00 - 117.00	35/21/13/13	34	48	381			Brown, wet, dense, fine to coarse SAND, trace silt, trace gravel.	G#244986 A-1-b, SP-SM WC=19.0%
							348				
							340				
							338				
							340				
120							344				
							375				
							382				
							388				
							432				
125	15D	24/24	125.00 - 127.00	WOR/WOR/WOR/WOR	---		324			Brown, wet, loose, fine to coarse SAND trace silt, trace gravel. 6.0 ft running sand, sample from running sand.	G#244987 A-1-b, SP WC=18.9%
							378				
							432				
							378				
							396				
130							432				
							436				
							378				
							334				
							362				
135	16D	24/10	135.00 - 137.00	WOR/WOR/WOR/WOR	---		280			Brown, wet, loose, fine to medium SAND trace coarse sand, trace gravel. 7.0 ft running sand, sample from running sand.	G#244988 A-1-b, SP WC=21.5%
							405				
							384				
							351				
							378				
140							378				
							405				
							351	258.20		142.00	
							370				
							324				
145	17D	24/14	145.00 - 147.00	20/11/9/12	20	28	178			Grey, wet, very stiff, fine to medium Sandy SILT, trace coarse sand.	G#244989 A-4, ML WC=21.4%
							221				
							235				
							279				
							337				
150											

Remarks:

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

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS
Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine
Boring No.: BB-FLP-101
WIN: 17872.00
Driller: MaineDOT | Elevation (ft.) 400.2 | Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 5/10/11-5/12/11 | Drilling Method: Cased Wash Boring | Core Barrel: BX
Boring Location: 425+67.1, 9.0 ft Rt. | Casing ID/OD: NW | Water Level*: 7.0 ft bgs.
Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐
Definitions: D = Split Spoon Sample; MD = Unsuccessful Split Spoon Sample attempt; U = Thin Wall Tube Sample; MU = Unsuccessful Thin Wall Tube Sample attempt; V = Insitu Vane Shear Test, PP = Pocket Penetrometer; MV = Unsuccessful Insitu Vane Shear Test attempt; R = Rock Core Sample; SSA = Solid Stem Auger; HSA = Hollow Stem Auger; RC = Roller Cone; WOH = weight of 140lb. hammer; WOR/C = weight of rods or casing; WO1P = Weight of one person; Su = Insitu Field Vane Shear Strength (psf); Tv = Pocket Torvane Shear Strength (psf); qp = Unconfined Compressive Strength (ksf); N-uncorrected = Raw field SPT N-value; Hammer Efficiency Factor = Annual Calibration Value; N60 = SPT N-uncorrected corrected for hammer efficiency; N60 = (Hammer Efficiency Factor/60%)*N-uncorrected; Su(lab) = Lab Vane Shear Strength (psf); WC = water content, percent; LL = Liquid Limit; PL = Plastic Limit; PI = Plasticity Index; G = Grain Size Analysis; C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150							340				
							392				
							383				
							440	247.20		153.00 TILL.	
							303				
155											
	R1	60/0	157.70 - 162.70	RQD = N/A%			BX Core	242.50		157.70 Top of Bedrock at Elev. 242.5 ft. R1: Bedrock: GRANITE (Sebago Pluton). R1: Core Times (min:sec) 157.7-158.7 ft (8:30) 158.7-159.7 ft (8:00) 159.7-160.7 ft (9:00) 160.7-161.7 ft (8:30) 161.7-162.7 ft (8:30) 0% Recovery Good solid drilling, no breaks, no seams, core barrel broke off in bottom of boring.	
160											
								237.50		162.70 Bottom of Exploration at 162.70 feet below ground surface.	
165											
170											
175											

Remarks:

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

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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1787(200)X

BRIDGE NO. 2464
WIN 17872.00
BRIDGE PLANS

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

SIGNATURE
P.E. NUMBER
DATE

LITTLE POND BRIDGE
LITTLE POND STREAM
FRYEBURG OXFORD COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\msta\009_BORING LOGS2.dgn   Division: GEOTECH   Username: terry.white   Date:10/27/2011

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	Elevation (ft.)	399.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Daggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	Casing ID/OD:	NW	Water Level*:	5.5 ft bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	398.92		7" Pavement ——0.58	
	1D	24/17	1.50 - 3.50	47/27/14/12	41	57		398.00 397.50		Brown, damp, Gravelly, fine to coarse SAND, trace silt, (Fill). ——1.50 Layer of Old Pavement from 1.5-2.0 ft bgs. ——2.00 Brown, damp, very dense, Gravelly, fine to coarse SAND, trace silt, (Fill).	G#245172 A-1-a, SW-SM WC=2.7%
5	2D	24/18	5.00 - 7.00	3/2/3/8	5	7				Brown, wet, loose, fine to coarse SAND, little silt, little gravel, (Fill).	G#245173 A-1-b, SM WC=18.2%
10	3D	24/3	10.00 - 12.00	WOR/WOR/1/1	1	1	3	389.50		——10.00 Grey, wet, very loose, fine to coarse SAND, trace silt, trace gravel, (Fill).	
							1				
							WOH				
							WOH	386.50		——13.00	
	4D	24/4	14.00 - 16.00	WOH/WOH/WOH/WOH	---		1			Brown, wet, very loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	
15							1				
							3				
							5				
							8	381.50		——18.00	
	5D	24/6	19.00 - 21.00	1/WOH/WOH/WOH	---		5			Grey, wet, very loose, fine to coarse SAND, trace gravel, trace silt.	
20							5				
							7				
							8				
							18	376.50		——23.00	
	6D	24/24	24.00 - 26.00	WOH/WOH/1/1	1	1	21			Olive-brown, wet, very loose, fine to medium SAND, some silt, trace coarse sand, trace gravel, little organics.	G#245174 A-2-4, SM WC=50.7%
25							20				
							18				
							35				
							35	371.50		——28.00	
	7D	24/24	29.00 - 31.00	WOH/WOH/WOH/WOH	---		40			Olive-brown, wet, very soft, SILT, some fine sand, trace clay.	G#245175 A-4, ML WC=50.2%
30							27				
							24				
							24				
							20	366.50		——33.00	
							19				
35	8D	24/24	35.00 - 37.00	WOH/WOH/WOH/WOH	---		32			Grey, wet, very soft, SILT, some fine sand.	G#244951 A-4, ML WC=32.9%
							26				
							20				
							20				
							21				
40	9D	24/18	40.00 - 42.00	WOR/WOR/WOR/WOR	---		29			Similar to above.	
							27				
							24				
							24				
							23				
45	10D	24/24	45.00 - 47.00	WOR/WOR/WOR/WOR	---		41			Grey, wet, very soft, SILT, some fine sand, trace clay, with black staining.	G#244952 A-4, ML WC=35.6%
							34				
							34				
							34				
							31				
50	11D	24/24	50.00 - 52.00	WOR/WOR/WOR/WOR	---		43			Grey, wet, very soft, SILT, little clay, little fine to medium sand.	G#244953 A-4, CL-ML WC=35.4%
							37				
							33				
							40				
							32				
55	12D	24/17	55.00 - 57.00	WOH/8/8/10	16	22	73	344.00		——55.50 Brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#244954 A-2-4, SM WC=18.9%
							84				
							186				
							196				
							189				
60	13D	24/20	60.00 - 62.00	3/9/12/20	21	29	144			Gold, wet, medium dense, fine to medium SAND, trace silt, trace coarse sand, trace gravel. 2.0 ft running sand. Kept a head of water in casing on all spoons from 60.0 ft bgs to bottom of boring.	G#244955 A-3, SP WC=22.5%
							190				
							221				
							216				
							221				
65	14D	24/15	65.00 - 67.00	1/4/9/15	13	18	135			Light brown, wet, medium dense, fine SAND, trace silt, trace medium to coarse sand, trace gravel.	G#244956 A-3, SP-SM WC=24.4%
							167				
							178				
							219				
							243				
70	15D	24/14	70.00 - 72.00	1/5/6/7	11	15	149			Light brown, moist, medium dense, fine SAND, some silt.	G#244957 A-2-4, SM WC=24.4%
							162				
							227				
							238				
75							294				

Remarks:

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

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Page 1 of 3

Boring No.: BB-FLP-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller: MaineDOT | Elevation (ft.) 399.5 | Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/14/11-5/3,5,9/11 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 426+16.8, 7.4 ft Lt. | Casing ID/OD: NW | Water Level*: 5.5 ft bgs.
Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	16D	24/15	75.50 - 77.50	4/4/7/8	11	15	226			Light brown, wet, medium dense, Silty fine SAND.	G#244958 A-4, SM WC=24.4%
							162				
							228				
							237				
							253	320.50		——79.00	
80	17D	24/16	80.00 - 82.00	WOH/4/7/7	11	15	146			Brown, wet, stiff, SILT, some fine sand, trace clay. 5.0 ft running sand.	G#244959 A-4, ML WC=22.4%
							169				
							306				
							303				
							308	315.50		——84.00	
85	18D	24/16	85.00 - 87.00	4/7/10/12	17	24	204			Grey, wet, medium dense, Silty fine SAND. 1.5 ft running sand.	G#244960 A-2-4, SM WC=25.2%
							259				
							338				
							305				
							300				
90	19D	24/17	90.00 - 92.00	5/18/20/21	38	53	244	309.50		——90.00 Brown, wet, very dense, fine to medium SAND, trace silt. 1.5 ft running sand.	G#244961 A-3, SP WC=23.5%
							267				
							257				
							285				
							310	305.50		——94.00	
95	20D	24/16	95.50 - 97.50	7/7/11/14	18	25	209			Brown, wet, very stiff, SILT, some fine sand.	G#244962 A-4, ML WC=25.6%
							212				
							245				
							260				
							249				
100	21D	24/17	100.00 - 102.00	21/13/11/11	24	34	208			Grey-brown, wet, hard, SILT, little fine sand, trace clay.	G#244963 A-4, ML WC=25.4%
							284				
							345				
							342				
							375				
105	22D	24/18	105.00 - 107.00	21/14/12/12	26	36	200			Grey, wet, hard, SILT, little fine sand, trace clay.	G#244964 A-4, ML WC=25.7%
							327				
							379				
							324				
							338				
110	23D	24/20	110.00 - 112.00	1/7/9/12	16	22	240			Grey, wet, very stiff, SILT, trace clay, trace fine sand.	G#244965 A-4, ML WC=26.6%
							262				
							378				
							432				
							400				
115	24D	24/18	115.00 - 117.00	25/17/25/17	42	59	324			Grey, wet, hard, SILT, some clay, trace fine sand in 1/2"-1" layers.	G#244966 A-4, CL-ML WC=25.5%
							378				
							380				
							483				
							486	280.50		——119.00	
120	25D	24/14	120.50 - 122.50	13/20/38/36	58	81	336			Grey, wet, very dense, fine to coarse SAND, trace gravel, trace silt. 2.7 ft running sand. Roller Coned ahead to 125.0 ft bgs.	G#244967 A-1-b, SP WC=18.3%
							338				
							297				
							459				
							351				
125	26D	24/14	125.00 - 127.00	21/15/18/18	33	46	350			Light brown, wet, dense, fine to coarse SAND, little gravel, trace silt. 5.0 ft running sand.	G#244968 A-1-b, SP-SM WC=11.2%
							460				
							542				
							573				
							433				
130	27D	24/6	130.00 - 132.00	25/13/12/13	25	35	400			Light brown, wet, dense, fine to coarse SAND, trace silt, trace gravel. 15.0 ft running sand.	G#244969 A-1-b, SP-SM WC=15.2%
							380				
							387				
							503				
							588				
135	MD	24/0	135.00 - 137.00	13/10/17/19	27	38	550			Similar to above, in wash, 5.0 ft running sand.	
							427				
							492				
							527				
							485				
140	28D	24/18	140.50 - 142.50	16/9/16/24	25	35	352			Light brown, wet, dense, fine to coarse SAND, trace gravel, trace silt. Roller Coned ahead to 150.0 ft bgs.	G#244970 A-3, SP WC=20.3%
							351				
							459				
							621				
							491				
145							486				
							548				
							454				
							351				
150							351				

Remarks:

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

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Page 2 of 3

Boring No.: BB-FLP-102

Maine Department of Transportation

Soil/Rock Exploration Log

US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller: MaineDOT | Elevation (ft.) 399.5 | Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles/Daggett | Datum: NAVD88 | Sampler: Standard Split Spoon
Logged By: B. Wilder | Rig Type: CME 45C | Hammer Wt./Fall: 140#/30"
Date Start/Finish: 4/14/11-5/3,5,9/11 | Drilling Method: Cased Wash Boring | Core Barrel: NQ-2"
Boring Location: 426+16.8, 7.4 ft Lt. | Casing ID/OD: NW | Water Level*: 5.5 ft bgs.
Hammer Efficiency Factor: 0.84 | Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150	MD	24/0	150.00 - 152.00	WOR/WOR/WOR/WOR	---		354			Similar to above, 5.0 ft running sand, spoon sunk into running sand.	
							303				
							405				
							491	246.50		——153.00 TILL. [a]508 blows for 0.8 ft.	
							[a]508	244.70		——154.80	
155	R1	60/57	155.00 - 160.00	RQD = 90%			NQ-2			Top of Bedrock at Elev. 244.7 ft. R1: Bedrock: Grey and salmon colored, medium grained, GRANITE with mica and iron staining, joints diping at approximately 60 degrees (Sebago Pluton). Rock Mass Quality = Good. R1: Core Times (min:sec) 155.0-156.0 ft (2:30) 156.0-157.0 ft (2:00) 157.0-158.0 ft (2:00) 158.0-159.0 ft (2:35) 159.0-160.0 ft (2:11) 95% Recovery	
160								239.50		——160.00 Bottom of Exploration at 160.00 feet below ground surface.	
165											
170											
175											

Remarks:

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

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

Page 3 of 3

Boring No.: BB-FLP-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1787(200)X

BRIDGE PLANS

WIN
17872.00

BRIDGE NO. 2464

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

SIGNATURE

P.E. NUMBER

DATE

LITTLE POND BRIDGE
LITTLE POND STREAM
FRYEBURG   OXFORD COUNTY

BORING LOGS

SHEET NUMBER

5

OF 5

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	399.53		8" Pavement —0.67	
5	1D	24/7	5.00 - 7.00	3/2/4/3	6	8				Brown, wet, loose, fine to coarse SAND, little silt, little gravel, (Fill).	
10	2D	24/4	10.00 - 12.00	WOH/WOH/WOH/ WOH	---		WOH			Brown, wet, very loose, fine to coarse SAND, little gravel, little silt, (Fill).	
							2				
							2				
							5				
							8				
15	3D	24/17	15.00 - 17.00	2/1/WOH/WOH	1	1	3			Grey-brown, wet, very loose, fine to coarse SAND, little silt, trace gravel, trace clay, little organics, old pavement, (Fill).	G#244976 A-2-4, SC-SM WC=31.4%
							3	383.20		—17.00	
							7				
							7				
							7				
20	4D	24/16	20.00 - 22.00	1/2/WOH/WOH	2	3	9			Grey-brown, wet, very loose, fine to coarse SAND, some silt, trace clay, trace gravel, some organics.	G#244977 A-2-4, SC-SM WC=52.0%
							9				
							17	377.70		—22.50	
							22				
25							20				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/20	25.00 - 27.00	WOH/WOH/WOH/ WOH	---		20			Grey, wet, very soft, fine to medium Sandy SILT, trace coarse sand, trace clay.	G#244978 A-4, ML WC=27.7%
							19				
							18				
							16				
							16				
30							18				
							15				
							18				
							16				
							18				
35	6D	24/18	35.00 - 37.00	WOH/WOH/WOH/ WOH	---		34			Grey, wet, very soft, fine to medium Sandy SILT, trace coarse sand, trace clay.	G#244979 A-4, ML WC=24.2%
							28				
							30				
							28				
							28				
40							47	360.20		40.00	
							44				
							42				
							35				
							37				
45	7D	24/18	45.00 - 47.00	WOH/WOH/3/5	3	4	78			Brown, wet, very loose, fine to medium SAND, little silt, trace coarse sand.	G#244980 A-2-4, SM WC=17.8%
							59				
							47				
							53				
							72				
50											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50							146				
							177				
							230				
							189				
							216				
55	8D	24/19	55.00 - 57.00	14/18/17/16	35	49	102			Brown, wet, dense, fine to medium SAND, little silt, trace coarse sand, trace gravel. 1.0 ft running sand. Kept casing full of water from 55.0 ft to bottom of boring.	G#244981 A-2-4, SM WC=22.0%
							116				
							132				
							138				
							138				
60							184				
							186				
							224				
							221				
							229				
65	9D	24/13	65.00 - 67.00	9/6/7/8	13	18	116	335.20		65.00 Brown, wet, very stiff, fine Sandy SILT.	G#244982 A-4, ML WC=22.1%
							108				
							111				
							132				
							178				
70							176				
							189				
							216				
							221				
							227	326.20		74.00	
75											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	10D	24/17	75.00 - 77.00	WOH/3/4/7	7	10	101			Brown, wet, loose, Silty fine SAND.	G#244983 A-4, SM WC=21.8%
							109				
							162				
							162				
							216				
80							217				
							221				
							324				
							225	317.20		83.00	
							230				
85	11D	24/14	85.00 - 87.00	19/10/7/9	17	24	216			Brown, wet, very stiff, SILT, little fine sand, trace clay.	G#244984 A-4, ML WC=22.0%
							162				
							270				
							216				
							189				
90							218				
							267				
							236				
							218				
							225	306.20		94.00	
95	12D	24/12	95.00 - 97.00	18/12/11/11	23	32	196			Brown, wet, dense, fine SAND, some silt.	
							184				
							216				
							219				
100							240				

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream Location: Fryeburg, Maine	Boring No.: BB-FLP-101 WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100							243				
							243				
							258	298.20		102.00	
							238				
							270				
105	13D		105.00 - 107.00	WOH/WOR/WOR/ WOR	---		310			Brown, wet, loose, fine to coarse SAND, trace gravel, trace silt. 6.0 ft running sand, sample from running sand.	G#244985 A-1-b, SP WC=20.3%
							327				
							328				
							330				
							281				
110							329				
							346				
							324				
							297				
							292				
115	14D	24/16	115.00 - 117.00	35/21/13/13	34	48	381			Brown, wet, dense, fine to coarse SAND, trace silt, trace gravel.	G#244986 A-1-b, SP-SM WC=19.0%
							348				
							340				
							338				
							340				
120							344				
							375				
							382				
							388				
							432				
125											

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream Location: Fryeburg, Maine	Boring No.: BB-FLP-101 WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
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S_u = Insitu Field Vane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
125	15D	24/24	125.00 - 127.00	WOR/WOR/WOR/ WOR	---		324			Brown, wet, loose, fine to coarse SAND trace silt, trace gravel. 6.0 ft running sand, sample from running sand.	G#244987 A-1-b, SP WC=18.9%
							378				
							432				
							378				
							396				
130							432				
							436				
							378				
							334				
							362				
135	16D	24/10	135.00 - 137.00	WOR/WOR/WOR/ WOR	---		280			Brown, wet, loose, fine to medium SAND trace coarse sand, trace gravel. 7.0 ft running sand, sample from running sand.	G#244988 A-1-b, SP WC=21.5%
							405				
							384				
							351				
							378				
140							378				
							405				
							351	258.20		142.00	
							370				
							324				
145	17D	24/14	145.00 - 147.00	20/11/9/12	20	28	178			Grey, wet, very stiff, fine to medium Sandy SILT, trace coarse sand.	G#244989 A-4, ML WC=21.4%
							221				
							235				
							279				
							337				
150											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-101
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	400.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/10/11-5/12/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	BX
Boring Location:	425+67.1, 9.0 ft Rt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150							340				
							392				
							383				
							440	247.20		TILL. (153.00)	
							303				
155											
	R1	60/0	157.70 - 162.70	RQD = N/A%			BX Core	242.50		Top of Bedrock at Elev. 242.5 ft. R1: Bedrock: GRANITE (Sebago Pluton). R1: Core Times (min:sec) 157.7-158.7 ft (8:30) 158.7-159.7 ft (8:00) 159.7-160.7 ft (9:00) 160.7-161.7 ft (8:30) 161.7-162.7 ft (8:30) 0% Recovery Good solid drilling, no breaks, no seams, core barrel broke off in bottom of boring. (157.70)	
160											
								237.50		(162.70) **Bottom of Exploration at 162.70 feet below ground surface.**	
165											
170											
175											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	398.92		7" Pavement —0.58	
	1D	24/17	1.50 - 3.50	47/27/14/12	41	57		398.00		Brown, damp, Gravelly, fine to coarse SAND, trace silt, (Fill). —1.50	G#245172 A-1-a, SW-SM WC=2.7%
								397.50		Layer of Old Pavement from 1.5-2.0 ft bgs. —2.00	
										Brown, damp, very dense, Gravelly, fine to coarse SAND, trace silt, (Fill).	
5	2D	24/18	5.00 - 7.00	3/2/3/8	5	7				Brown, wet, loose, fine to coarse SAND, little silt, little gravel, (Fill).	G#245173 A-1-b, SM WC=18.2%
10	3D	24/3	10.00 - 12.00	WOR/WOR/1/1	1	1	3	389.50		—10.00 Grey, wet, very loose, fine to coarse SAND, trace silt, trace gravel, (Fill).	
							1				
							WOH				
							WOH	386.50		—13.00	
	4D	24/4	14.00 - 16.00	WOH/WOH/WOH/ WOH	---		1			Brown, wet, very loose, fine to coarse SAND, trace gravel, trace silt, (Fill).	
15							1				
							3				
							5				
							8	381.50		—18.00	
	5D	24/6	19.00 - 21.00	1/WOH/WOH/WOH	---		5			Grey, wet, very loose, fine to coarse SAND, trace gravel, trace silt.	
20							5				
							7				
							8				
							18	376.50		—23.00	
25	6D	24/24	24.00 - 26.00	WOH/WOH/1/1	1	1	21			Olive-brown, wet, very loose, fine to medium SAND, some silt, trace coarse sand, trace gravel, little organics.	G#245174 A-2-4, SM WC=50.7%

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							20				
							18				
							35				
							35	371.50		28.00	
	7D	24/24	29.00 - 31.00	WOH/WOH/WOH/ WOH	---		40			Olive-brown, wet, very soft, SILT, some fine sand, trace clay.	G#245175 A-4, ML WC=50.2%
30							27				
							24				
							24				
							20	366.50		33.00	
							19				
35	8D	24/24	35.00 - 37.00	WOH/WOH/WOH/ WOH	---		32			Grey, wet, very soft, SILT, some fine sand.	G#244951 A-4, ML WC=32.9%
							26				
							20				
							20				
							21				
40	9D	24/18	40.00 - 42.00	WOR/WOR/WOR/ WOR	---		29			Similar to above.	
							27				
							24				
							24				
							23				
45	10D	24/24	45.00 - 47.00	WOR/WOR/WOR/ WOR	---		41			Grey, wet, very soft, SILT, some fine sand, trace clay, with black staining.	G#244952 A-4, ML WC=35.6%
							34				
							34				
							34				
50							31				

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Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/24	50.00 - 52.00	WOR/WOR/WOR/ WOR	---		43			Grey, wet, very soft, SILT, little clay, little fine to medium sand.	G#244953 A-4, CL-ML WC=35.4%
							37				
							33				
							40				
							32				
55	12D	24/17	55.00 - 57.00	WOH/8/8/10	16	22	73	344.00		55.50 Brown, wet, medium dense, fine to coarse SAND, little silt, trace gravel.	G#244954 A-2-4, SM WC=18.9%
							84				
							186				
							196				
							189				
60	13D	24/20	60.00 - 62.00	3/9/12/20	21	29	144			Gold, wet, medium dense, fine to medium SAND, trace silt, trace coarse sand, trace gravel.	G#244955 A-3, SP WC=22.5%
							190			2.0 ft running sand. Kept a head of water in casing on all spoons from 60.0 ft bgs to bottom of boring.	
							221				
							216				
							221				
65	14D	24/15	65.00 - 67.00	1/4/9/15	13	18	135			Light brown, wet, medium dense, fine SAND, trace silt, trace medium to coarse sand, trace gravel.	G#244956 A-3, SP-SM WC=24.4%
							167				
							178				
							219				
							243				
70	15D	24/14	70.00 - 72.00	1/5/6/7	11	15	149			Light brown, moist, medium dense, fine SAND, some silt.	G#244957 A-2-4, SM WC=24.4%
							162				
							227				
							238				
75							294				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	16D	24/15	75.50 - 77.50	4/4/7/8	11	15	226			Light brown, wet, medium dense, Silty fine SAND.	G#244958 A-4, SM WC=24.4%
							162				
							228				
							237				
							253	320.50		79.00	
80	17D	24/16	80.00 - 82.00	WOH/4/7/7	11	15	146			Brown, wet, stiff, SILT, some fine sand, trace clay. 5.0 ft running sand.	G#244959 A-4, ML WC=22.4%
							169				
							306				
							303				
							308	315.50		84.00	
85	18D	24/16	85.00 - 87.00	4/7/10/12	17	24	204			Grey, wet, medium dense, Silty fine SAND. 1.5 ft running sand.	G#244960 A-2-4, SM WC=25.2%
							259				
							338				
							305				
							300	309.50		90.00	
90	19D	24/17	90.00 - 92.00	5/18/20/21	38	53	244			Brown, wet, very dense, fine to medium SAND, trace silt. 1.5 ft running sand.	G#244961 A-3, SP WC=23.5%
							267				
							257				
							285				
							310	305.50		94.00	
95	20D	24/16	95.50 - 97.50	7/7/11/14	18	25	209			Brown, wet, very stiff, SILT, some fine sand.	G#244962 A-4, ML WC=25.6%
							212				
							245				
							260				
100							249				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100	21D	24/17	100.00 - 102.00	21/13/11/11	24	34	208			Grey-brown, wet, hard, SILT, little fine sand, trace clay.	G#244963 A-4, ML WC=25.4%
							284				
							345				
							342				
							375				
105	22D	24/18	105.00 - 107.00	21/14/12/12	26	36	200			Grey, wet, hard, SILT, little fine sand, trace clay.	G#244964 A-4, ML WC=25.7%
							327				
							379				
							324				
							338				
110	23D	24/20	110.00 - 112.00	1/7/9/12	16	22	240			Grey, wet, very stiff, SILT, trace clay, trace fine sand.	G#244965 A-4, ML WC=26.6%
							262				
							378				
							432				
							400				
115	24D	24/18	115.00 - 117.00	25/17/25/17	42	59	324			Grey, wet, hard, SILT, some clay, trace fine sand in 1/2"-1" layers.	G#244966 A-4, CL-ML WC=25.5%
							378				
							380				
							483				
							486	280.50		119.00	
120	25D	24/14	120.50 - 122.50	13/20/38/36	58	81	336			Grey, wet, very dense, fine to coarse SAND, trace gravel, trace silt. 2.7 ft running sand. Roller Coned ahead to 125.0 ft bgs.	G#244967 A-1-b, SP WC=18.3%
							338				
							297				
							459				
125							351				

