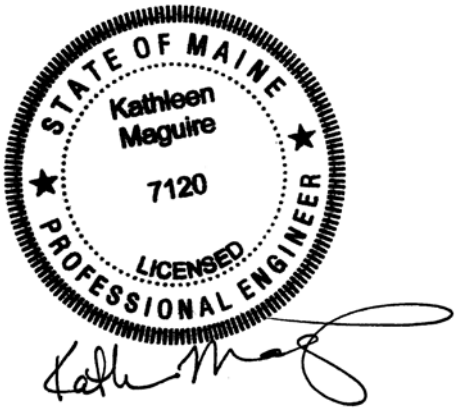
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

SECOND OTTER BRIDGE
OVER OTTER CHAIN STREAM
MILFORD, MAINE

Prepared by:
Kathleen Maguire, P.E.
Geotechnical Engineer



Reviewed by:
Laura Krusinski, P.E.
Senior Geotechnical Engineer

Penobscot County
WIN 16667.00

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this Geotechnical Design Report is to present subsurface information and make geotechnical recommendations for the replacement of the Second Otter Bridge over Otter Chain Stream in Milford, Maine. The replacement structure will consist of a single, 150-foot long span, steel superstructure founded on H-pile supported integral abutments constructed in front of the existing abutments. Embankments will be constructed in order to accommodate the proposed shortened span length. The existing abutments will be entirely removed. 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. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, K_p, of 3.25 anticipating that integral abutments will experience some movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.005, then the calculation of lateral earth pressure should assume a Coulomb passive earth pressure coefficient, K_p, of 6.89. For designing the integral abutment reinforcing steel to resist passive earth pressures, use a maximum load factor (γ_{EH}) of 1.50. 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 proposed vertical alignment is approximately 3.25 feet higher than the existing. The placement of the fills to shorten the bridge span will require the placement of up to approximately 15 feet of fill. It is known that a previous bridge structure had fills in this area. Potential settlement due the placement of the proposed fill is estimated to be between 3 and 4 inches. It is anticipated that the settlement will occur during construction. Due to the past fill conditions at the site, the site soils have been previously exposed to loading conditions similar to those planned for this structure. Therefore, settlements could be less than those calculated. 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 6.5 feet below finished exterior grade for frost protection.

Seismic Design Considerations – A seismic analysis is not required for single-span bridges regardless of seismic zone. Second Otter Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criteria eliminates the MaineDOT Bridge Design Guide (BDG) requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per AASHTO LRFD Bridge Design Specifications 6th Edition (LRFD) Articles 3.10.9 and 4.7.4.4, respectively.

Construction Considerations – Construction of the proposed abutments will require fill placement, soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The construction of the proposed abutments will require the placement of embankment fills fill prior to pile driving.

It is possible that the presence of cobbles and boulders at either abutment will impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations and installation of sheet piles for cofferdams. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling or down-hole hammers. Alternative methods to clear obstructions may be used as approved by the Resident.

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.

1.0 INTRODUCTION

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

The existing Second Otter Bridge carries County Road over Otter Chain Stream and was constructed in 1936. The bridge consists of an approximately 216-foot long, single span riveted through truss structure. The bridge substructures consist of full height, mass concrete abutments and wingwalls supported on timber piles. The 2011 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and superstructure are in poor condition (rating of 4) and the substructure is in satisfactory condition (rating of 6). The Bridge Sufficiency Rating is 17. The structure has a scour critical rating of "8 – Stable Above Footing" meaning that the bridge foundation have been determined to be stable for the assessed or calculated scour condition. Inspection records note that the structure is in overall poor condition with moderate, isolated areas of heavy deterioration. There is minor to moderate cracking of the backwalls and abutments. A 1/16 to 1/8 inch crack is present in the center of the west abutment.

The replacement structure will consist of a single, 150-foot long span, steel superstructure founded on H-pile supported integral abutments constructed in front of the location of the existing abutments. Embankments extending out into Otter Chain Stream will be constructed in order to accommodate the proposed shortened span length. The proposed horizontal alignment will approximately match the existing alignment. The roadway profile will be raised approximately 3.25 feet at proposed abutments. The proposed bridge will be constructed on the existing alignment using a temporary bridge to maintain traffic during construction.

2.0 GEOLOGIC SETTING

Second Otter Bridge in Milford carries County Road over Otter Chain Stream 1.5 miles east of State Route 2 as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic map entitled Orono Quadrangle, Maine Open File No. 81-6 (1981) published by the Maine Geological Survey the surficial soils in the vicinity of the site consist of glacial-marine deposits (Presumpscot Formation) comprised of mostly silt and clay with low permeability and poor drainage. These soils are typically deposited in flat to gently sloping topography except where dissected by streams.

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 calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling three (3) test borings. Test boring BB-MOS-101 was conducted approximately 52 feet behind proposed Abutment No. 1 (west). Test boring BB-MOS-102 was conducted in the center of Otter Chain Stream to obtain information for a possible bridge pier. BB-MOS-103 was conducted approximately 47feet behind proposed Abutment No. 2 (east). 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 July 19 and 22, 2010 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 4 – Boring Logs found end of this report.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in 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. Undisturbed tube samples were attempted in the soft soil deposits in borings BB-MOS-101and BB-MOS-102 but were unsuccessful. In-situ vane shear tests were made where possible in soft soil deposits to measure the shear strength of the strata. 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 consultant geotechnical engineer hired by MaineDOT logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the exploration program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of nine (9) standard grain size analyses with water content, four (4) grain size analyses with hydrometer and water content and four (4) Atterberg Limits tests. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 4 – Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the borings generally consisted of fill over glaciomarine sediments over glacial till all underlain by bedrock. The exploration locations are shown on Sheet 2 - Boring Location Plan and an interpretive subsurface profile depicting the generalized 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

A layer of fill was encountered beneath the pavement in borings BB-MOS-101 and BB-MOS-103. The fill consisted of:

- Brown, damp, fine to coarse sand, little to some gravel, trace to some silt;
- Brown, damp, silt, little to some gravel, little to some fine to coarse sand, trace organics;
- Brown, damp, clayey silt, trace fine to course sand, trace gravel; and
- Brown, wet, silty fine to coarse sand, trace gravel, trace wood.

The thickness of the fill was approximately 11.4 feet in boring BB-MOS-101 and approximately 11.0 feet in boring BB-MOS-103. Corrected SPT N-values in the fill ranged from 28 to 49 blows per foot (bpf) in the sand samples indicating that the sand fill is medium dense to dense in consistency. Corrected SPT N-values in the silt fill ranged from 10 to 25 blows per foot (bpf) in the silt samples indicating that the silt fill is stiff to very stiff in consistency. Water contents obtained from fill samples ranged from approximately 4% to 23%. Grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-1-b, A-4, A-2-4 or A-6 by the AASHTO Classification System and an SM, ML or CL by the Unified Soil Classification System.

5.2 Stream Alluvium

A thin, discontinuous layer of reworked stream alluvium was encountered beneath the fill in boring BB-MOS-101. The stream alluvium consisted of:

- Grey brown, wet, gravel, some silt.

Fragments of peat were note din the wash water when advancing drilling equipment through the lower 1 foot of the stream alluvium.

The thickness of the stream alluvium layer was approximately 6.6 feet. One corrected SPT N-value in the stream alluvium was 7 bpf indicating that the stream alluvium is loose in consistency.

5.3 Glaciomarine Deposit

Glaciomarine deposits were encountered in borings BB-MOS-101 and BB-MOS-102. The glaciomarine deposits consisted of:

- Grey, wet, silty clay, trace fine sand,
- Grey, wet, silty clay, some wood fragments, trace fine to coarse sand, trace gravel, trace organics; and
- Grey, wet, silt, some gravel, some fine to coarse sand, occasional cobbles.

The thickness of the glaciomarine deposits was approximately 19.0 feet in boring BB-MOS-101 and approximately 15.0 feet in boring BB-MOS-102. Vane shear testing conducted within the glaciomarine deposits showed undrained shear strengths ranging from approximately 45 psf to 402 pounds per square foot (psf) while the remolded shear strengths ranged from approximately 45 psf to 67 psf. These shear strength values indicate that the undisturbed glaciomarine deposits are very soft to soft in consistency. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the glaciomarine deposits were determined to have sensitivities ranging from approximately 1 to 8.9 and is classified as insensitive to very sensitive. Water contents from samples obtained within the glaciomarine layer range from approximately 12% to 52%. Grain size analyses conducted on the glaciomarine samples indicate that the soil is classified as an A-4 or A-6 by the AASHTO Classification System and a CL or ML by the Unified Soil Classification System.

Table 5-1 below summarizes the results of the Atterberg Limits tests from samples of the glaciomarine deposits:

Sample No.	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-MOS-101 5D	28.9	32	22	10	0.69
BB-MOS-101 6D	38.8	35	23	12	1.32
BB-MOS-102 2D	51.2	32	23	9	3.13

Table 5-1 – Summary of Atterberg Limits Testing Results

The plasticity indices of the samples indicate that the soils have low to medium plasticity. Interpretation of these results indicates that the soils with liquidity indices of 1 or less are normally consolidated while those with liquidity indices in excess of 1 are on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. Soils with liquidity indices in excess of 1 have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as “quick”.

5.4 Glacial Till

A layer of glacial till was encountered in all the borings. The glacial till consisted of:

- Grey, wet, gravelly, fine to coarse sand, trace silt, with cobbles;
- Grey, wet, silty fine to coarse sand, little to some gravel;
- Grey, wet, fine to coarse sand, some silt, some gravel;
- Grey, wet, silt, little sand, occasional wood fragments; and
- Grey, wet, silt, some fine to coarse sand, little gravel, occasional cobbles.

The thickness of the glacial till layer ranged from approximately 13.5 feet in boring BB-MOS-102 to 48.1 feet in boring BB-MOS-103. Corrected SPT N-values in the cohesionless glacial till ranged from 10 to 111 bpf indicating that the cohesionless glacial till is loose to very dense in consistency. Corrected SPT N-values in the cohesive glacial till ranged from 3 to 70 bpf indicating that the cohesive glacial till is soft to hard in consistency. Water contents from samples obtained within the glacial till range from approximately 9% to 11%. Grain size analyses conducted on the glacial till samples indicate that the soil is classified as an A-2-4 or A-4 by the AASHTO Classification System and a SM or CL by the Unified Soil Classification System.

5.5 Bedrock

Bedrock was encountered and cored in all of the borings. The Table 5-2 summarizes the depths to bedrock corresponding elevations of the top of bedrock and RQD:

Boring Number	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
BB-MOS-101	52.0 feet	58.4 feet	16 to 43%
BB-MOS-102	28.5 feet	58.4 feet	60 to 67%
BB-MOS-103	59.1 feet	51.3 feet	35 to 46%

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

The bedrock is identified as grey, fine grained, sandstone, hard, fresh to slightly weathered, with joints close to moderately close, and tight fractures from horizontal to nearly vertical and minor silt infilling. The rock quality designation (RQD) of the bedrock was determined to range from 16 to 67 percent indicating a rock mass quality of very poor to fair.

5.6 Groundwater

Groundwater was observed at depths of approximately 11.2 to 13.0 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 alternatives were considered for the bridge replacement:

- A two-span, approximately 230-foot long structure with abutments located behind the existing abutments founded on integral, driven H-pile supported abutments and a pile bent pier in the center of the structure;
- A two-span, approximately 190-foot long structure with abutments located in front of the existing abutments founded on integral, driven H-pile supported abutments and a pile bent pier in the center of the structure, embankment construction would be required for this alternative; and
- A single-span, approximately 150-foot long structure with abutments located in front of the existing abutments founded on integral, driven H-pile supported abutments, embankment construction would be required for this alternative.

Due to cost, the single-span, approximately 150-foot long structure with integral abutments located in front of the existing abutments founded on driven H-piles, with proposed embankments was selected. This report addresses only this foundation type. The proposed horizontal alignment will match the existing. The proposed vertical alignment is approximately 3.25 feet higher than the existing vertical alignment. The placement of the embankment fills to shorten the span will have wetland impacts and will require mitigation. The proposed bridge will be constructed using a temporary bridge to maintain traffic during construction.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for H-pile supported integral abutments driven to bedrock located in front of the existing abutments and embankment construction to shorten the bridge span length.

7.1 Integral Abutment H-Piles

The proposed abutments will be founded on a single row of driven, integral H-piles. 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. Pile sizes HP12x53 and HP 14x73 are not allowed for bridges with a span length of 150 feet and a fixed head abutment per MaineDOT Bridge Design Guide (BDG) Table 5-3. 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 of ASTM A 148 Grade 90-60 steel 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-MOS-101	102.0 feet	52.0 feet	58.4 feet	44 feet
Abutment #2 BB-MOS-103	102.0 feet	59.1 feet	51.3 feet	51 feet

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

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

7.1.1 Strength Limit State Design

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

- Structural resistance of individual piles in axial compression,
- Structural resistance of individual piles in combined axial loading and flexure, and
- Compressive axial geotechnical resistance of individual piles bearing on rock.

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

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

Structural Resistance. The nominal axial structural compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. Preliminary estimates of the factored axial structural compressive resistances of the five (5) H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 (good driving conditions) and an unbraced length (ℓ) of 1 foot and an effective length factor (K) of 1.2 for the fixed head condition and 0.8 for the pinned head condition. These factored axial structural compressive resistances are presented in Table 7-2 below. It is the responsibility

of the structural engineer to recalculate the nominal axial structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e".

Geotechnical Resistance. The nominal axial geotechnical compressive resistance in the strength limit state was calculated using the guidance in LRFD Article 10.7.3.2.3 which states that "The nominal bearing resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The nominal bearing resistance shall not exceed the values obtained from Article 6.9.4.1 with the resistance factors specified in Article 6.5.4.2 and Article 6.15 for severe driving (φ_c=0.50)." These factored axial geotechnical compressive resistances are presented in Table 7-2 below.

Drivability Resistance. The drivability of five (5) 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. These factored drivability resistances are presented in Table 7-2 below.

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

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)				
	Structural Resistance[1] ϕ_c=0.60		Controlling Geotechnical Resistance[2] φ_c=0.50	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
	Fixed Head K=1.2	Pinned Head K=0.8			
HP 12x53	N/A	465	387	280	280
HP 12x74	653	653	545	421	421
HP 14x73	N/A	642	535	411	411
HP 14x89	782	783	652	469	469
HP 14x117	1031	1031	860	598	598

[1] Calculated using a resistance factor for good driving conditions, unbraced length (ℓ) of 1 foot and a K as shown in the Table.
[2] Calculated using LRFD Article 10.7.3.2.3 Piles Driven to Hard Rock
N/A – Not Applicable - Pile sizes HP12x53 and HP 14x73 are not allowed for bridges with a span length of 150 feet and a fixed head abutment per MaineDOT BDG Table 5-3.

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

The estimated factored axial pile resistances from the drivability analyses for all pile sizes considered are less than the factored controlling geotechnical resistances and the factored structural resistances. Therefore, it is recommended that the maximum factored axial pile

load used in design for the strength limit state should not exceed the governing resistance shown in the rightmost column of Table 7-2 above.

