MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

GREAT HILL BRIDGE
OVER GREAT WORKS RIVER
SOUTH BERWICK, MAINE

Prepared by:

Kathleen Maguire, P.E.
Geotechnical Engineer

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

York County
PIN 16749.00

Soils Report No. 2010-20
Bridge No. 1236

Fed No. BR-1674(900)X
August 6, 2010

TEDOCS No. 1058241

Table of Contents

Tables

Sheets

Appendices

GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Great Hill Bridge on Great Hill Road over Great Works River in South Berwick, Maine. The proposed replacement bridge will consist of a 75 foot single span; steel superstructure supported on H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with driving points to protect the tips and improve penetration. It is recommended that the maximum factored axial pile load used in design for the strength, service and extreme limit states should not exceed the factored drivability resistance. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Downdrag – Settlement analyses indicate that approximately 9.4 inches of settlement will occur at the site due to the placement of a maximum of 12 feet of fill. Settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag (negative skin friction) forces on piles. The magnitude of downdrag has been estimated to range between 83 and 101 kips depending upon pile size. It is recommended that a load factor, γ_p=1.0, be applied to downdrag forces in cohesive and cohesionless downdrag zones.

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. The Coulomb passive earth pressure coefficient, K_p, of 6.89 is recommended. Developing full passive requires displacements of the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced Coulomb passive earth pressure coefficient, but not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p, of 3.25. A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, γ_{EH} = 1.50. All abutment designs shall include a drainage system behind the abutments to intercept any groundwater.

Bearing Resistance - Bearing resistance for foundations on fill or native sand soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 4 ksf for wall system bases less than 8 feet wide and 5 ksf for bases from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 3 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing. In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as 0.3 *f'c*. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

Scour and Riprap - The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load. For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Settlement - Evaluation of the potential settlement due to the placement of the approximately 12 feet of fill resulted in approximately 9.4 inches of settlement. The majority of this settlement is consolidation settlement within the compressible silt and clay soils underlying the site. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag forces on piles. This settlement is anticipated to occur over a long period of time (on the order of 5 to 6 years) and may require attention by a maintenance crew.

Frost Protection - Any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. Integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

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

Construction Considerations - Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the Plans. Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill prior to pile driving. Using the excavated native soils as structural backfill should not be permitted. 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.

1.0 INTRODUCTION

A subsurface investigation and geotechnical design for the replacement of Great Hill Bridge on Great Hill Road over Great Works River in South Berwick, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the subsurface information obtained at the site, geotechnical design parameters and foundation recommendations.

The existing bridge superstructure was constructed in 1983 and consists of a single lane, 45 foot long single-span structure with rolled steel girders and a timber deck. The bridge abutments are comprised of stacked granite blocks (with some mortar) founded on soil. The date the abutments were constructed is unknown. The 2008 Maine Department of Transportation (MaineDOT) maintenance inspection report indicates that the substructure is in poor condition. The east abutment has full height vertical cracks in the return wings causing the breast wall to rotate towards the channel. The west abutment stones have shifted causing mortar and backfill to fall out between the blocks. In 2007 a recommendation to monitor the abutments for movement was made. The maintenance inspection report indicates that the bridge superstructure is in "satisfactory" condition (rating of 6), the substructure is in "poor" condition (rating of 4) and the deck is in "fair" condition (rating of 5). The Bridge Sufficiency Rating is 26.0. The bridge has a scour critical rating of "U" meaning that the bridge has unknown foundations that have not been evaluated for scour. It is understood that the existing bridge will be completely removed and replaced.

The proposed bridge will consist of a two-lane, 75 foot long, single-span, superstructure founded on H-pile supported integral abutments on a new alignment. Both the superstructure and substructure design will be a detail-build option in the final contract. In order to improve the roadway alignment, the new roadway centerline will move upstream approximately 10 feet at the east approach and downstream approximately 20 feet at the west approach. Both of the proposed abutments will be located approximately 25 feet behind the existing abutments on the new alignment. The vertical alignment will be raised approximately 2.0 feet at the east abutment and approximately 3.5 feet at the west abutment. Two large fill areas will be required behind the abutments. Approximately 12 feet of fill will be required behind Abutment No. 1 at the southeast end and approximately 11 feet of fill will be required behind Abutment No. 2 at the northwest end to construct the roadway on the proposed alignment. Retaining walls may be constructed along the relocated roadway to retain the widened roadway section and minimize impacts. The existing bridge will be closed to traffic during construction.

2.0 GEOLOGIC SETTING

Great Hill Bridge on Great Hill Road in South Berwick crosses Great Works River approximately 0.5 miles east of Hooper Sands Road as shown on Sheet 1 - Location Map found at the end of this report. Great Works River flows in a southwesterly direction to the Salmon Falls River.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits. Soils in the site area are generally comprised of silt, clay, sand and minor amounts of gravel. Sand is dominant in some areas, but may be underlain by finer-grained sediments. The unit contains small areas of till not completely covered by marine sediments. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams and commonly has a branching network of steep-walled stream gullies. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine, published by the Maine Geological Survey (1985), the site lies at the interface of two identified bedrock formations. To the northeast the bedrock is identified as Silurian-Precambrian age calcareous pelite of the Eliot Formation. To the southeast the bedrock is identified as Silurian-Precambrian age calcareous feldspathic sandstone of the Kittery Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling three (3) test borings at the site. Test boring BB-SBGW-101 was drilled behind the location of existing Abutment No. 1 (south). Test boring BB-SBGW-102 was drilled behind the location of existing Abutment No. 2 (north). Test boring BB-SBGW-103 was drilled on the north river bank at the potential location of a proposed abutment if and alternate alignment is chosen. The exploration locations are shown on Sheet 2 - Boring Location Plan found at the end of this report. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The borings were drilled on November 3 and December 2, 2009 by the Maine Department of Transportation (MaineDOT) drill crew and Northern Test Boring (NTB) of Gorham, Maine. 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 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. Both of the drill rigs used at the site are equipped with automatic hammers to drive the split spoon. The hammers were calibrated in February of 2009. The MaineDOT automatic hammer was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. The NTB automatic hammer was found to deliver approximately 13 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 to the raw field N-values. This hammer efficiency factor (0.84 for MaineDOT and 0.68 for NTB) and both the raw field N-value and the corrected N-value are shown on the boring logs.

Undisturbed tube samples were obtained in the soft soil deposits where possible. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ-2" core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion if the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of four (4) standard grain size analyses with natural moisture content, twenty-one (21) grain size analysis with hydrometer and natural moisture content, twelve (12) Atterberg Limits tests, one (1) consolidation test, and two (2) standard tube openings with laboratory vanes. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 4 - Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the site consisted of fill materials overlying silt and clay overlying sand all overlying bedrock. An interpretive subsurface profile depicting the site stratigraphy is show on Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

Fill Materials. A surficial layer of fill was encountered in all of the borings. The fill materials encountered were:

- Brown and light brown, damp, silty, fine to coarse sand, with trace gravel
- Brown, damp, gravelly, fine to coarse sand, with little silt
- Red-brown, damp, fine to coarse sand, with little silt, trace gravel and trace organics
- Light brown, moist, fine to coarse sand, some silt, little gravel, trace clay, and trace organics
- Riprap (6 inches thick) underlain by cobbles and gravel was encountered in boring BB-SBGW-103.

The overall thickness of the fill layer ranged from approximately 5.0 feet to 11.0 feet. Corrected SPT N-values in the fill layer ranged from 3 to 22 blows per foot (bpf) indicating that the fill soil is very loose to medium dense in consistency. Water contents from five (5) samples obtained within the fill range from approximately 14% to 22%. Five (5) grain size analyses conducted on samples from the fill indicate that the soil is classified as an A-4 or A-2-4 by the AASHTO Classification System and a SM or SC-SM by the Unified Soil Classification System.

Silt and Clay. Beneath the fill materials, interbedded layers of silt, clayey silt and silty clay were encountered. Soils encountered consisted of:

- Grey-brown, damp, mottled, silt with little sand and trace organics
- Grey, wet, silt with little clay, little sand and trace organics
- Light brown, wet, silt with some sand and little clay
- Grey, wet, clayey silt, with trace fine sand and trace gravel
- Grey, wet, silty clay, with trace sand

A discontinuous layer of sand was encountered at a depth of 13.5 feet bgs within the interbedded silt and clay in boring BB-SBGW-102.

The thickness of the silt and clay layer ranged from approximately 18.7 to 39.0 feet. Corrected SPT N-values obtained in the silt and clay ranged from weight of rods (WOR) to 7 bpf indicating that the soil is very soft to medium stiff in consistency.

Vane shear testing conducted on silt and clay samples showed measured undrained shear strengths ranging from approximately 268 to 1062 psf while the remolded shear strength ranged from approximately 45 to 134 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silty clay was determined to have sensitivity ranging from approximately 4.0 to 11.0 and is classified as moderately sensitive to very sensitive.

Water contents from fifteen (15) samples obtained within this layer ranged from approximately 33% to 45%. Fifteen (15) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-6, A-4 or A-2-4 by the AASHTO Classification System and a CL or CL-ML by the Unified Soil Classification System.

Table 5-1 below summarizes the results of the Atterberg Limits testing on the silt and clay samples:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-SBGW-101 4D	Clayey Silt	33.5	34	22	12	0.96
BB-SBGW-101 1U	Silty Clay	44.9	37	23	14	1.56
BB-SBGW-101 5D	Clayey Silt	43.9	35	22	13	1.68
BB-SBGW-102 5D	Clayey Silt	40.5	36	23	13	1.35
BB-SBGW-102 6D	Silty Clay	45.0	37	24	13	1.62
BB-SBGW-102 1U	Clayey Silt	41.8	37	23	14	1.34
BB-SBGW-103 3D	Clayey Silt	37.0	34	23	11	1.27
BB-SBGW-103 4D	Silty Clay	39.5	34	23	11	1.50
BB-SBGW-103 5D	Clayey Silt	38.7	37	24	13	1.13
BB-SBGW-103 6D	Clayey Silt	42.2	37	23	14	1.37
BB-SBGW-103 7D	Silty Clay	42.7	37	23	14	1.41
BB-SBGW-103 8D	Clayey Silt	43.0	37	24	13	1.46

Table 5-1 – Summary of Atterberg Limits Testing Results

Interpretation of these results indicates that silt and clay is generally on the verge of becoming a viscous liquid if disturbed. For all but one of the samples the natural water content exceeds the liquid limit. This indicates that the soil has 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”. One (1) of the samples has a liquidity index of approximately 1 indicating a soil which is normally consolidated.

One-dimensional (1-D) consolidation testing was conducted on one (1) tube sample taken from the layer. The results of this test were used to calculate the anticipated settlements at the site and are included in Appendix B - Laboratory Data.

A discontinuous layer of sand was encountered at a depth of 13.5 feet bgs within the silt and clay in boring BB-SBGW-102. The sand layer was approximately 5 feet thick. One corrected SPT N-value obtained within the sand was 6 bpf indicating that the sand is loose in consistency. One (1) water content from a sample obtained within this layer was approximately 30%. One (1) grain size analysis conducted on a sample of the sand indicates that the soil is classified as an A-2-4 by the AASHTO Classification System and a SC-SM by the Unified Soil Classification System.

Sand. Beneath the silt and clay materials a layer of sand was encountered. Soils encountered consisted of:

- Grey, wet, silty fine to coarse sand with trace broken rock
- Grey, wet, fine to coarse sand with trace to some gravel, trace to some silt, trace clay and trace broken rock fragments

The overall thickness of the sand layer ranged from approximately 13.5 to 9.3 feet. Corrected SPT N-values in the sand layer ranged from 19 to greater than 50 bpf indicating that the soil is medium dense to very dense in consistency. Water contents from four (4) samples obtained within the sand ranged from approximately 10% to 14%. Four (4) grain size analyses conducted on samples from the sand indicate that the soil is classified as an A-1-b or A-2-4 by the AASHTO Classification System and a SW-SC, SM or SC-SM by the Unified Soil Classification System.

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

Boring Number/ Location	Approximate Depth to Bedrock	Approximate Bedrock Elevation	RQD
BB-SBGW-101	43.2 feet	58.3 feet	10 – 41%
BB-SBGW-102	46.4 feet	55.1 feet	0 – 50%
BB-SBGW-103	53.3 feet	42.6 feet	0 – 38%

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

The bedrock at the site can be identified as grey, fine-grained, highly fractured, sandstone. The RQD of the bedrock ranged from 0 to 50% indicating a rock of very poor to poor quality.

Groundwater. Groundwater was observed at a depths ranging from approximately 5.5 to 14.0 feet below the ground surface at the boring locations. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

6.0 FOUNDATION ALTERNATIVES

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

- A single-span structure utilizing cast-in-place or precast concrete integral stub abutments supported on driven steel H-piles
- A two-span structure utilizing cast-in-place or precast concrete integral stub abutments supported on driven steel H-piles and a pipe pile pier bent
- A two-span structure utilizing precast cast-in-place or concrete integral stub abutments supported on driven steel H-piles and a mass concrete pier supported on H-piles

After consideration of all of the alternatives, the Bridge Program has chosen an alignment which will allow for a single span, integral structure supported on driven H-piles.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

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

7.1 Integral Abutment H-piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with driving points 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 Free Length
Abutment #1 BB-SBGW-101	92.5 feet	43.7 feet	58.3 feet	35 feet
Abutment #2 BB-SBGW-102	92.2 feet	46.4 feet	55.1 feet	37 feet

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

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

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 7.1.1 below.

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

Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2. An L-Pile® analysis is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, movements and pile head displacements.

7.1.1 Strength Limit State

The nominal compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. For preliminary analyses the H-piles were assumed fully embedded and the column slenderness factor, λ, was taken as 0. The factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60 and a λ of 0. It is the responsibility of the structural designer to recalculate λ for the upper and lower portions of the H-pile based on unbraced length and K-values from project specific L-Pile® analyses and recalculate structural resistances.

For the portion of the pile which is theoretically in pure compression, i.e. below the point of fixity, the factored structural axial resistances of five (5) H-pile sections were calculated using a resistance factor, ϕ_c, of 0.60. The factored structural axial resistance may be controlled by the combined axial and flexural resistance of the pile. This is the responsibility of the structural designer.

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

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

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

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* $\phi_c = 0.60$ $\lambda = 0$	Geotechnical Resistance $\varphi_{stat} = 0.45$	Drivability Resistance $\varphi_{dyn} = 0.65$	Governing Resistance Based on Drivability Analyses
12 x 53	465	236	347	236
12 x 74	654	329	385	329
14 x 73	642	297	384	297
14 x 89	783	361	446	361
14 x 117	1032	473	579	473

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

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

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

Since the abutment piles will be modeled with a fixed pile head and subjected to lateral and axial loads, bending moments and displacements, the piles should be analyzed for combined axial compression and flexure resistance per LRFD Articles 6.9.2.2 and 6.15. An L-Pile®

analysis by the project geotechnical engineer is recommended to evaluate the soil-pile interaction for combined axial and flexure, with factored axial loads, movements and pile head displacements applied. The resistance for the piles should be determined for compliance with the interaction equation. The upper portion of the pile is defined per LRFD Figure C6.15.2-1 as that portion of the pile above the point of second infection in the movement vs. pile depth curve, or at the lowest point of zero infection. 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. The resistance of the pile in the lower zone need only be checked against axial load.

7.1.2 Service and Extreme Limit States

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

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

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

Pile Section	Service and Extreme Limit States Factored Axial Pile Resistance (kips)			
	Structural Resistance* $\phi = 1.0$ $\lambda = 0$	Geotechnical Resistance $\varphi = 1.0$	Drivability Resistance $\varphi = 1.0$	Governing Resistance Based on Drivability Analyses
12 x 53	775	524	534	524
12 x 74	1090	732	593	593
14 x 73	1070	660	590	590
14 x 89	1305	803	686	686
14 x 117	1720	1052	891	891

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

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

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, it is recommended that the governing resistance used in service and extreme limit state design be the resistances shown in the last column of Table 7-3 above. It should be noted that the factored geotechnical resistance governs for the HP 12x53 pile section while the remaining pile sections are governed by the factored drivability resistance.

7.1.3 Pile Resistance and Pile Quality Control

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

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 8 to 13 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Downdrag

Settlement analyses discussed later in this report indicate that approximately 9.4 inches of settlement will occur at the site due to the placement of a maximum of 12 feet of fill in order to straighten out the roadway alignment. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag (negative skin friction) forces on piles. The magnitude of downdrag has been estimated based on the effective vertical stress and empirical β factors obtained from full scale tests. The calculated downdrag values are:

Pile Section	Unfactored Downdrag Loads (DD) (kips)
HP 12 x 53	72
HP 12 x 74	74
HP 14 x 73	85
HP 14 x 89	86
HP 14 x 117	88

Table 7-4 – Unfactored Downdrag Loads

Calculations for the pile downdrag loads are included the Appendix C- Calculations. Based on past practice, it is recommended that a load factor, γ_p=1.0, is applied to downdrag forces in cohesive and cohesionless downdrag zones.

7.3 Integral Stub Abutment Design

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

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

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

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

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 7-4 below:

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

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

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. The approach slab should be positively connected to the abutment.

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

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

7.4 Bearing Resistance

In the event that any structure foundation is founded on spread footings bearing on fill or native sand, the footings shall be proportioned to provide stability against bearing capacity failure. Application of permanent and transient loads is specified in LRFD Article 11.5.5. The stress distribution for spread footings on bedrock may be assumed to be a triangular or trapezoidal distribution over the effective base as shown in LRFD Figure 11.6.3.2-2. Bearing resistance for foundations on fill or native sand soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 4 ksf for wall footings less than 8 feet wide and 5 ksf for footings from 10 to 12 feet wide. Based on presumptive bearing resistance values a factored bearing resistance of 3 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing.

See Appendix C – Calculations, for supporting documentation.

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

7.5 Scour and Riprap

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

- Average diameter of particle at 50 percent passing, D_{50} = 0.14 mm
- Average diameter of particle at 95 percent passing, D_{95} = 12.3 mm
- Soil Classification AASHTO Soil Type A-2-4 or 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 riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

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

7.6 Settlement

In order to improve the roadway alignment, the new roadway centerline will move upstream approximately 10 feet at the east approach and downstream approximately 20 feet at the west approach. Both of the proposed abutments will be located approximately 25 feet behind the existing abutments on the new alignment. The vertical alignment will be raised approximately 2.0 feet at the east abutment and approximately 3.5 feet at the west abutment. Two large fill areas will be required behind the abutments. Approximately 12 feet of fill will be required behind Abutment No. 1 at the southeast corner and approximately 11 feet of fill will be required behind Abutment No. 2 at the northwest corner to construct the roadway on the proposed alignment.