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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LL = Liquid Limit
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C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
125	26D	24/14	125.00 - 127.00	21/15/18/18	33	46	350			Light brown, wet, dense, fine to coarse SAND, little gravel, trace silt. 5.0 ft running sand.	G#244968 A-1-b, SP-SM WC=11.2%
							460				
							542				
							573				
							433				
130	27D	24/6	130.00 - 132.00	25/13/12/13	25	35	400			Light brown, wet, dense, fine to coarse SAND, trace silt, trace gravel. 15.0 ft running sand.	G#244969 A-1-b, SP-SM WC=15.2%
							380				
							387				
							503				
							588				
135	MD	24/0	135.00 - 137.00	13/10/17/19	27	38	550			Similar to above, in wash, 5.0 ft running sand.	
							427				
							492				
							527				
							485				
140	28D	24/18	140.50 - 142.50	16/9/16/24	25	35	352			Light brown, wet, dense, fine to coarse SAND, trace gravel, trace silt.	G#244970 A-3, SP WC=20.3%
							351				
							459			Roller Coned ahead to 150.0 ft bgs.	
							621				
							491				
145							486				
							548				
							454				
							351				
							351				
150											

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Little Pond Bridge #2464 carries Rt. 302 over Little Pond Stream
Location: Fryeburg, Maine

Boring No.: BB-FLP-102
WIN: 17872.00

Driller:	MaineDOT	**Elevation (ft.)**	399.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/11-5/3,5,9/11	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	426+16.8, 7.4 ft Lt.	**Casing ID/OD:**	NW	**Water Level*:**	5.5 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒	Hydraulic ☐	Rope & Cathead ☐

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WOH = weight of 140lb. hammer
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WO1P = Weight of one person
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
150	MD	24/0	150.00 - 152.00	WOR/WOR/WOR/ WOR	---		354			Similar to above, 5.0 ft running sand, spoon sunk into running sand.	
							303				
							405				
							491	246.50		153.00 TILL.	
							a508			a508 blows for 0.8 ft.	
155	R1	60/57	155.00 - 160.00	RQD = 90%			NQ-2	244.70		154.80 Top of Bedrock at Elev. 244.7 ft. R1: Bedrock: Grey and salmon colored, medium grained, GRANITE with mica and iron staining, joints diping at approximately 60 degrees (Sebago Pluton). Rock Mass Quality = Good. R1: Core Times (min:sec) 155.0-156.0 ft (2:30) 156.0-157.0 ft (2:00) 157.0-158.0 ft (2:00) 158.0-159.0 ft (2:35) 159.0-160.0 ft (2:11) 95% Recovery	
160								239.50		160.00 **Bottom of Exploration at 160.00 feet below ground surface.**	
165											
170											
175											

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Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Fryeburg Work Number: 17872.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-FLP-101, 3D	425+67.1	9.0 Rt.	15.0-17.0	244976	1	31.4			SC-SM	A-2-4	II
BB-FLP-101, 4D	425+67.1	9.0 Rt.	20.0-22.0	244977	1	52.0			SC-SM	A-2-4	II
BB-FLP-101, 5D	425+67.1	9.0 Rt.	25.0-27.0	244978	1	27.7			ML	A-4	IV
BB-FLP-101, 6D	425+67.1	9.0 Rt.	35.0-37.0	244979	1	24.2			ML	A-4	IV
BB-FLP-101, 7D	425+67.1	9.0 Rt.	45.0-47.0	244980	1	17.8			SM	A-2-4	II
BB-FLP-101, 8D	425+67.1	9.0 Rt.	55.0-57.0	244981	2	22.0			SM	A-2-4	II
BB-FLP-101, 9D	425+67.1	9.0 Rt.	65.0-67.0	244982	2	22.1			ML	A-4	IV
BB-FLP-101, 10D	425+67.1	9.0 Rt.	75.0-77.0	244983	2	21.8			SM	A-4	III
BB-FLP-101, 11D	425+67.1	9.0 Rt.	85.0-87.0	244984	2	22.0			ML	A-4	IV
BB-FLP-101, 13D	425+67.1	9.0 Rt.	105.0-107.0	244985	2	20.3			SP	A-1-b	0
BB-FLP-101, 14D	425+67.1	9.0 Rt.	115.0-117.0	244986	3	19.0			SP-SM	A-1-b	0
BB-FLP-101, 15D	425+67.1	9.0 Rt.	125.0-127.0	244987	3	18.9			SP	A-1-b	0
BB-FLP-101, 16D	425+67.1	9.0 Rt.	135.0-137.0	244988	3	21.5			SP	A-1-b	0
BB-FLP-101, 17D	425+67.1	9.0 Rt.	145.0-147.0	244989	3	21.4			ML	A-4	IV
BB-FLP-102, 1D	426+16.8	7.4 Lt.	1.5-3.5	245172	4	2.7			SW-SM	A-1-a	0
BB-FLP-102, 2D	426+16.8	7.4 Lt.	5.0-7.0	245173	4	18.2			SM	A-1-b	II
BB-FLP-102, 6D	426+16.8	7.4 Lt.	24.0-26.0	245174	4	50.7			SM	A-2-4	II
BB-FLP-102, 7D	426+16.8	7.4 Lt.	29.0-31.0	245175	4	50.2			ML	A-4	IV
BB-FLP-102, 8D	426+16.8	7.4 Lt.	35.0-37.0	244951	4	32.9			ML	A-4	IV
BB-FLP-102, 10D	426+16.8	7.4 Lt.	45.0-47.0	244952	4	35.6			ML	A-4	IV
BB-FLP-102, 11D	426+16.8	7.4 Lt.	50.0-52.0	244953	5	35.4			CL-ML	A-4	IV
BB-FLP-102, 12D	426+16.8	7.4 Lt.	55.5-57.0	244954	5	18.9			SM	A-2-4	II
BB-FLP-102, 13D	426+16.8	7.4 Lt.	60.0-62.0	244955	5	22.5			SP	A-3	0
BB-FLP-102, 14D	426+16.8	7.4 Lt.	65.0-67.0	244956	5	24.4			SP-SM	A-3	0
BB-FLP-102, 15D	426+16.8	7.4 Lt.	70.0-72.0	244957	5	24.4			SM	A-2-4	II
BB-FLP-102, 16D	426+16.8	7.4 Lt.	75.5-77.5	244958	5	24.4			SM	A-4	III
BB-FLP-102, 17D	426+16.8	7.4 Lt.	80.0-82.0	244959	6	22.4			ML	A-4	IV
BB-FLP-102, 18D	426+16.8	7.4 Lt.	85.0-87.0	244960	6	25.2			SM	A-2-4	II
BB-FLP-102, 19D	426+16.8	7.4 Lt.	90.0-92.0	244961	6	23.5			SP	A-3	0
BB-FLP-102, 20D	426+16.8	7.4 Lt.	95.5-97.5	244962	6	25.6			ML	A-4	IV
BB-FLP-102, 21D	426+16.8	7.4 Lt.	100.0-102.0	244963	6	25.4			ML	A-4	IV
BB-FLP-102, 22D	426+16.8	7.4 Lt.	105.0-107.0	244964	6	25.7			ML	A-4	IV
BB-FLP-102, 23D	426+16.8	7.4 Lt.	110.0-112.0	244965	7	26.6			ML	A-4	IV
BB-FLP-102, 24D	426+16.8	7.4 Lt.	115.0-117.0	244966	7	25.5			CL-ML	A-4	IV
BB-FLP-102, 25D	426+16.8	7.4 Lt.	120.5-122.5	244967	7	18.3			SP	A-1-b	0
BB-FLP-102, 26D	426+16.8	7.4 Lt.	125.0-127.0	244968	7	11.2			SP-SM	A-1-b	0
BB-FLP-102, 27D	426+16.8	7.4 Lt.	130.0-132.0	244969	7	15.2			SP-SM	A-1-b	0
BB-FLP-102, 28D	426+16.8	7.4 Lt.	140.5-142.5	244970	7	20.3			SP	A-3	0

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

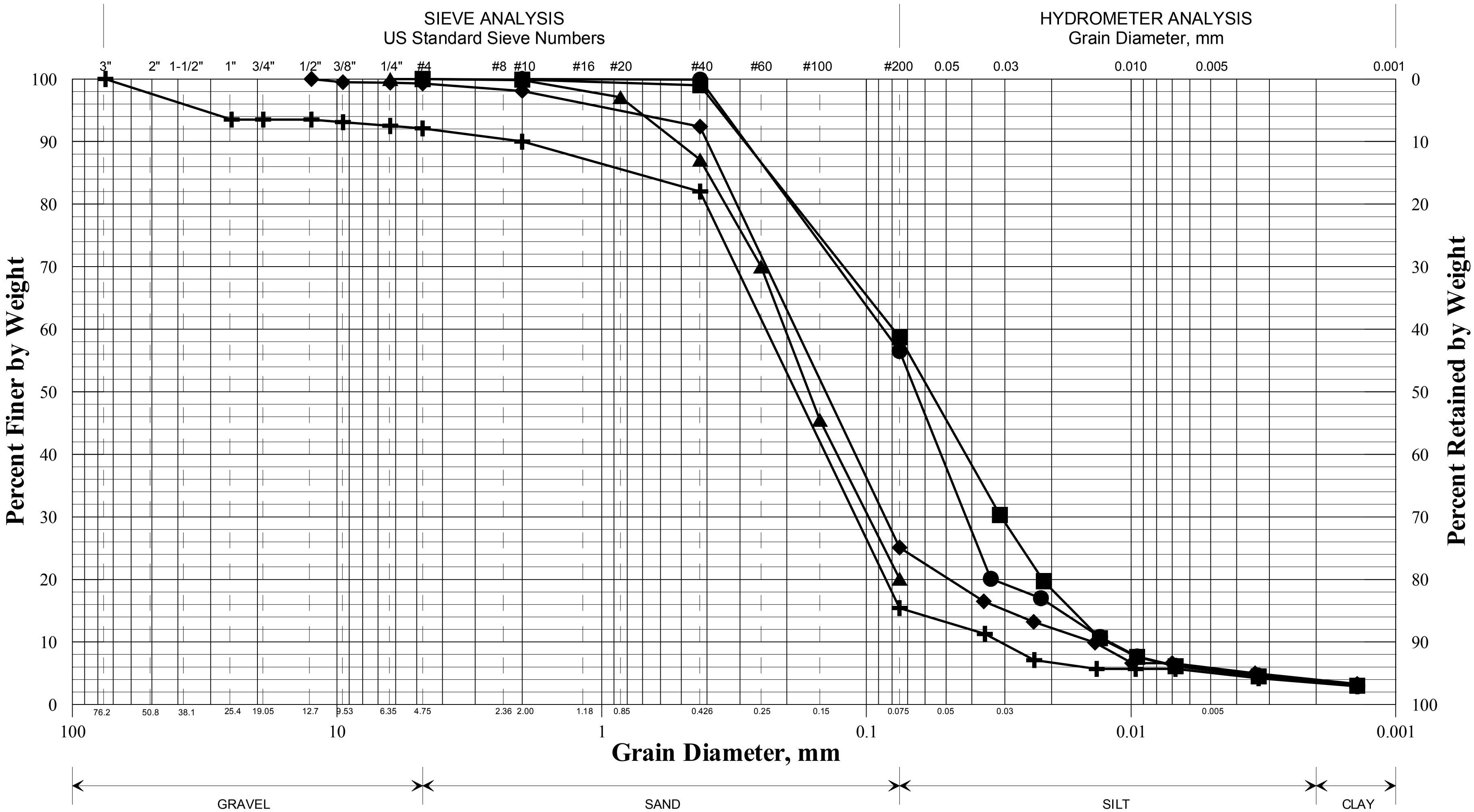
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-101/3D	425+67.1	9.0 RT	15.0-17.0	SAND, little silt, trace gravel, trace clay.	31.4			
◆	BB-FLP-101/4D	425+67.1	9.0 RT	20.0-22.0	SAND, some silt, trace clay, trace gravel.	52.0			
■	BB-FLP-101/5D	425+67.1	9.0 RT	25.0-27.0	Sandy SILT, trace clay.	27.7			
●	BB-FLP-101/6D	425+67.1	9.0 RT	35.0-37.0	Sandy SILT, trace clay.	24.2			
▲	BB-FLP-101/7D	425+67.1	9.0 RT	45.0-47.0	SAND, little silt.	17.8			
✖									