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

7.1.2 Service and Extreme Limit State Design

The design of the H-piles at the service limit state shall consider tolerable transverse and longitudinal movement of the piles, overall stability of the pile group and pile group movements/stability considering changes in foundation conditions due to scour at the design flood event. For the service limit state a resistance factor, φ, of 1.0 should be used.

Extreme limit state design checks for the H-piles shall include pile axial bearing resistance, failure of the pile group by overturning (eccentricity), pile failure by uplift in tension and structural failure. The extreme event load combinations are those related to ice loads, debris loads, the check flood for scour and certain hydraulic events. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. The design and check floods for scour are defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

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

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

Pile Section	Service and Extreme Limit States Factored Axial Pile Resistance (kips)				
	Structural Resistance[1] ϕ=1.0		Controlling Geotechnical Resistance[2] ϕ=1.0	Drivability Resistance ϕ=1.0	Governing Resistance
	Fixed Head K=1.2	Pinned Head K=0.8			
HP 12x53	N/A	774	774	430	430
HP 12x74	1088	1089	1089	647	647
HP 14x73	N/A	1069	1069	632	632
HP 14x89	1303	1304	1304	722	722
HP 14x117	1718	1719	1719	920	920

[1] Calculated using a resistance factor for good driving conditions, unbraced length (ℓ) of 1 foot and a K as shown in the Table.
[2] Calculated using LRFD Article 10.7.3.2.3 Piles Driven to Hard Rock
N/A – Not Applicable - Pile sizes HP12x53 and HP 14x73 are not allowed for bridges with a span length of 150 feet and a fixed head abutment per MaineDOT BDG Table 5-3.

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

The estimated factored axial pile resistances from the drivability analyses for all pile sizes considered are less than the factored controlling geotechnical resistances and the factored structural resistances. Therefore, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the governing resistance shown in the rightmost 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 signal matching at each integral abutment. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the Contractor in the wave equation analysis. Restrikes will not be required as a part of the field quality control program unless pile behavior indicates the pile has refused on a cobble, is not seated firmly on bedrock or if piles "walk" out of position. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The maximum factored axial pile load should be shown on the plans.

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

resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Integral Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads and lateral forces transferred through the integral superstructure. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider changes in foundation conditions and pile group resistance after scour due to the design flood.

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

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

The Designer may assume Soil Type 4 (MaineDOT Bridge Design Guide [BDG] Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction angle of 20 degrees. Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive earth pressure state. Calculation of passive earth pressures should assume a Rankine passive earth pressure coefficient, K_p, of 3.25 anticipating that integral abutments will experience some movements. Should the ratio of lateral abutment movement to abutment height (y/H) exceed 0.005, then the calculation of lateral earth pressure should assume a Coulomb passive earth pressure coefficient, K_p, of 6.89. For designing the integral abutment backwall reinforcing steel, use a maximum load factor (γ_{EH}) of 1.50 to calculate factored passive earth pressures.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 7-4 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. Weep holes should be constructed approximately 6 inches above the Q1.1 elevation (normal high water). The approach slab should be positively connected to the integral abutment. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.1.4.

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

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

7.3 Scour and Riprap

A grain size analysis was performed on a soil sample taken from the streambed to generate a grain size curve for determining parameters to be used in scour analyses. The samples were 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} = 0.0018 mm
- Average diameter of particle at 95 percent passing, D_{95} = 0.03 mm
- Soil Classification AASHTO Soil Type A-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 conforming to MaineDOT Supplemental Specification Section 703.26 Plain and Heavy Riprap 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 proposed vertical alignment is approximately 3.25 feet higher than the existing vertical alignment. The placement of the fills to shorten the bridge span will require the placement of up to approximately 15 feet of fill into Otter Chain Stream. It is known that a previous bridge structure had fills in this area. The fill materials were removed when the existing structure was constructed and the bridge span was lengthened. With the proposed reduction in span length, the placement of fill materials into Otter Chain Stream will be required again.

Potential settlement due the placement of the proposed fill is estimated to be between 3 and 4 inches. It is anticipated that the settlement will occur during construction. Due to the past fill conditions at the site, the site soils have been previously exposed to loading conditions similar to those planned for this structure. Therefore, settlements could be less than those calculated.

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.

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory, the site has an air design-freezing index of approximately 1588 F-degree days. Considering the site soils and natural water contents determined in the laboratory, this correlates to a frost depth of approximately 6.5 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 6.5 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) = 0.068g
- Site Class D (Stiff soil with 15 < N average < 50 blows per foot)
- Acceleration coefficient (A_s) = 0.110g
- Design spectral acceleration coefficient at 0.2-second period, S_{DS} = 0.236g
- Design spectral acceleration coefficient at 1.0-second period, S_{D1} = 0.106g
- Seismic Zone 1, based on: $S_{D1} \leq 0.15$g (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 Second Otter Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. This criterion eliminates the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be designed per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

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

7.7 Construction Considerations

Construction of the proposed abutments will require fill placement, soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The construction of the proposed abutments will require the placement of embankment fills fill prior to pile driving.

Cobbles and boulders were encountered in the test borings. It is possible that the presence of cobbles and boulders at either abutment will impact pile driving and installation operations. These impacts include, but are not limited to, driving H-piles for abutment foundations and installation of sheet piles for cofferdams. Obstructions may be cleared by conventional excavation methods, pre-augering, predrilling or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident. The potential for obstructions to slow construction activities should be considered by the Contractor.

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.

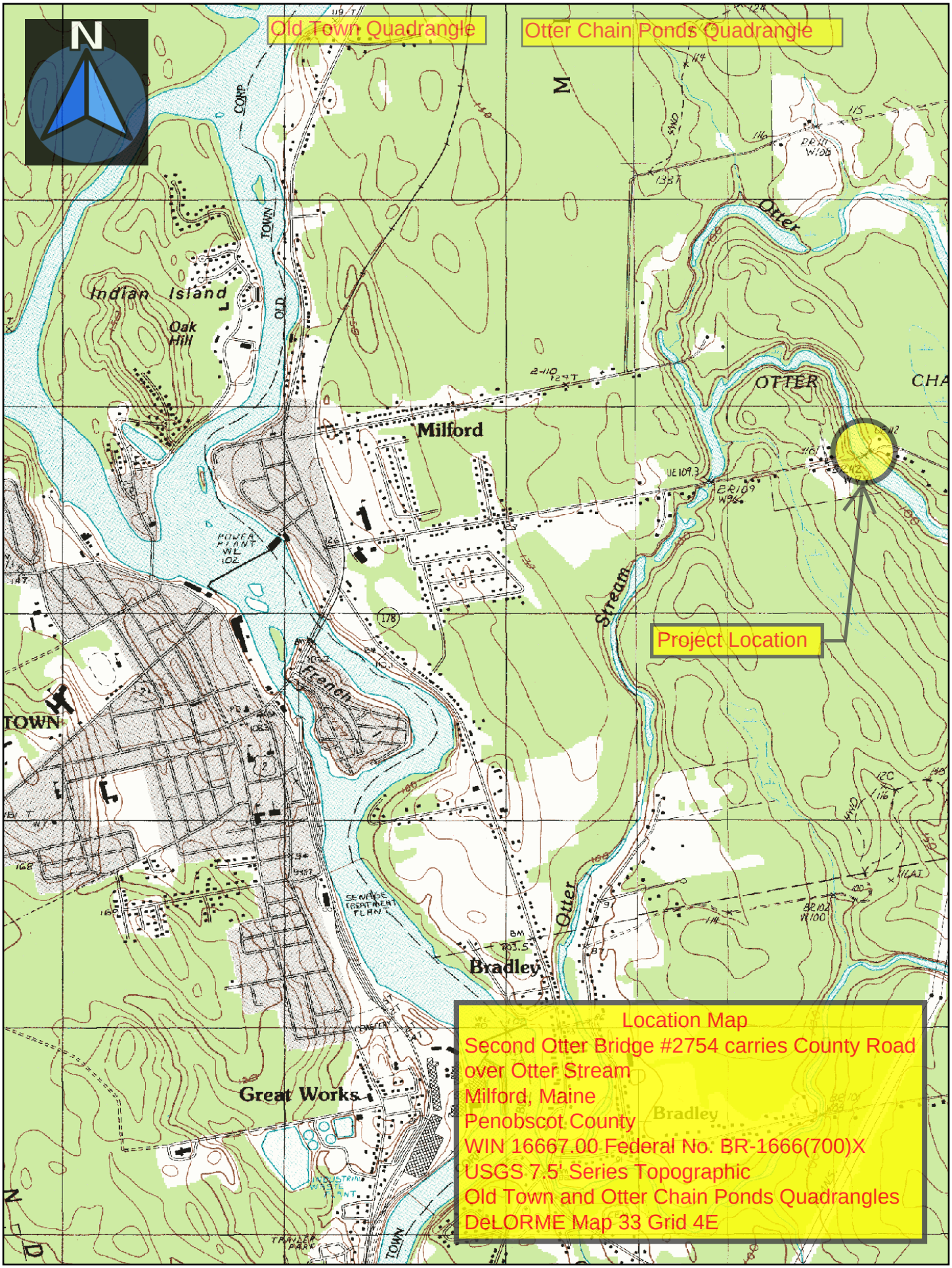
The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches. Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements 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 Second Otter Bridge in Milford 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.

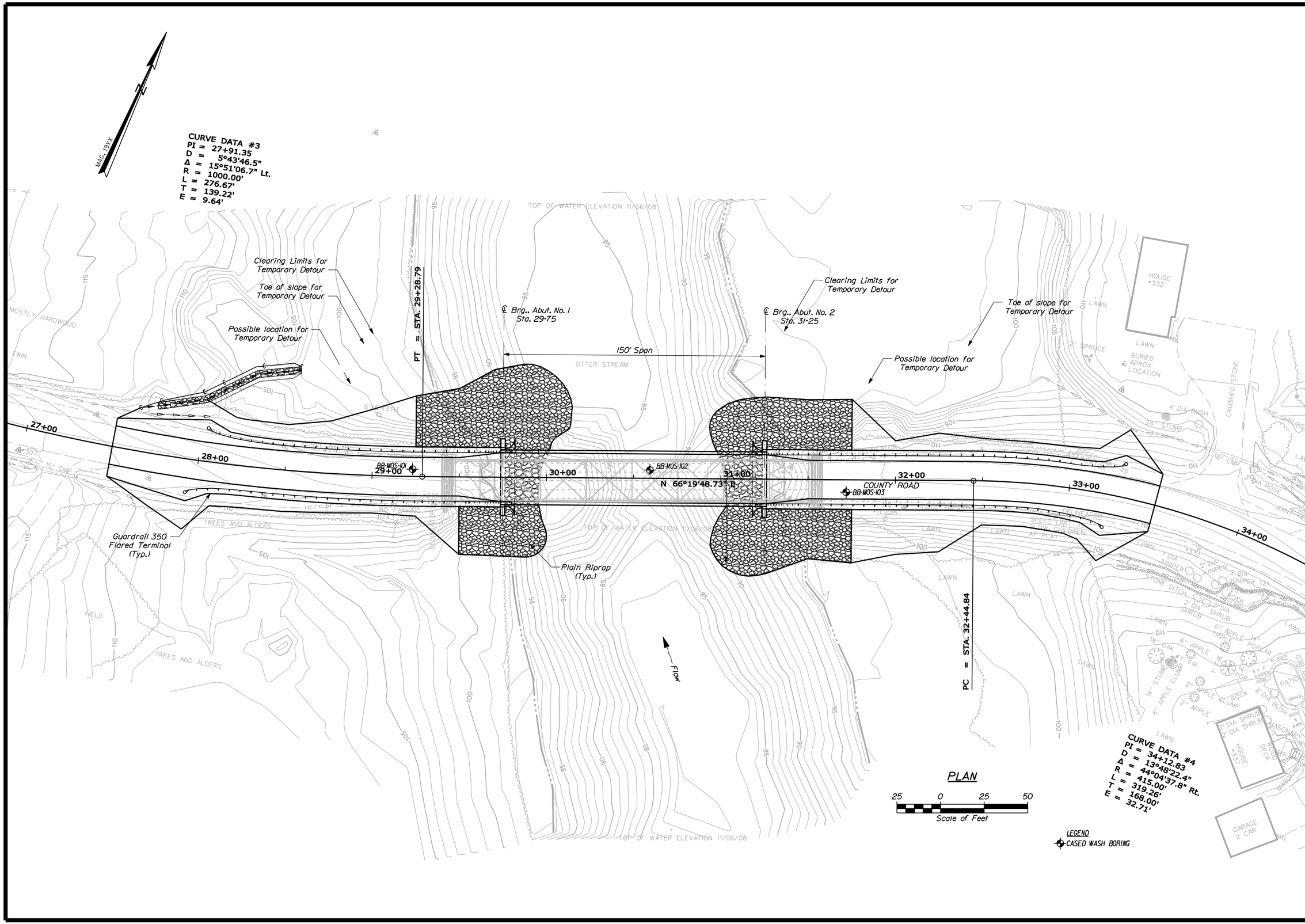
It is also recommend that the geotechnical engineer be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



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.