A one dimensional consolidation test was performed on an undisturbed tube sample which indicated that the silt and clay deposit is over consolidated. The soils are highly compressible and are susceptible to consolidation if the in-situ stresses are increased above the maximum past pressure (i.e., consolidation will occur if fill is placed, or if structures are supported on clay). Evaluation of the potential settlement due to the placement of the approximately 12 feet of fill resulted in approximately 9.4 inches of settlement. The majority of this settlement is consolidation settlement within the compressible silt and clay soils underlying the site. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag forces on piles. This settlement is anticipated to occur over a long period of time (on the order of 5 to 6 years) and may require attention by a maintenance crew.

7.7 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT BDG Design Freezing Index map (MaineDOT BDG Figure 5-1) the site has a design freezing index of approximately 1100 F-degree days. A laboratory water content of 20% was used for granular soils above the water table. This correlates to a frost depth of 5.0 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis the site was assigned a design freezing index of 1123 F-degree days. A laboratory water content of 20% was used for granular soils above the water table. This results in a calculated frost depth of 5.3 feet.

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

7.8 Seismic Design Considerations

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

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

- Peak Ground Acceleration coefficient (PGA) = 0.101g
- Site Class E (soils with an average N-value less than 15 bpf or S_u less than 1.0 ksf)
- Acceleration coefficient (A_s) = 0.251
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.481g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.159g
- Seismic Zone 2 (based on S_{D1} greater than 0.15g but less than 0.30g)

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

7.9 Precast Concrete Modular Block Retaining Wall

Precast Concrete Modular Gravity (PCMG) walls may be constructed along the relocated roadway to retain the widened roadway section and minimize impacts. These walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The walls shall be designed in accordance with LRFD and Special Provision 635 which is included in Appendix D found at the end of this report.

The PCMG wall designs shall consider a live load surcharge estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 7-6 below:

Wall Height (feet)	h_{eq} (feet)	
	Distance from wall backface to edge of traffic = 0 feet	Distance from wall backface to edge of traffic ≥ 1 foot
5	5.0	2.0
10	3.5	2.0
≥20	2.0	2.0

Table 7-6 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Bearing resistance for PCMG walls founded on a leveling slab on fill or native sand soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 4 ksf for wall system bases less than 8 feet wide and 5 ksf for bases from 8.5 to 12 feet wide. The bearing resistance factor, $\phi_{b,}$ for spread footings on soil is 0.45. Based on presumptive bearing resistance values a factored bearing resistance of 3 ksf may be used to control settlement when analyzing the service limit state and for preliminary footing sizing assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

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

The designer shall apply a sliding resistance factor ϕ_T of 0.85 to the nominal sliding resistance of precast concrete wall segments founded on spread footings on clay. For footings on soil the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed one-fourth (1/4th) of the footing dimensions in either direction (LRFD Article 10.6.3.3). Sliding computations for resistance to lateral loads shall assume a maximum frictional coefficient of 0.36x(tan 20º) at the foundation soil to soil interface. Recommended values of sliding frictional coefficients are based on LRFD Article 11.11.4.2, Table 10.5.5.2.2-1 and Table 3.11.5.3-1.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635.

7.10 Construction Considerations

Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the Plans.

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

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

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

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

8.0 CLOSURE

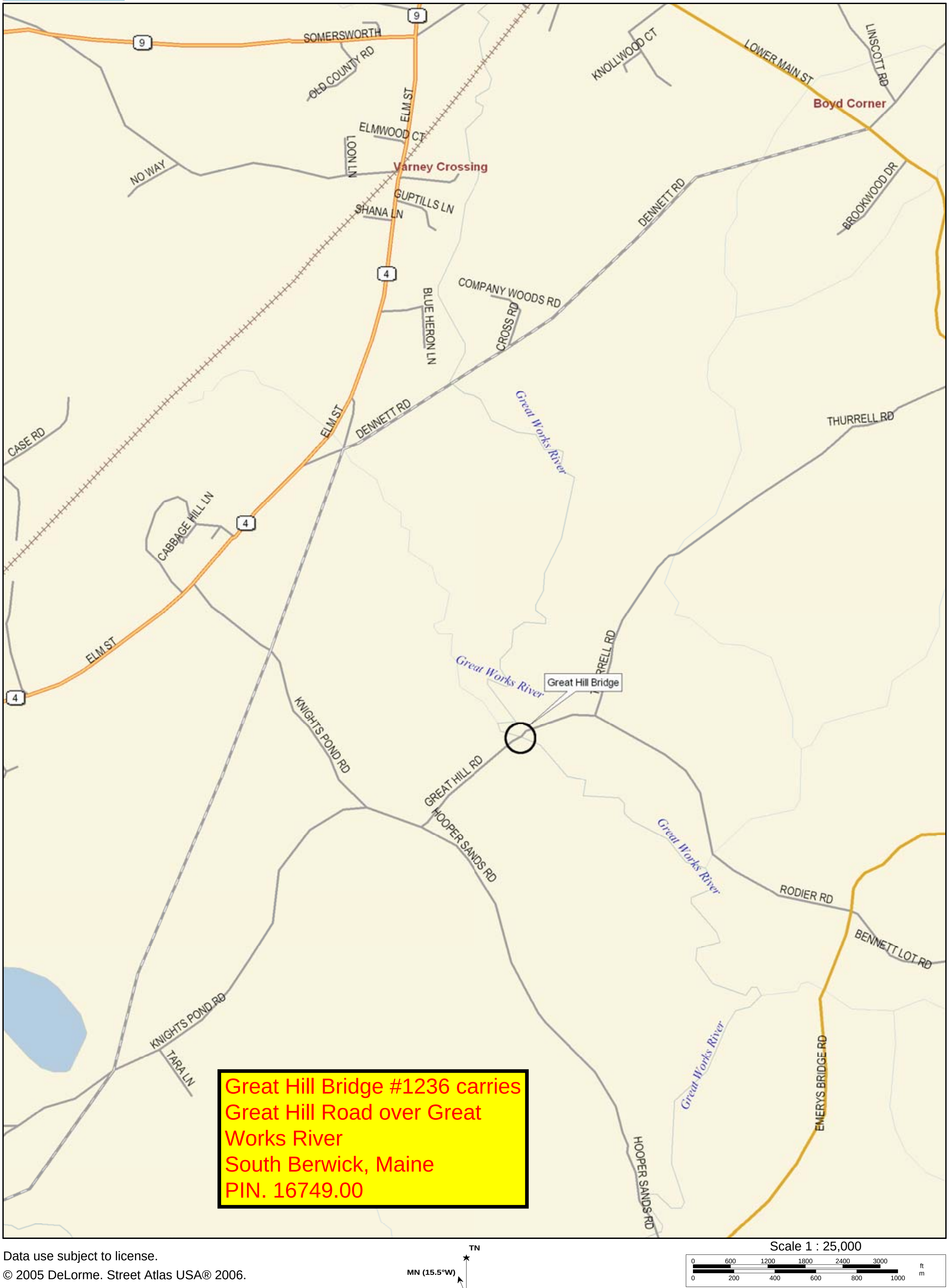
This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Great Hill Bridge in South Berwick, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is implied.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

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

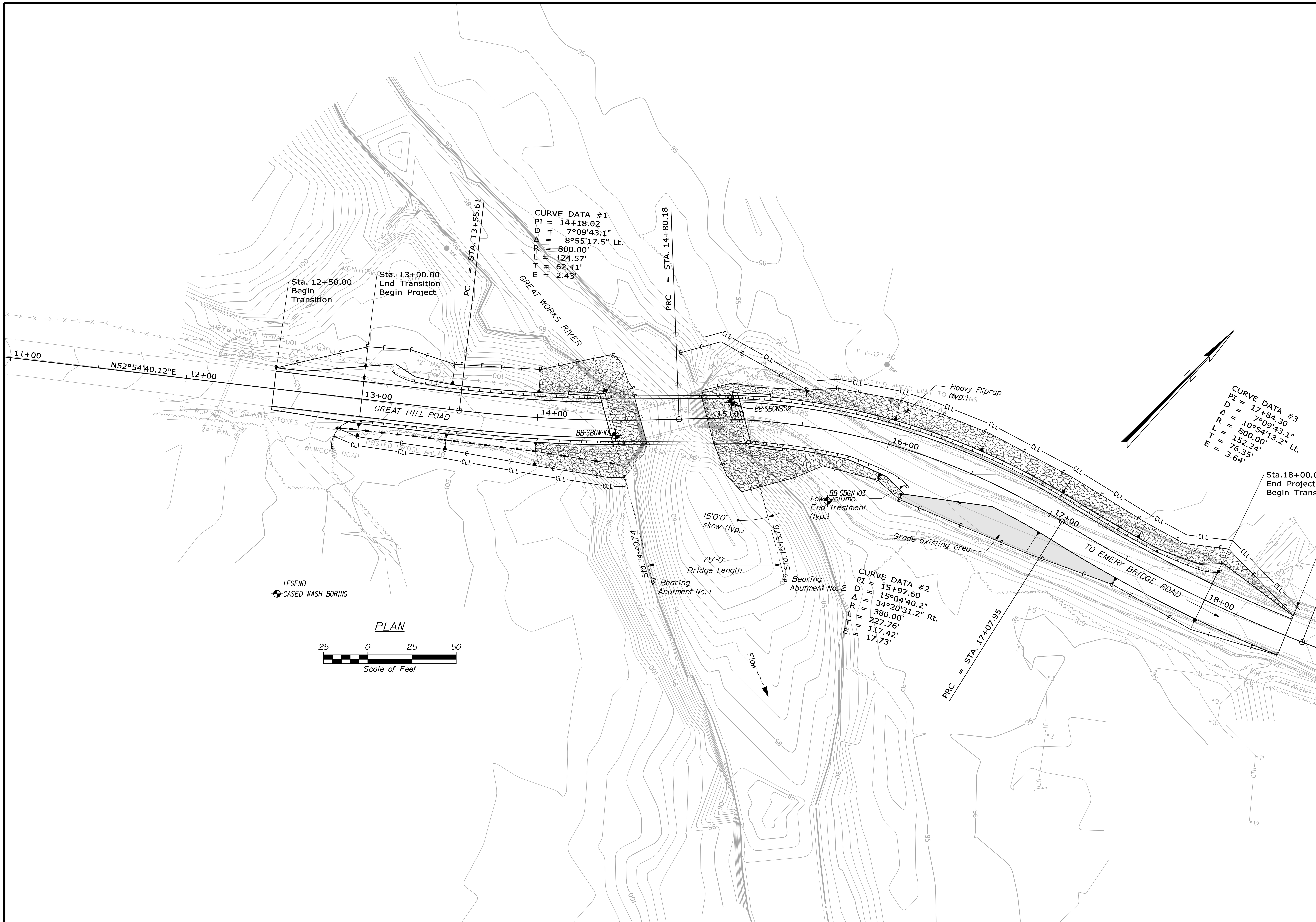
Sheets

DELORME

Street Atlas USA® 2006



Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:8/4/2010



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1674(900)X

BRIDGE NO. 1236

PIN
16749.00

BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	J. Wentworth	
DESIGN-DETAILED	K.MAGUIRE	
CHECKED-REVIEWED	T.WHITE	MAY 2010
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

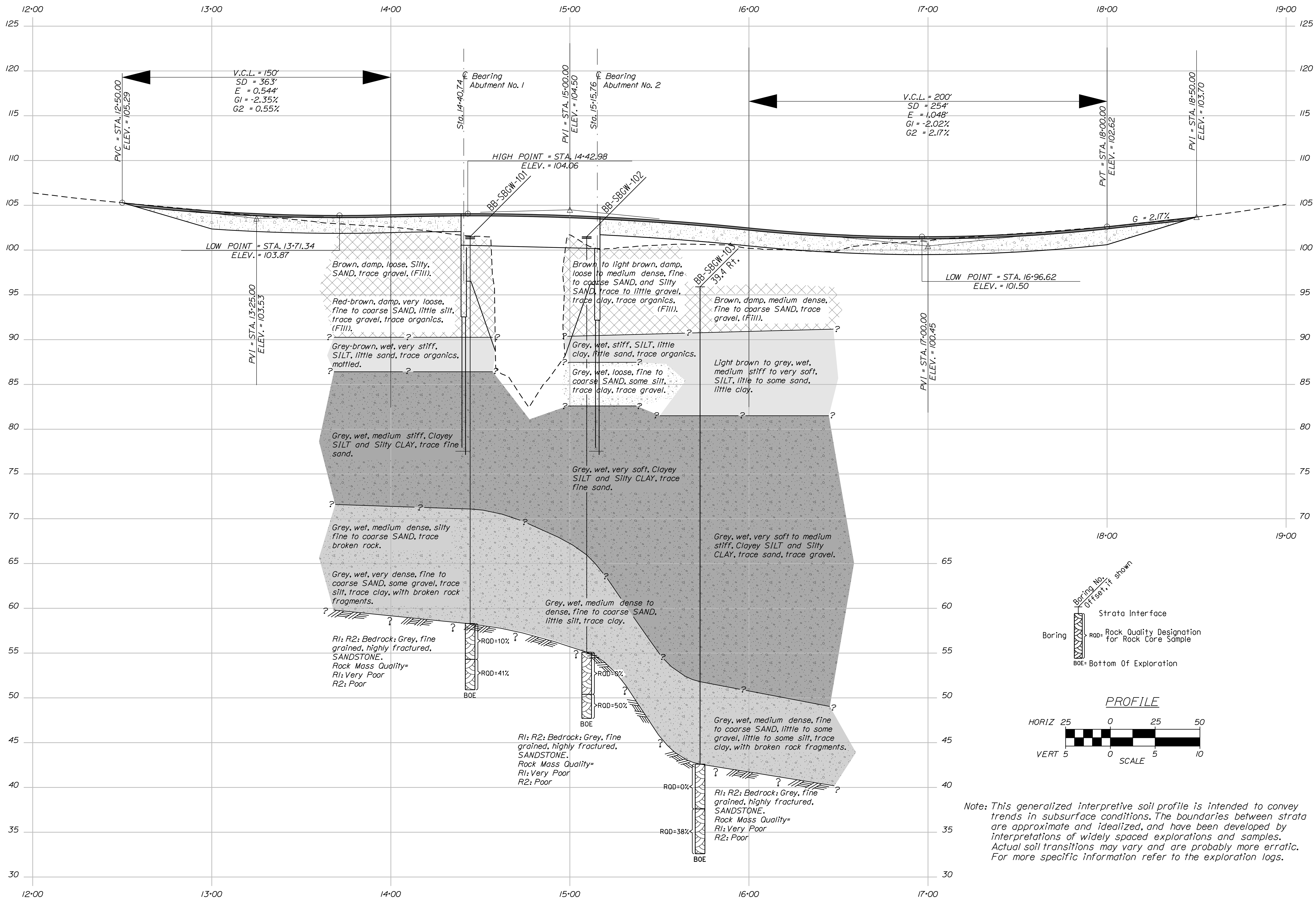
P.E. NUMBER

DATE

GREAT HILL BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK YORK COUNTY

BORING LOCATION PLAN

SHEET NUMBER
2
OF 4



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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1674(900)X

PIN 16749.00

BRIDGE NO. 1236

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. Wentworth		
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	NOV 2009
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

GREAT HILL BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK YORK COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 4

Filename: ...\00\GEOTECH\MSTA\006_ISP1.dgn Division: GEOTECH Username: terry.white Date: 8/4/2010

Date:8/4/2010 Username: terry.white Division: GEOTECH Filename: ...\msta\008_BORING LOGS1.dgn

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works
Location: South Berwick, Maine

Boring No.: BB-SBGW-101
PIN: 16749.00

Driller:	MaineDOT	Elevation (ft.)	101.5	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	14+44.3, 8.6 Rt.	Casing ID/OD:	HW	Water Level*:	5.5' 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							SSA	101.25	Pavement 0.25	
	1D	24/16	1.00 - 3.00	3/2/2/3	4	6			Brown, damp, loose, Silty SAND, trace gravel, (Fill).	G#236828 A-4, SM WC=18.7%
5	2D	24/12	5.00 - 7.00	1/1/1/1	2	3			Red-brown, damp, very loose, fine to coarse SAND, little silt, trace gravel, trace organics, (Fill).	G#236829 A-2-4, SM WC=14.3%
10	3D	24/12	10.00 - 12.00	4/11/5/6	16	22	33			G#236830 A-4, SC-SM WC=14.6%
							45	90.50	11.00 Grey-brown, very stiff, mottled, SILT, some gravel, little sand, little clay, trace organics.	
							54			
							79			
							89			
15	4D	24/24	15.00 - 17.00	2/2/3/2	5	7	53	86.50	15.00 Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#236831 A-6, CL WC=33.5% LL=34 PL=22 PI=12
							54			
	V1		17.63 - 18.00	Su=580/89 psf			34		55x110 mm raw torque readings: V1: 13.0/2.0 ft-lbs	
	V2		18.63 - 19.00	Su=589/80 psf			30		V2: 13.2/1.8 ft-lbs	
							24			
20	1U	24/24	20.00 - 22.00	Hydraulic Push			20		Grey, wet, medium stiff, Silty CLAY, trace fine sand.	G,C#236832 A-6, CL WC=44.9% LL=37 PL=23 PI=14
							20			
	V3		22.63 - 23.00	Su=402/89 psf			21		55x110 mm raw torque readings: V3: 9.0/2.0 ft-lbs	
	V4		23.60 - 23.97	Su=446/89 psf			21		V4: 10.0/2.0 ft-lbs	
					--		20			
25	5D V5	24/24	25.00 - 27.00 25.60 - 25.97	Through Vane Su=946/125 psf	---		18		Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm raw torque readings: V5: 21.2/2.8 ft-lbs	G#236833 A-6, CL WC=43.9% LL=35 PL=22 PI=13
	V6		26.60 - 26.97	Su=1062/112 psf			19		V6: 23.8/2.5 ft-lbs	
							20			
							19			
							36	71.80	29.70	
30	6D	24/4	30.00 - 32.00	9/12/9/9	21	29	29		Grey, wet, medium dense, silty, fine to coarse SAND, trace broken rock.	
							32			
							47			
							42			
							40			
35	7D	24/1	35.00 - 37.00	18/10/9/9	19	27	20		Similar to above.	
							22			
							48			
							74			
							a95		a95 blows for 0.7', 6" cobble at 39.7' bgs.	
40	8D	24/15	40.20 - 42.20	4/33/29/24	62	87	89		Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, trace clay, with broken rock fragments.	G#236834 A-1-b, SW-SC WC=9.7%
							96			
							221		b177 blows for 0.2'.	
	R1	48/41	43.20 - 47.20	RQD = 10%			b177 NQ-2	58.30	43.20 Top of Bedrock at Elev. 58.3'. Bedrock: Grey, fine-grained, highly fractured, SANDSTONE. Rock Mass Quality: Very Poor R1:Core Times (min:sec) 43.2-44.2' (4:39) 44.2-45.2' (3:29) 45.2-46.2' (4:12) 46.2-47.2' (8:47) 85% Recovery Core Blocked	
45										
	R2	40.8/40.8	47.20 - 50.60	RQD = 41%					Bedrock: Grey, fine-grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor R2:Core Times (min:sec) 47.2-48.2' (4:19) 48.2-49.2' (4:30) 49.2-50.2' (4:21) 50.2-50.6' (7:00) 100% Recovery Core Blocked	
50								50.90	50.60 Bottom of Exploration at 50.60 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:

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

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

Page 1 of 1

Boring No.: BB-SBGW-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works
Location: South Berwick, Maine

Boring No.: BB-SBGW-102
PIN: 16749.00

Driller:	Northern Test Boring	Elevation (ft.)	101.5	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Mike	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50 Trailer	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:45	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+09.4, 9.4 Lt.	Casing ID/OD:	HW	Water Level*:	11.8' bgs.
Hammer Efficiency Factor: 0.68		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	101.30	Pavement 0.20	
	1D	24/10	1.00 - 3.00	5/11/7/9	18	20			Brown, damp, medium dense, gravelly, fine to coarse SAND, little silt, (Fill).	
								98.50	3.00	
5	2D	24/17	5.00 - 7.00	1/2/3/4	5	6			Light-brown, damp, loose, Silty, fine to coarse SAND, trace gravel, (Fill).	G#236835 A-4, SM WC=22.3%
								93.00	8.50	
10	3D/A	24/19	10.00 - 12.00	6/6/5/3	11	12			(3D) 10.0-11.0' bgs. Light-brown, moist, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, trace organics, (Fill).	G#236836 A-2-4, SC-SM WC=21.8%
								90.50	11.00 (3D/A) 11.0-12.0' bgs. Grey, wet, stiff, SILT, little clay, little sand, trace organics.	G#236837 A-4, CL-ML WC=33.1%
								88.00	13.50	
15	4D	24/16	15.00 - 17.00	1/1/4/6	5	6	29		Grey, wet, loose, silty, fine to coarse SAND, some silt, trace clay, trace gravel.	G#236838 A-2-4, SC-SM WC=30.3%
							42			
							40			
							36	83.00	18.50	
							35			
20	5D	24/22	20.00 - 22.00	WOH/WOH/1/1	1	1	35		Grey, wet, very soft, Clayey SILT, trace sand, plastic.	G#236839 A-6, CL WC=40.5% LL=36 PL=23 PI=13
							39			
							38			
							38			
							36			
25	MU/6D	24/24	25.00 - 27.00	WOR/WOR/WOR/WOR	---		38		Failed Piston Sample attempt. Grey, wet, very soft, Silty CLAY, trace fine sand. Roller Coned ahead to 27.0' bgs, took vane.	G#236840 A-6, CL WC=45.0% LL=37 PL=24 PI=13
							29			
	V1		27.00 - 27.37	Su=357/89 psf			25		55x110 mm raw torque readings: V1: 8.0/2.0 ft-lbs	
	V2		28.00 - 28.37	Su=491/67 psf			32		V2: 11.0/1.5 ft-lbs	
							33			
30	1U	24/20	30.00 - 32.00	Piston Sampler	---		22		Grey, wet, very soft, Clayey SILT, trace fine sand.	G,C#236841 A-6, CL WC=41.8% LL=37 PL=23 PI=14
							30			
	V3		32.00 - 32.37	Su=670/89 psf			30		55x110 mm raw torque readings: V3: 15.0/2.0 ft-lbs	
	V4		33.00 - 33.37	Su=804/134 psf			33		V4: 18.0/3.0 ft-lbs	
							36			
35	MV/MD	2.4/0	35.00 - 35.20	40(2.4")			44	66.50 66.30	35.00 Failed 55x110 mm Vane attempt, would not push. Failed Spoon attempt, no recovery. Grey, wet, dense, fine to coarse SAND, little silt. 35.20 Boulder from 35.2-37.3' bgs.	
							69			
							118	64.20	37.30	
							88			
							76			
40	7D	24/1	40.00 - 42.00	21/10/7/7	17	19	27		Grey, wet, medium dense, fine to coarse SAND, little silt. Roller Coned ahead to 45.0' bgs.	
							19			
							19			
							28			
							76			
45	8D	16.8/14	45.00 - 46.40	24/14/50(4.8")	---		81		Grey, wet, dense, fine to coarse SAND, some gravel, little silt, trace clay. a100 blows for 0.4'.	G#236842 A-2-4, SC-SM WC=9.7%
	R1	56.4/19	46.40 - 51.10	RQD = 0%			a100 NQ-2	55.10	46.40 Top of Bedrock at Elev. 55.1'. Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Very poor. R1:Core Times (min:sec) 46.4-47.4' (2:25) 47.4-48.4' (2:40) 48.4-49.4' (0:30) 49.4-50.4' (1:25) 50.4-51.1' (4:54) 34% Recovery Core Blocked	
50										
	R2	32.4/25	51.10 - 53.80	RQD = 50%					Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor. R2:Core Times (min:sec) 51.1-52.1' (3:38) 52.1-53.1' (3:26) 53.1-53.8' (4:50) 78% Recovery Core Blocked	
								47.70	53.80 Bottom of Exploration at 53.80 feet below ground surface.	
55										
60										
65										
70										
75										

Remarks:

Auto Hammer #149

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-SBGW-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works
Location: South Berwick, Maine

Boring No.: BB-SBGW-103
PIN: 16749.00

Driller:	MaineDOT	Elevation (ft.)	95.9	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Wright	Datum:	NAVD 88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	11/3/09; 07:00-14:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	15+72.7, 39.4 Rt.	Casing ID/OD:	HW	Water Level*:	14.0' 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	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	95.40	6" layer of Riprap. 0.50 COBBLES and GRAVEL, (Fill).	
5	1D	24/19	5.00 - 7.00	WOH/1/3/3	4	6		90.90	5.00 Light brown, wet, medium stiff, SILT, some sand, little clay.	G#236843 A-4, CL-ML WC=31.6%
10	2D	24/22	10.00 - 12.00	WOR/WOH/WOH/WOH	---		aHP		aHP = Hydraulic Push Grey, saturated, very soft, SILT, little sand, little clay.	G#236844 A-4, CL-ML WC=31.9%
							aHP			
							aHP			
							aHP			
							aHP	81.90	14.00	
15	3D	24/20	15.00 - 17.00	WOH/WOH/WOH/WOH	---		8		Grey, wet, very soft, Clayey SILT, trace fine sand.	G#236845 A-6, CL WC=37.0% LL=34 PL=23 PI=11
							8			
							8			
							10			
							15			
20	4D V1	24/22	20.00 - 22.00 20.63 - 21.00	Hydraulic Push Su=625/112 psf	---		aHP		Grey, wet, medium stiff, Silty CLAY, trace fine sand, highly plastic. 55x110 mm vane raw torque readings: V1: 14.0/2.5 ft-lbs	G#236846 A-6, CL WC=39.5% LL=34 PL=23 PI=11
	V2		21.63 - 22.00	Su=580/89 psf					V2: 13.0/2.0 ft-lbs	
25	5D V3	24/24	25.00 - 27.00 25.63 - 26.00	Hydraulic Push Su=513/67 psf					Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand, trace gravel. 55x110 mm vane raw torque readings: V3: 11.5/1.5 ft-lbs 1/2" sand layer at 26.0' bgs.	G#236847 A-6, CL WC=38.7% LL=37 PL=24 PI=13
	V4		26.63 - 27.00	Su=446/45 psf					V4: 10.0/1.0 ft-lbs	
30	6D V5	24/20	30.00 - 32.00 30.63 - 31.00	Hydraulic Push Su=268/67 psf					Grey, wet, soft, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V5: 6.0/1.5 ft-lbs	G#236848 A-6, CL WC=42.2% LL=37 PL=23 PI=14
	MV6		31.00 - 31.37						Failed 55x110 mm vane attempt, could only push 0.37'. 1" sand layer at 31.4' bgs.	
35	MV7 7D	24/24	35.00 - 35.37 35.00 - 37.00	Hydraulic Push					Failed 55x110 mm vane attempt, could only push 0.5'. 1/2" sand layer. Grey, wet, soft, Silty CLAY, trace fine sand.	G#236849 A-6, CL WC=42.7% LL=37 PL=23 PI=14
40	8D V8	24/24	40.00 - 42.00 40.63 - 41.00	WOR/WOR/WOR/WOR Su=647/134 psf	---				Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V8: 14.5/3.0 ft-lbs	G#236850 A-6, CL WC=43.0% LL=37 PL=24 PI=13
	V9		41.63 - 42.00	Su=714/89 psf					V9: 16.0-2.0 ft-lbs	
							44	51.90	44.00	
45	9D	24/14	45.00 - 47.00	8/9/10/7	19	27	50		Grey, wet, medium dense, fine to coarse SAND, some gravel, rock fragments, little silt, trace clay.	G#236851 A-1-b, SC-SM WC=9.6%
							55			
							49			
							47			
							38			
50	10D	24/20	50.00 - 52.00	4/10/6/6	16	22	26		Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel.	G#236852 A-2-4, SM WC=13.8%
							36			
							56		b98 blows for 0.3'.	
	R1	60/60	53.30 - 58.30	RQD = 0%			b98 NQ-2	42.60	53.30 Top of Bedrock at Elev. 42.6'. Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Very Poor. R1:Core Times (min:sec) 53.3-54.3' (4:52) 54.3-55.3' (4:51) 55.3-56.3' (4:02) 56.3-57.3' (3:35) 57.3-58.3' (4:05) 100% Recovery	
55										
	R2	60/60	58.30 - 63.30	RQD = 38%					Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor. R2:Core Times (min:sec) 58.3-59.3' (4:23) 59.3-60.3' (3:27) 60.3-61.3' (4:55) 61.3-62.3' (4:07) 62.3-63.3' (3:00) 100% Recovery	
60										
								32.60	63.30 Bottom of Exploration at 63.30 feet below ground surface.	
65										
70										
75										

Remarks:

700# down pressure on Core Barrel.

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-SBGW-103

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-1674(900)X

BRIDGE NO. 1236

PIN 16749.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	J. Wentworth		
DESIGN-DETAILED	K.MAGUIRE	T.WHITE	NOV 2009
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	-----	-----	-----
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE

P.E. NUMBER

DATE

GREAT HILL BRIDGE
GREAT WORKS RIVER
SOUTH BERWICK YORK COUNTY

BORING LOGS

SHEET NUMBER

4

OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

PIN	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: Great Hill Bridge #1236 carries Great Hill Road over Great Works River Location: South Berwick, Maine	Boring No.: BB-SBGW-101 PIN: 16749.00

Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+44.3, 8.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	5.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	101.25		Pavement 0.25	
	1D	24/16	1.00 - 3.00	3/2/2/3	4	6				Brown, damp, loose, Silty SAND, trace gravel, (Fill).	G#236828 A-4, SM WC=18.7%
5	2D	24/12	5.00 - 7.00	1/1/1/1	2	3				Red-brown, damp, very loose, fine to coarse SAND, little silt, trace gravel, trace organics, (Fill).	G#236829 A-2-4, SM WC=14.3%
10	3D	24/12	10.00 - 12.00	4/11/5/6	16	22	33				G#236830 A-4, SC-SM WC=14.6%
							45	90.50		11.00 Grey-brown, very stiff, mottled, SILT, some gravel, little sand, little clay, trace organics.	
							54				
							79				
							89				
15	4D	24/24	15.00 - 17.00	2/2/3/2	5	7	53	86.50		15.00 Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#236831 A-6, CL WC=33.5% LL=34 PL=22 PI=12
							54				
	V1		17.63 - 18.00	Su=580/89 psf			34			55x110 mm raw torque readings: V1: 13.0/2.0 ft-lbs	
	V2		18.63 - 19.00	Su=589/80 psf			30			V2: 13.2/1.8 ft-lbs	
							24				
20	1U	24/24	20.00 - 22.00	Hydraulic Push			20			Grey, wet, medium stiff, Silty CLAY, trace fine sand.	G,C#236832 A-6, CL WC=44.9% LL=37 PL=23 PI=14
							20				
	V3		22.63 - 23.00	Su=402/89 psf			21			55x110 mm raw torque readings: V3: 9.0/2.0 ft-lbs	
	V4		23.60 - 23.97	Su=446/89 psf			21			V4: 10.0/2.0 ft-lbs	
					--		20				
25											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-101
PIN: 16749.00

Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+44.3, 8.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	5.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D ~~V5~~	24/24	25.00 - 27.00 ~~25.60 - 25.97~~	Through Vane ~~Su=946/125 psf~~	---		18			Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm raw torque readings: V5: 21.2/2.8 ft-lbs V6: 23.8/2.5 ft-lbs	G#236833 A-6, CL WC=43.9% LL=35 PL=22 PI=13
	V6		26.60 - 26.97	Su=1062/112 psf			19				
							20				
							19				
							36	71.80		29.70	
30	6D	24/4	30.00 - 32.00	9/12/9/9	21	29	29			Grey, wet, medium dense, silty, fine to coarse SAND, trace broken rock.	
							32				
							47				
							42				
							40				
35	7D	24/1	35.00 - 37.00	18/10/9/9	19	27	20			Similar to above.	
							22				
							48				
							74				
							a95			a95 blows for 0.7', 6" cobble at 39.7' bgs.	
40	8D	24/15	40.20 - 42.20	4/33/29/24	62	87	89			Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, trace clay, with broken rock fragments.	G#236834 A-1-b, SW-SC WC=9.7%
							96				
							221				
	R1	48/41	43.20 - 47.20	RQD = 10%			b177 NQ-2	58.30		b177 blows for 0.2'. 43.20 Top of Bedrock at Elev. 58.3'. Bedrock: Grey, fine-grained, highly fractured, SANDSTONE. Rock Mass Quality: Very Poor R1:Core Times (min:sec) 43.2-44.2' (4:39) 44.2-45.2' (3:29) 45.2-46.2' (4:12) 46.2-47.2' (8:47) 85% Recovery Core Blocked	
45											
	R2	40.8/40.8	47.20 - 50.60	RQD = 41%						Bedrock: Grey, fine-grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor R2:Core Times (min:sec) 47.2-48.2' (4:19)	
50											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-101
PIN: 16749.00

Driller:	MaineDOT	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder/K. Maguire	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	14+44.3, 8.6 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	5.5' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50								50.90		48.2-49.2' (4:30) 49.2-50.2' (4:21) 50.2-50.6' (7:00) 100% Recovery Core Blocked	
										50.60 **Bottom of Exploration at 50.60 feet below ground surface.**	
55											
60											
65											
70											
75											

Remarks:

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-102
PIN: 16749.00

Driller:	Northern Test Boring	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+09.4, 9.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.8' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

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.)
0							SSA	101.30
	1D	24/10	1.00 - 3.00	5/11/7/9	18	20		
								98.50
5	2D	24/17	5.00 - 7.00	1/2/3/4	5	6		
								93.00
10	3D/A	24/19	10.00 - 12.00	6/6/5/3	11	12		90.50
								88.00
15	4D	24/16	15.00 - 17.00	1/1/4/6	5	6	29	
							42	
							40	
							36	83.00
							35	
20	5D	24/22	20.00 - 22.00	WOH/WOH/1/1	1	1	35	
							39	
							38	
							38	
							36	
25								

Elevation (ft.)	Visual Description and Remarks	Depth (ft.)	Laboratory Testing Results/ AASHTO and Unified Class.
101.30	Pavement	0.20	
	Brown, damp, medium dense, gravelly, fine to coarse SAND, little silt, (Fill).		
98.50		3.00	
	Light-brown, damp, loose, Silty, fine to coarse SAND, trace gravel, (Fill).		G#236835 A-4, SM WC=22.3%
93.00		8.50	
	(3D) 10.0-11.0' bgs. Light-brown, moist, medium dense, fine to coarse SAND, some silt, little gravel, trace clay, trace organics, (Fill).		G#236836 A-2-4, SC-SM WC=21.8%
90.50		11.00	
	(3D/A) 11.0-12.0' bgs. Grey, wet, stiff, SILT, little clay, little sand, trace organics.		G#236837 A-4, CL-ML WC=33.1%
88.00		13.50	
	Grey, wet, loose, silty, fine to coarse SAND, some silt, trace clay, trace gravel.		G#236838 A-2-4, SC-SM WC=30.3%
83.00		18.50	
	Grey, wet, very soft, Clayey SILT, trace sand, plastic.		G#236839 A-6, CL WC=40.5% LL=36 PL=23 PI=13

Remarks:

Auto Hammer #149

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: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-102
PIN: 16749.00

Driller:	Northern Test Boring	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+09.4, 9.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.8' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	MU/6D	24/24	25.00 - 27.00	WOR/WOR/WOR/ WOR	---		38			Failed Piston Sample attempt. Grey, wet, very soft, Silty CLAY, trace fine sand. Roller Coned ahead to 27.0' bgs, took vane.	G#236840 A-6, CL WC=45.0% LL=37 PL=24 PI=13
							29				
	V1		27.00 - 27.37	Su=357/89 psf			25			55x110 mm raw torque readings: V1: 8.0/2.0 ft-lbs	
	V2		28.00 - 28.37	Su=491/67 psf			32			V2: 11.0/1.5 ft-lbs	
							33				
30	1U	24/20	30.00 - 32.00	Piston Sampler	---		22			Grey, wet, very soft, Clayey SILT, trace fine sand.	G,C#236841 A-6, CL WC=41.8% LL=37 PL=23 PI=14
							30				
	V3		32.00 - 32.37	Su=670/89 psf			30			55x110 mm raw torque readings: V3: 15.0/2.0 ft-lbs	
	V4		33.00 - 33.37	Su=804/134 psf			33			V4: 18.0/3.0 ft-lbs	
							36	66.50		35.00	
35	MV/MD	2.4/0	35.00 - 35.20	40(2.4")			44	66.30		Failed 55x110 mm Vane attempt, would not push. Failed Spoon attempt, no recovery. Grey, wet, dense, fine to coarse SAND, little silt. 35.20	
							69			Boulder from 35.2-37.3' bgs.	
							118	64.20		37.30	
							88				
							76				
40	7D	24/1	40.00 - 42.00	21/10/7/7	17	19	27			Grey, wet, medium dense, fine to coarse SAND, little silt. Roller Coned ahead to 45.0' bgs.	
							19				
							19				
							28				
							76				
45	8D	16.8/14	45.00 - 46.40	24/14/50(4.8")	---		81			Grey, wet, dense, fine to coarse SAND, some gravel, little silt, trace clay.	G#236842 A-2-4, SC-SM WC=9.7%
	R1	56.4/19	46.40 - 51.10	RQD = 0%			a100 NQ-2	55.10		a100 blows for 0.4'. 46.40	
										Top of Bedrock at Elev. 55.1'. Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Very poor. R1:Core Times (min:sec) 46.4-47.4' (2:25) 47.4-48.4' (2:40) 48.4-49.4' (0:30)	
50											

Remarks:

Auto Hammer #149

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:	Great Hill Bridge #1236 carries Great Hill Road over Great Works River	**Location:**	South Berwick, Maine	**Boring No.:**	BB-SBGW-102
				PIN:	16749.00

Driller:	Northern Test Boring	**Elevation (ft.)**	101.5	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Mike	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Trailer	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	12/2/09; 07:00-15:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+09.4, 9.4 Lt.	**Casing ID/OD:**	HW	**Water Level*:**	11.8' bgs.
Hammer Efficiency Factor:	0.68	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50										49.4-50.4' (1:25) 50.4-51.1' (4:54) 34% Recovery Core Blocked	
	R2	32.4/25	51.10 - 53.80	RQD = 50%						Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor. R2:Core Times (min:sec) 51.1-52.1' (3:38) 52.1-53.1' (3:26) 53.1-53.8' (4:50) 78% Recovery Core Blocked	
								47.70		53.80	
55										**Bottom of Exploration at 53.80 feet below ground surface.**	
60											
65											
70											
75											

Remarks:

Auto Hammer #149

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: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-103
PIN: 16749.00

Driller:	MaineDOT	**Elevation (ft.)**	95.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Wright	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/3/09; 07:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+72.7, 39.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	14.0' 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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	95.40		6" layer of Riprap. 0.50 COBBLES and GRAVEL, (Fill).	
5	1D	24/19	5.00 - 7.00	WOH/1/3/3	4	6		90.90		5.00 Light brown, wet, medium stiff, SILT, some sand, little clay.	G#236843 A-4, CL-ML WC=31.6%
10	2D	24/22	10.00 - 12.00	WOR/WOH/WOH/ WOH	---		aHP			aHP = Hydraulic Push Grey, saturated, very soft, SILT, little sand, little clay.	G#236844 A-4, CL-ML WC=31.9%
							aHP				
							aHP				
							aHP				
							aHP	81.90		14.00	
15	3D	24/20	15.00 - 17.00	WOH/WOH/WOH/ WOH	---		8			Grey, wet, very soft, Clayey SILT, trace fine sand.	G#236845 A-6, CL WC=37.0% LL=34 PL=23 PI=11
							8				
							8				
							10				
							15				
20	4D	24/22	20.00 - 22.00	Hydraulic Push	---		aHP			Grey, wet, medium stiff, Silty CLAY, trace fine sand, highly plastic.	G#236846 A-6, CL WC=39.5% LL=34 PL=23 PI=11
	V1		20.63 - 21.00	Su=625/112 psf						55x110 mm vane raw torque readings: V1: 14.0/2.5 ft-lbs	
	V2		21.63 - 22.00	Su=580/89 psf						V2: 13.0/2.0 ft-lbs	
25											

Remarks:

700# down pressure on Core Barrel.