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Fryeburg
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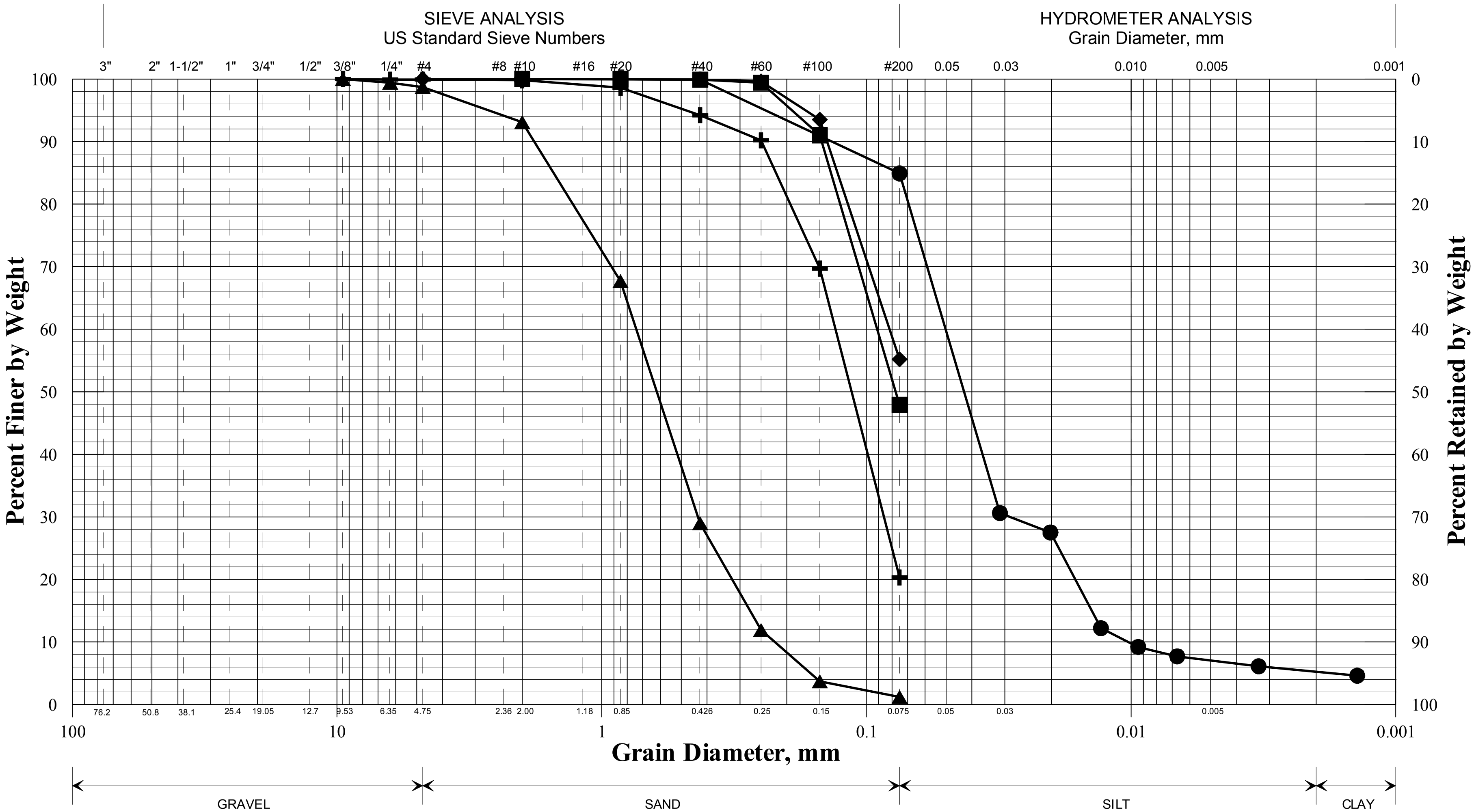
WHITE, TERRY A 6/21/2011

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-101/8D	425+67.1	9.0 RT	55.0-57.0	SAND, little silt, trace gravel.	22.0			
◆	BB-FLP-101/9D	425+67.1	9.0 RT	65.0-67.0	Sandy SILT.	22.1			
■	BB-FLP-101/10D	425+67.1	9.0 RT	75.0-77.0	Silty SAND.	21.8			
●	BB-FLP-101/11D	425+67.1	9.0 RT	85.0-87.0	SILT, little sand, trace clay.	22.0			
▲	BB-FLP-101/13D	425+67.1	9.0 RT	105.0-107.0	SAND, trace gravel, trace silt.	20.3			
✖									

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Fryeburg
Reported by/Date
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SHEET 2

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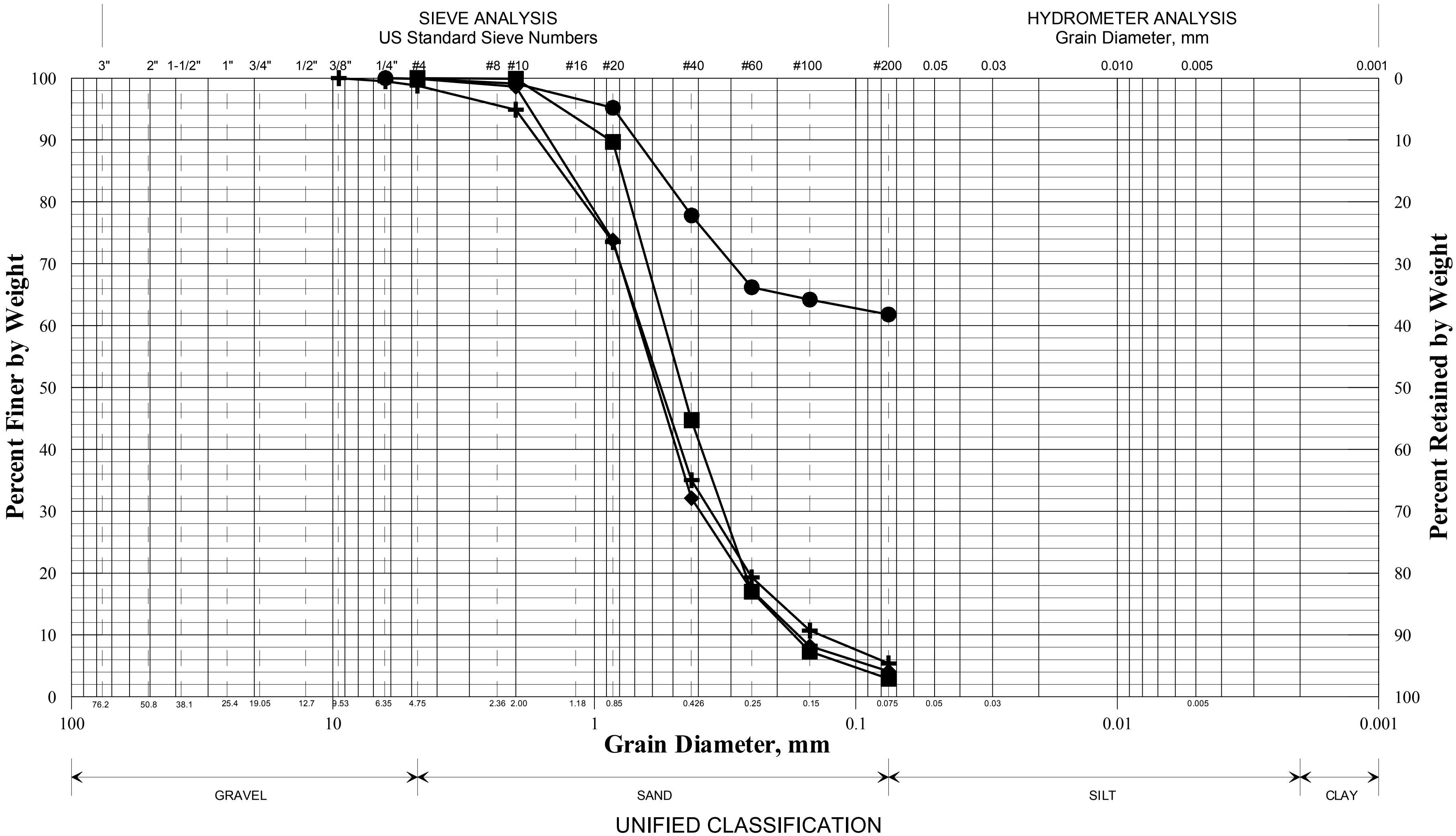
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-101/14D	425+67.1	9.0 RT	115.0-117.0	SAND, trace silt, trace gravel.	19.0			
◆	BB-FLP-101/15D	425+67.1	9.0 RT	125.0-127.0	SAND, trace silt, trace gravel.	18.9			
■	BB-FLP-101/16D	425+67.1	9.0 RT	135.0-137.0	SAND, trace silt.	21.5			
●	BB-FLP-101/17D	425+67.1	9.0 RT	145.0-147.0	Sandy SILT.	21.4			
▲									
✖									

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SHEET 3

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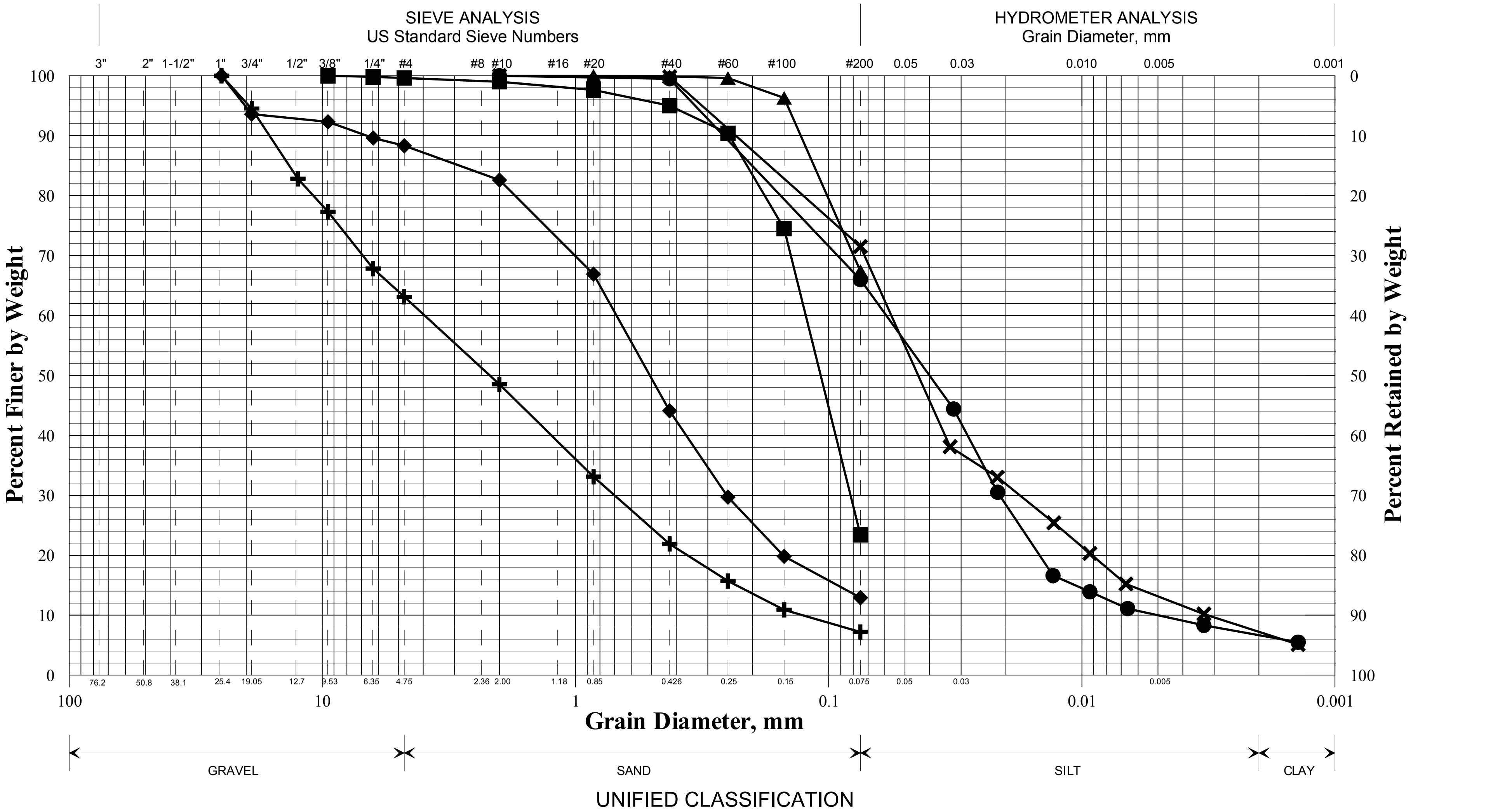
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-102/1D	426+16.8	7.4 LT	1.5-3.5	Gravelly SAND, trace silt.	2.7			
◆	BB-FLP-102/2D	426+16.8	7.4 LT	5.0-7.0	SAND, little silt, little gravel.	18.2			
■	BB-FLP-102/6D	426+16.8	7.4 LT	24.0-26.0	SAND, some silt, trace gravel.	50.7			
●	BB-FLP-102/7D	426+16.8	7.4 LT	29.0-31.0	SILT, some sand, trace clay.	50.2			
▲	BB-FLP-102/8D	426+16.8	7.4 LT	35.0-37.0	SILT, some sand.	32.9			
✖	BB-FLP-102/10D	426+16.8	7.4 LT	45.0-47.0	SILT, some sand, trace clay.	35.6			