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1666(700)X

PIN 16667.00

BRIDGE NO. 2754

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	S. BOOKER	
DESIGN-DETAILED	M.MORE AU	T.WHITE	AUG 2010
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SECOND OTTER BRIDGE
OTTER STREAM
MILFORD PENOBSCOT COUNTY

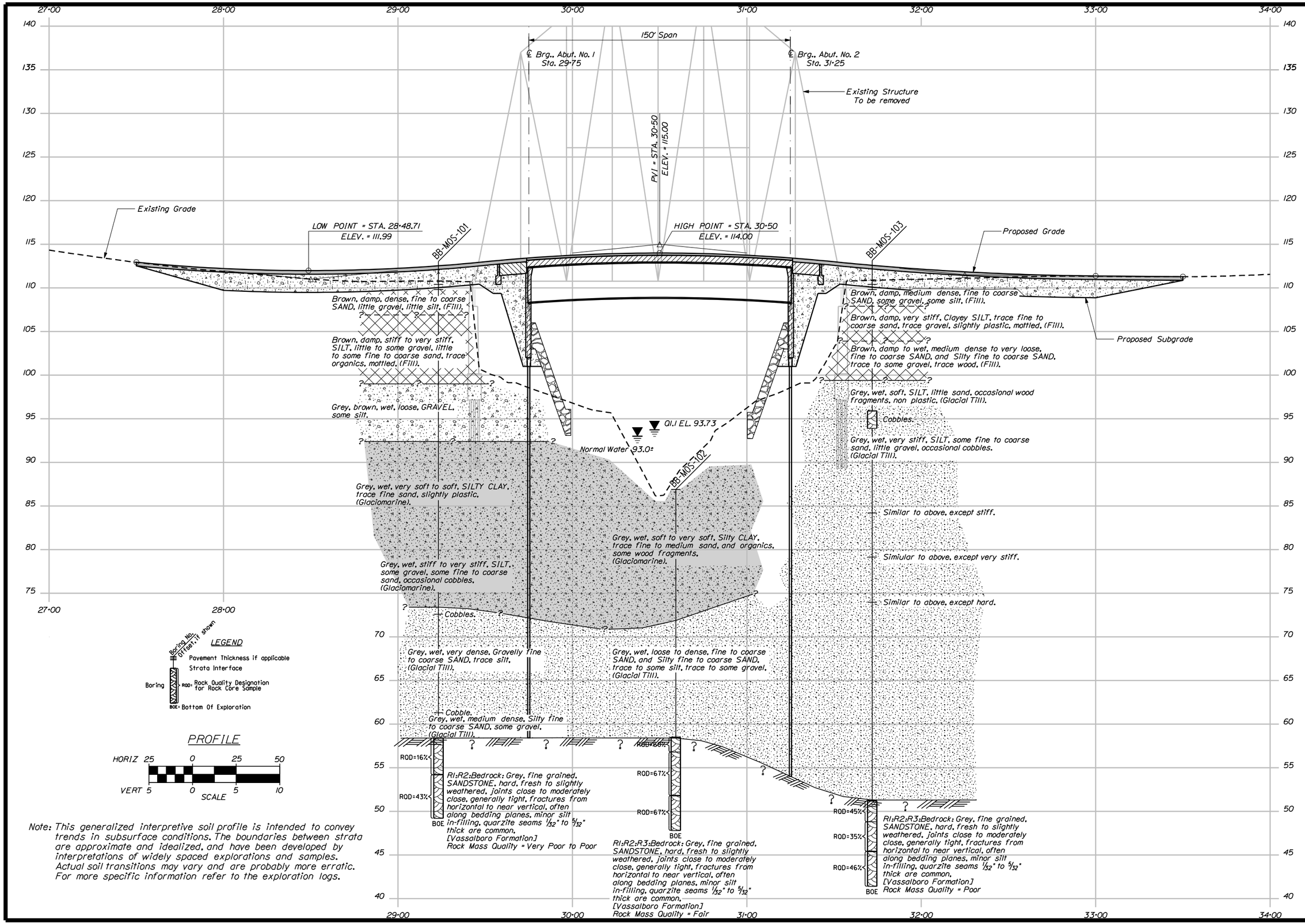
BORING LOCATION PLAN

SHEET NUMBER

2

OF 4

Filename: ...\00\geotech\msta\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012



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-1666(700)X

PIN 16667.00

BRIDGE NO. 2754

BRIDGE PLANS

PROJ. MANAGER	S. BOOGE		
DESIGN-DETAILED	M.MOREAU	T.WHITE	DEC 2012
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

SECOND OTTER BRIDGE
OTTER STREAM
MILFORD PENOBSCOT COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\00\geotech\msta\007_ISP1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012

Filename: ...\msta\008_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/20/2012

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

Project: Second Otter Bridge #2754 carries County Road over Otter Stream		Location: Milford, Maine		Boring No.: BB-MOS-101	WIN: 16667.00
Driller:	MaineDOT	Elevation (ft.)	110.4	Auger ID/OD:	5" Solid Stem
Operator:	Giguere	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	M. Foley	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/19/10-7/20/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	29+23.2, 4.1 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	13.0' bgs
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	109.80	7" Pavement (0.60)	
	1D	24/3	1.00 - 3.00	8/14/21/14	35	49			Brown, damp, dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#239836 A-1-b, SM WC=3.8%
								106.90	(3.50)	
5	2D	24/6	5.00 - 7.00	6/3/4/5	7	10			Brown, damp, stiff, SILT, some gravel, some fine to coarse sand, (Fill).	G#239837 A-4, SM WC=9.2%
10	3D	24/6	10.00 - 12.00	10/8/10/6	18	25	PUSH		Brown, damp, very stiff, SILT, little gravel, little fine to coarse sand, trace organics, mottled, (Fill).	G#239838 A-4, ML WC=19.7%
							11	99.00	(11.40) Based on survey, original ground is at approximately 11.4' bgs, at approximate elevation 99.0'.	
							61		Casing blows indicate a change at 13.0' bgs.	
							110			
							105			
15	4D	24/2	15.00 - 17.00	2/3/2/3	5	7	33		Grey brown, wet, loose, GRAVEL, some silt.	
							47			
							51		Small chunks of PEAT noted in wash water coming up at 17.0' bgs.	
							65	92.40	(18.00) Wash water changes from brown to grey at 18.0' bgs.	
							74			
20	MV/5D	24/22	20.00 - 22.00	WOH/WOH/WOH/WOH	---		62		Failed vane attempt at 20.0' bgs, would not push. Grey, wet, very soft, Silty CLAY, trace fine sand, slightly plastic, (Glaciomarine).	G#239839 A-4, CL WC=28.9% LL=32 PL=22 PI=10
							63			
							74			
							72			
							68			
25	6D V2A	24/24	25.00 - 27.00 25.60 - 25.97	push thru vane Su=402/67 psf	---		58		Grey, wet, soft, Silty CLAY, trace fine sand, slightly plastic, (Glaciomarine).	G#239840 A-6, CL WC=38.8% LL=35 PL=23 PI=12
	V2B		26.60 - 26.97	Su=402/45 psf			62		55x110 mm vane raw torque readings: V2A: 9.0/1.5 ft-lbs V2B: 9.0/1.0 ft-lbs	
							52			
							53			
							54	81.40	(29.00)	
30	7D/MU	24/12	30.00 - 32.00	3/3/5/6	8	11	52		Failed tube attempt, tube would not push. Grey, wet, stiff, SILT, some gravel, some fine to coarse sand, occasional cobbles, (Glaciomarine).	
							72			
							71			
							67			
							92			
35	8D	24/13	35.00 - 37.00	4/5/7/7	12	17	62		Grey, wet, very stiff, SILT, some gravel, some fine to coarse sand, occasional cobbles, (Glaciomarine).	G#239841 A-4, ML WC=12.0%
							56			
							a67	73.40	(37.00) a67 blows for 0.8'. Probable Cobble/Boulder at 37.8' bgs. Roller Coned down to 38.5' bgs. Cored from 38.5-42.0' bgs, cored through 0.5' cobble and 1.2' hard Till.	
							OPEN			
							HOLE			
40							18			
							49			
							63		Washed ahead to 45.0' bgs, then telescoped NW Casing down hole.	
							108			
							81			
45	9D	24/8	45.00 - 47.00	47/47/32/21	79	111	27		Grey, wet, very dense, Gravelly fine to coarse SAND, trace silt, (Glacial Till).	
							78			
							80			
							158			
							64		Cored through 0.6' Quartzite Cobble at 49.1' bgs.	
50	10D	18/10	50.50 - 52.00	7/8/12	20	28	60		(50.8-52.0' bgs) Grey, wet, medium dense, Silty fine to coarse SAND, some gravel, (Glacial Till).	
							60			
	R1	50.4/50.4	52.00 - 56.20	RQD = 16%			NQ-2 CORE	58.40	(52.00) Spoon refusal at 52.0' bgs. Top of Bedrock at Elev. 58.4'. Bedrock: Grey, fine grained, SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation). Rock Mass Quality = Very Poor to Poor. R1:Core Times (min:sec) 52.0-53.0' (8:00) 53.0-54.0' (3:00) 54.0-55.0' (3:15) 55.0-56.0' (5:00) 56.0-56.2' (3:45) 100% Recovery	
55	R2	60/60	56.20 - 61.20	RQD = 43%					R2:Core Times (min:sec) 56.2-57.2' (4:00) 57.2-58.2' (6:15) 58.2-59.2' (6:30) 59.2-60.2' (6:30) 60.2-61.2' (6:45) 100% Recovery	
60								49.20	(61.20) Bottom of Exploration at 61.20 feet below ground surface.	
65										
70										
75										

Remarks:

Rock Coring Down Pressure 500 lbs.

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

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

Page 1 of 1

Boring No.: BB-MOS-101

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

Project: Second Otter Bridge #2754 carries County Road over Otter Stream		Location: Milford, Maine		Boring No.: BB-MOS-102	WIN: 16667.00
Driller:	MaineDOT	Elevation (ft.)	86.9	Auger ID/OD:	5" Solid Stem
Operator:	Giguere	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	M. Foley	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/21/10-7/22/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	30+59.3, 4.7 ft Lt.	Casing ID/OD:	HW & NW	Water Level*:	Water Boring
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/a3	0.00 - 2.00	5/2/1-12"	3	4	HW PUSH		a3" recovery 1st attempt, sent spoon down a 2nd time 24" recovery. Grey, wet, soft, Silty CLAY, trace fine sand and organics, some wood fragments, (Glaciomarine).	
5	2D V1A	24/12	5.00 - 7.00 5.60 - 5.97	push thru vane Su=45/45 psf	---				Grey, wet, very soft, Silty CLAY, trace fine to medium sand.	G#239842 A-4, CL WC=51.2% LL=32 PL=23 PI=9
	V1B		6.60 - 6.97	Su=134/45 psf					55x110 mm raw torque readings: V1A: 1.0/1.0 ft-lbs V1B: 3.0/1.0 ft-lbs	
10	bMU								bPushed casing to 10.3' bgs on 21 July, had to push below bridge deck for overnight, missed shelby tube at 10.3'.	
	3D MV	24/24	14.00 - 16.00 14.00 - 14.00	WOR/WOR/8/11	8	11			Telescoped NW Casing at 14.0' bgs. Grey, wet, stiff, Silty CLAY, trace fine to coarse sand, trace gravel, (Glaciomarine). Failed 55x110 mm vane attempt, would not push.	
15								71.90	(15.00) Grey, wet, medium dense, Silty fine to coarse SAND, little gravel, (Glacial Till).	
							66			
							34			
							68			
							31			
20	4D	24/10	20.60 - 22.60	38/13/21/17	34	48	37		Grey, wet, dense, fine to coarse SAND, some silt, some gravel, (Glacial Till).	G#239843 A-2-4, SM WC=8.7%
							45			
							56			
							43			
							SUNK			
25	5D	24/7	25.00 - 27.00	4/3/4/4	7	10	26		Grey, wet, loose, fine to coarse SAND, some gravel, some silt, (Glacial Till).	
							24			
							22			
	R1	20.4/16	28.40 - 30.10	RQD = 60%			NQ-2	58.40	(28.50) Top of Bedrock at Elev. 58.4'. Bedrock: Grey, fine grained, SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 28.4-29.4' (4:18) 29.4-30.1' (6:00) 78% Recovery	
30	R2	60/62	30.10 - 35.10	RQD = 67%			CORE		R2:Core Times (min:sec) 30.1-31.1' (3:00) 31.1-32.1' (3:00) 32.1-33.1' (5:30) 33.1-34.1' (4:45) 34.1-35.1' (3:45) 103% Recovery	
35	R3	48/48	35.10 - 39.10	RQD = 67%					R3:Core Times (min:sec) 35.1-36.1' (5:00) 36.1-37.1' (4:30) 37.1-38.1' (3:45) 38.1-39.1' (3:18) 100% Recovery	
								47.80	(39.10) Bottom of Exploration at 39.10 feet below ground surface.	
40										
45										
50										

Remarks:

13.6' from Bridge Deck to top of water.
25.3' from Bridge Deck to Mudline. Mudline is 0 ft on this log.
7" thick Bridge Deck.

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

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

Page 1 of 1

Boring No.: BB-MOS-102

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

Project: Second Otter Bridge #2754 carries County Road over Otter Stream		Location: Milford, Maine		Boring No.: BB-MOS-103	WIN: 16667.00
Driller:	MaineDOT	Elevation (ft.)	110.4	Auger ID/OD:	5" Solid Stem
Operator:	Giguere	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	M. Foley	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	7/20/10-7/21/10	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	31+71.9, 7.1 ft Rt.	Casing ID/OD:	HW & NW	Water Level*:	11.2' 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							aSHW	109.80	7" Pavement aSSA to 10.0' bgs, then pushed HW Casing to 10.0' bgs. (0.60)	
	1D	24/11	1.00 - 3.00	6/10/10/4	20	28			Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt, (Fill).	G#239844 A-2-4, SM WC=7.3%
								107.90	(2.50)	
5	2D/AB	24/19	5.00 - 7.00	2/4/5/12	9	13			Brown, damp, stiff, Clayey SILT, trace fine to coarse sand, trace gravel, slightly plastic, mottled, (Fill).	G#239845 A-6, CL WC=23.1% LL=33 PL=20 PI=13
								103.90	(6.50) 2D/B (6.5-7.0') Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
10	3D/AB	24/14	10.00 - 12.00	2/1/1/1	2	3	65		3D/A (10.0-11.0') Brown, wet, very loose, Silty fine to coarse SAND, trace gravel, 2 inch wood chunk, with iron stained layers, (Fill).	
							21	99.40	(11.00) 3D/B (11.0-12.0') Grey, wet, soft, SILT, little sand, occasional wood fragments, non-plastic, (Glacial Till). Based on survey, original ground is at approximately 12.4' bgs, at approximate elevation 98.0'.	
							58			
							219			
							104 347		Probable Cobble/Boulder at 14.5'bgs. Roller Coned ahead to 15.0' bgs, then began Core run at 15.0' to 16.5'bgs. 0.8' Cobble stacked on 0.3' Cobble with Till in Core Barrel.	
15							NQ-2			
	4D	24/13	16.50 - 18.50	6/8/8/8	16	22	bWA		Grey, wet, very stiff, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till). bWashed ahead of Casing from 15.0-31.0' bgs.	G#239846 A-4, CL WC=10.8%
20	5D	24/16	20.00 - 22.00	4/11/6/9	17	24			Similar to above.	
25	6D	24/10	25.00 - 27.00	8/5/3/4	8	11			Grey, wet, stiff, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till).	
30	7D	24/13	30.00 - 32.00	5/6/11/9	17	24			Grey, wet, very stiff, SILT, some fine to coarse sand, little gravel, (Glacial Till). Telescoped NW Casing at 31.0' bgs.	G#239847 A-4, CL WC=11.0%
							70			
							78			
							89			
							101			
35	8D	24/10	35.00 - 37.00	16/12/18/24	30	42	94		Similar to above, hard.	
							128			
							160			
							174			
							203			
40	9D	24/13	40.00 - 42.00	8/13/12/14	25	35	130		Similar to above.	
							165			
							216			
							270			
							324			
45	10D	24/19	45.00 - 47.00	11/11/13/14	24	34	245		Grey, wet, hard, SILT, some fine to coarse sand, little gravel, (Glacial Till).	G#239848 A-4, CL WC=9.7%
							256			
							337			
							348			
							486			
50	11D	24/10	50.00 - 52.00	15/24/15/8	39	55	bWA		Grey, wet, hard, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till).	
55	12D	24/14	55.00 - 57.00	11/18/32/25	50	70			Similat to above, with larger gravel pieces (Max Particle Size 1.75").	
	R1	28.8/28.8	59.20 - 61.60	RQD = 45%			NQ-2 CORE	51.30	(59.10) Top of Bedrock at Elev. 51.3'. Roller Coned ahead to 59.2' bgs. Bedrock: Grey, fine grained SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation). Rock Mass Quality = Poor. R1:Core Times (min:sec) 59.2-60.2' (3:45) 60.2-61.2' (4:00) 61.2-61.6' (1:00) 100% Recovery	
60	R2	40.8/38	61.60 - 65.00	RQD = 35%					R2:Core Times (min:sec) 61.6-62.6' (3:45) 62.6-63.6' (4:18) 63.6-64.6' (5:00) 64.6-65.0' (4:30) 93% Recovery	
65	R3	48/48	65.00 - 69.00	RQD = 46%					R3:Core Times (min:sec) 65.0-66.0' (5:30) 66.0-67.0' (3:18) 67.0-68.0' (3:00) 68.0-69.0' (3:30) 100% Recovery	
								41.40	(69.00) Bottom of Exploration at 69.00 feet below ground surface.	
70										
75										

Remarks:

Rock Coring Down Pressure 500 lbs.