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-SBGW-103

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Great Hill Bridge #1236 carries Great Hill Road over Great Works River
Location: South Berwick, Maine

Boring No.: BB-SBGW-103
PIN: 16749.00

Driller:	MaineDOT	**Elevation (ft.)**	95.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Wright	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/3/09; 07:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+72.7, 39.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	14.0' 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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/24	25.00 - 27.00	Hydraulic Push						Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand, trace gravel. 55x110 mm vane raw torque readings: V3: 11.5/1.5 ft-lbs 1/2" sand layer at 26.0' bgs. V4: 10.0/1.0 ft-lbs	G#236847 A-6, CL WC=38.7% LL=37 PL=24 PI=13
	V3		25.63 - 26.00	Su=513/67 psf							
	V4		26.63 - 27.00	Su=446/45 psf							
30	6D	24/20	30.00 - 32.00	Hydraulic Push						Grey, wet, soft, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V5: 6.0/1.5 ft-lbs Failed 55x110 mm vane attempt, could only push 0.37'. 1" sand layer at 31.4' bgs.	G#236848 A-6, CL WC=42.2% LL=37 PL=23 PI=14
	V5		30.63 - 31.00	Su=268/67 psf							
	MV6		31.00 - 31.37								
35	MV7	24/24	35.00 - 35.37	Hydraulic Push						Failed 55x110 mm vane attempt, could only push 0.5'. 1/2" sand layer. Grey, wet, soft, Silty CLAY, trace fine sand.	G#236849 A-6, CL WC=42.7% LL=37 PL=23 PI=14
	7D		35.00 - 37.00								
40	8D	24/24	40.00 - 42.00	WOR/WOR/WOR/ WOR	---					Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V8: 14.5/3.0 ft-lbs V9: 16.0-2.0 ft-lbs	G#236850 A-6, CL WC=43.0% LL=37 PL=24 PI=13
	V8		40.63 - 41.00	Su=647/134 psf							
	V9		41.63 - 42.00	Su=714/89 psf							
44							44	51.90		44.00	
45	9D	24/14	45.00 - 47.00	8/9/10/7	19	27	50			Grey, wet, medium dense, fine to coarse SAND, some gravel, rock fragments, little silt, trace clay.	G#236851 A-1-b, SC-SM WC=9.6%
							55				
							49				
							47				
							38				
50											

Remarks:

700# down pressure on Core Barrel.

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:	Great Hill Bridge #1236 carries Great Hill Road over Great Works River	**Boring No.:**	BB-SBGW-103		
Location:	South Berwick, Maine	**PIN:**	16749.00		
Driller:	MaineDOT	**Elevation (ft.)**	95.9	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Wright	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/3/09; 07:00-14:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	15+72.7, 39.4 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	14.0' 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	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	10D	24/20	50.00 - 52.00	4/10/6/6	16	22	26			Grey, wet, medium dense, fine to coarse SAND, some silt, little gravel.	G#236852 A-2-4, SM WC=13.8%
							36				
							56				
	R1	60/60	53.30 - 58.30	RQD = 0%			b98 NQ-2	42.60		b98 blows for 0.3'. 53.30	
										Top of Bedrock at Elev. 42.6'. Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Very Poor. R1:Core Times (min:sec) 53.3-54.3' (4:52) 54.3-55.3' (4:51) 55.3-56.3' (4:02) 56.3-57.3' (3:35) 57.3-58.3' (4:05) 100% Recovery	
55											
	R2	60/60	58.30 - 63.30	RQD = 38%						Bedrock: Grey, fine grained, highly fractured, SANDSTONE. Rock Mass Quality: Poor. R2:Core Times (min:sec) 58.3-59.3' (4:23) 59.3-60.3' (3;27) 60.3-61.3' (4:55) 61.3-62.3' (4:07) 62.3-63.3' (3:00) 100% Recovery	
60											
								32.60		63.30 **Bottom of Exploration at 63.30 feet below ground surface.**	
65											
70											
75											

Remarks:

700# down pressure on Core Barrel.

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

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

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): South Berwick   Project Number: 16749.00

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-SBGW-101, 1D	14+44.3	8.6 Rt.	1.0-3.0	236828	1	18.7			SM	A-4	III
BB-SBGW-101, 2D	14+44.3	8.6 Rt.	5.0-7.0	236829	1	14.3			SM	A-2-4	II
BB-SBGW-101, 3D	14+44.3	8.6 Rt.	10.0-12.0	236830	1	14.6			SC-SM	A-4	IV
BB-SBGW-101, 4D	14+44.3	8.6 Rt.	15.0-17.0	236831	1	33.5	34	12	CL	A-6	III
BB-SBGW-101, 1U	14+44.3	8.6 Rt.	20.0-22.0	236832	2	44.9	37	14	CL	A-6	III
BB-SBGW-101, 5D	14+44.3	8.6 Rt.	25.0-27.0	236833	2	43.9	35	13	CL	A-6	III
BB-SBGW-101, 8D	14+44.3	8.6 Rt.	40.2-42.2	236834	2	9.7			SW-SC	A-1-b	II
BB-SBGW-102, 2D	15+09.4	9.4 Lt.	5.0-7.0	236835	3	22.3			SM	A-4	III
BB-SBGW-102, 3D	15+09.4	9.4 Lt.	10.0-11.0	236836	3	21.8			SC-SM	A-2-4	II
BB-SBGW-102, 3D/A	15+09.4	9.4 Lt.	11.0-12.0	236837	3	33.1			CL-ML	A-4	III
BB-SBGW-102, 4D	15+09.4	9.4 Lt.	15.0-17.0	236838	3	30.3			SC-SM	A-2-4	III
BB-SBGW-102, 5D	15+09.4	9.4 Lt.	20.0-22.0	236839	4	40.5	36	13	CL	A-6	III
BB-SBGW-102, 6D	15+09.4	9.4 Lt.	25.0-27.0	236840	4	45.0	37	13	CL	A-6	III
BB-SBGW-102, 1U	15+09.4	9.4 Lt.	30.0-32.0	236841	4	41.8	37	14	CL	A-6	III
BB-SBGW-102, 8D	15+09.4	9.4 Lt.	45.0-46.4	236842	4	9.7			SC-SM	A-2-4	II
BB-SBGW-103, 1D	15+72.7	39.4 Rt.	5.0-7.0	236843	5	31.6			CL-ML	A-4	III
BB-SBGW-103, 2D	15+72.7	39.4 Rt.	10.0-12.0	236844	5	31.9			CL-ML	A-4	III
BB-SBGW-103, 3D	15+72.7	39.4 Rt.	15.0-17.0	236845	5	37.0	34	11	CL	A-6	IV
BB-SBGW-103, 4D	15+72.7	39.4 Rt.	20.0-22.0	236846	5	39.5	34	11	CL	A-6	IV
BB-SBGW-103, 5D	15+72.7	39.4 Rt.	25.0-27.0	236847	5	38.7	37	13	CL	A-6	III
BB-SBGW-103, 6D	15+72.7	39.4 Rt.	30.0-32.0	236848	6	42.2	37	14	CL	A-6	III
BB-SBGW-103, 7D	15+72.7	39.4 Rt.	35.0-37.0	236849	6	42.7	37	14	CL	A-6	III
BB-SBGW-103, 8D	15+72.7	39.4 Rt.	40.0-42.0	236850	6	43.0	37	13	CL	A-6	III
BB-SBGW-103, 9D	15+72.7	39.4 Rt.	45.0-47.0	236851	6	9.6			SC-SM	A-1-b	II
BB-SBGW-103, 10D	15+72.7	39.4 Rt.	50.0-52.0	236852	6	13.8			SM	A-2-4	II

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

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

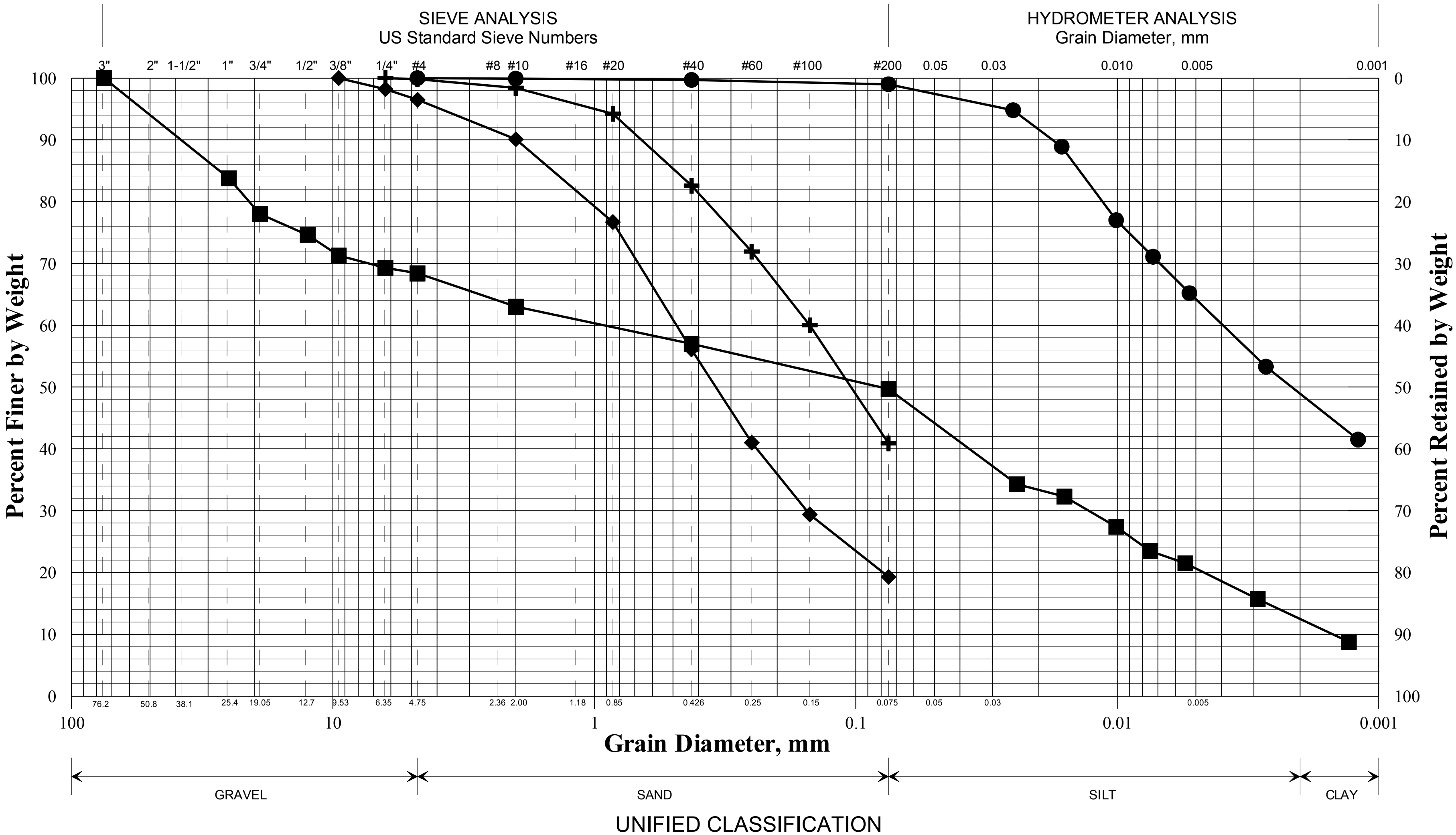
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE

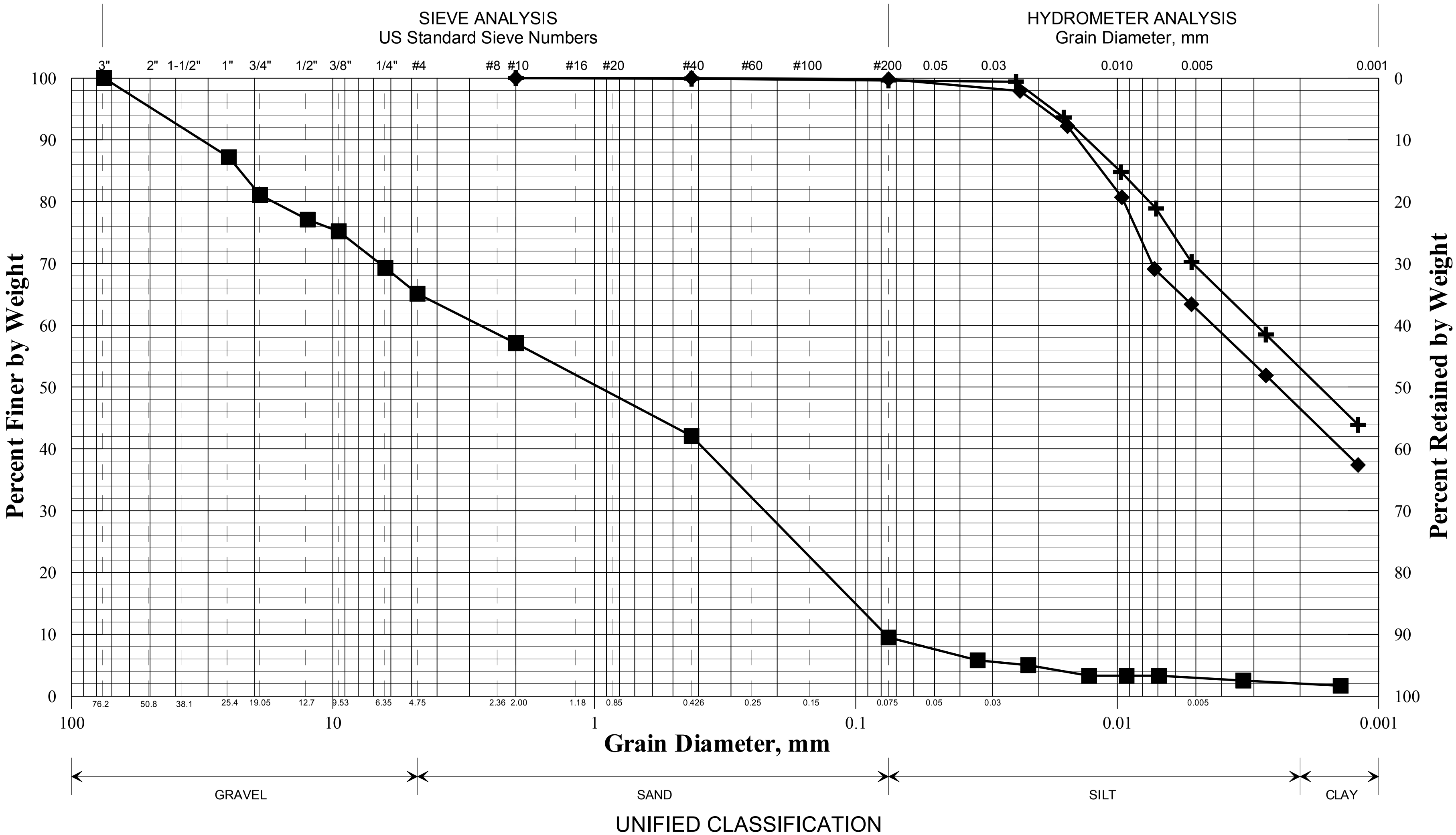


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-101/1D	14+44.3	8.6 RT	1.0-3.0	Silty SAND, trace gravel.	18.7			
◆	BB-SBGW-101/2D	14+44.3	8.6 RT	5.0-7.0	SAND, little silt, trace gravel.	14.3			
■	BB-SBGW-101/3D	14+44.3	8.6 RT	10.0-12.0	SILT, some gravel, little sand, little clay.	14.6			
●	BB-SBGW-101/4D	14+44.3	8.6 RT	15.0-17.0	Clayey SILT, trace sand.	33.5	34	22	12
▲									
✖									

PIN	
016749.00	
Town	
South Berwick	
Reported by/Date	
WHITE, TERRY A	1/28/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