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Fryeburg
Reported by/Date
WHITE, TERRY A 6/9/2011

SHEET 4

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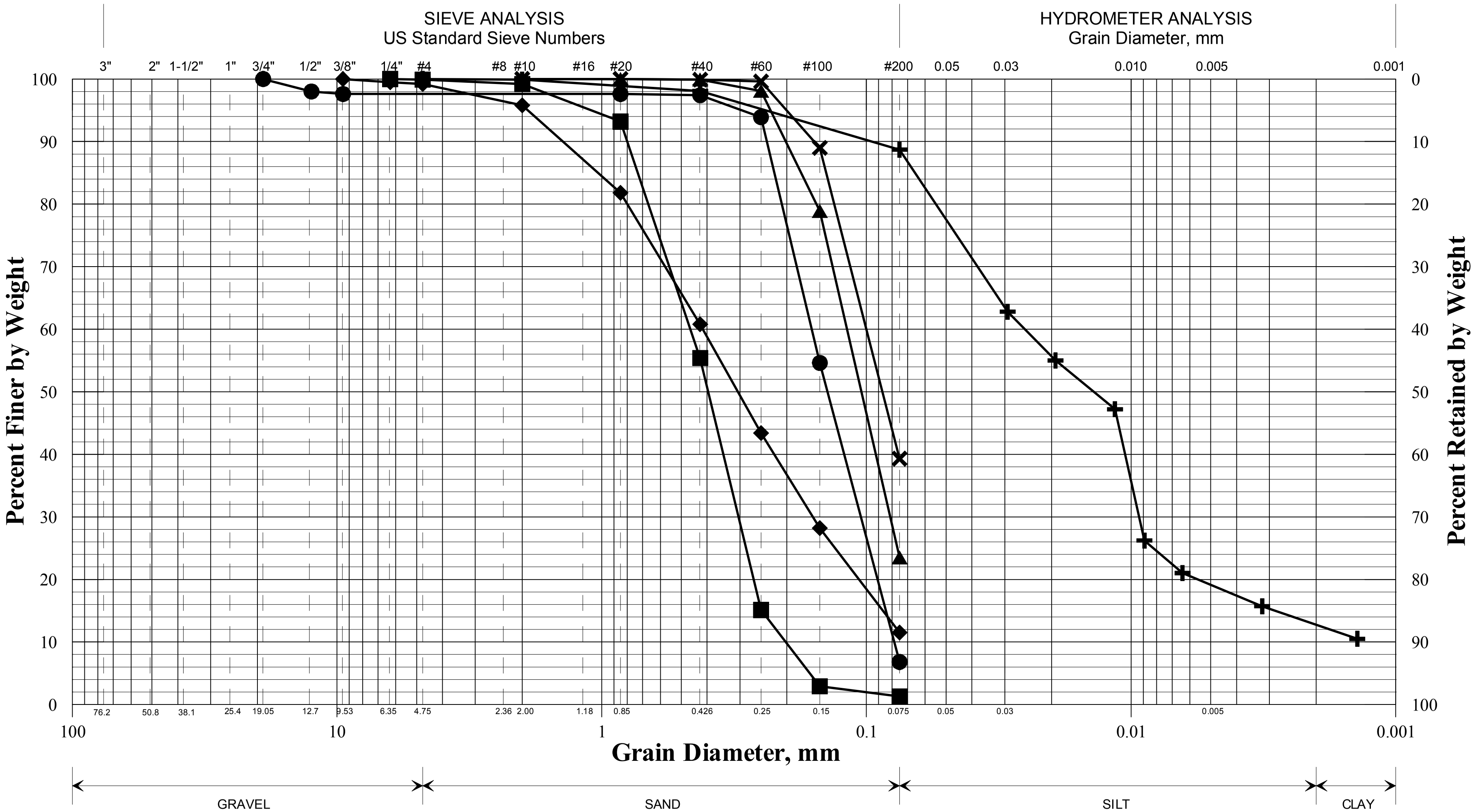
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	HB-FLP-102/11D	426+16.8	7.4 LT	50.0-52.0	SILT, little clay, little sand.	35.4			
◆	BB-FLP-102/12D	426+16.8	7.4 LT	55.5-57.0	SAND, little silt, trace gravel.	18.9			
■	BB-FLP-102/13D	426+16.8	7.4 LT	60.0-62.0	SAND, trace silt, trace gravel.	22.5			
●	BB-FLP-102/14D	426+16.8	7.4 LT	65.0-67.0	SAND, trace silt, trace gravel.	24.4			
▲	BB-FLP-102/15D	426+16.8	7.4 LT	70.0-72.0	SAND, some silt.	24.4			
✖	BB-FLP-102/16D	426+16.8	7.4 LT	75.5-77.5	Silty SAND.	24.4			

PIN
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Town
Fryeburg
Reported by/Date
WHITE, TERRY A 6/9/2011

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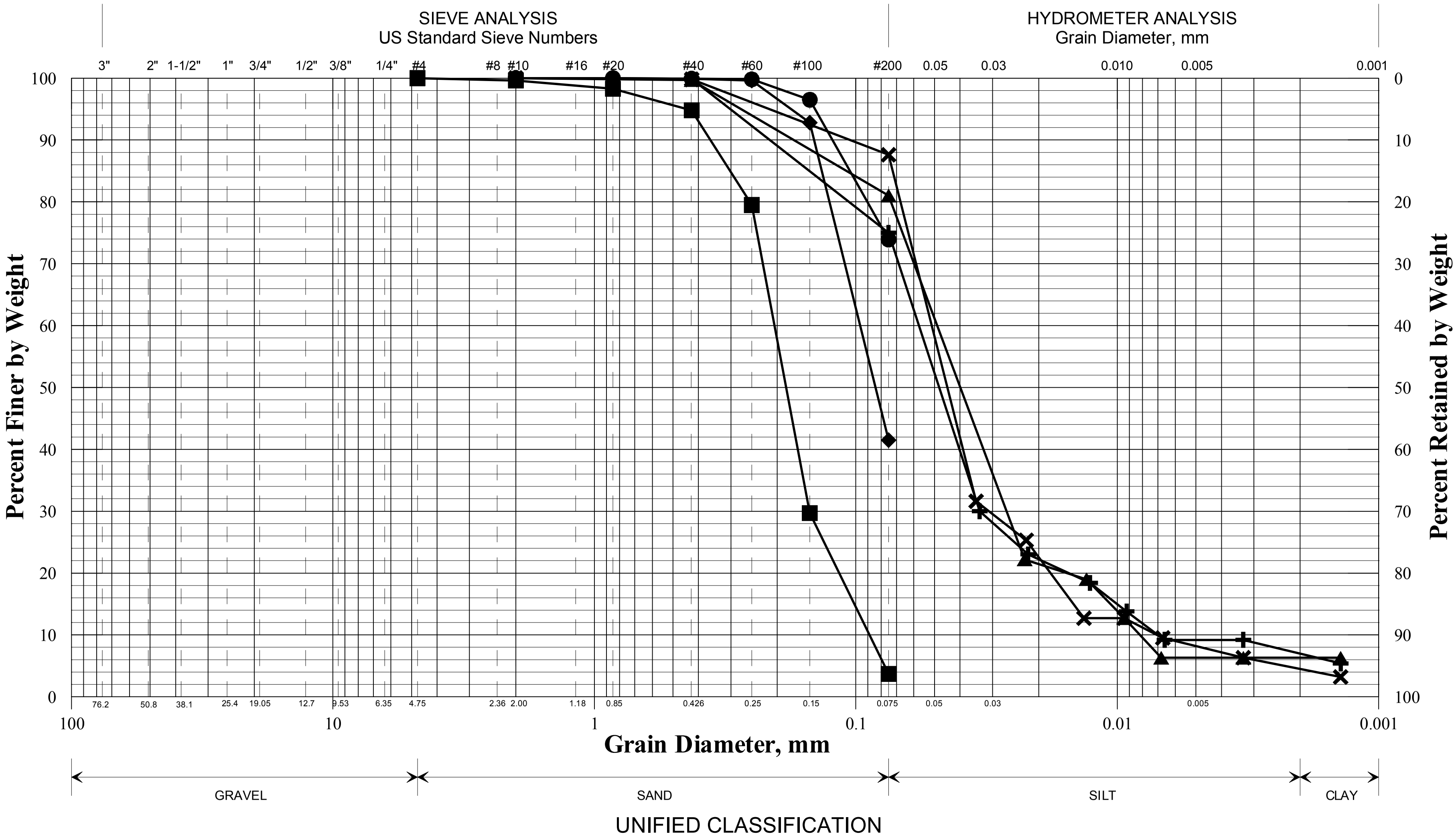
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-102/17D	426+16.8	7.4 LT	80.0-82.0	SILT, some sand, trace clay.	22.4			
◆	BB-FLP-102/18D	426+16.8	7.4 LT	85.0-87.0	Silty SAND.	25.2			
■	BB-FLP-102/19D	426+16.8	7.4 LT	90.0-92.0	SAND, trace silt.	23.5			
●	BB-FLP-102/20D	426+16.8	7.4 LT	95.5-97.5	SILT, some sand.	25.6			
▲	BB-FLP-102/21D	426+16.8	7.4 LT	100.0-102.0	SILT, little sand, trace clay.	25.4			
✖	BB-FLP-102/22D	426+16.8	7.4 LT	105.0-107.0	SILT, little sand, trace clay.	25.7			