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

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

Page 1 of 1

Boring No.: BB-MOS-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1666(700)X

PIN 16667.00

BRIDGE PLANS

BRIDGE NO. 2754

	BY	DATE
PROJ. MANAGER	S. BODGE	
DESIGN-DETAILED	M.MOREAU	T.WHITE DEC 2012
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

SECOND OTTER BRIDGE
OTTER STREAM
MILFORD PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-101
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/19/10-7/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	29+23.2, 4.1 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	13.0' bgs
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	109.80	7" Pavement — 0.60	
	1D	24/3	1.00 - 3.00	8/14/21/14	35	49			Brown, damp, dense, fine to coarse SAND, little gravel, little silt, (Fill).	G#239836 A-1-b, SM WC=3.8%
								106.90	3.50	
5	2D	24/6	5.00 - 7.00	6/3/4/5	7	10			Brown, damp, stiff, SILT, some gravel, some fine to coarse sand, (Fill).	G#239837 A-4, SM WC=9.2%
10	3D	24/6	10.00 - 12.00	10/8/10/6	18	25	PUSH		Brown, damp, very stiff, SILT, little gravel, little fine to coarse sand, trace organics, mottled, (Fill).	G#239838 A-4, ML WC=19.7%
							11	99.00	11.40	
							61		Based on survey, original ground is at approximately 11.4' bgs, at approximate elevation 99.0'.	
							110		Casing blows indicate a change at 13.0' bgs.	
							105			
15	4D	24/2	15.00 - 17.00	2/3/2/3	5	7	33		Grey brown, wet, loose, GRAVEL, some silt.	
							47			
							51		Small chunks of PEAT noted in wash water coming up at 17.0' bgs.	
							65	92.40	18.00	
							74		Wash water changes from brown to grey at 18.0' bgs.	
20	MV/5D	24/22	20.00 - 22.00	WOH/WOH/WOH/ WOH	---		62		Failed vane attempt at 20.0' bgs, would not push.	G#239839 A-4, CL WC=28.9% LL=32 PL=22 PI=10
							63		Grey, wet, very soft, Silty CLAY, trace fine sand, slightly plastic, (Glaciomarine).	
							74			
							72			
25							68			

Remarks:

Rock Coring Down Pressure 500 lbs.

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.

Boring No.: BB-MOS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-101
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/19/10-7/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	29+23.2, 4.1 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	13.0' bgs
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	push thru vane	---		58			Grey, wet, soft, Silty CLAY, trace fine sand, slightly plastic, (Glaciomarine).	G#239840 A-6, CL WC=38.8% LL=35 PL=23 PI=12
	V2A		25.60 - 25.97	Su=402/67 psf							
	V2B		26.60 - 26.97	Su=402/45 psf			62			55x110 mm vane raw torque readings: V2A: 9.0/1.5 ft-lbs V2B: 9.0/1.0 ft-lbs	
							52				
							53				
							54	81.40		29.00	
30	7D/MU	24/12	30.00 - 32.00	3/3/5/6	8	11	52			Failed tube attempt, tube would not push. Grey, wet, stiff, SILT, some gravel, some fine to coarse sand, occasional cobbles, (Glaciomarine).	
							72				
							71				
							67				
							92				
35	8D	24/13	35.00 - 37.00	4/5/7/7	12	17	62			Grey, wet, very stiff, SILT, some gravel, some fine to coarse sand, occasional cobbles, (Glaciomarine).	G#239841 A-4, ML WC=12.0%
							56	73.40		37.00	
							[a]67			[a]67 blows for 0.8'.	
							OPEN			Probable Cobble/Boulder at 37.8' bgs. Roller Coned down to 38.5' bgs. Cored from 38.5-42.0' bgs, cored through 0.5' cobble and 1.2' hard Till.	
							HOLE				
40							18				
							49				
							63			Washed ahead to 45.0' bgs, then telescoped NW Casing down hole.	
							108				
							81				
45	9D	24/8	45.00 - 47.00	47/47/32/21	79	111	27			Grey, wet, very dense, Gravelly fine to coarse SAND, trace silt, (Glacial Till).	
							78				
							80				
							158				
50							64			Cored through 0.6' Quartzite Cobble at 49.1' bgs.	

Remarks:

Rock Coring Down Pressure 500 lbs.

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: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-101
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/19/10-7/20/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	29+23.2, 4.1 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	13.0' bgs
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	18/10	50.50 - 52.00	7/8/12	20	28	60			(50.8-52.0' bgs) Grey, wet, medium dense, Silty fine to coarse SAND, some gravel, (Glacial Till).	
							60	58.40		52.00	
	R1	50.4/50.4	52.00 - 56.20	RQD = 16%			NQ-2 CORE			Spoon refusal at 52.0' bgs. Top of Bedrock at Elev. 58.4'. Bedrock: Grey, fine grained, SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation). Rock Mass Quality = Very Poor to Poor. R1:Core Times (min:sec) 52.0-53.0' (8:00) 53.0-54.0' (3:00) 54.0-55.0' (3:15) 55.0-56.0' (5:00) 56.0-56.2' (3:45) 100% Recovery R2:Core Times (min:sec) 56.2-57.2' (4:00) 57.2-58.2' (6:15) 58.2-59.2' (6:30) 59.2-60.2' (6:30) 60.2-61.2' (6:45) 100% Recovery	
55											
	R2	60/60	56.20 - 61.20	RQD = 43%							
60								49.20		61.20 **Bottom of Exploration at 61.20 feet below ground surface.**	
65											
70											
75											

Remarks:

Rock Coring Down Pressure 500 lbs.

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.

Boring No.: BB-MOS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-102
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	86.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/21/10-7/22/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	30+59.3, 4.7 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Water Boring
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/[a]3	0.00 - 2.00	5/2/1-12"	3	4	HW PUSH		[a]3" recovery 1st attempt, sent spoon down a 2nd time 24" recovery. Grey, wet, soft, Silty CLAY, trace fine sand and organics, some wood fragments, (Glaciomarine).	
5	2D V1A	24/12	5.00 - 7.00 5.60 - 5.97	push thru vane Su=45/45 psf	---				Grey, wet, very soft, Silty CLAY, trace fine to medium sand.	G#239842 A-4, CL WC=51.2% LL=32 PL=23 PI=9
	V1B		6.60 - 6.97	Su=134/45 psf					55x110 mm raw torque readings: V1A: 1.0/1.0 ft-lbs V1B: 3.0/1.0 ft-lbs	
10	[b]MU								[b]Pushed casing to 10.3' bgs on 21 July, had to push below bridge deck for overnight, missed shelby tube at 10.3'.	
15	3D MV	24/24	14.00 - 16.00 14.00 - 14.00	WOR/WOR/8/11	8	11		71.90	Telescoped NW Casing at 14.0' bgs. Grey, wet, stiff, Silty CLAY, trace fine to coarse sand, trace gravel, (Glaciomarine). Failed 55x110 mm vane attempt, would not push. 15.00	
							66		Grey, wet, medium dense, Silty fine to coarse SAND, little gravel, (Glacial Till).	
							34			
							68			
							31			
20	4D	24/10	20.60 - 22.60	38/13/21/17	34	48	37		Grey, wet, dense, fine to coarse SAND, some silt, some gravel, (Glacial Till).	G#239843 A-2-4, SM WC=8.7%
							45			
							56			
							43			
25							SUNK			

Remarks:

13.6' from Bridge Deck to top of water.
25.3' from Bridge Deck to Mudline. Mudline is 0 ft on this log.
7" thick Bridge Deck.

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.

Boring No.: BB-MOS-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-102
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	86.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/21/10-7/22/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	30+59.3, 4.7 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Water Boring
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/7	25.00 - 27.00	4/3/4/4	7	10	26			Grey, wet, loose, fine to coarse SAND, some gravel, some silt, (Glacial Till).	
							24				
							22				
	R1	20.4/16	28.40 - 30.10	RQD = 60%			NQ-2	58.40		28.50	
										Top of Bedrock at Elev. 58.4'. Bedrock: Grey, fine grained, SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation). Rock Mass Quality = Fair. R1:Core Times (min:sec) 28.4-29.4' (4:18) 29.4-30.1' (6:00) 78% Recovery R2:Core Times (min:sec) 30.1-31.1' (3:00) 31.1-32.1' (3:00) 32.1-33.1' (5:30) 33.1-34.1' (4:45) 34.1-35.1' (3:45) 103% Recovery R3:Core Times (min:sec) 35.1-36.1' (5:00) 36.1-37.1' (4:30) 37.1-38.1' (3:45) 38.1-39.1' (3:18) 100% Recovery	
30	R2	60/62	30.10 - 35.10	RQD = 67%			CORE				
35	R3	48/48	35.10 - 39.10	RQD = 67%				47.80		39.10	
										Bottom of Exploration at 39.10 feet below ground surface.	
40											
45											
50											

Remarks:

13.6' from Bridge Deck to top of water.
25.3' from Bridge Deck to Mudline. Mudline is 0 ft on this log.
7" thick Bridge Deck.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-103
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/20/10-7/21/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	31+71.9, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	11.2' 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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							[a]SHW	109.80	7" Pavement [a]SSA to 10.0' bgs, then pushed HW Casing to 10.0' bgs.	
	1D	24/11	1.00 - 3.00	6/10/10/4	20	28			0.60 Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt, (Fill).	G#239844 A-2-4, SM WC=7.3%
								107.90	2.50	
5	2D/AB	24/19	5.00 - 7.00	2/4/5/12	9	13			Brown, damp, stiff, Clayey SILT, trace fine to coarse sand, trace gravel, slightly plastic, mottled, (Fill).	G#239845 A-6, CL WC=23.1% LL=33 PL=20 PI=13
								103.90	6.50 2D/B (6.5-7.0') Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
10	3D/AB	24/14	10.00 - 12.00	2/1/1/1	2	3	65		3D/A (10.0-11.0') Brown, wet, very loose, Silty fine to coarse SAND, trace gravel, 2 inch wood chunk, with iron stained layers, (Fill).	
							21	99.40	11.00 3D/B (11.0-12.0') Grey, wet, soft, SILT, little sand, occasional wood fragments, non-plastic, (Glacial Till).	
							58		Based on survey, original ground is at approximately 12.4' bgs, at approximate elevation 98.0'.	
							219			
15							104 347 NQ-2		Probable Cobble/Boulder at 14.5'bgs. Roller Coned ahead to 15.0' bgs, then began Core run at 15.0' to 16.5'bgs. 0.8' Cobble stacked on 0.3' Cobble with Till in Core Barrel.	
	4D	24/13	16.50 - 18.50	6/8/8/8	16	22	[b]WA		Grey, wet, very stiff, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till).	G#239846 A-4, CL WC=10.8%
									[b]Washed ahead of Casing from 15.0-31.0' bgs.	
20	5D	24/16	20.00 - 22.00	4/11/6/9	17	24			Similar to above.	
25										

Remarks:

Rock Coring Down Pressure 500 lbs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-103
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/20/10-7/21/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	31+71.9, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	11.2' 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)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/10	25.00 - 27.00	8/5/3/4	8	11				Grey, wet, stiff, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till).	
30	7D	24/13	30.00 - 32.00	5/6/11/9	17	24				Grey, wet, very stiff, SILT, some fine to coarse sand, little gravel, (Glacial Till). Telescoped NW Casing at 31.0' bgs.	G#239847 A-4, CL WC=11.0%
							70				
							78				
							89				
							101				
35	8D	24/10	35.00 - 37.00	16/12/18/24	30	42	94			Similar to above, hard.	
							128				
							160				
							174				
							203				
40	9D	24/13	40.00 - 42.00	8/13/12/14	25	35	130			Similar to above.	
							165				
							216				
							270				
							324				
45	10D	24/19	45.00 - 47.00	11/11/13/14	24	34	245			Grey, wet, hard, SILT, some fine to coarse sand, little gravel, (Glacial Till).	G#239848 A-4, CL WC=9.7%
							256				
							337				
							348				
50							486				

Remarks:

Rock Coring Down Pressure 500 lbs.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Second Otter Bridge #2754 carries County Road over Otter Stream
Location: Milford, Maine

Boring No.: BB-MOS-103
WIN: 16667.00

Driller:	MaineDOT	**Elevation (ft.)**	110.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	M. Foley	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	7/20/10-7/21/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	31+71.9, 7.1 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	11.2' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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WC = water content, percent
LL = Liquid Limit
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C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	24/10	50.00 - 52.00	15/24/15/8	39	55	bWA		Grey, wet, hard, SILT, some fine to coarse sand, little gravel, occasional cobbles, (Glacial Till).	
55	12D	24/14	55.00 - 57.00	11/18/32/25	50	70			Similat to above, with larger gravel pieces (Max Particle Size 1.75").	
	R1	28.8/28.8	59.20 - 61.60	RQD = 45%			NQ-2 CORE	51.30	59.10 Top of Bedrock at Elev. 51.3'. Roller Coned ahead to 59.2' bgs.	
60	R2	40.8/38	61.60 - 65.00	RQD = 35%					Bedrock: Grey, fine grained SANDSTONE, hard, fresh to slightly weathered, joints close to moderately close, generally tight, fractures from horizontal to near vertical often along bedding planes, minor silt in-filling, quartzite seams 1/32" to 5/32" thick are common. (Vassalboro Formation).	
65	R3	48/48	65.00 - 69.00	RQD = 46%				41.40	Rock Mass Quality = Poor. R1:Core Times (min:sec) 59.2-60.2' (3:45) 60.2-61.2' (4:00) 61.2-61.6' (1:00) 100% Recovery R2:Core Times (min:sec) 61.6-62.6' (3:45) 62.6-63.6' (4:18) 63.6-64.6' (5:00) 64.6-65.0' (4:30) 93% Recovery R3:Core Times (min:sec) 65.0-66.0' (5:30) 66.0-67.0' (3:18) 67.0-68.0' (3:00) 68.0-69.0' (3:30) 100% Recovery	
70									69.00 **Bottom of Exploration at 69.00 feet below ground surface.**	
75										

Remarks:

Rock Coring Down Pressure 500 lbs.