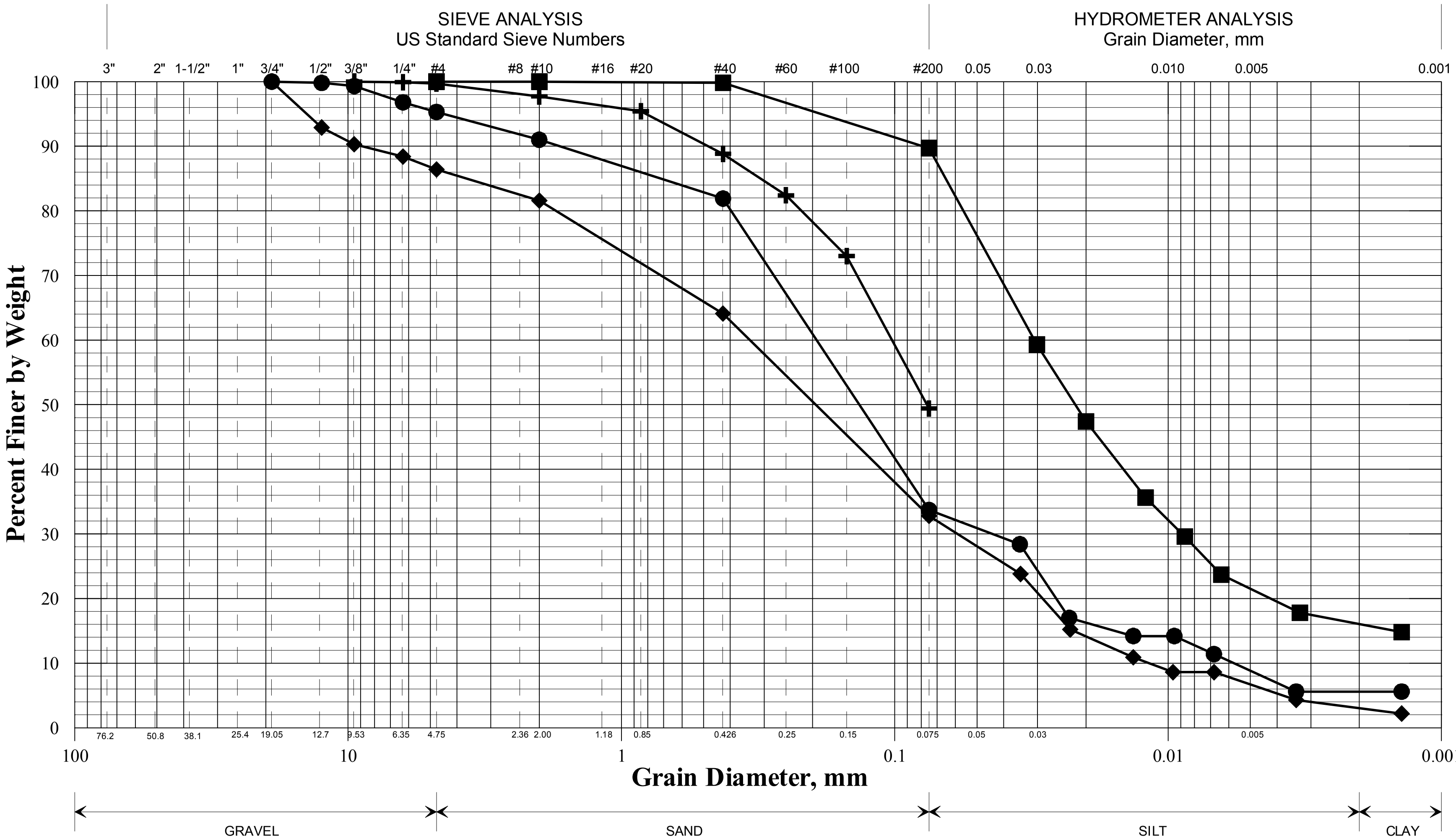


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-101/1U	14+44.3	8.6 RT	20.0-22.0	Silty CLAY, trace sand.	44.9	37	23	14
◆	BB-SBGW-101/5D	14+44.3	8.6 RT	25.0-27.0	Clayey SILT, trace sand.	43.9	35	22	13
■	BB-SBGW-101/8D	14+44.3	8.6 RT	40.2-42.2	SAND, some gravel, trace silt, trace clay.	9.7			
●									
▲									
✖									

PIN
016749.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 1/29/2010

SHEET 2

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



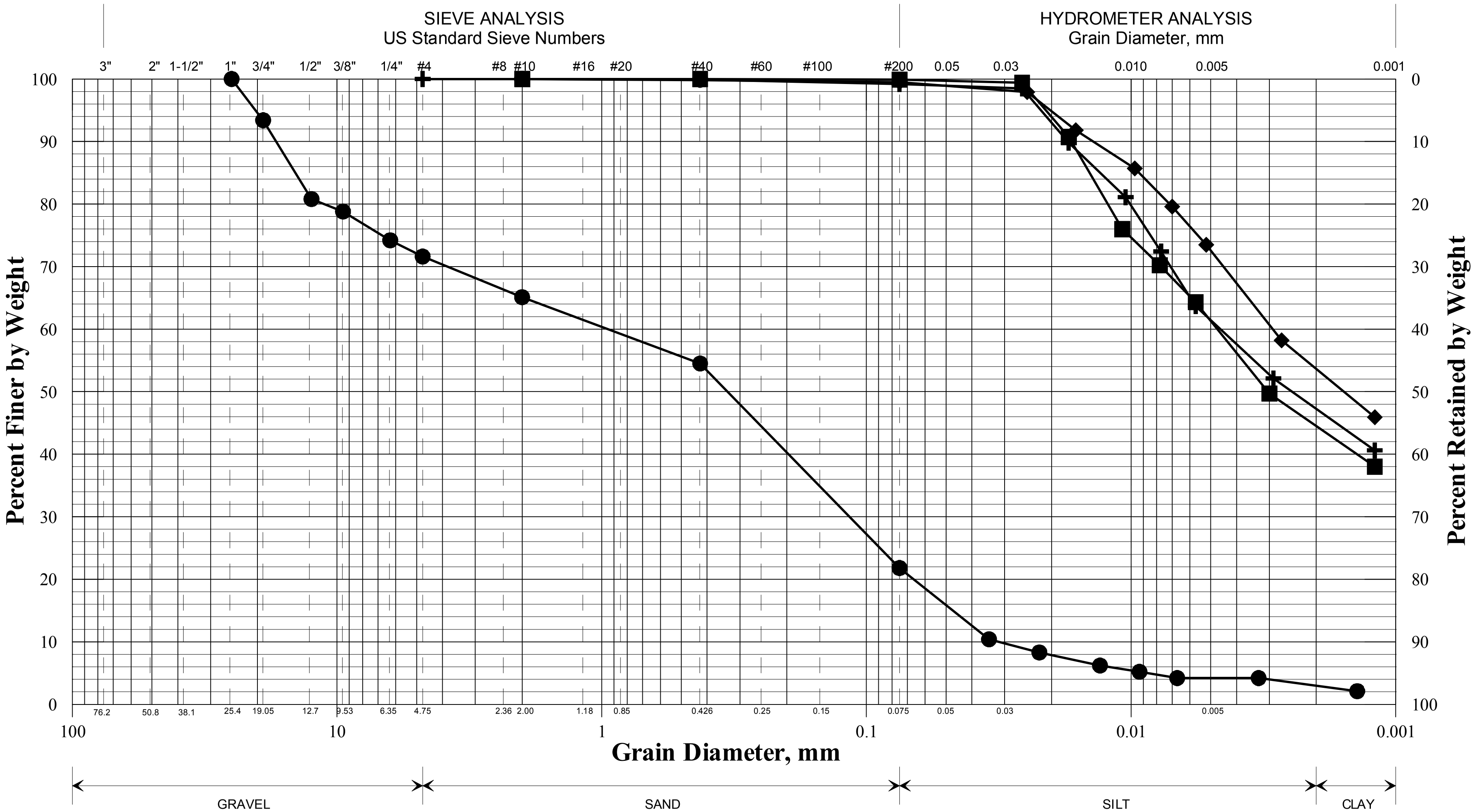
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-102/2D	15+09.4	9.4 LT	5.0-7.0	Silty SAND, trace gravel.	22.3			
◆	BB-SBGW-102/3D	15+09.4	9.4 LT	10.0-11.0	SAND, some silt, little gravel, trace clay.	21.8			
■	BB-SBGW-102/3DA	15+09.4	9.4 LT	11.0-12.0	SILT, little clay, little sand.	33.1			
●	BB-SBGW-102/4D	15+09.4	9.4 LT	15.0-17.0	SAND, some silt, trace clay, trace gravel.	30.3			
▲									
✖									

PIN	
016749.00	
Town	
South Berwick	
Reported by/Date	
WHITE, TERRY A	1/28/2010

SHEET 3

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



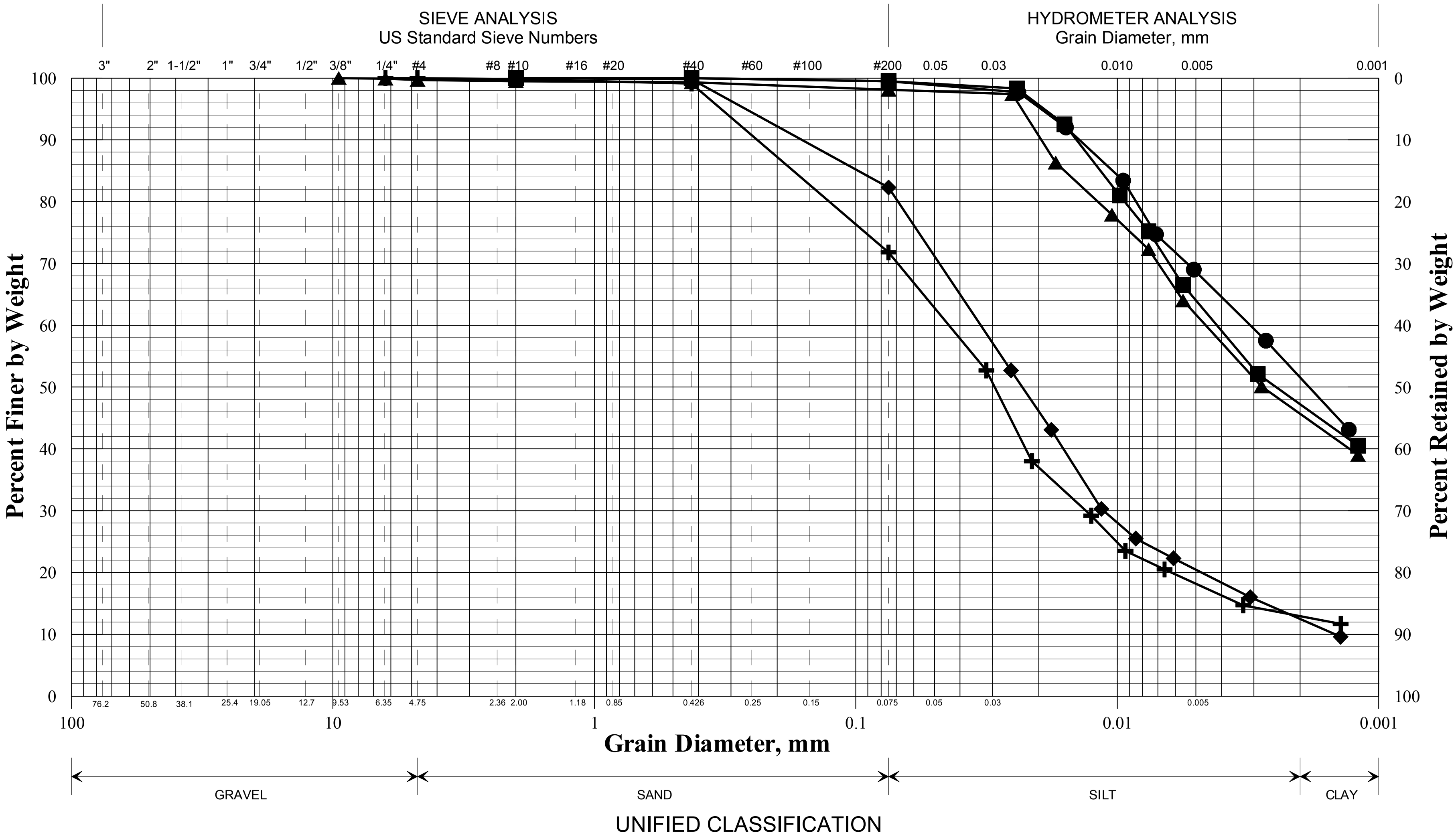
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-102/5D	15+09.4	9.4 LT	20.0-22.0	Clayey SILT, trace sand.	40.5	36	23	13
◆	BB-SBGW-102/6D	15+09.4	9.4 LT	25.0-27.0	Silty CLAY, trace sand.	45.0	37	24	13
■	BB-SBGW-102/1U	15+09.4	9.4 LT	30.0-32.0	Clayey SILT, trace sand.	41.8	37	23	14
●	BB-SBGW-102/8D	15+09.4	9.4 LT	45.0-46.4	SAND, some gravel, little silt, trace clay.	9.7			
▲									
✖									

PIN
016749.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 1/28/2010

SHEET 4

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

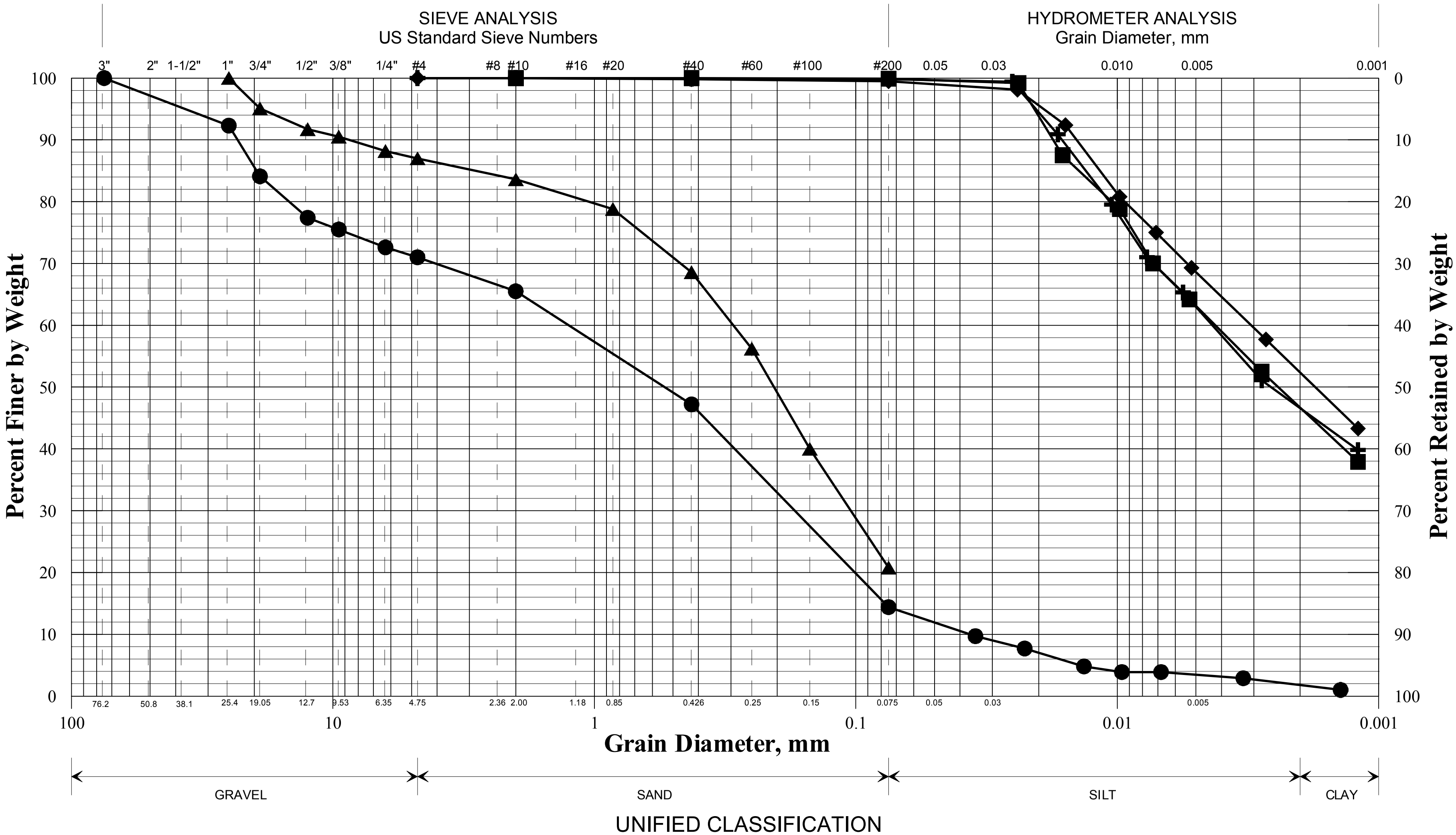


	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-103/1D	15+72.7	39.4 RT	5.0-7.0	SILT, some sand, little clay.	31.6			
◆	BB-SBGW-103/2D	15+72.7	39.4 RT	10.0-12.0	SILT, little sand, little clay.	31.9			
■	BB-SBGW-103/3D	15+72.7	39.4 RT	15.0-17.0	Clayey SILT, trace sand,	37.0	34	23	11
●	BB-SBGW-103/4D	15+72.7	39.4 RT	20.0-22.0	Silty CLAY, trace sand.	39.5	34	23	11
▲	BB-SBGW-103/5D	15+72.7	39.4 RT	25.0-27.0	Clayey SILT, trace sand, trace gravel.	38.7	37	24	13
✖									