PIN
017872.00
Town
Fryeburg
Reported by/Date
WHITE, TERRY A 6/9/2011

SHEET 6

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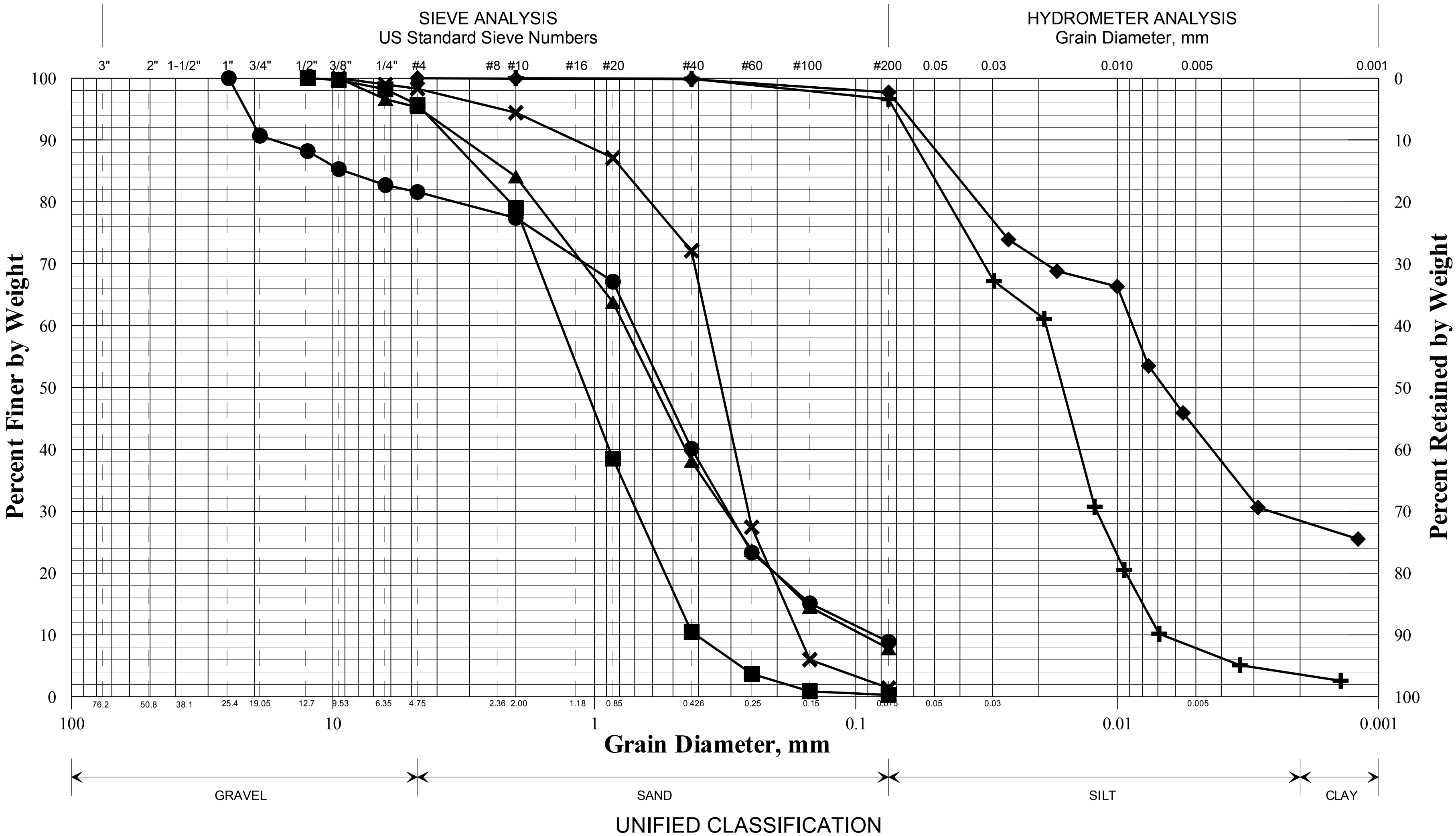
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-FLP-102/23D	426+16.8	7.4 LT	110.0-112.0	SILT, trace clay, trace sand.	26.6			
◆	BB-FLP-102/24D	426+16.8	7.4 LT	115.0-117.0	SILT, some clay, trace sand.	25.5			
■	BB-FLP-102/25D	426+16.8	7.4 LT	120.5-122.5	SAND, trace gravel, trace silt.	18.3			
●	BB-FLP-102/26D	426+16.8	7.4 LT	125.0-127.0	SAND, little gravel, trace silt.	11.2			
▲	BB-FLP-102/27D	426+16.8	7.4 LT	130.0-132.0	SAND, trace silt, trace gravel.	15.2			
✖	BB-FLP-102/28D	426+16.8	7.4 LT	140.5-142.5	SAND, trace gravel, trace silt.	20.3			

PIN
017872.00
Town
Fryeburg
Reported by/Date
WHITE, TERRY A 6/9/2011

SHEET 7

Appendix C

Calculations

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at the following piles:

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

Note: All matrices set up in this order

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

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

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

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

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

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

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

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

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

l = unbraced length $l_{unbraced} := 48 \cdot in$ Assume 4 feet unbraced - scour

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 10937 \\ 16035 \\ 22486 \\ 28057 \\ 38247 \end{pmatrix} \cdot kip$$

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

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 14.1129 \\ 14.7112 \\ 21.0152 \\ 21.4997 \\ 22.2368 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

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

$$P_n = \begin{pmatrix} 752 \\ 1059 \\ 1049 \\ 1280 \\ 1688 \end{pmatrix} \cdot \text{kip}$$

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 "good".

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: eq. 6.9.2.1-1

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

$$P_r = \begin{pmatrix} 451 \\ 636 \\ 629 \\ 768 \\ 1013 \end{pmatrix} \cdot \text{kip}$$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

$$\phi := 1.0$$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

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

$$P_r = \begin{pmatrix} 752 \\ 1059 \\ 1049 \\ 1280 \\ 1688 \end{pmatrix} \cdot \text{kip}$$

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

Service/Extreme Limit States

Geotechnical Resistance

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

Bedrock Type:

Granite RQD 90%

Use RQD = 90% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

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

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

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_c := 25000 \cdot psi$

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

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

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

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

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

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

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

K_{sp} includes a factor of safety of 3

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

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

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

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

Nominal Geotechnical Tip Resistance, R_p:

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

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

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

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

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

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

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

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

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

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1 Pile Drivability Analysis, Steel piles and 6.5.4.2 resistance during pile driving

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

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

Pile Size = 12 x 53

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

State of Maine Dept. Of Transportation — 11-Jul-2011
Fryeburg Little Pond 12x53 Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
505.0	44.79	7.50	11.9	6.38	38.75
506.0	44.83	7.50	12.0	6.38	38.77
507.0	44.84	7.50	12.2	6.38	38.75
508.0	44.88	7.50	12.3	6.38	38.75
509.0	44.96	7.51	12.4	6.39	38.82
510.0	45.00	7.51	12.6	6.39	38.84
511.0	45.03	7.52	12.7	6.40	38.86
512.0	45.06	7.52	12.8	6.40	38.88
513.0	45.08	7.52	13.1	6.40	38.85
514.0	45.11	7.52	13.2	6.40	38.87

APE D 36-26

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	150.00	ft
Pile Penetration	145.00	ft
Pile Top Area	15.50	in2

Limited driving stress to 45 ksi

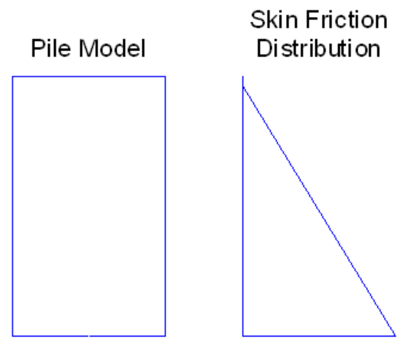
Strength Limit State:

$R_{dr\_12x53\_factored} := 510 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_12x53\_factored} = 332 \cdot kip$

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

$R_{dr\_12x53\_servext} := 510 \cdot kip$



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

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

State of Maine Dept. Of Transportation — 11-Jul-2011
Fryeburg Little Pond 12x74 Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
650.0	44.58	7.57	13.2	6.98	40.62
655.0	44.75	7.60	13.7	6.99	40.68
660.0	44.88	7.63	14.3	7.00	40.74
665.0	44.99	7.65	15.0	7.01	40.74
670.0	45.16	7.68	15.5	7.02	40.86
675.0	45.26	7.71	16.2	7.03	40.86
680.0	45.40	7.73	17.0	7.04	40.92
685.0	45.52	7.75	17.7	7.05	40.98
690.0	45.65	7.78	18.4	7.06	41.04
695.0	45.75	7.80	19.3	7.06	41.05

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_12x74\_factored} := 665 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_12x74\_factored} = 432 \cdot kip$

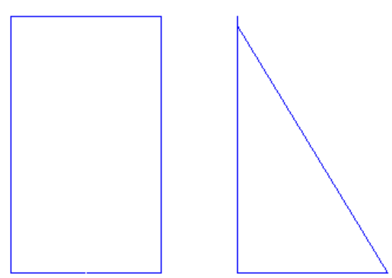
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_12x74\_servext} := 665 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	150.00	ft
Pile Penetration	145.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

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

State of Maine Dept. Of Transportation — 11-Jul-2011
Fryeburg Little Pond 14x73 Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
645.0	44.90	7.67	13.2	6.99	40.93
646.0	44.93	7.68	13.2	6.99	40.97
647.0	44.94	7.68	13.4	6.99	40.94
648.0	45.00	7.69	13.4	6.99	40.99
649.0	45.00	7.69	13.6	6.99	40.97
650.0	45.03	7.70	13.8	6.99	40.95
651.0	45.04	7.71	13.8	7.00	40.98
652.0	45.11	7.72	13.9	7.00	41.02
653.0	45.14	7.72	14.0	7.01	41.05
654.0	45.16	7.73	14.2	7.00	41.03

Limited driving stress to 45 ksi

Strength Limit State:

$R_{dr\_14x73\_factored} := 649 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x73\_factored} = 422 \cdot kip$

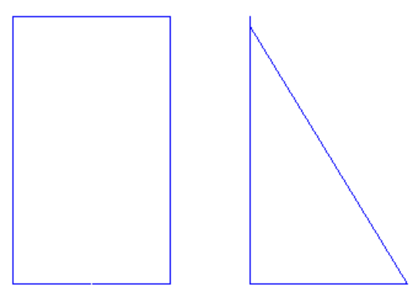
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x73\_servext} := 649 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	150.00	ft
Pile Penetration	145.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

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

State of Maine Dept. Of Transportation — 11-Jul-2011
Fryeburg Little Pond 14x89 Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
720.0	42.48	6.16	14.7	7.03	38.39
721.0	42.53	6.16	14.8	7.03	38.41
722.0	42.53	6.16	14.9	7.03	38.43
723.0	42.57	6.17	15.1	7.03	38.40
724.0	42.56	6.17	15.2	7.03	38.41
725.0	42.62	6.17	15.2	7.03	38.43
726.0	42.65	6.18	15.3	7.04	38.45
727.0	42.66	6.18	15.4	7.04	38.46
728.0	42.68	6.19	15.6	7.04	38.42
729.0	42.69	6.19	15.7	7.04	38.44

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x89\_factored} := 722 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x89\_factored} = 469 \cdot kip$

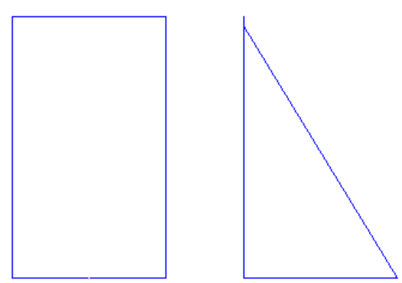
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x89\_servext} := 722 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	150.00	ft
Pile Penetration	145.00	ft
Pile Top Area	26.10	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a APE D36-26 hammer on second highest fuel setting

State of Maine Dept. Of Transportation — 11-Jul-2011
Fryeburg Little Pond 14x117 Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
895.0	42.58	5.70	14.7	7.86	41.66
896.0	42.60	5.70	14.8	7.86	41.66
897.0	42.62	5.69	14.8	7.86	41.65
898.0	42.64	5.69	14.8	7.87	41.71
899.0	42.68	5.69	14.9	7.87	41.72
900.0	42.66	5.70	15.1	7.86	41.65
901.0	42.70	5.69	15.0	7.87	41.70
902.0	42.72	5.69	15.1	7.87	41.70
903.0	42.74	5.70	15.2	7.87	41.70
904.0	42.73	5.69	15.3	7.87	41.69

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x117\_factored} := 901 \cdot kip \cdot \phi_{dyn}$

$R_{dr\_14x117\_factored} = 586 \cdot kip$

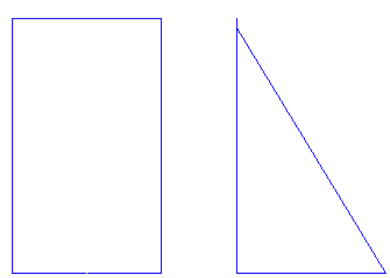
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x117\_servext} := 901 \cdot kip$

APE D 36-26

Efficiency	0.800	
Helmet	2.56	kips
Hammer Cushion	52988	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	150.00	ft
Pile Penetration	145.00	ft
Pile Top Area	34.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

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

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

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

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

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

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

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

$K_p = 6.89$

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

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

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

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

$K_{p\_rank} = 3.25$

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

Settlement Analysis:

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

The roadway grade at centerline may be raised by as much as 1.2 feet.
Look at a simplified soil profile based on BB-FLP-101

Finished Grade

Proposed Fill - Look at 1.2 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Existing Grade

Existing Fill - fine to coarse sand $H_{1fill} := 17.0 \cdot ft$ $\gamma_{fill} := 125 \cdot pcf$ $N_{fill} := 4$

Groundwater at 7.0 ft bgs
$\gamma_w := 62.4pcf$

Sand - fine to coarse sand $H_{2sand} := 5.5 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{2sand} := 3$