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

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Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Milford Project Number: 16667.00

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-MOS-101, 1D	29+23.2	4.1 Lt.	1.0-3.0	239836	1	3.8			SM	A-1-b	II
BB-MOS-101, 2D	29+23.2	4.1 Lt.	5.0-7.0	239837	1	9.2			SM	A-4	III
BB-MOS-101, 3D	29+23.2	4.1 Lt.	10.0-12.0	239838	1	19.7			ML	A-4	IV
BB-MOS-101, 5D	29+23.2	4.1 Lt.	20.0-22.0	239839	1	28.9	32	10	CL	A-4	IV
BB-MOS-101, 6D	29+23.2	4.1 Lt.	25.0-27.0	239840	1	38.8	35	12	CL	A-6	III
BB-MOS-101, 8D	29+23.2	4.1 Lt.	35.0-37.0	239841	1	12.0			ML	A-4	IV
BB-MOS-102, 2D	30+59.3	4.7 Lt.	5.0-7.0	239842	2	51.2	32	9	CL	A-4	IV
BB-MOS-102, 4D	30+59.3	4.7 Lt.	20.6-22.6	239843	2	8.7			SM	A-2-4	II
BB-MOS-103, 1D	31+71.9	7.1 Rt.	1.0-3.0	239844	3	7.3			SM	A-2-4	II
BB-MOS-103, 2D/A	31+71.9	7.1 Rt.	5.0-6.5	239845	3	23.1	33	13	CL	A-6	III
BB-MOS-103, 4D	31+71.9	7.1 Rt.	16.5-18.5	239846	3	10.8			CL	A-4	IV
BB-MOS-103, 7D	31+71.9	7.1 Rt.	30.0-32.0	239847	3	11.0			CL	A-4	IV
BB-MOS-103, 10D	31+71.9	7.1 Rt.	45.0-47.0	239848	3	9.7			CL	A-4	IV

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.

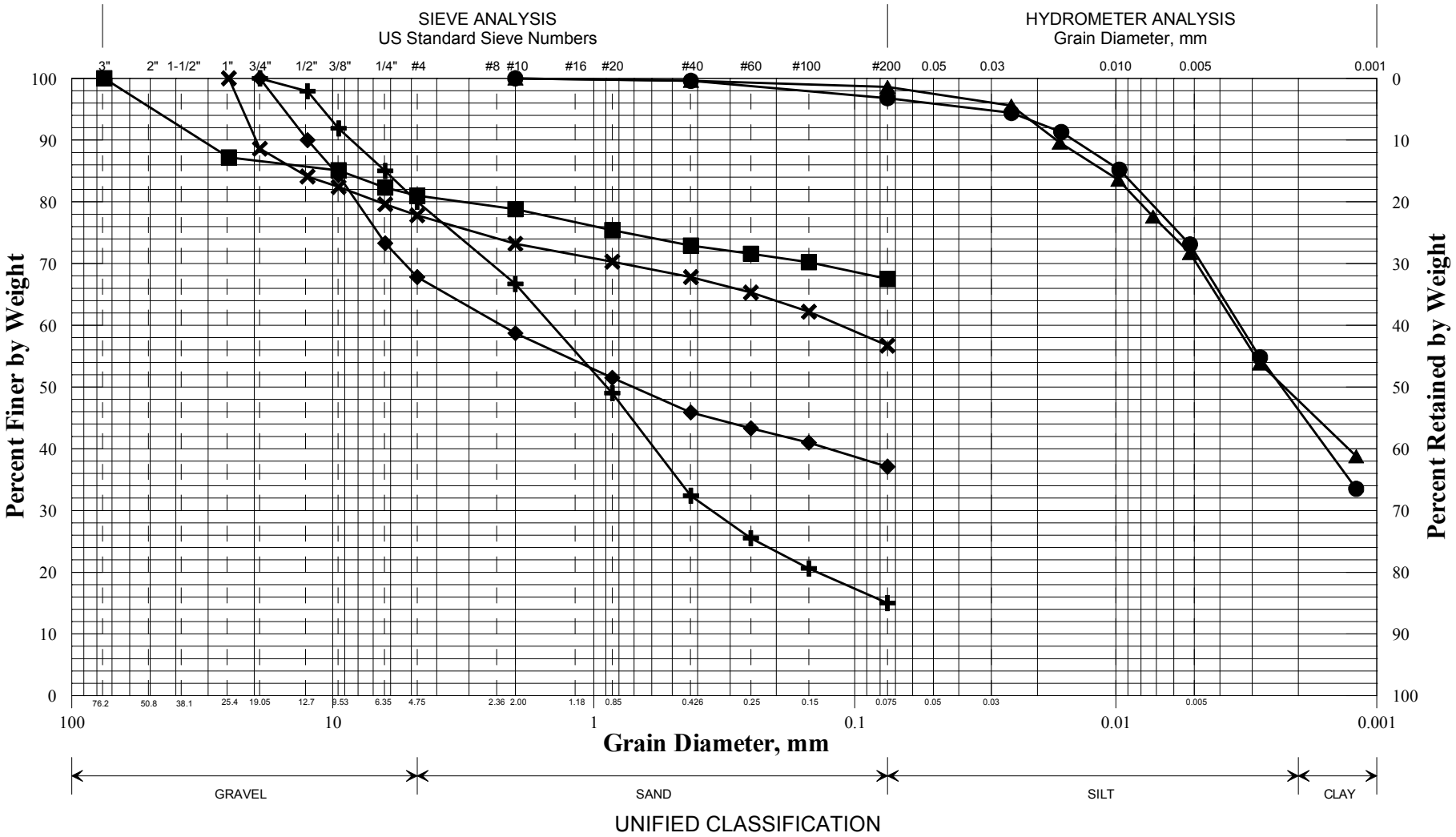
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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-MOS-101/1D	29+23.2	4.1 LT	1.0-3.0	SAND, little gravel, little silt.	3.8			
◆	BB-MOS-101/2D	29+23.2	4.1 LT	5.0-7.0	SILT, some gravel, some sand.	9.2			
■	BB-MOS-101/3D	29+23.2	4.1 LT	10.0-12.0	SILT, little gravel, little sand.	19.7			
●	BB-MOS-101/5D	29+23.2	4.1 LT	20.0-22.0	Silty CLAY, trace sand.	28.9	32	22	10
▲	BB-MOS-101/6D	29+23.2	4.1 LT	25.0-27.0	Silty CLAY, trace sand.	38.8	35	23	12
✖	BB-MOS-101/8D	29+23.2	4.1 LT	35.0-37.0	SILT, some gravel, some sand.	12.0			

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WHITE, TERRY A 11/9/2010

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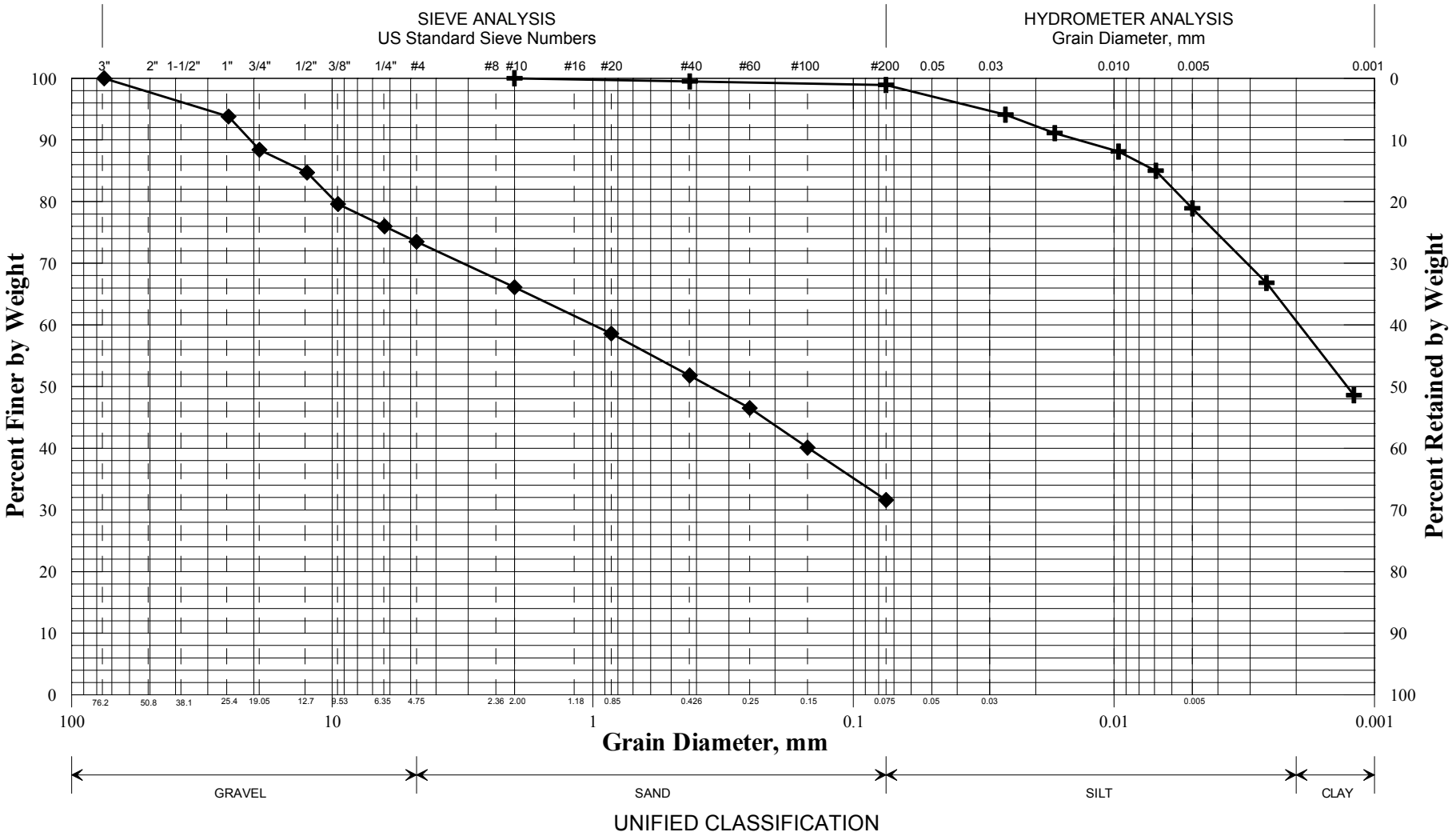
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-MOS-102/2D	30+59.3	4.7 LT	5.0-7.0	Silty CLAY, trace sand.	51.2	32	23	9
◆	BB-MOS-102/4D	30+59.3	4.7 LT	20.6-22.6	SAND, some silt, some gravel.	8.7			
■									
●									
▲									
✖									

WIN
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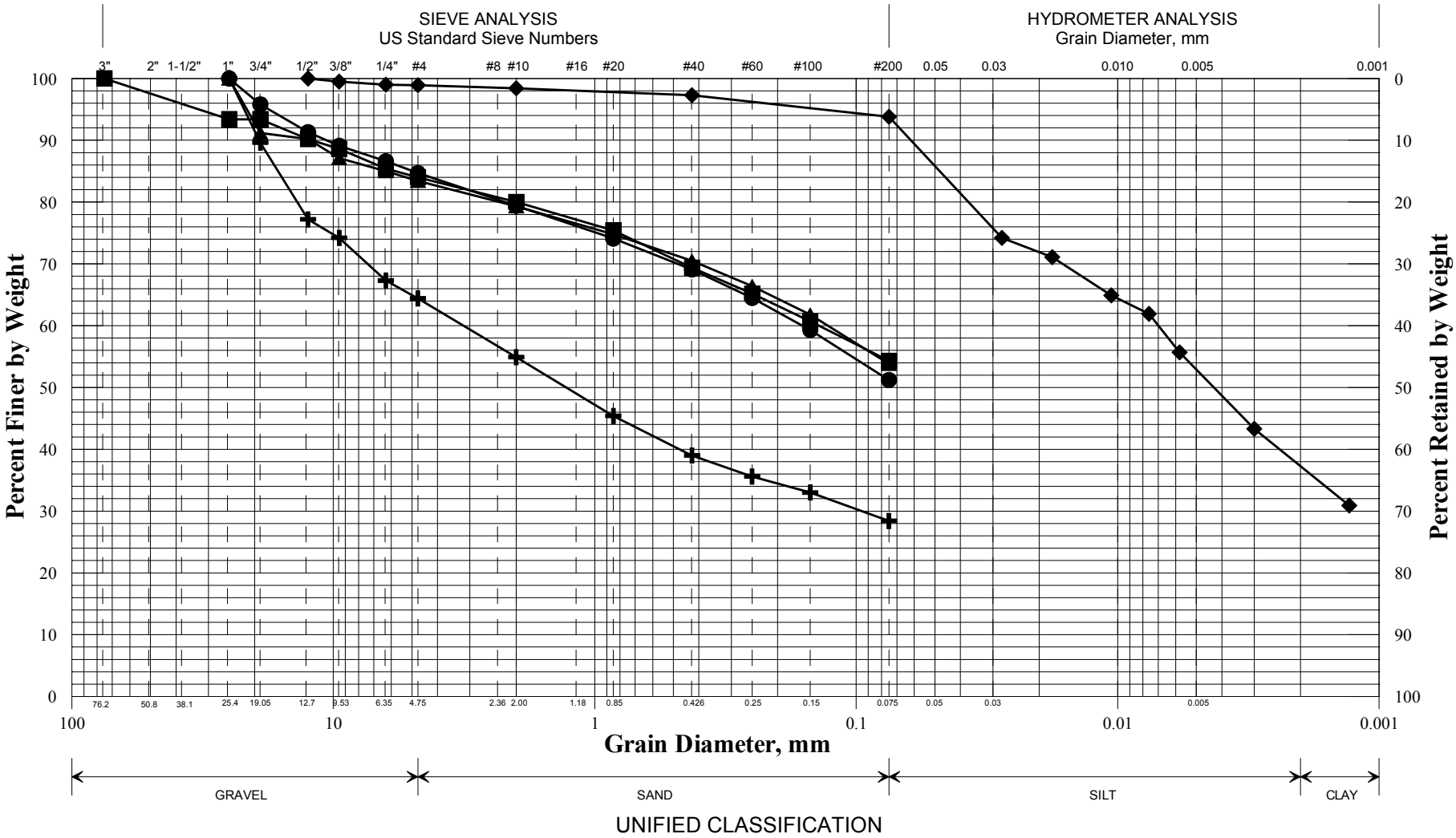
Town
Milford
Reported by/Date
WHITE, TERRY A 11/9/2010

SHEET 2

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-MOS-103/1D	31+71.9	7.1 RT	1.0-3.0	SAND, some gravel, some silt.	7.3			
◆	BB-MOS-103/2DA	31+71.9	7.1 RT	5.0-6.5	Clayey SILT, trace sand, trace gravel.	23.1	33	20	13
■	BB-MOS-103/4D	31+71.9	7.1 RT	16.5-18.5	SILT, some sand, little gravel.	10.8			
●	BB-MOS-103/7D	31+71.9	7.1 RT	30.0-32.0	SILT, some sand, little gravel.	11.0			
▲	BB-MOS-103/10D	31+71.9	7.1 RT	45.0-47.0	SILT, some sand, little gravel.	9.7			
✖									