PIN
016749.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 1/28/2010

SHEET 5

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

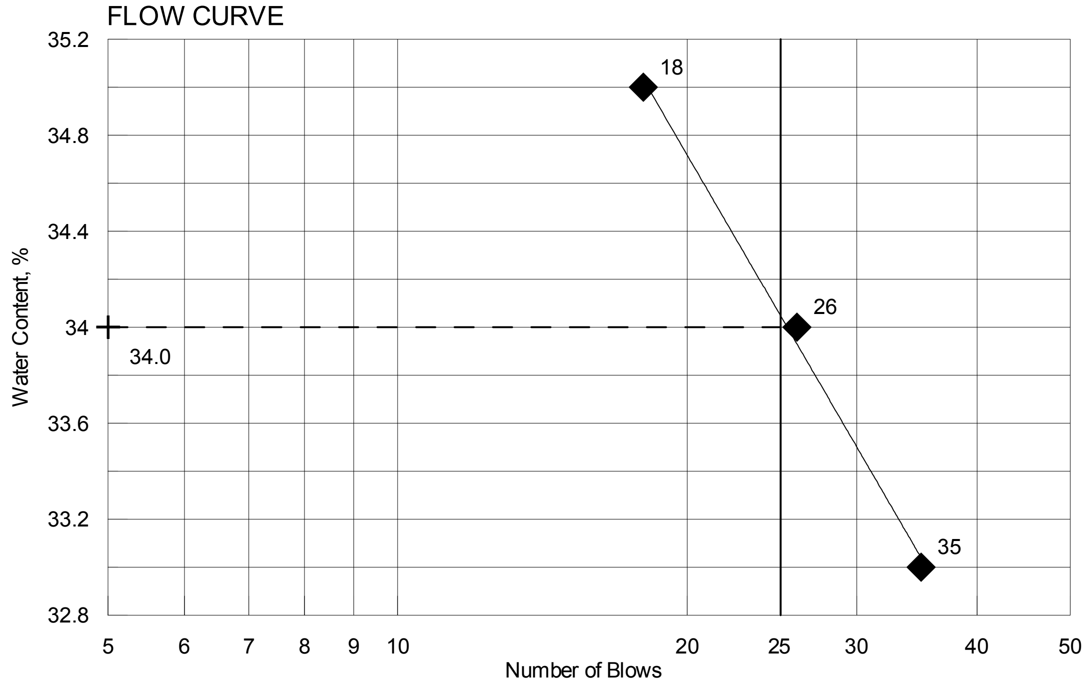


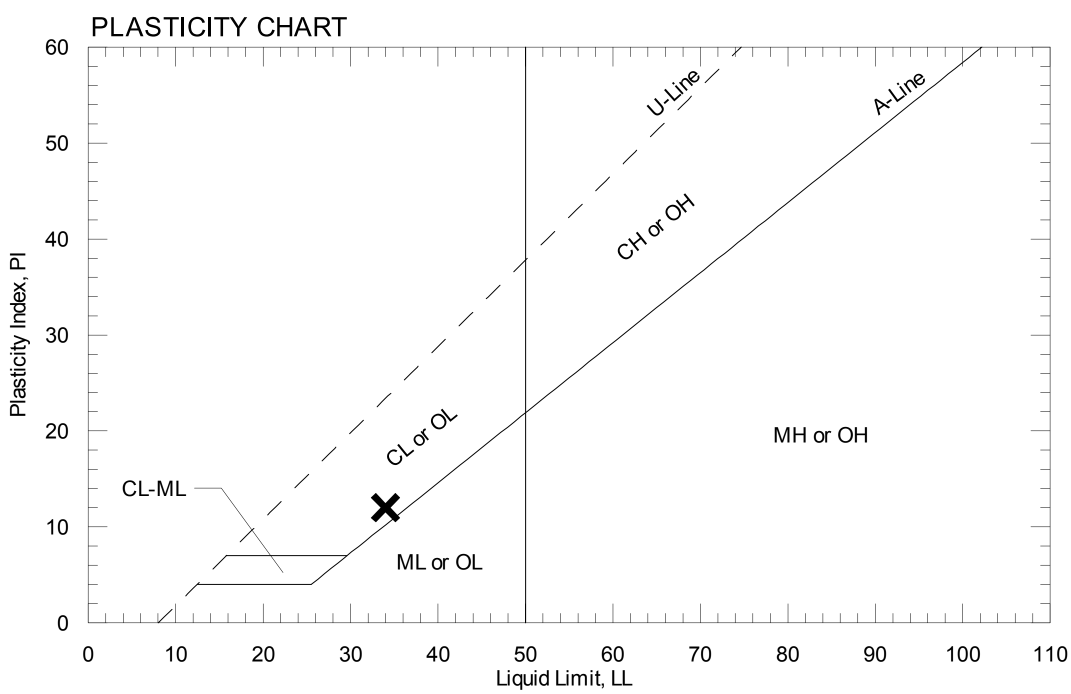
	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-SBGW-103/6D	15+72.7	39.4 RT	30.0-32.0	Clayey SILT, trace sand.	42.2	37	23	14
◆	BB-SBGW-103/7D	15+72.7	39.4 RT	35.0-37.0	Silty CLAY, trace sand.	42.7	37	23	14
■	BB-SBGW-103/8D	15+72.7	39.4 RT	40.0-42.0	Clayey SILT, trace sand.	43.0	37	24	13
●	BB-SBGW-103/9D	15+72.7	39.4 RT	45.0-47.0	SAND, some gravel, little silt, trace clay.	9.6			
▲	BB-SBGW-103/10D	15+72.7	39.4 RT	50.0-52.0	SAND, some silt, little gravel.	13.8			
✖									

PIN
016749.00
Town
South Berwick
Reported by/Date
WHITE, TERRY A 1/28/2010

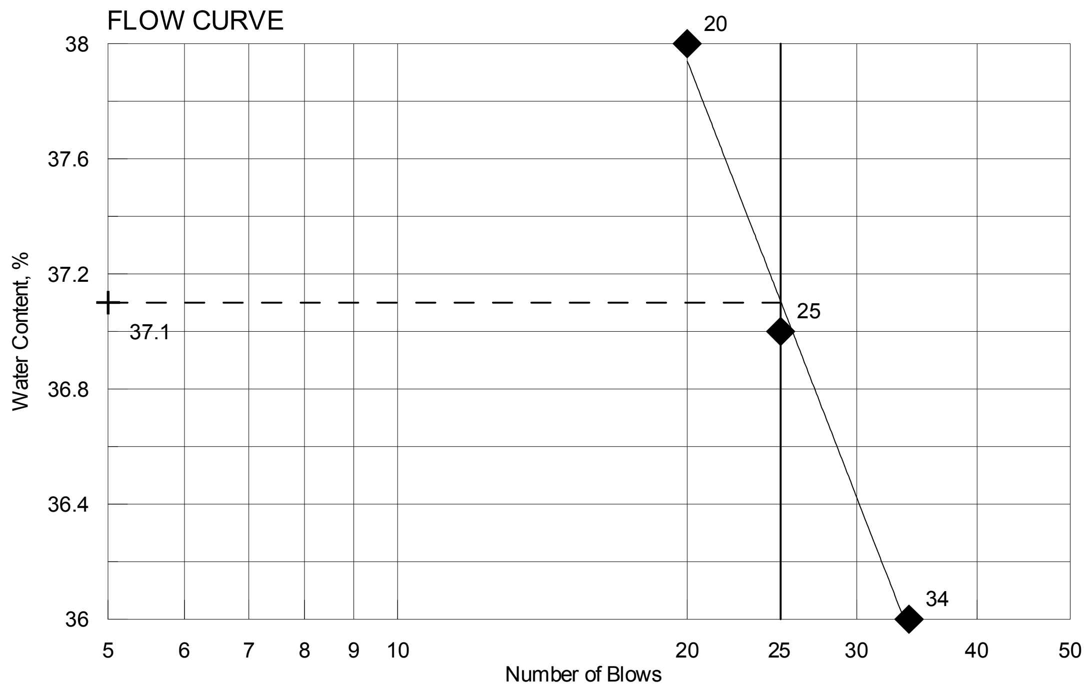
SHEET 6

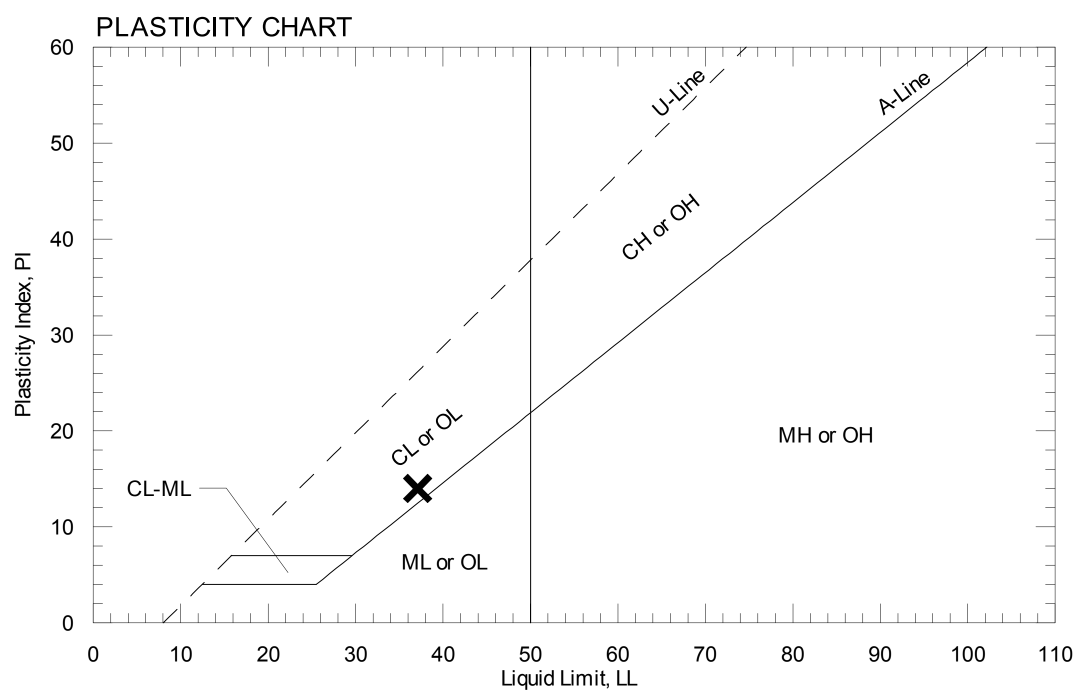
TOWN	South Berwick	Reference No.	236831
PIN	016749.00	Water Content, %	33.5
Sampled	12/2/2009	Plastic Limit	22
Boring No./Sample No.	BB-SBGW-101/4D	Liquid Limit	34
Station	14+44.3	Plasticity Index	12
Depth	15.0-17.0	Tested By	BBURR



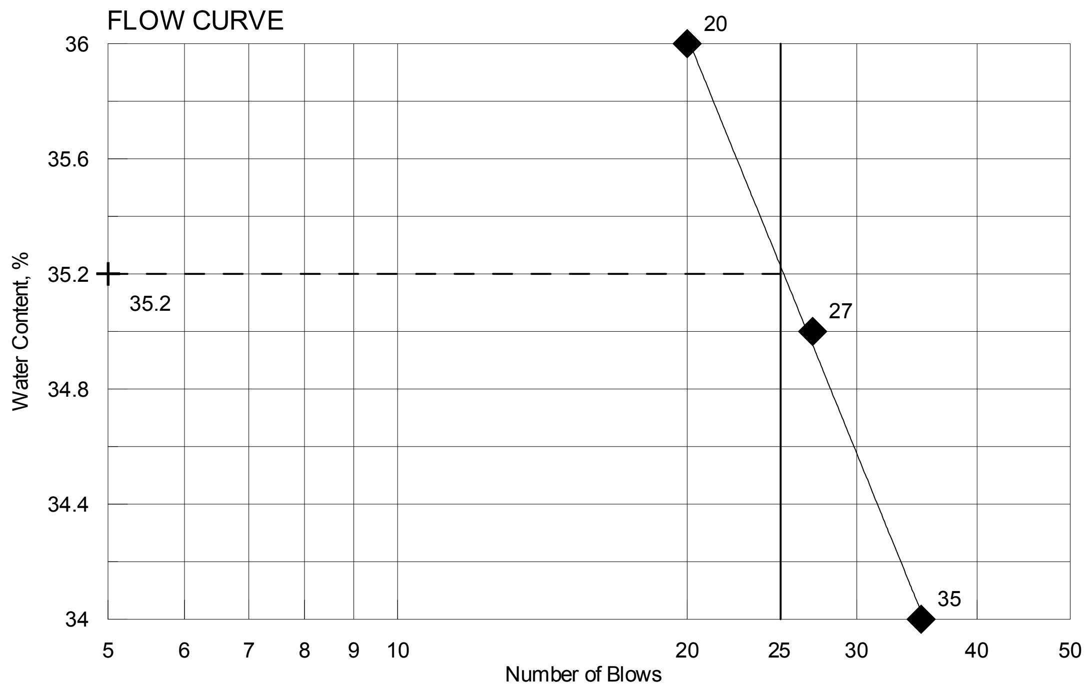


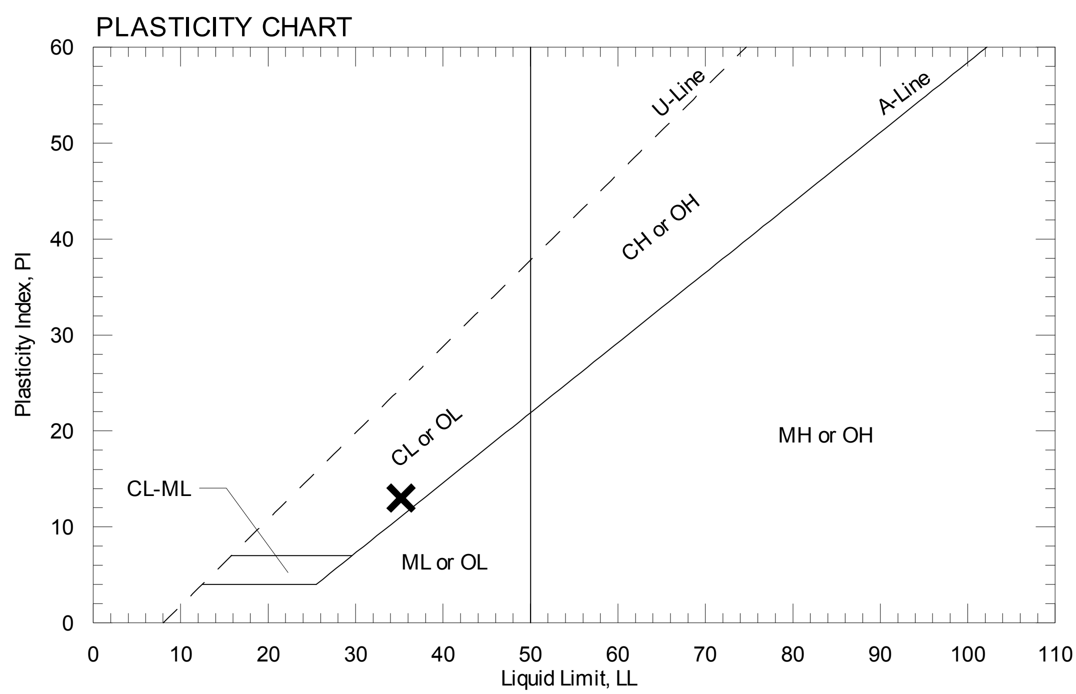
TOWN	South Berwick	Reference No.	236832
PIN	016749.00	Water Content, %	44.9
Sampled	12/2/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-101/1U	Liquid Limit	37
Station	14+44.3	Plasticity Index	14
Depth	20.0-22.0	Tested By	BBURR



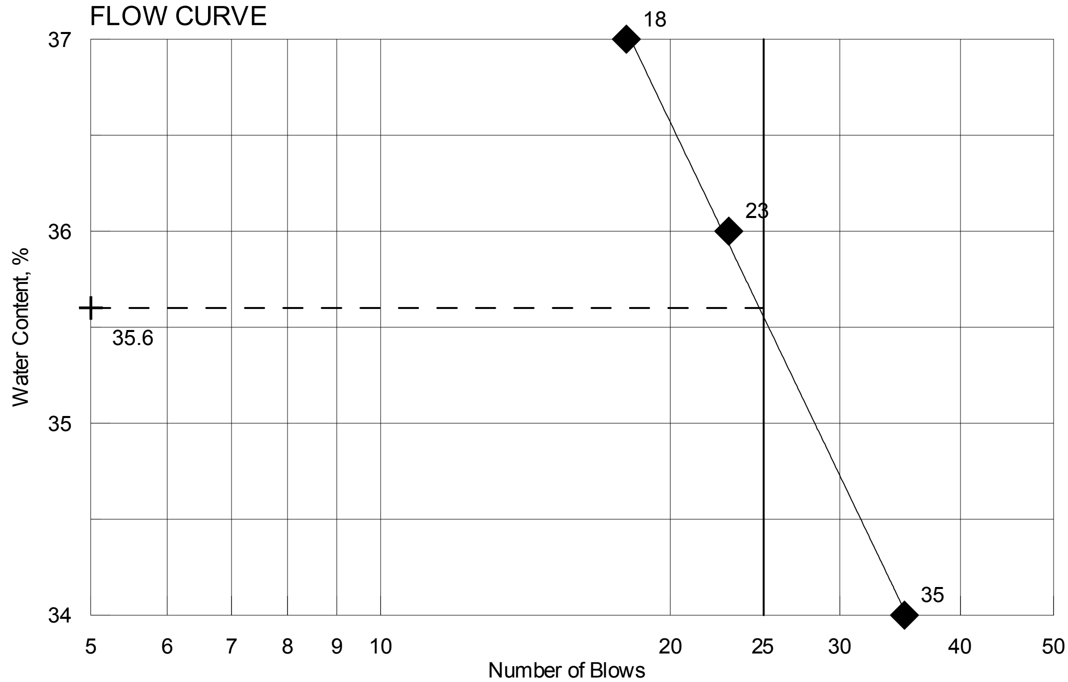


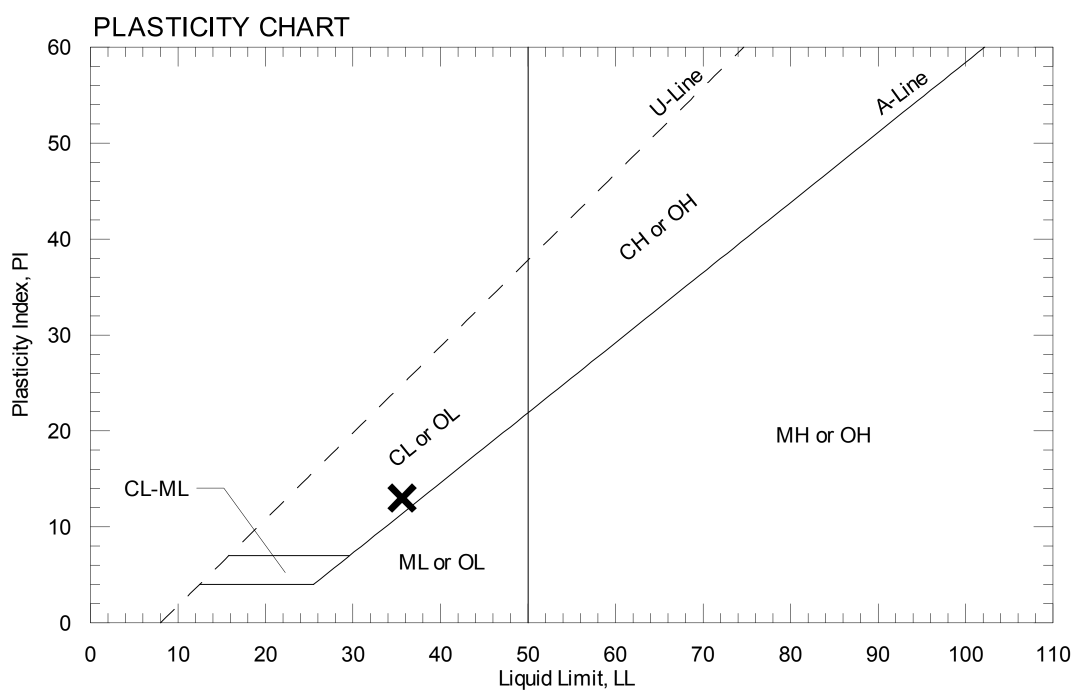
TOWN	South Berwick	Reference No.	236833
PIN	016749.00	Water Content, %	43.9
Sampled	12/2/2009	Plastic Limit	22
Boring No./Sample No.	BB-SBGW-101/5D	Liquid Limit	35
Station	14+44.3	Plasticity Index	13
Depth	25.0-27.0	Tested By	BBURR