Sandy Silt - soft
Total Layer height: H = 17.5 ft - divide into 3 layers

$H_3 := 17.5 \cdot ft$

$H_{3silt1} := 6.0 \cdot ft$ $\gamma_{silt} := 115 \cdot pcf$ $N_{3silt1} := 1$

$H_{3silt2} := 6.0 \cdot ft$ $N_{3silt2} := 1$

$H_{3silt3} := 5.5 \cdot ft$ $N_{3silt3} := 1$

Sand - fine to coarse sand, loose to medium dense
Total Layer height: H = 25.0 ft - divide into 3 layers

$H_4 := 25.0 \cdot ft$

$H_{4sand1} := 8.0 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{4sand1} := 4$

$H_{4sand2} := 8.0 \cdot ft$ $N_{4sand2} := 30$

$H_{4sand3} := 9.0 \cdot ft$ $N_{4sand3} := 49$

Sandy Silt/Silty Sand
Total Layer height: H = 29.0 ft - divide into 3 layers

$H_5 := 29.0 \cdot ft$

$H_{5silt1} := 10.0 \cdot ft$ $\gamma_{silt} := 115 \cdot pcf$ $N_{5silt1} := 18$

$H_{5silt2} := 10.0 \cdot ft$ $N_{5silt2} := 10$

$H_{5silt3} := 9.0 \cdot ft$ $N_{5silt3} := 24$

Sand/Sandy Silt/Till
Total Layer height: H = 64.0 ft - divide into 3 layers

$H_{6sand1} := 22.0 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{6sand1} := 32$

$H_{6sand2} := 22.0 \cdot ft$ $N_{6sand2} := 48$

$H_{6sand3} := 24.0 \cdot ft$ $N_{6sand3} := 30$

Bedrock - granite

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name: Little Pond Bridge — Client: Fryeburg
Project Number: 17872.00 — Project Manager: MParlin
Date: 08/02/11 — Computed by: km

Embank. slope a = 10.00(ft)
Embank. width b = 30.00(ft)
p load/unit area = 150.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 20.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	150.00
1.00	149.96
8.00	136.96
9.00	133.36
19.00	97.13
20.00	94.10
25.00	80.90
26.00	78.61
30.00	70.47
31.00	68.66
37.00	59.35
38.00	58.02
44.00	51.06
52.00	43.92
60.00	38.47
61.00	37.88
70.00	33.26
80.00	29.27
89.00	26.41
90.00	26.12
105.00	22.48
127.00	18.65
150.00	15.83

at 8.5 ft $\Delta\sigma_{z1fill} := 135.16 \cdot psf$

at 19.75 ft $\Delta\sigma_{z2sand} := 94.86 \cdot psf$

at 25.5 ft $\Delta\sigma_{z3silt1} := 79.76 \cdot psf$

at 31.5 ft $\Delta\sigma_{z3silt2} := 67.8 \cdot psf$

at 37.25 ft $\Delta\sigma_{z3silt3} := 59.02 \cdot psf$

at 44.0 ft $\Delta\sigma_{z4sand1} := 51.06 \cdot psf$

at 52.0 ft $\Delta\sigma_{z4sand2} := 43.92 \cdot psf$

at 60.5 ft $\Delta\sigma_{z4sand3} := 38.18 \cdot psf$

at 70.0 ft $\Delta\sigma_{z5silt1} := 33.26 \cdot psf$

at 80.0 ft $\Delta\sigma_{z5silt2} := 29.27 \cdot psf$

at 89.5 ft $\Delta\sigma_{z5silt3} := 26.27 \cdot psf$

at 105.0 ft $\Delta\sigma_{z6sand1} := 22.48 \cdot psf$

at 127.0 ft $\Delta\sigma_{z6sand2} := 18.65 \cdot psf$

at 150.0 ft $\Delta\sigma_{z6sand3} := 15.83 \cdot psf$

Existing Fill

$tsf := psf \cdot 1000$

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

Calculate vertical stress at mid point: $\sigma_{1fill\_o} := 7 \cdot ft \cdot (\gamma_{fill}) + 1.5 \cdot ft \cdot (\gamma_{fill} - \gamma_w)$ $\sigma_{1fill\_o} = 0.9689 \cdot tsf$

Corrected SPT N_{60}-value (bpf) $N_{fill} = 4$

At P_o = 0.97 tsf $C_{N\_1fill} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{1fill\_o}}\right)$ $C_{N\_1fill} = 1.2442$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_1fill} \cdot N_{fill}$ $N1_{60} = 5$
From LRFD Eq 10.4.6.2.4-1

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

Bearing Capacity Index: $C_{1fill} := 37$

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

$\Delta\sigma_{z1fill} = 135.16 \cdot psf$

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

Calculate vertical stress at mid point:

$$\sigma_{2sand\_o} := \left[\frac{H_{2sand}}{2}\cdot(\gamma_{sand}-\gamma_w)\right] + 7.0\cdot ft\cdot(\gamma_{fill}) + 10.0\cdot ft\cdot(\gamma_{fill}-\gamma_w) \qquad \sigma_{2sand\_o} = 1.6731\cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{2sand} = 3$

At P_o = 1.7 tsf $C_{N\_2sand} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{2sand\_o}}\right)$ $C_{N\_2sand} = 1.0615$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_2sand}\cdot N_{2sand}$ $N1_{60} = 3$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $C_{2sand} := 35$

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

$$\Delta\sigma_{z2sand} = 94.86\cdot psf$$

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

Layer 1:

Calculate vertical stress at mid point:

$$\sigma_{3silt1\_o} := \left[\frac{H_{3silt1}}{2}\cdot(\gamma_{silt}-\gamma_w)\right] + H_{2sand}\cdot(\gamma_{sand}-\gamma_w) + 7.0\cdot ft\cdot(\gamma_{fill}) + 10.0\cdot ft\cdot(\gamma_{fill}-\gamma_w)$$

$$\sigma_{3silt1\_o} = 2.0031\cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{3silt1} = 1$

At P_o = 2.0 tsf $C_{N\_3silt1} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{3silt1\_o}}\right)$ $C_{N\_3silt1} = 1.0013$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3silt1}\cdot N_{3silt1}$ $N1_{60} = 1$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $C_{3silt1} := 18$

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

$$\Delta\sigma_{z3silt1} = 79.76\cdot psf$$

Layer 2: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3silt2\_o} := \left[\frac{H_{3silt2}}{2}\cdot(\gamma_{silt}-\gamma_w)\right] + H_{3silt1}\cdot(\gamma_{silt}-\gamma_w) + H_{2sand}\cdot(\gamma_{sand}-\gamma_w) + 7.0\cdot ft\cdot(\gamma_{fill}) + 10.0\cdot ft\cdot(\gamma_{fill}-\gamma_w)$$

$$\sigma_{3silt2\_o} = 2.3187\cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{3silt2} = 1$

At P_o = 2.3 tsf $C_{N\_3silt2} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{3silt2\_o}}\right)$ $C_{N\_3silt2} = 0.9523$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3silt2}\cdot N_{3silt2}$ $N1_{60} = 1$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $C_{3silt2} := 18$

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

$$\Delta\sigma_{z3silt2} = 67.8\cdot psf$$

Layer 3: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{3silt3\_o} := \left[\frac{H_{3silt3}}{2}\cdot(\gamma_{silt}-\gamma_w)\right] + (H_{3silt1}+H_{3silt2})\cdot(\gamma_{silt}-\gamma_w) + H_{2sand}\cdot(\gamma_{sand}-\gamma_w) + 7.0\cdot ft\cdot(\gamma_{fill}) + 10.0\cdot ft\cdot(\gamma_{fill}-\gamma_w)$$

$$\sigma_{3silt3\_o} = 2.6212\cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{3silt3} = 1$

At P_o = 2.6 tsf $C_{N\_3silt3} := 0.77\cdot\log\left(\frac{40\cdot ksf}{\sigma_{3silt3\_o}}\right)$ $C_{N\_3silt3} = 0.9113$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_3silt3}\cdot N_{3silt3}$ $N1_{60} = 1$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $C_{3silt3} := 18$

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

$$\Delta\sigma_{z3silt3} = 59.02\cdot psf$$

Calculate vertical stress at bottom of silt layer (at 40.0 ft bgs):

$$\sigma_{40ft} := H_3\cdot(\gamma_{silt}-\gamma_w) + H_{2sand}\cdot(\gamma_{sand}-\gamma_w) + 7\cdot ft\cdot(\gamma_{fill}) + 10\cdot ft\cdot(\gamma_{fill}-\gamma_w)$$

$\sigma_{40ft} = 2.7658\cdot tsf$ use to shorten vertical stress equations

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

Layer 1: Calculate vertical stress at mid point:

$$\sigma_{4sand1\_o} := \frac{H_{4sand1}}{2} \cdot (\gamma_{sand} - \gamma_w) + \sigma_{40ft} \qquad \sigma_{4sand1\_o} = 3.0162 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{4sand1} = 4$

At P_o = 3.0 tsf $C_{N\_4sand1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{4sand1\_o}}\right)$ $C_{N\_4sand1} = 0.8644$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_4sand1} \cdot N_{4sand1}$ $N1_{60} = 3$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $C_{4sand1} := 35$

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

$$\Delta\sigma_{z4sand1} = 51.06 \cdot psf$$

Layer 2: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{4sand2\_o} := \frac{H_{4sand2}}{2} \cdot (\gamma_{sand} - \gamma_w) + H_{4sand1} \cdot (\gamma_{sand} - \gamma_w) + \sigma_{40ft} \qquad \sigma_{4sand2\_o} = 3.517 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{4sand2} = 30$

At P_o = 3.5 tsf $C_{N\_4sand2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{4sand2\_o}}\right)$ $C_{N\_4sand2} = 0.813$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_4sand2} \cdot N_{4sand2}$ $N1_{60} = 24$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $C_{4sand2} := 72$

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

$$\Delta\sigma_{z4sand2} = 43.92 \cdot psf$$

Layer 3: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{4sand3\_o} := \frac{H_{4sand3}}{2} \cdot (\gamma_{sand} - \gamma_w) + (H_{4sand1} + H_{4sand2}) \cdot (\gamma_{sand} - \gamma_w) + \sigma_{40ft} \qquad \sigma_{4sand3\_o} = 4.0491 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $\quad N_{4sand3} = 49$

At P_o = 4.0 tsf $\quad C_{N\_4sand3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{4sand3\_o}}\right) \quad C_{N\_4sand3} = 0.7659 \quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $\quad N1_{60} := C_{N\_4sand3} \cdot N_{4sand3} \quad N1_{60} = 38$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $\quad C_{4sand3} := 108$

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

$$\Delta\sigma_{z4sand3} = 38.18 \cdot psf$$

Calculate vertical stress at bottom of sand layer (at 65.0 ft bgs):

$$\sigma_{65ft} := H_4 \cdot (\gamma_{sand} - \gamma_w) + H_3 \cdot (\gamma_{silt} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + 7 \cdot ft \cdot (\gamma_{fill}) + 10 \cdot ft \cdot (\gamma_{fill} - \gamma_w)$$

$\sigma_{65ft} = 4.3308 \cdot tsf \quad$ use to shorten vertical stress equations

Sandy Silt/Silty Sand

Layer 1: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{5silt1\_o} := \frac{H_{5silt1}}{2} \cdot (\gamma_{silt} - \gamma_w) + \sigma_{65ft} \qquad \sigma_{5silt1\_o} = 4.5938 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $\quad N_{5silt1} = 18$

At P_o = 4.6 tsf $\quad C_{N\_5silt1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{5silt1\_o}}\right) \quad C_{N\_5silt1} = 0.7237 \quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $\quad N1_{60} := C_{N\_5silt1} \cdot N_{5silt1} \quad N1_{60} = 13$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $\quad C_{5silt1} := 33$

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

$$\Delta\sigma_{z5silt1} = 33.26 \cdot psf$$

Layer 2: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{5silt2\_o} := \frac{H_{5silt2}}{2} \cdot (\gamma_{silt} - \gamma_w) + H_{5silt1} \cdot (\gamma_{silt} - \gamma_w) + \sigma_{65ft} \qquad \sigma_{5silt2\_o} = 5.1198 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{5silt2} = 10$

At P_o = 5.1 tsf

$$C_{N\_5silt2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{5silt2\_o}}\right) \qquad C_{N\_5silt2} = 0.6875$$

LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_5silt2} \cdot N_{5silt2}$ $\qquad N1_{60} = 7$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $C_{5silt2} := 25$

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

$$\Delta\sigma_{z5silt2} = 29.27 \cdot psf$$

Layer 3: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{5silt3\_o} := \frac{H_{5silt3}}{2} \cdot (\gamma_{silt} - \gamma_w) + (H_{5silt1} + H_{5silt2}) \cdot (\gamma_{silt} - \gamma_w) + \sigma_{65ft} \qquad \sigma_{5silt3\_o} = 5.6195 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{5silt3} = 24$