WIN
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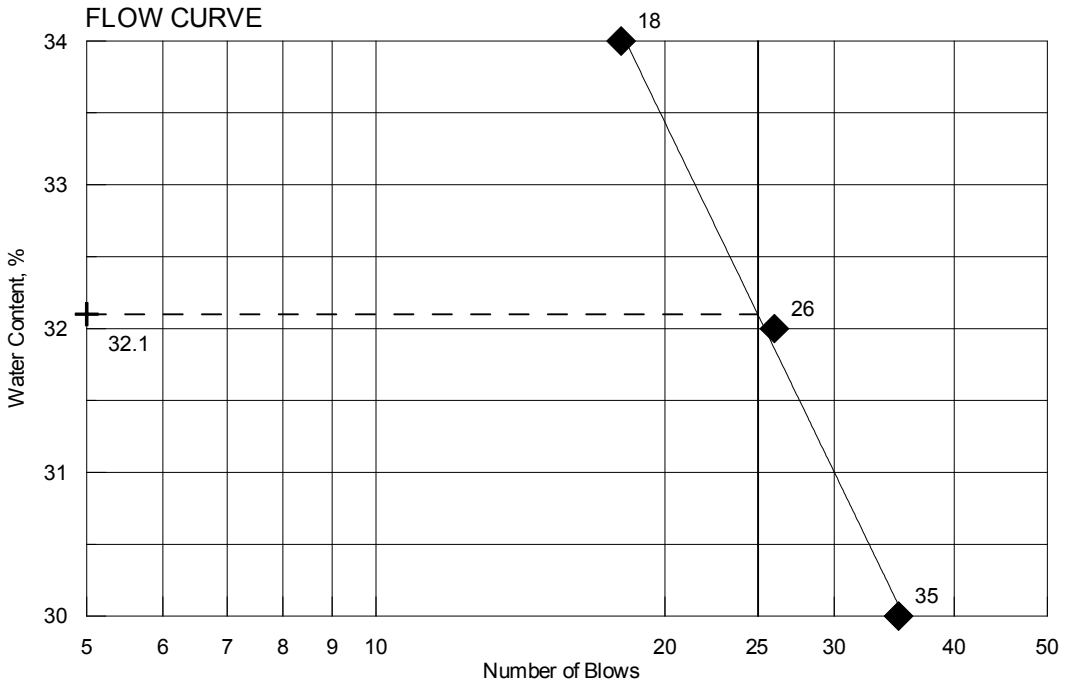
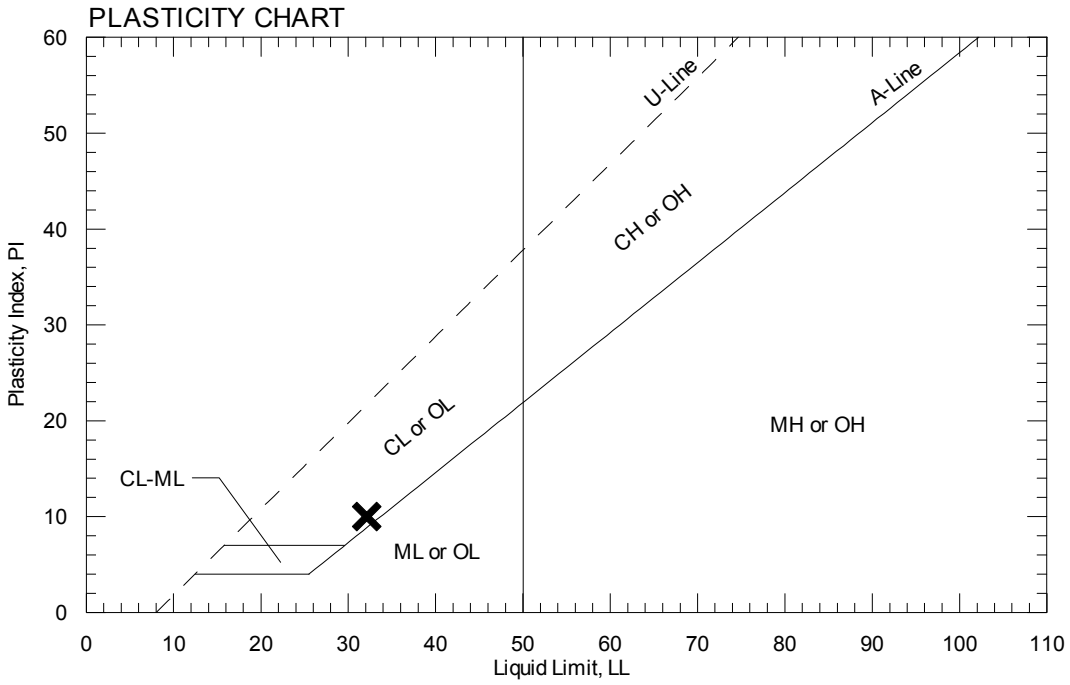
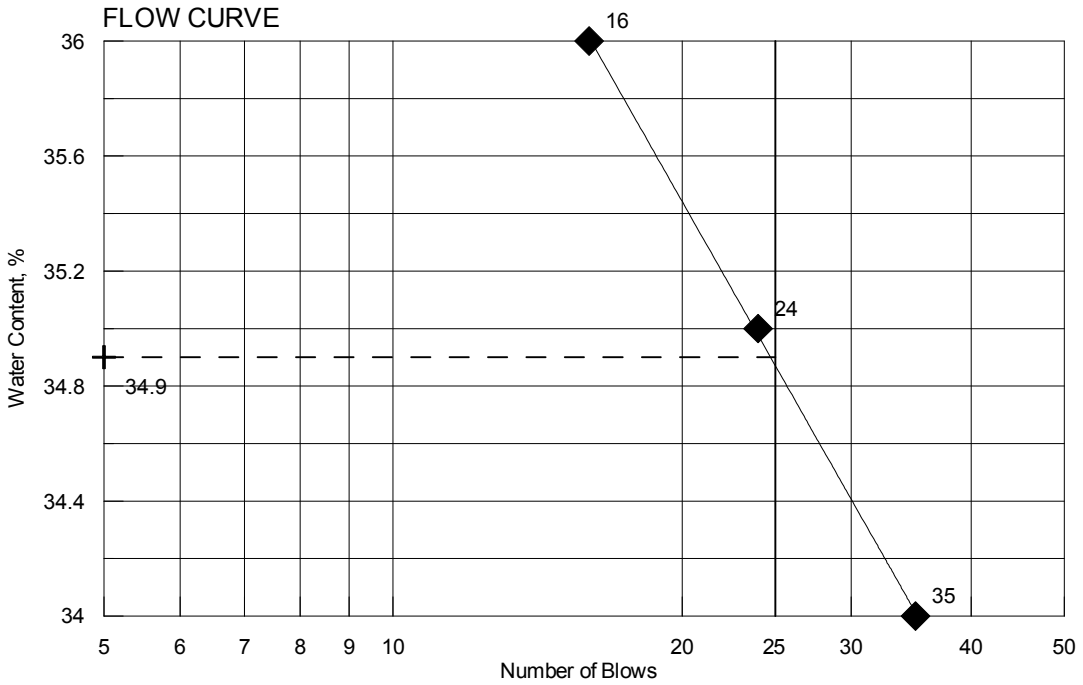
Town
Milford
Reported by/Date
WHITE, TERRY A 11/9/2010

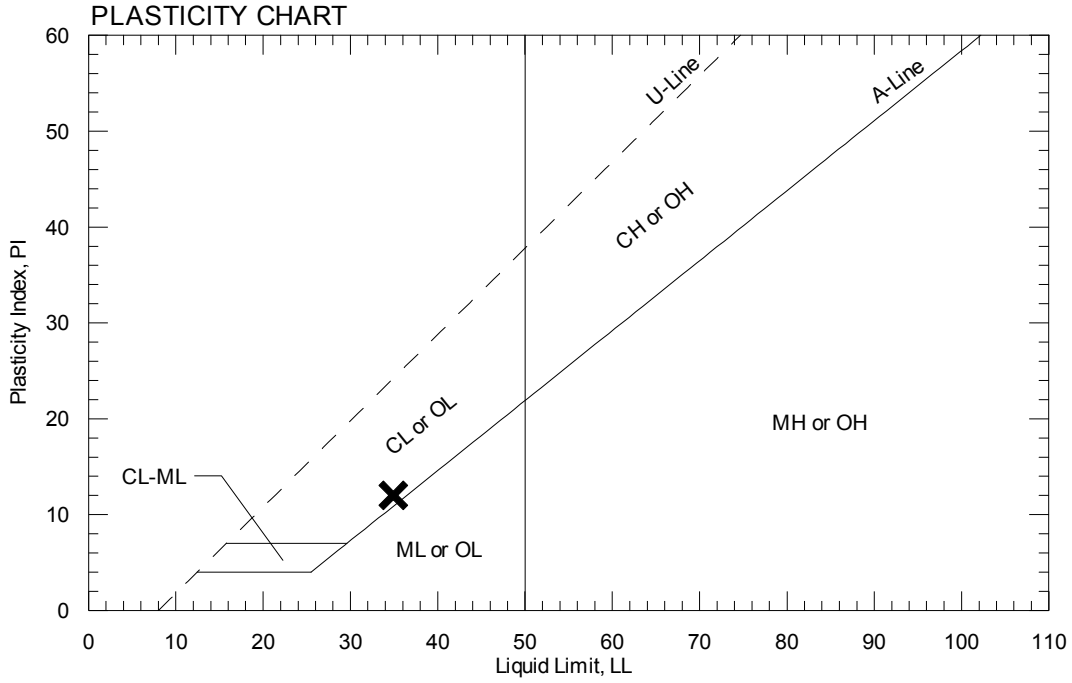
TOWN	Milford	Reference No.	239839
WIN	016667.00	Water Content, %	28.9
Sampled	7/19/2010	Liquid Limit @ 25 blows (T 89), %	32
Boring No./Sample No.	BB-MOS-101/5D	Plastic Limit (T 90), %	22
Station	29+23.2	Plasticity Index (T 90), %	10
Depth	20.0-22.0	Tested By	BBURR



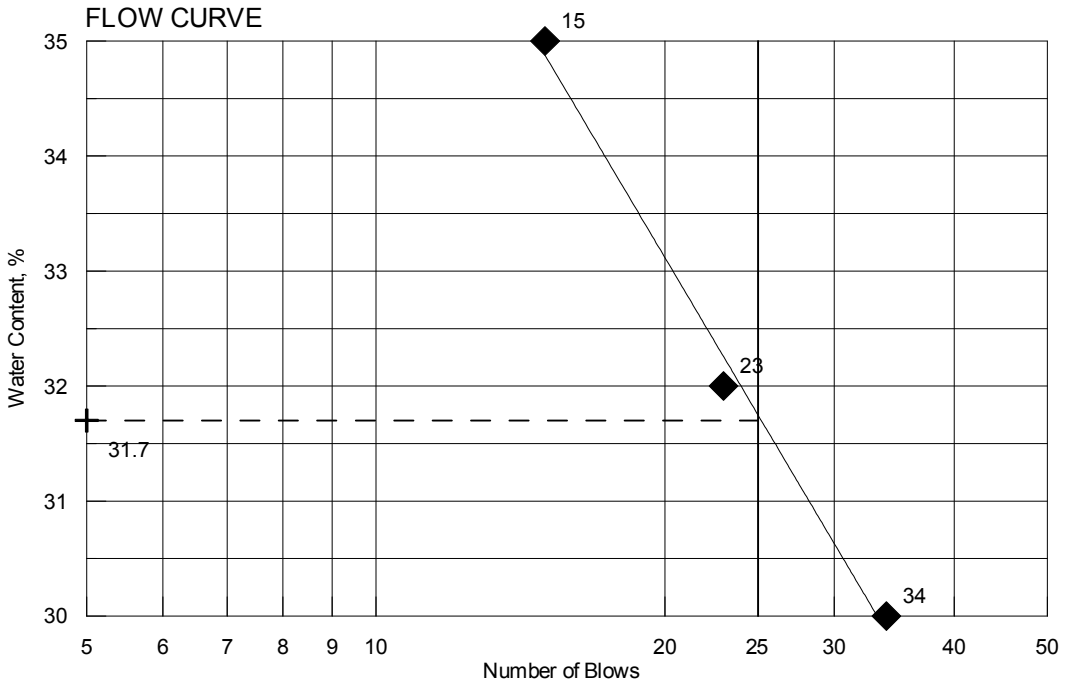


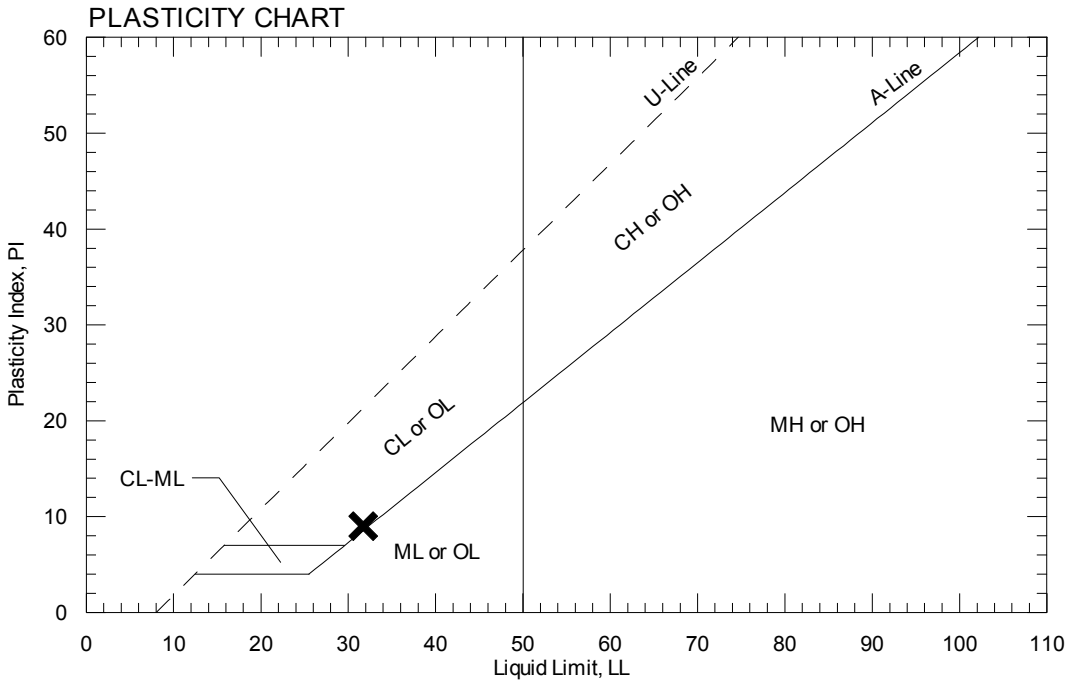
TOWN	Milford	Reference No.	239840
WIN	016667.00	Water Content, %	38.8
Sampled	7/19/2010	Liquid Limit @ 25 blows (T 89), %	35
Boring No./Sample No.	BB-MOS-101/6D	Plastic Limit (T 90), %	23
Station	29+23.2	Plasticity Index (T 90), %	12
Depth	25.0-27.0	Tested By	BBURR