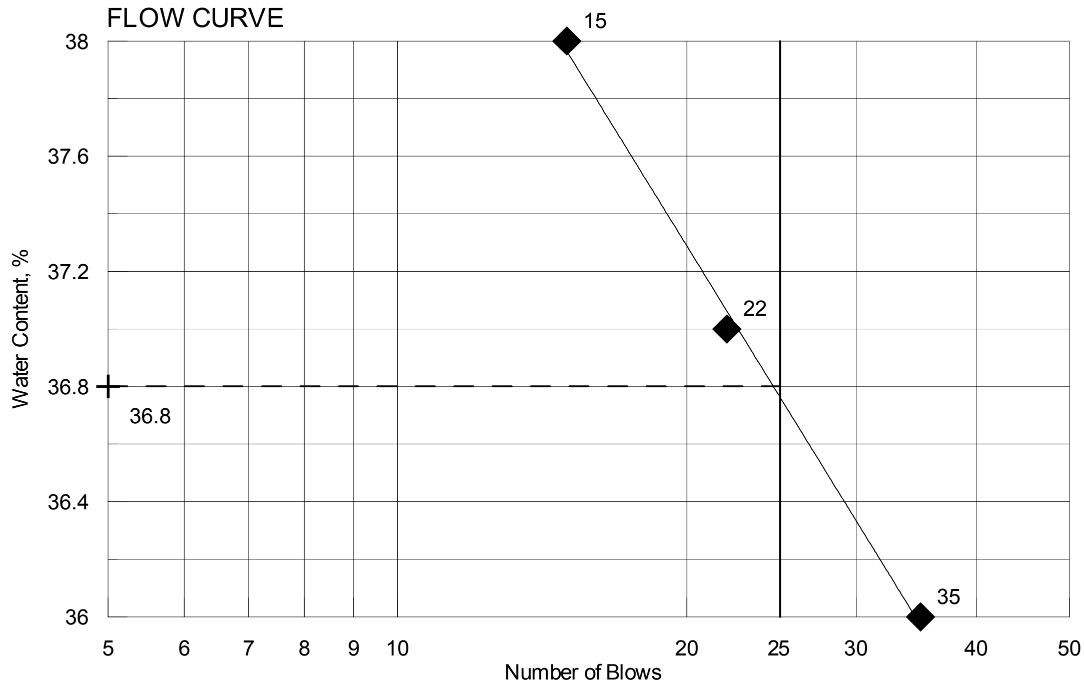


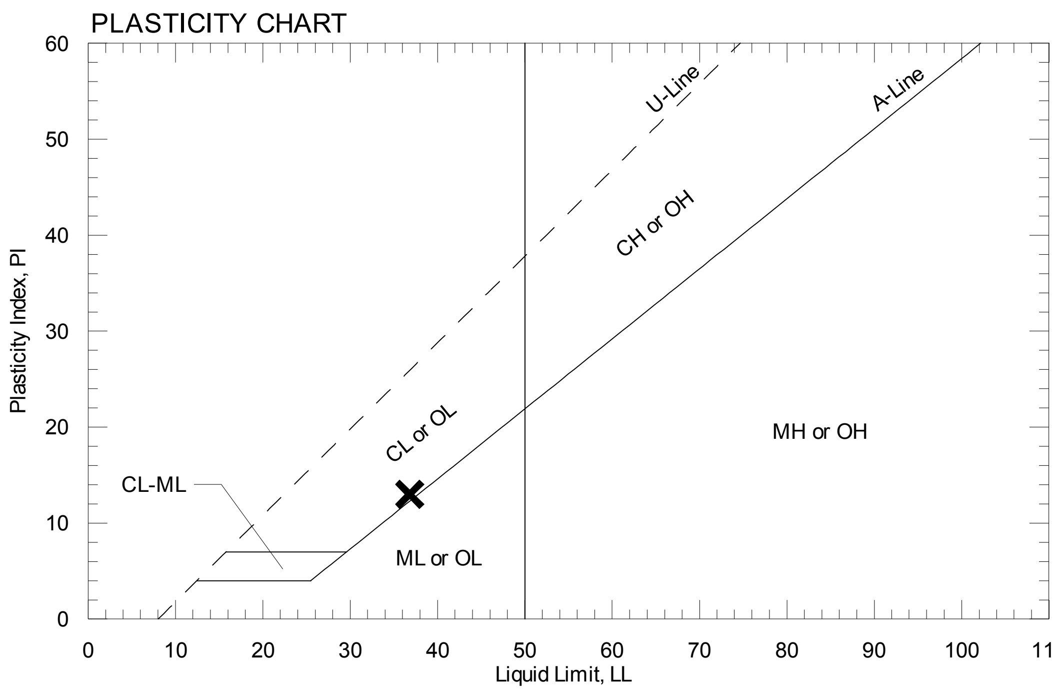
TOWN	South Berwick	Reference No.	236839
PIN	016749.00	Water Content, %	40.5
Sampled	12/2/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-102/5D	Liquid Limit	36
Station	15+09.4	Plasticity Index	13
Depth	20.0-22.0	Tested By	BBURR



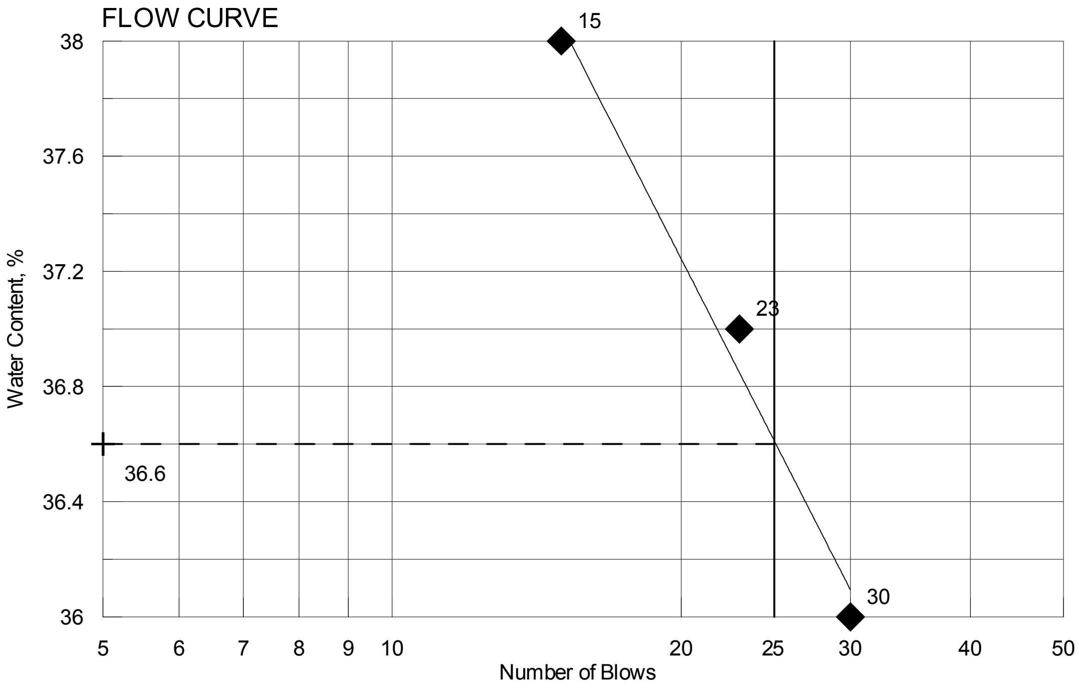


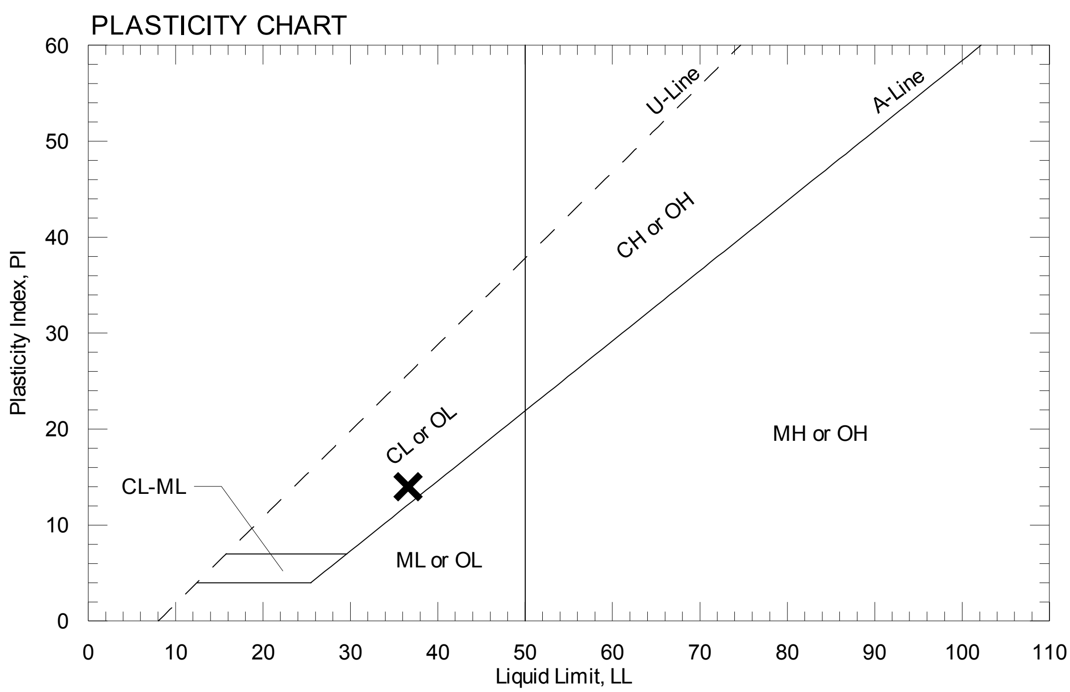
TOWN	South Berwick	Reference No.	236840
PIN	016749.00	Water Content, %	45
Sampled	12/2/2009	Plastic Limit	24
Boring No./Sample No.	BB-SBGW-102/6D	Liquid Limit	37
Station	15+09.4	Plasticity Index	13
Depth	25.0-27.0	Tested By	BBURR



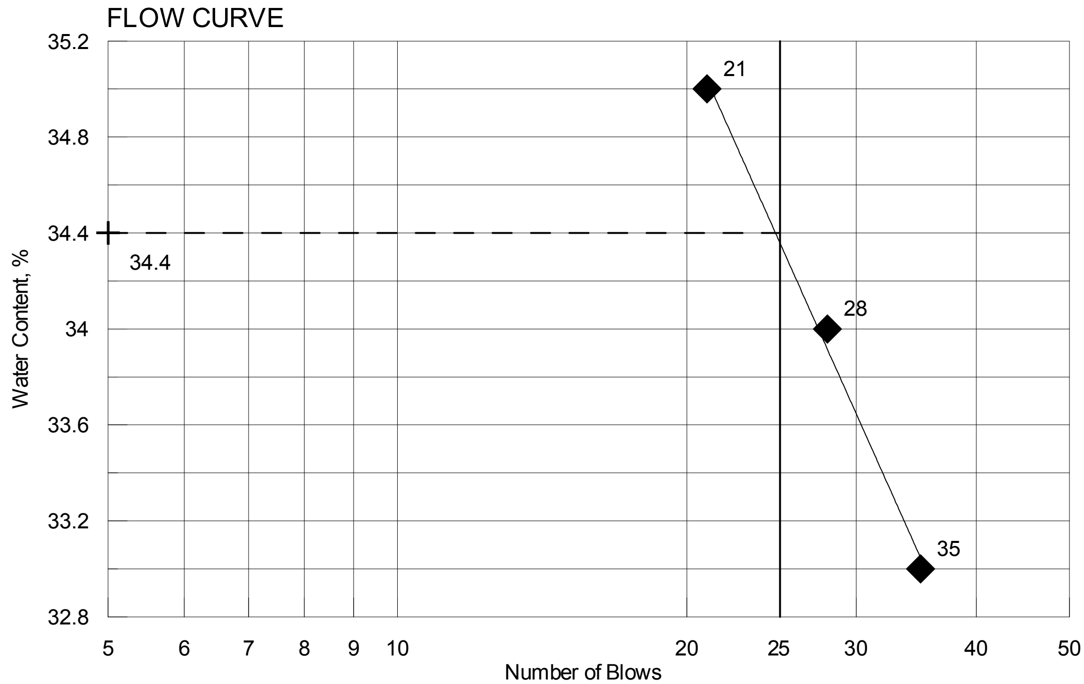


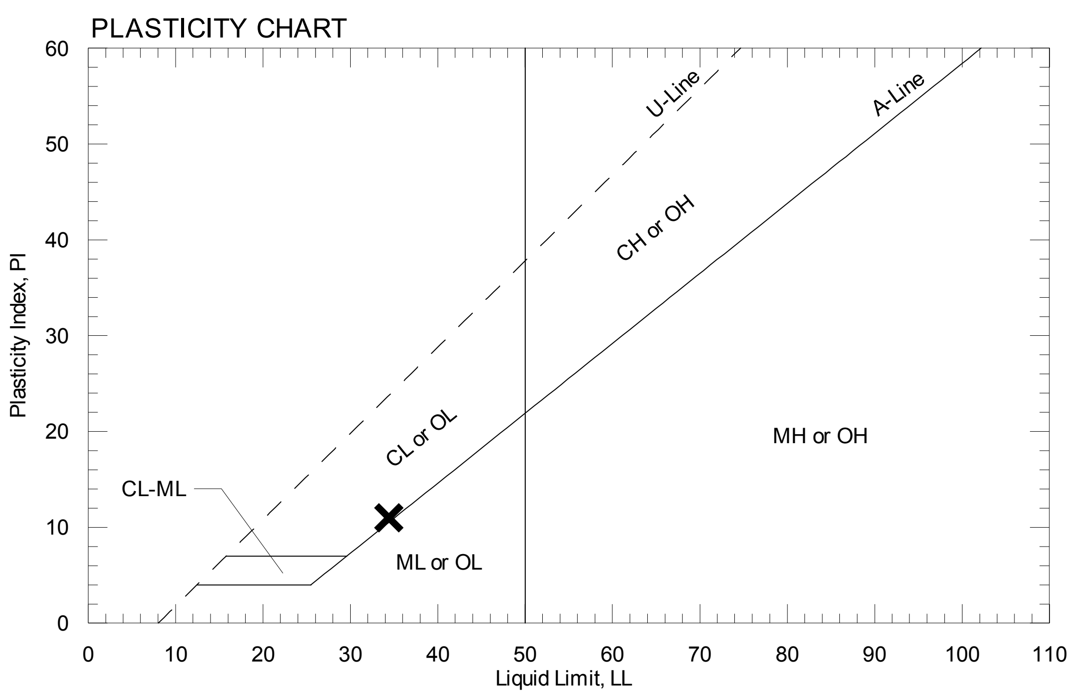
TOWN	South Berwick	Reference No.	236841
PIN	016749.00	Water Content, %	41.8
Sampled	12/2/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-102/1U	Liquid Limit	37
Station	15+09.4	Plasticity Index	14
Depth	30.0-32.0	Tested By	BBURR



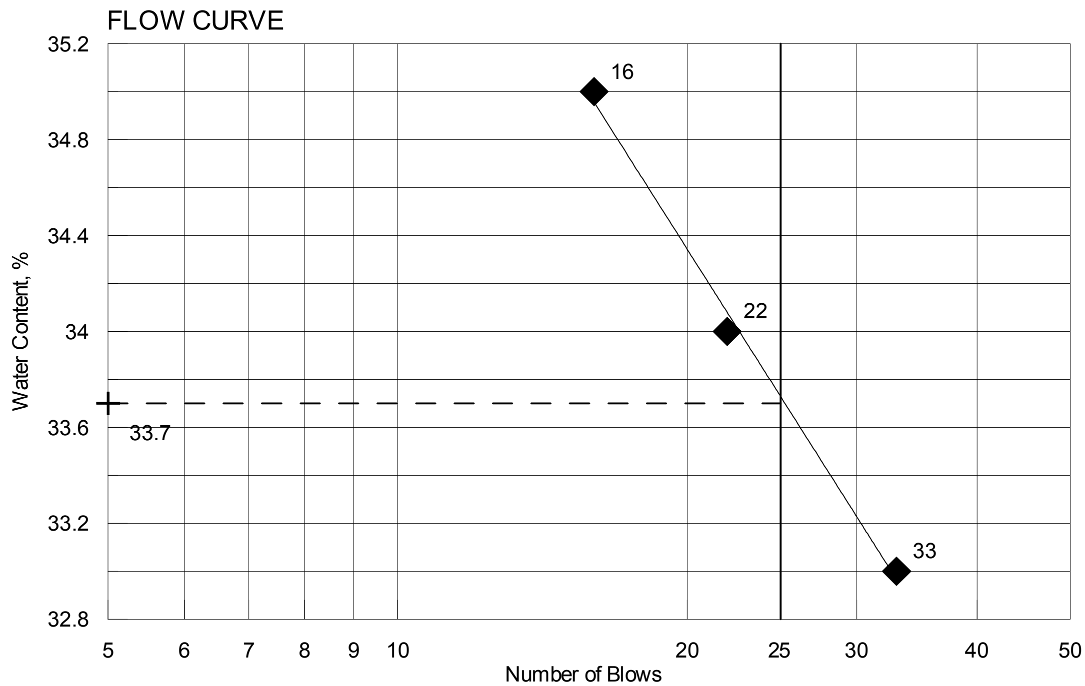


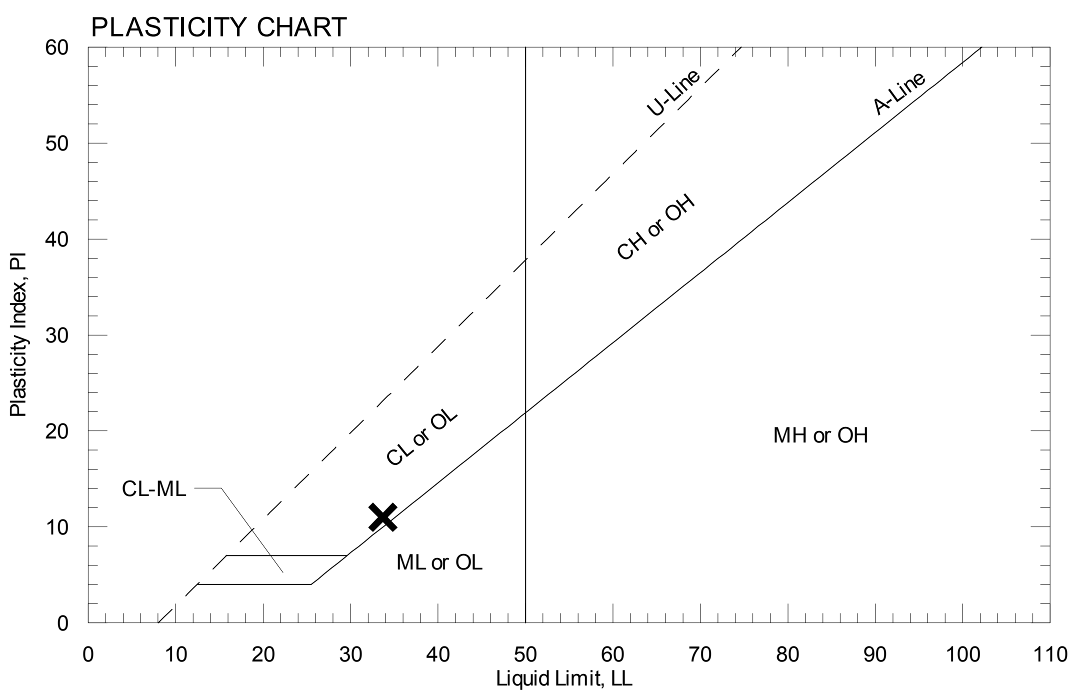
TOWN	South Berwick	Reference No.	236845
PIN	016749.00	Water Content, %	37
Sampled	11/4/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-103/3D	Liquid Limit	34
Station	15+72.7	Plasticity Index	11
Depth	15.0-17.0	Tested By	BBURR