At P_o = 5.6 tsf

$$C_{N\_5silt3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{5silt3\_o}}\right) \qquad C_{N\_5silt3} = 0.6563$$

LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_5silt3} \cdot N_{5silt3}$ $\qquad N1_{60} = 16$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Inorganic Silt" curve

Bearing Capacity Index: $C_{5silt3} := 37$

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

$$\Delta\sigma_{z5silt3} = 26.27 \cdot psf$$

Calculate vertical stress at bottom of silt layer (at 94.0 ft bgs):

$$\sigma_{94ft} := H_5 \cdot (\gamma_{silt} - \gamma_w) + H_4 \cdot (\gamma_{sand} - \gamma_w) + H_3 \cdot (\gamma_{silt} - \gamma_w) + H_{2sand} \cdot (\gamma_{sand} - \gamma_w) + 7 \cdot ft \cdot (\gamma_{fill}) + 10 \cdot ft \cdot (\gamma_{fill} - \gamma_w)$$

$$\sigma_{94ft} = 5.8562 \cdot tsf$$

use to shorten vertical stress equations

Sand/Sandy Silt/Till

Layer 1: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{6sand1\_o} := \frac{H_{6sand1}}{2} \cdot \left(\gamma_{sand} - \gamma_w\right) + \sigma_{94ft} \qquad \sigma_{6sand1\_o} = 6.5448 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $\quad N_{6sand1} = 32$

At P_o = 6.5 tsf $\quad C_{N\_6sand1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{6sand1\_o}}\right) \quad C_{N\_6sand1} = 0.6053 \quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $\quad N1_{60} := C_{N\_6sand1} \cdot N_{6sand1} \quad N1_{60} = 19$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $\quad C_{6sand1} := 67$

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

$$\Delta\sigma_{z6sand1} = 22.48 \cdot psf$$

Layer 2: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{6sand2\_o} := \frac{H_{6sand2}}{2} \cdot \left(\gamma_{sand} - \gamma_w\right) + H_{6sand1} \cdot \left(\gamma_{sand} - \gamma_w\right) + \sigma_{94ft} \qquad \sigma_{6sand2\_o} = 7.922 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $\quad N_{6sand2} = 48$

At P_o = 7.9 tsf $\quad C_{N\_6sand2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{6sand2\_o}}\right) \quad C_{N\_6sand2} = 0.5415 \quad$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $\quad N1_{60} := C_{N\_6sand2} \cdot N_{6sand2} \quad N1_{60} = 26$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $\quad C_{6sand2} := 82$

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

$$\Delta\sigma_{z6sand2} = 18.65 \cdot psf$$

Layer 3: Determine corrected N-value normalized for overburden $N1_{60}$:

Calculate vertical stress at mid point:

$$\sigma_{6sand3\_o} := \frac{H_{6sand3}}{2} \cdot (\gamma_{sand} - \gamma_w) + (H_{6sand1} + H_{6sand2}) \cdot (\gamma_{sand} - \gamma_w) + \sigma_{94ft} \qquad \sigma_{6sand3\_o} = 9.3618 \cdot tsf$$

Corrected SPT N_{60}-value (bpf) $N_{6sand3} = 30$

At P_o = 9.3 tsf $C_{N\_6sand3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{6sand3\_o}}\right)$ $C_{N\_6sand3} = 0.4856$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_6sand3} \cdot N_{6sand3}$ $N1_{60} = 15$
From LRFD Eq. 10.4.6.2.4-1

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

Bearing Capacity Index: $C_{6sand3} := 58$

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

$$\Delta\sigma_{z6sand3} = 15.83 \cdot psf$$

Existing Fill: $\Delta H_{1fill} := H_{1fill} \cdot \frac{1}{C_{1fill}} \cdot \log\left(\frac{\sigma_{1fill\_o} + \Delta\sigma_{z1fill}}{\sigma_{1fill\_o}}\right)$ $\Delta H_{1fill} = 0.3127 \cdot in$

Native Sand: $\Delta H_{2sand} := H_{2sand} \cdot \frac{1}{C_{2sand}} \cdot \log\left(\frac{\sigma_{2sand\_o} + \Delta\sigma_{z2sand}}{\sigma_{2sand\_o}}\right)$ $\Delta H_{2sand} = 0.0452 \cdot in$

Sandy Silt Layer 1: $\Delta H_{3silt1} := H_{3silt1} \cdot \frac{1}{C_{3silt1}} \cdot \log\left(\frac{\sigma_{3silt1\_o} + \Delta\sigma_{z3silt1}}{\sigma_{3silt1\_o}}\right)$ $\Delta H_{3silt1} = 0.0678 \cdot in$

Sandy Silt Layer 2: $\Delta H_{3silt2} := H_{3silt2} \cdot \frac{1}{C_{3silt2}} \cdot \log\left(\frac{\sigma_{3silt2\_o} + \Delta\sigma_{z3silt2}}{\sigma_{3silt2\_o}}\right)$ $\Delta H_{3silt2} = 0.0501 \cdot in$

Sandy Silt Layer 3: $\Delta H_{3silt3} := H_{3silt3} \cdot \frac{1}{C_{3silt3}} \cdot \log\left(\frac{\sigma_{3silt3\_o} + \Delta\sigma_{z3silt3}}{\sigma_{3silt3\_o}}\right)$ $\Delta H_{3silt3} = 0.0355 \cdot in$

$$\Delta H_{3silt} := \Delta H_{3silt1} + \Delta H_{3silt2} + \Delta H_{3silt3} \qquad \Delta H_{3silt} = 0.1534 \cdot in$$

Sand Layer 1: $\Delta H_{4sand1} := H_{4sand1} \cdot \frac{1}{C_{4sand1}} \cdot \log\left(\frac{\sigma_{4sand1\_o} + \Delta\sigma_{z4sand1}}{\sigma_{4sand1\_o}}\right)$ $\Delta H_{4sand1} = 0.02 \cdot in$

Sand Layer 2: $\Delta H_{4sand2} := H_{4sand2} \cdot \frac{1}{C_{4sand2}} \cdot \log\left(\frac{\sigma_{4sand2\_o} + \Delta\sigma_{z4sand2}}{\sigma_{4sand2\_o}}\right)$ $\Delta H_{4sand2} = 0.0072 \cdot in$

Sand Layer 3: $\Delta H_{4sand3} := H_{4sand3} \cdot \frac{1}{C_{4sand3}} \cdot \log\left(\frac{\sigma_{4sand3\_o} + \Delta\sigma_{z4sand3}}{\sigma_{4sand3\_o}}\right)$ $\Delta H_{4sand3} = 0.0041 \cdot in$

$\Delta H_{4sand} := \Delta H_{4sand1} + \Delta H_{4sand2} + \Delta H_{4sand3}$ $\Delta H_{4sand} = 0.0313 \cdot in$

Sandy Silt/Silty Sand Layer 1: $\Delta H_{5silt1} := H_{5silt1} \cdot \frac{1}{C_{5silt1}} \cdot \log\left(\frac{\sigma_{5silt1\_o} + \Delta\sigma_{z5silt1}}{\sigma_{5silt1\_o}}\right)$ $\Delta H_{5silt1} = 0.0114 \cdot in$

Sandy Silt/Silty Sand Layer 2: $\Delta H_{5silt2} := H_{5silt2} \cdot \frac{1}{C_{5silt2}} \cdot \log\left(\frac{\sigma_{5silt2\_o} + \Delta\sigma_{z5silt2}}{\sigma_{5silt2\_o}}\right)$ $\Delta H_{5silt2} = 0.0119 \cdot in$

Sandy Silt/Silty Sand Layer 3: $\Delta H_{5silt3} := H_{5silt3} \cdot \frac{1}{C_{5silt3}} \cdot \log\left(\frac{\sigma_{5silt3\_o} + \Delta\sigma_{z5silt3}}{\sigma_{5silt3\_o}}\right)$ $\Delta H_{5silt3} = 0.0059 \cdot in$

$\Delta H_{5silt} := \Delta H_{5silt1} + \Delta H_{5silt2} + \Delta H_{5silt3}$ $\Delta H_{5silt} = 0.0292 \cdot in$

Sand/Sandy Silt/Till Layer 1: $\Delta H_{6sand1} := H_{6sand1} \cdot \frac{1}{C_{6sand1}} \cdot \log\left(\frac{\sigma_{6sand1\_o} + \Delta\sigma_{z6sand1}}{\sigma_{6sand1\_o}}\right)$ $\Delta H_{6sand1} = 0.0059 \cdot in$

Sand/Sandy Silt/Till Layer 2: $\Delta H_{6sand2} := H_{6sand2} \cdot \frac{1}{C_{6sand2}} \cdot \log\left(\frac{\sigma_{6sand2\_o} + \Delta\sigma_{z6sand2}}{\sigma_{6sand2\_o}}\right)$ $\Delta H_{6sand2} = 0.0033 \cdot in$

Sand/Sandy Silt/Till Layer 3: $\Delta H_{6sand3} := H_{6sand3} \cdot \frac{1}{C_{6sand3}} \cdot \log\left(\frac{\sigma_{6sand3\_o} + \Delta\sigma_{z6sand3}}{\sigma_{6sand3\_o}}\right)$ $\Delta H_{6sand3} = 0.0036 \cdot in$

$\Delta H_{6sand} := \Delta H_{6sand1} + \Delta H_{6sand2} + \Delta H_{6sand3}$ $\Delta H_{6sand} = 0.0128 \cdot in$

$$\Delta H_T := \Delta H_{1fill} + \Delta H_{2sand} + \Delta H_{3silt} + \Delta H_{4sand} + \Delta H_{5silt} + \Delta H_{6sand}$$

$\Delta H_T = 0.5845 \cdot in$ Say less than 1 inch of settlement will occur during construction

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:
Fryeburg, Maine
DFI = 1400 degree-days

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

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

$\text{Frost\_depth} := 79.2\text{in}$ $\qquad$ $\text{Frost\_depth} = 6.6 \cdot \text{ft}$

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

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Bridgton

--- ModBerg Results ---

Project Location: Bridgton 3 NW, Maine

Air Design Freezing Index	=	1600 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	1280 F-days
Mean Annual Temperature	=	43.9 deg F
Design Length of Freezing Season	=	133 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	72.0	10.0	120.0	26	32	1.7	1.5	1,728

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

Total Depth of Frost Penetration = 6.00 ft = 72.0 in.

$\text{Frost\_depth}_{\text{modberg}} := 72.0 \cdot \text{in}$ $\qquad$ $\text{Frost\_depth}_{\text{modberg}} = 6\,\text{ft}$

Use Modberg Frost Depth = 6.0 feet for design

Seismic:

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

BB-FLP-101					BB-FLP-102				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
6	8		7	0.875	1.5	57		3	0.052632
11	1	weight of hammer	7	7	6	7		5	0.714286
16	1		5	5	11	1		5	5
21	3		5	1.666667	15	1	weight of hammer	5	5
26	1	weight of hammer	10	10	20	1	weight of hammer	5	5
36	1	weight of hammer	10	10	25	1		5	5
46	4		10	2.5	30	1	weight of hammer	5	5
56	49		10	0.204082	36	1	weight of hammer	5	5
66	18		10	0.555556	41	1	weight of hammer	5	5
76	10		10	1	46	1	weight of hammer	5	5
86	24		10	0.416667	51	1	weight of hammer	5	5
96	32		6	0.1875	56	22		5	0.227273
					61	29		5	0.172414
					66	18		5	0.277778
					71	15		5	0.333333
					76.5	15		5	0.333333
					81	15		5	0.333333
					86	24		5	0.208333
					91	53		5	0.09434
					96.5	25		7	0.28
SUM			**100**	**39.40547**				**100**	**48.02705**

di/di/N **2.537719** **di/di/N** **2.08216**

SUM	Nav.	2.309939

Nav <15 bpf; Site Class E

Note: Weight of rod (WOR) and weight of hammer (WOH) values are taken as N=1.

```
17872.00 Fryeburg Little Pond Bridge
Date and Time:  8/2/2011 1:20:32 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
 State - Maine
 Zip Code - 04037
 Zip Code Latitude    =    44.000500
 Zip Code Longitude  = -070.963200
 Site Class B
 Data are based on a 0.05 deg grid spacing.
   Period         Sa
    (sec)          (g)
     0.0         0.102     PGA - Site Class B
     0.2         0.198     Ss    - Site Class B
     1.0         0.049     S1    - Site Class B


Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
 State - Maine
 Zip Code - 04037
 Zip Code Latitude    =    44.000500
 Zip Code Longitude  = -070.963200
 As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
 Site Class E  -  Fpga =  2.48,  Fa =  2.50,  Fv =  3.50
 Data are based on a 0.05 deg grid spacing.
   Period         Sa
    (sec)          (g)
     0.0         0.254     As   - Site Class E
     0.2         0.495     SDs - Site Class E
     1.0         0.173     SD1 - Site Class E
```