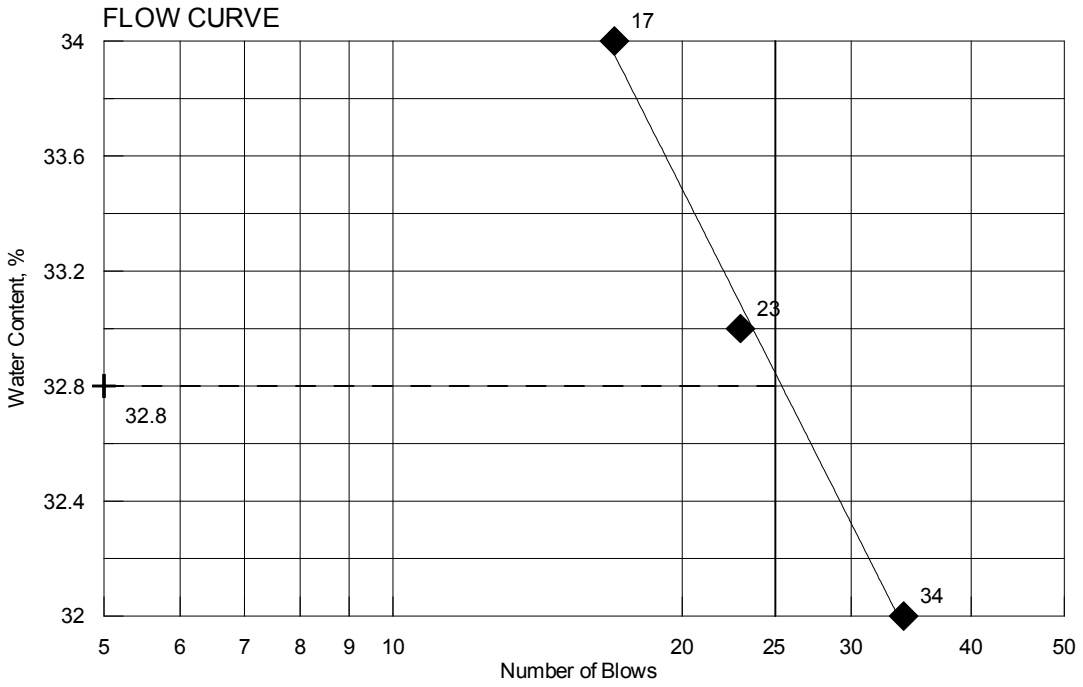


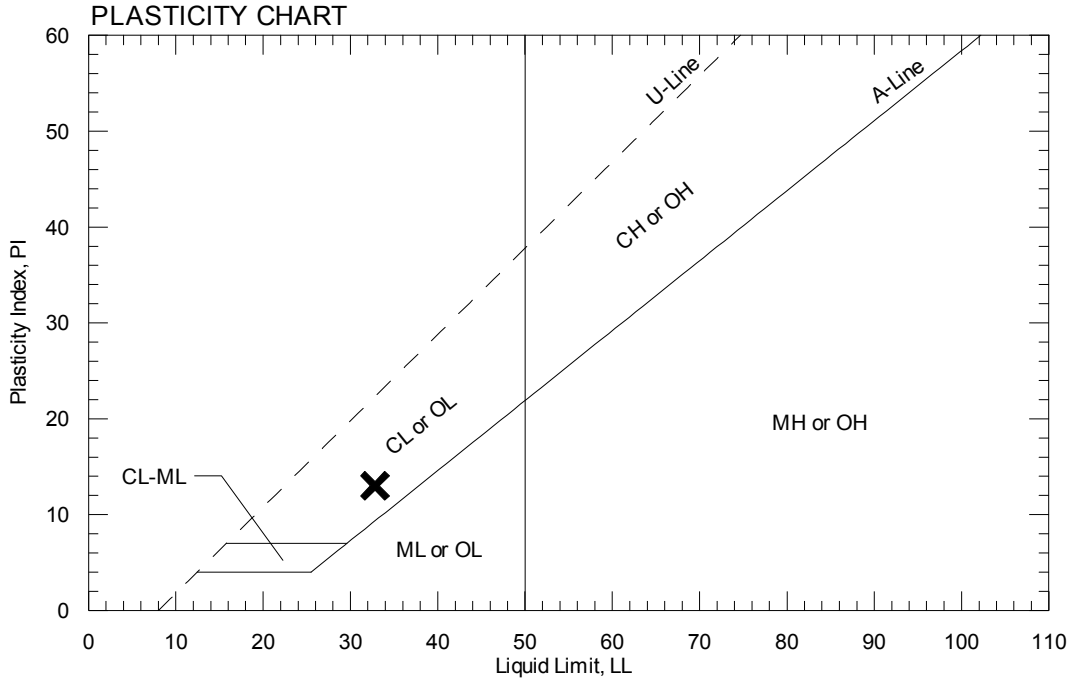
TOWN	Milford	Reference No.	239842
WIN	016667.00	Water Content, %	51.2
Sampled	7/21/2010	Liquid Limit @ 25 blows (T 89), %	32
Boring No./Sample No.	BB-MOS-102/2D	Plastic Limit (T 90), %	23
Station	30+59.3	Plasticity Index (T 90), %	9
Depth	5.0-7.0	Tested By	BBURR





TOWN	Milford	Reference No.	239845
WIN	016667.00	Water Content, %	23.1
Sampled	7/20/2010	Liquid Limit @ 25 blows (T 89), %	33
Boring No./Sample No.	BB-MOS-103/2DA	Plastic Limit (T 90), %	20
Station	31+71.9	Plasticity Index (T 90), %	13
Depth	5.0-6.5	Tested By	BBURR





Appendix C

Calculations

LIQUIDITY INDEX (LI):

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

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

Sample	WC	LL	PL	PI	Plasticity	LI	
BB-MOS-101/5D	28.9	32	22	10	low plasticity	0.69	overconsolidated
BB-MOS-101/6D	38.8	35	23	12	medium plasticity	1.32	viscous liquid when remolded
BB-MOS-102/2D	51.2	32	23	9	low plasticity	3.13	viscous liquid when remolded

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles (Fixed Head)

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

Look at the following piles:

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

Note: All matrices set up in this order

Pile sizes HP12x53 and 14x73 are not allowed for bridges with a spane length of 150 feet and a fixed head abutment per MaineDOT BDG Table 5-3.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

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

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

LRFD Article 6.9.4.1.1

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

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

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

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

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

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "good" based on borings.

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: eq. 6.9.2.1-1

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

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

Strength Limit State
Fixed head

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

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

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

Service/Extreme Limit States
Fixed head

Axial Structural Resistance of H-piles (Pinned Head)

Look at the following piles:

HP 12 x 53
HP 12 x 74 Note: All matrices set up in this order
HP 14 x 73
HP 14 x 89
HP 14 x 117

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

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

K = effective length factor $K_{eff} := 0.8$ LRFD Table C4.6.2.5-1 Design value: ideal conditions, rotation free, translation free at head; rotation fixed, translation fixed at tip

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

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

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

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

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

LRFD eq. 6.9.4.1.2-1

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

$$P_e = \begin{pmatrix} 393749 \\ 577268 \\ 809506 \\ 1010055 \\ 1376901 \end{pmatrix} \cdot kip$$

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

$$\frac{P_e}{P_o} = \begin{pmatrix} 508 \\ 530 \\ 757 \\ 774 \\ 801 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

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

$$P_n = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "good" based on borings.

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

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance: eq. 6.9.2.1-1

$$P_r := \phi_c \cdot P_n \qquad P_r = \begin{pmatrix} 465 \\ 653 \\ 642 \\ 783 \\ 1031 \end{pmatrix} \cdot kip$$

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

Strength Limit State
Pinned head

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_r := \phi \cdot P_n \qquad P_r = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit States
Pinned head

Geotechnical Resistance - by Canadian Geotechnical Method

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

Bedrock Type:

Sandstone Average RQD 50%

Use RQD = 50% and ϕ = 27 to 34 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 6th Edition 2012

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

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

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

q_u for sandstone compressive strength ranges from 9700 to 25000 psi

use $\sigma_c := 12000 \cdot psi$

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

Spacing of discontinuities: $c := 24 \cdot in$ 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

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

$$K_{sp} = \begin{pmatrix} 0.4566 \\ 0.4541 \\ 0.4249 \\ 0.4238 \\ 0.4219 \end{pmatrix}$$

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

$q_a := \sigma_c \cdot K_{sp} \cdot d_f$ $q_a = \begin{pmatrix} 789 \\ 785 \\ 734 \\ 732 \\ 729 \end{pmatrix} \cdot ksf$

Nominal Geotechnical Tip Resistance, R_p:

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

$R_p := \overrightarrow{(3q_a \cdot A_s)}$ $R_p = \begin{pmatrix} 255 \\ 356 \\ 327 \\ 398 \\ 522 \end{pmatrix} \cdot kip$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

$R_f := \phi_{stat} \cdot R_p$ $R_f = \begin{pmatrix} 115 \\ 160 \\ 147 \\ 179 \\ 235 \end{pmatrix} \cdot kip$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

$R_{fse} := \phi \cdot R_p$ $R_{fse} = \begin{pmatrix} 255 \\ 356 \\ 327 \\ 398 \\ 522 \end{pmatrix} \cdot kip$

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

Service/Extreme Limit States

Axial Geotechnical Resistance Piles Driven to Hard Rock per LRFD Article 10.7.3.2.3

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

Nominal Structural Resistance:
previously calculated

$$P_n = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

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

Determine *Factored* Axial Geotechnical Resistance per LRFD Article 10.7.3.2.3

Apply resistance factor for severe driving from LRFD Article 6.5.4.2

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

Factored Axial Geotechnical Resistance
Strength Limit State

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

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

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

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

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

$$\phi = 1.0$$

Factored Axial Geotechnical Resistance -
Service and Extreme Limit States

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

$$P_{serv\_ext} = \begin{pmatrix} 774 \\ 1089 \\ 1069 \\ 1304 \\ 1719 \end{pmatrix} \cdot kip$$

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

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (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 Delmag 19-42 hammer

State of Maine Dept. Of Transportation — 13-Dec-2012
16667 Milford Drivability 12x53 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
425.0	43.20	6.18	5.8	8.94	21.08
430.0	43.53	6.26	6.0	9.00	21.17
435.0	43.81	6.35	6.1	9.05	21.28
440.0	44.14	6.43	6.3	9.12	21.44
445.0	44.42	6.51	6.4	9.17	21.56
450.0	44.73	6.58	6.6	9.23	21.72
455.0	45.01	6.63	6.7	9.28	21.83
460.0	45.28	6.70	6.9	9.33	21.94
465.0	45.57	6.74	7.1	9.39	22.03
470.0	45.83	6.79	7.3	9.45	22.15

Limit driving stress to 45 ksi - blow count limited to 6 bpi as 7 bpi exceeds 45 ksi

$R_{dr\_12x53} := 430 \cdot kip$

Strength Limit State:

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

$R_{dr\_12x53\_strength} = 280 \cdot kip$

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

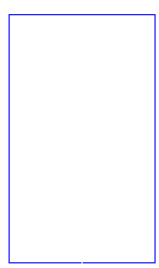
$R_{dr\_12x53\_servext} := R_{dr\_12x53} \cdot \phi$

$R_{dr\_12x53\_servext} = 430 \cdot kip$

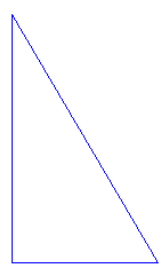
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	50.00	ft
Pile Penetration	50.00	ft
Pile Top Area	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag 19-42 hammer

State of Maine Dept. Of Transportation — 13-Dec-2012
16667 Milford Drivability 12x74 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
645.0	44.67	5.88	14.8	9.88	22.03
647.0	44.70	5.89	15.0	9.89	22.02
649.0	44.78	5.93	15.1	9.90	22.07
651.0	44.84	5.93	15.4	9.91	22.06
653.0	44.88	5.98	15.5	9.92	22.11
655.0	44.94	5.99	15.8	9.93	22.09
657.0	45.02	6.04	15.9	9.94	22.15
659.0	45.05	6.04	16.1	9.95	22.13
661.0	45.12	6.08	16.3	9.95	22.18
663.0	45.18	6.13	16.4	9.96	22.22

Limit driving stress to 45 ksi - blow count limited to 15 bpi

$R_{dr\_12x74} := 647 \cdot kip$

Strength Limit State:

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

$R_{dr\_12x74\_strength} = 421 \cdot kip$

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

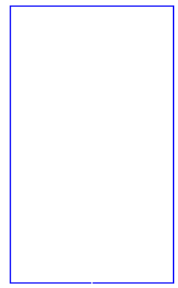
$R_{dr\_12x74\_servext} := R_{dr\_12x74} \cdot \phi$

$R_{dr\_12x74\_servext} = 647 \cdot kip$

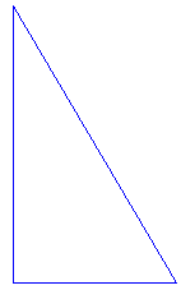
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	50.00	ft
Pile Penetration	50.00	ft
Pile Top Area	21.80	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag 19-42 hammer

State of Maine Dept. Of Transportation — 13-Dec-2012
16667 Milford Drivability 14x73 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
630.0	44.69	5.58	13.9	9.86	21.96
632.0	44.73	5.60	14.0	9.87	22.01
634.0	44.80	5.60	14.2	9.88	22.00
636.0	44.87	5.62	14.3	9.89	22.06
638.0	44.92	5.62	14.5	9.90	22.05
640.0	44.96	5.66	14.6	9.91	22.10
642.0	45.04	5.65	14.9	9.92	22.09
644.0	45.10	5.67	15.0	9.93	22.14
646.0	45.14	5.67	15.2	9.94	22.14
648.0	45.23	5.68	15.3	9.95	22.18

Limit driving stress to 45 ksi - blow count limited to 14 bpi as 15 bpi exceeds 45 ksi

$R_{dr\_14x73} := 632 \cdot kip$

Strength Limit State:

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

$R_{dr\_14x73\_strength} = 411 \cdot kip$

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

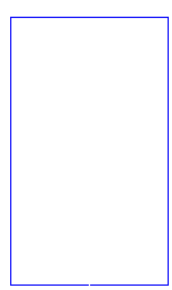
$R_{dr\_14x73\_servext} := R_{dr\_14x73} \cdot \phi$

$R_{dr\_14x73\_servext} = 632 \cdot kip$

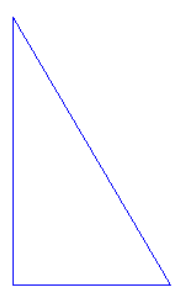
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	50.00	ft
Pile Penetration	50.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

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

State of Maine Dept. Of Transportation — 13-Dec-2012
16667 Milford Drivability 14x89 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
720.0	44.92	4.94	8.8	7.17	29.06
721.0	44.96	4.94	8.9	7.17	29.08
722.0	45.00	4.95	8.9	7.18	29.17
723.0	45.03	4.95	9.0	7.18	29.10
724.0	45.07	4.95	9.0	7.18	29.11
725.0	45.12	4.96	9.0	7.19	29.20
726.0	45.15	4.96	9.0	7.19	29.20
727.0	45.16	4.96	9.1	7.20	29.21
728.0	45.18	4.97	9.1	7.20	29.21
729.0	45.24	4.97	9.2	7.20	29.24

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

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

Strength Limit State:

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

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

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

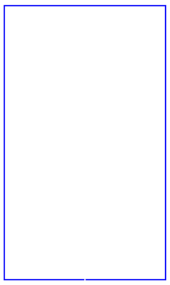
$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \phi$

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

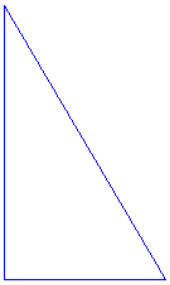
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	50.00	ft
Pile Penetration	50.00	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 Delmag 36-32 hammer on lowest fuel setting

State of Maine Dept. Of Transportation — 13-Dec-2012
16667 Milford Drivability 14x117 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
910.0	43.63	3.79	14.5	7.48	29.64
915.0	43.74	3.81	14.7	7.49	29.68
920.0	43.87	3.85	15.0	7.51	29.72
925.0	43.99	3.92	15.2	7.52	29.84
930.0	44.09	3.96	15.5	7.53	29.89
935.0	43.90	3.98	16.3	7.47	29.56
940.0	44.02	4.05	16.5	7.49	29.68
945.0	44.13	4.08	16.8	7.50	29.72
950.0	44.18	4.11	17.2	7.51	29.71
955.0	44.30	4.18	17.5	7.52	29.84

Limit blow count to 15 bpi

$R_{dr\_14x117} := 920 \cdot kip$

Strength Limit State:

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

$R_{dr\_14x117\_strength} = 598 \cdot kip$

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

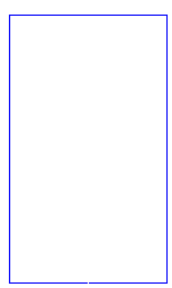
$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \phi$

$R_{dr\_14x117\_servext} = 920 \cdot kip$

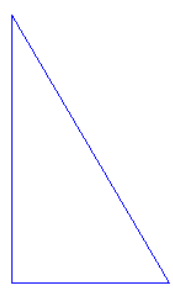
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	50.00	ft
Pile Penetration	50.00	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

For construction of causeways as much as 15 feet of fill will be placed.
Look at a simplified soil profile based on BB-MOS-101

Proposed Fill - Look at 15 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Groundwater at top of gravel

$\gamma_w := 62.4pcf$

Existing Gravel $H_{1gravel} := 6.6 \cdot ft$ $\gamma_{gravel} := 125 \cdot pcf$ $N_{gravel} := 7$

Glaciomarine - Silty Clay and Silt $H_{2GM} := 19.0 \cdot ft$ $\gamma_{GM} := 115 \cdot pcf$

Total Layer height: H = 19.0 ft - divide into 4 layers

$H_{2GM1} := 4.0 \cdot ft$ $N_{2GM1} := 1$ $C_{c\_GM} := 0.4$ $C_{r\_GM} := 0.03$ $e_{o\_GM} := 0.77$

$H_{2GM2} := 5.0 \cdot ft$ $N_{2GM2} := 1$

$H_{2GM3} := 5.0 \cdot ft$ $N_{2GM3} := 11$

$H_{2GM4} := 5.0 \cdot ft$ $N_{2GM4} := 17$

Assumed Values based on Lab Data and "A Summary of Geotechnical Engineering Information on the Presumpscot Formation Silty Clay" 1986 by David W. Andrews:

Glacial Till -
Total Layer height: H = 15 ft

$H_3 := 15.0 \cdot ft$ $\gamma_{Till} := 125 \cdot pcf$ $N_{Till} := 50$

Bedrock - Sandstone

LOADING ON AN INFINITE STRIP
VERTICAL EMBANKMENT LOADING

Project Name: Second Otter Bridge Client: Milford
Project Number : 16667.00 Project Manager: Coombs
Date: 12/18/12 Computed by: km

Embank. slope a = 20.00(ft)
Embank. width b = 44.00(ft)
p load/unit area = 1875.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 30.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	1875.00
1.00	1874.77
2.00	1873.18
3.00	1869.04
4.00	1861.44
5.00	1849.83
6.00	1833.99
7.00	1814.04
8.00	1790.30
9.00	1763.25
10.00	1733.43
11.00	1701.42
12.00	1667.76
13.00	1632.95
14.00	1597.43
15.00	1561.59
16.00	1525.72
17.00	1490.08
18.00	1454.88
19.00	1420.27
20.00	1386.37
21.00	1353.28
22.00	1321.04
23.00	1289.71
24.00	1259.31
25.00	1229.85
26.00	1201.33
27.00	1173.74
28.00	1147.07
29.00	1121.30
30.00	1096.41
31.00	1072.38
32.00	1049.17
33.00	1026.77
34.00	1005.14
35.00	984.25

by linear interpolation

at 3.3 ft $\Delta\sigma_{z1gravel} := 1866.76 \cdot psf$

at 8.6 ft $\Delta\sigma_{z2GM1} := 1774.07 \cdot psf$

at 13.1ft $\Delta\sigma_{z2GM2} := 1629.40 \cdot psf$

at 18.1 ft $\Delta\sigma_{z2GM3} := 1451.42 \cdot psf$

at 23.1 ft $\Delta\sigma_{z2GM4} := 1286.67 \cdot psf$

at 33.1 ft $\Delta\sigma_{z3Till} := 1024.61 \cdot psf$

Existing Gravel

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

Calculate vertical stress at mid point: $\sigma_{gravel\_o} := \frac{H_{1gravel}}{2} \cdot (\gamma_{gravel} - \gamma_w)$ $\sigma_{gravel\_o} = 0.2066 \cdot ksf$

Corrected SPT N_{60}-value (bpf) $N_{gravel} = 7$

At P_o = 0.21 tsf $C_{N\_gravel} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{gravel\_o}}\right)$ $C_{N\_gravel} = 1.761$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_gravel} \cdot N_{gravel}$ $N1_{60} = 12$
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: $C_{gravel} := 55$

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

$$\Delta\sigma_{z1gravel} = 1866.76 \cdot psf$$

Glaciomarine - 4 Layers

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

Assumed Values based on Lab Data and "A Summary of Geotechnical Engineering Information on the Presumpscot Formation Silty Clay" 1986 by David W. Andrews:

<u>Layer 1 - Silty Clay:</u> Assumed Values: $e_{o\_GM} = 0.77$ $C_{r\_GM} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{2GM1\_o} := \left[\frac{H_{2GM1}}{2} \cdot (\gamma_{GM} - \gamma_w)\right] + \left[H_{1gravel} \cdot (\gamma_{gravel} - \gamma_w)\right] \qquad \sigma_{2GM1\_o} = 0.5184 \cdot ksf$$

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

$$\Delta\sigma_{z2GM1} = 1774.07 \cdot psf$$

<u>Layer 2 - Silty Clay:</u> Assumed Values: $e_{o\_GM} = 0.77$ $C_{r\_GM} = 0.03$

Calculate vertical stress at mid point:

$$\sigma_{2GM2\_o} := \left[\frac{H_{2GM2}}{2} \cdot (\gamma_{GM} - \gamma_w)\right] + H_{2GM1} \cdot (\gamma_{GM} - \gamma_w) + H_{1gravel} \cdot (\gamma_{gravel} - \gamma_w) \qquad \sigma_{2GM2\_o} = 0.7551 \cdot ksf$$

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

$$\Delta\sigma_{z2GM2} = 1629.4 \cdot psf$$

Layer 3 -Silt :

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

Calculate vertical stress at mid point:

$$\sigma_{2GM3\_o} := \left[\frac{H_{2GM3}}{2} \cdot (\gamma_{GM} - \gamma_w)\right] + \left[(H_{2GM1} + H_{2GM2}) \cdot (\gamma_{GM} - \gamma_w)\right] + H_{1gravel} \cdot (\gamma_{gravel} - \gamma_w) \qquad \sigma_{2GM3\_o} = 1.0181 \cdot ksf$$

Corrected SPT N_{60}-value (bpf) $N_{2GM3} = 11$

At P_o = 1.02 tsf $C_{N\_2GM3} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{2GM3\_o}}\right)$ $C_{N\_2GM3} = 1.2276$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_2GM3} \cdot N_{2GM3}$ $N1_{60} = 14$
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: $C_{2GM3} := 38$

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

$$\Delta\sigma_{z2GM3} = 1451.42 \cdot psf$$

Layer 4 -Silt:

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

Calculate vertical stress at mid point:

$$\sigma_{2GM4\_o} := \left[\frac{H_{2GM4}}{2} \cdot (\gamma_{GM} - \gamma_w)\right] + \left[(H_{2GM1} + H_{2GM2} + H_{2GM3}) \cdot (\gamma_{GM} - \gamma_w)\right] + H_{1gravel} \cdot (\gamma_{gravel} - \gamma_w)$$

$$\sigma_{2GM4\_o} = 1.2811 \cdot ksf$$

Corrected SPT N_{60}-value (bpf) $N_{2GM4} = 17$

At P_o = 1.28 tsf $C_{N\_2GM4} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{2GM4\_o}}\right)$ $C_{N\_2GM4} = 1.1508$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_2GM4} \cdot N_{2GM4}$ $N1_{60} = 20$
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: $C_{2GM4} := 42$

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

$$\Delta\sigma_{z2GM4} = 1286.67 \cdot psf$$

Glacial Till

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

Calculate vertical stress at mid point:

$$\sigma_{Till\_o} := \frac{H_3}{2} \cdot (\gamma_{Till} - \gamma_w) + H_{2GM} \cdot (\gamma_{GM} - \gamma_w) + H_{1gravel} \cdot (\gamma_{gravel} - \gamma_w) \qquad \sigma_{Till\_o} = 1.8821 \cdot ksf$$

Corrected SPT N_{60}-value (bpf) $N_{Till} = 50$

At P_o = 1.88 tsf $C_{N\_Till} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{Till\_o}}\right)$ $C_{N\_Till} = 1.0221$ LRFD Article 10.4.6.2.4

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60} := C_{N\_Till} \cdot N_{Till}$ $N1_{60} = 51$
From LRFD Eq. 10.4.6.2.4-1

From Hough Figure 7-7 pg 7-17 using the "Well graded fine to medium silty sand" curve

Bearing Capacity Index: $C_{Till} := 120$

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

$$\Delta\sigma_{z3Till} = 1024.61 \cdot psf$$

Calculate Settlement:

Existing Gravel: $\Delta H_{1gravel} := H_{1gravel} \cdot \frac{1}{C_{gravel}} \cdot \log\left(\frac{\sigma_{gravel\_o} + \Delta\sigma_{z1gravel}}{\sigma_{gravel\_o}}\right)$ $\Delta H_{1gravel} = 1.4423 \cdot in$

Glaciomarine Layer 1: Silty Clay $\Delta H_{2GM1} := H_{2GM1} \cdot \left(\frac{C_{r\_GM}}{1 + e_{o\_GM}}\right) \cdot \log\left(\frac{\sigma_{2GM1\_o} + \Delta\sigma_{z2GM1}}{\sigma_{2GM1\_o}}\right)$ $\Delta H_{2GM1} = 0.5253 \cdot in$

Glaciomarine Layer 2: Silty Clay $\Delta H_{2GM2} := H_{2GM2} \cdot \left(\frac{C_{r\_GM}}{1 + e_{o\_GM}}\right) \cdot \log\left(\frac{\sigma_{2GM2\_o} + \Delta\sigma_{z2GM2}}{\sigma_{2GM2\_o}}\right)$ $\Delta H_{2GM2} = 0.5079 \cdot in$

Glaciomarine Layer 3: Silt $\Delta H_{2GM3} := H_{2GM3} \cdot \left(\frac{1}{C_{2GM3}}\right) \cdot \log\left(\frac{\sigma_{2GM3\_o} + \Delta\sigma_{z2GM3}}{\sigma_{2GM3\_o}}\right)$ $\Delta H_{2GM3} = 0.6076 \cdot in$

Glaciomarine Layer 4: Silt $\Delta H_{2GM4} := H_{2GM4} \cdot \left(\frac{1}{C_{2GM4}}\right) \cdot \log\left(\frac{\sigma_{2GM4\_o} + \Delta\sigma_{z2GM4}}{\sigma_{2GM4\_o}}\right)$ $\Delta H_{2GM4} = 0.4314 \cdot in$

Glacial Till: $\Delta H_{3Till} := H_3 \cdot \frac{1}{C_{Till}} \cdot \log\left(\frac{\sigma_{Till\_o} + \Delta\sigma_{z3Till}}{\sigma_{Till\_o}}\right)$ $\Delta H_{3Till} = 0.2831 \cdot in$

TOTAL SETTLEMENT:

$$\Delta H_T := \Delta H_{1gravel} + \Delta H_{2GM1} + \Delta H_{2GM2} + \Delta H_{2GM3} + \Delta H_{2GM4} + \Delta H_{3Till} \qquad \Delta H_T = 3.7976 \cdot in$$

Say 3 to 4 inches of settlement will occur

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:
Milford, Maine
DFI = 1750 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 1750 frost penetration = 88.8 inches

$Frost\_depth := 88.8in$ $\quad$ $Frost\_depth = 7.4 \cdot ft$

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

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Orono

--- ModBerg Results ---

Project Location: Orono, Maine

Air Design Freezing Index	=	1588 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	1270 F-days
Mean Annual Temperature	=	43.5 deg F
Design Length of Freezing Season	=	132 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	77.3	10.0	125.0	28	34	2.0	1.6	1,800

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

****
Total Depth of Frost Penetration = 6.45 ft = 77.3 in.
****

$Frost\_depth_{modberg} := 77.3 \cdot in$ $\quad$ $Frost\_depth_{modberg} = 6.4417\,ft$

Use Frost Depth = 6.5 feet for design

Seismic:

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

BB-MOS-101					BB-MOS-102				
Depth	**SPT N**		**di**	**di/N**	**Depth**	**SPT N**		**di**	**di/N**
2	49		2.5	0.05102	2	28		2.5	0.0892857
6	10		5	0.5	6	13		5	0.3846154
11	25		5	0.2	11	3		5	1.6666667
16	7		5	0.714286	17.5	22		6	0.2727273
21	1		5	5	21	24		5	0.2083333
26	1		5	5	26	11		5	0.4545455
31	11		5	0.454545	31	24		5	0.2083333
36	17		5	0.294118	36	42		5	0.1190476
46	111		12.5	0.112613	41	35		5	0.1428571
51	28		2	0.071429	46	34		5	0.1470588
52	100	**Bedrock**	48	0.48	51	55		5	0.0909091
					56	70		5.5	0.0785714
					59	100	**Bedrock**	41	0.41
SUM			**100**	**12.87801**	**SUM**			**100**	**4.2729513**
			di/di/N	**7.765175**				**di/di/N**	**23.403028**
							SUM	Nav.	15.584102
						15<Nav<50 blows per foot = Site Class D			

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

16667.00 Milford Second Otter Bridge

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04461
Zip Code Latitude = 44.959800
Zip Code Longitude = -068.614200
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.068	PGA - Site Class B
0.2	0.147	Ss - Site Class B
1.0	0.044	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04461
Zip Code Latitude = 44.959800
Zip Code Longitude = -068.614200
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.110	As - Site Class D
0.2	0.236	SDs - Site Class D
1.0	0.106	SD1 - Site Class D