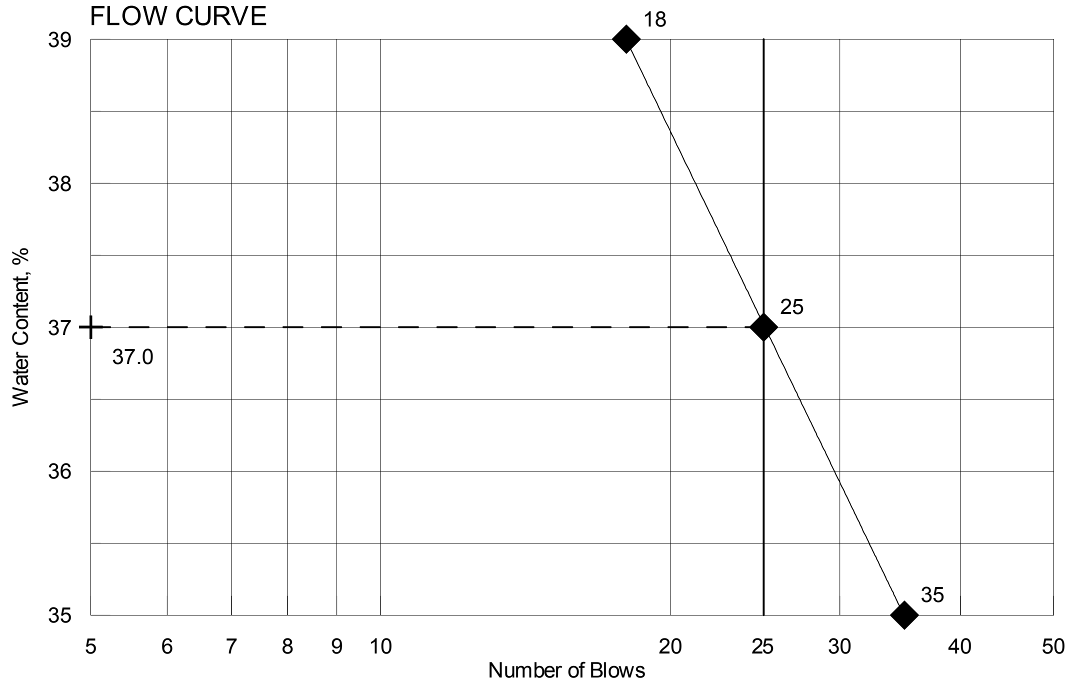


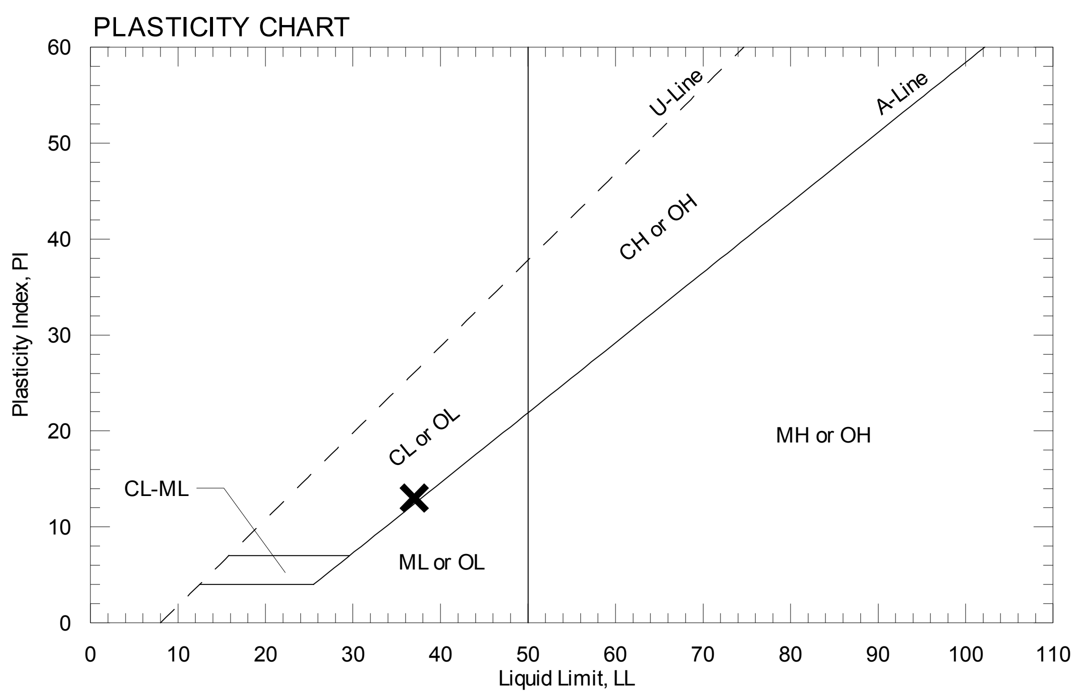
TOWN	South Berwick	Reference No.	236846
PIN	016749.00	Water Content, %	39.5
Sampled	11/4/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-103/4D	Liquid Limit	34
Station	15+72.7	Plasticity Index	11
Depth	20.0-22.0	Tested By	BBURR



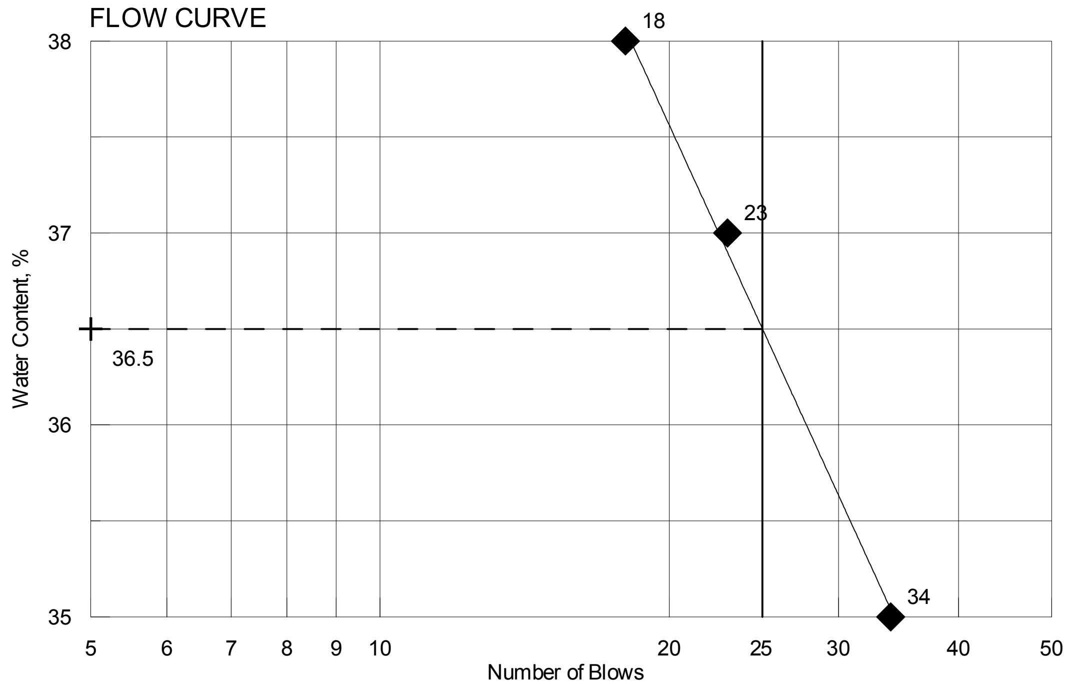


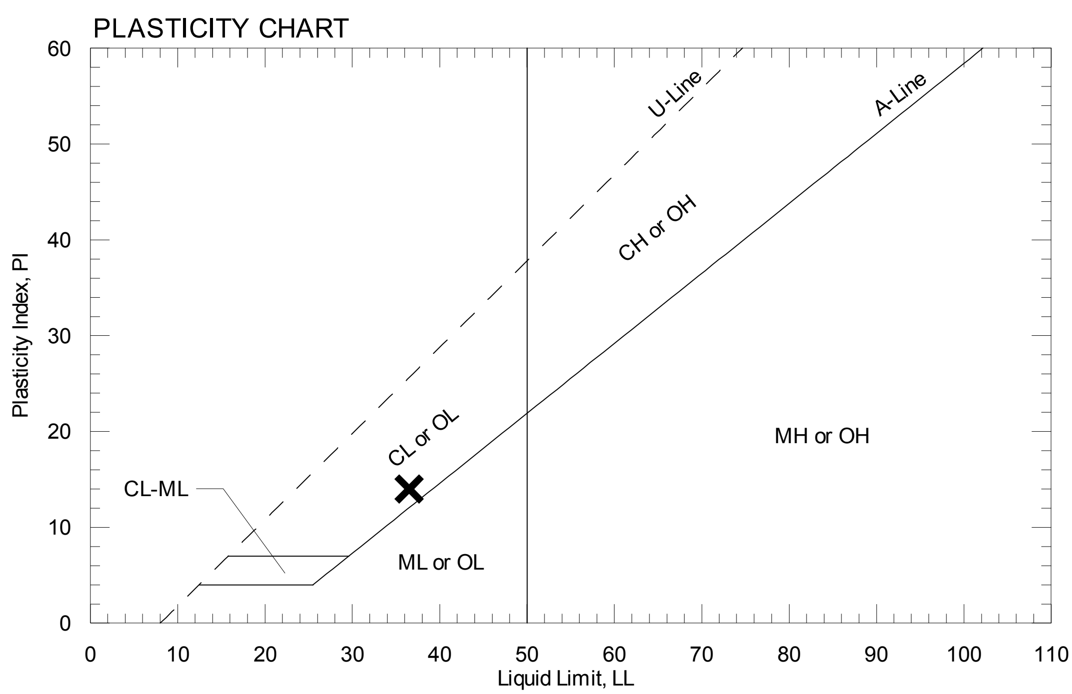
TOWN	South Berwick	Reference No.	236847
PIN	016749.00	Water Content, %	38.7
Sampled	11/4/2009	Plastic Limit	24
Boring No./Sample No.	BB-SBGW-103/5D	Liquid Limit	37
Station	15+72.7	Plasticity Index	13
Depth	25.0-27.0	Tested By	BBURR



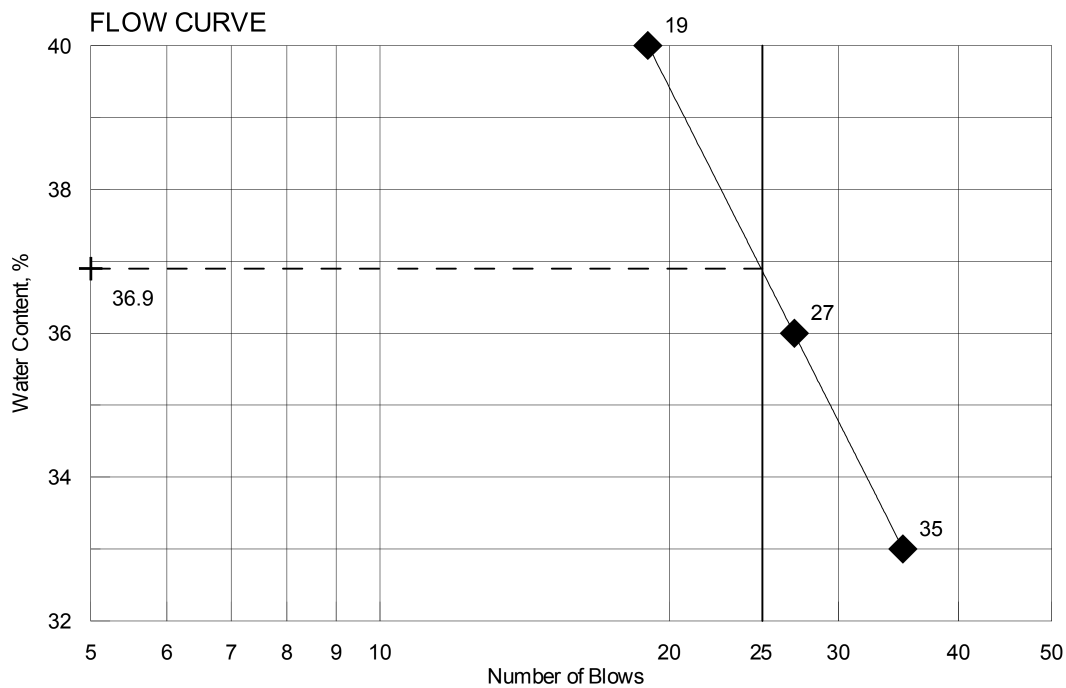


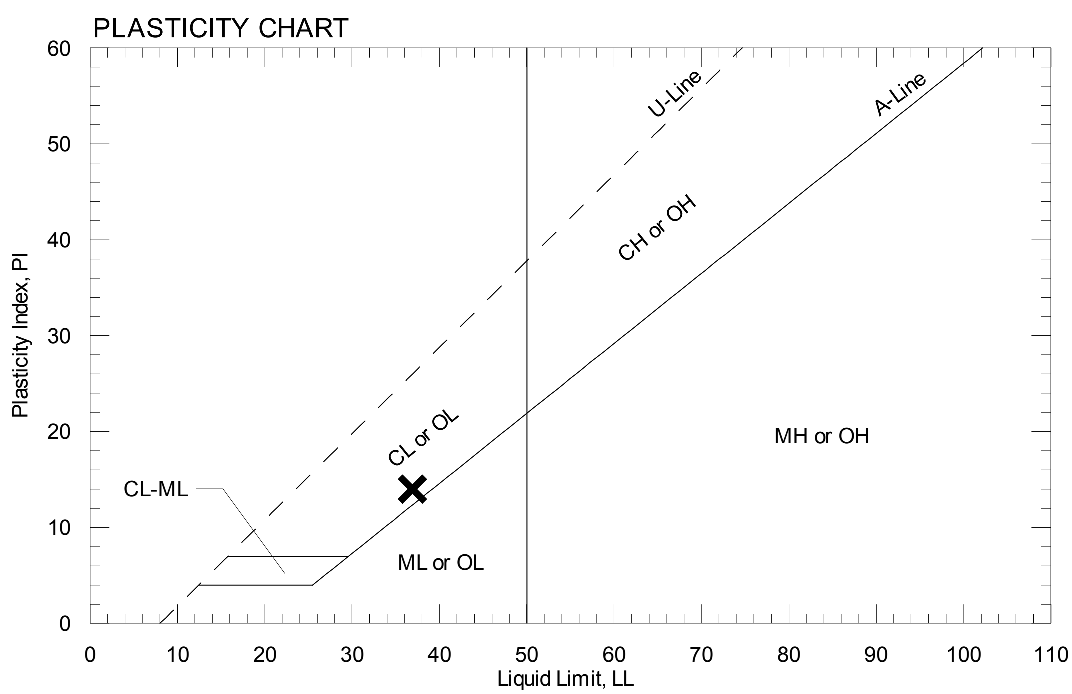
TOWN	South Berwick	Reference No.	236848
PIN	016749.00	Water Content, %	42.2
Sampled	11/4/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-103/6D	Liquid Limit	37
Station	15+72.7	Plasticity Index	14
Depth	30.0-32.0	Tested By	BBURR



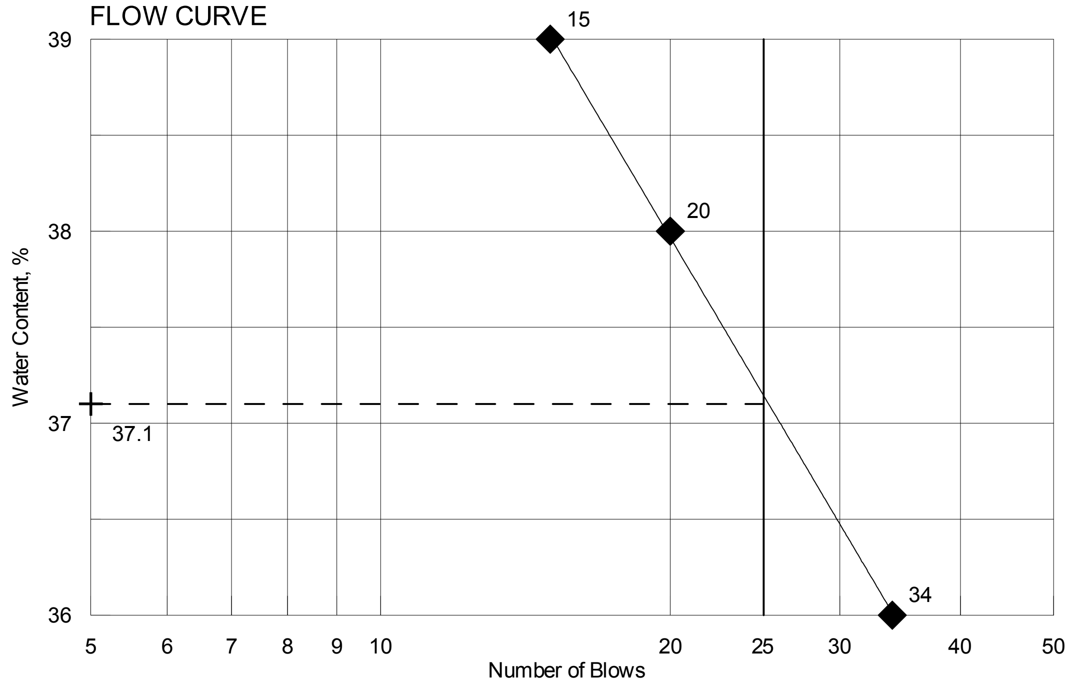


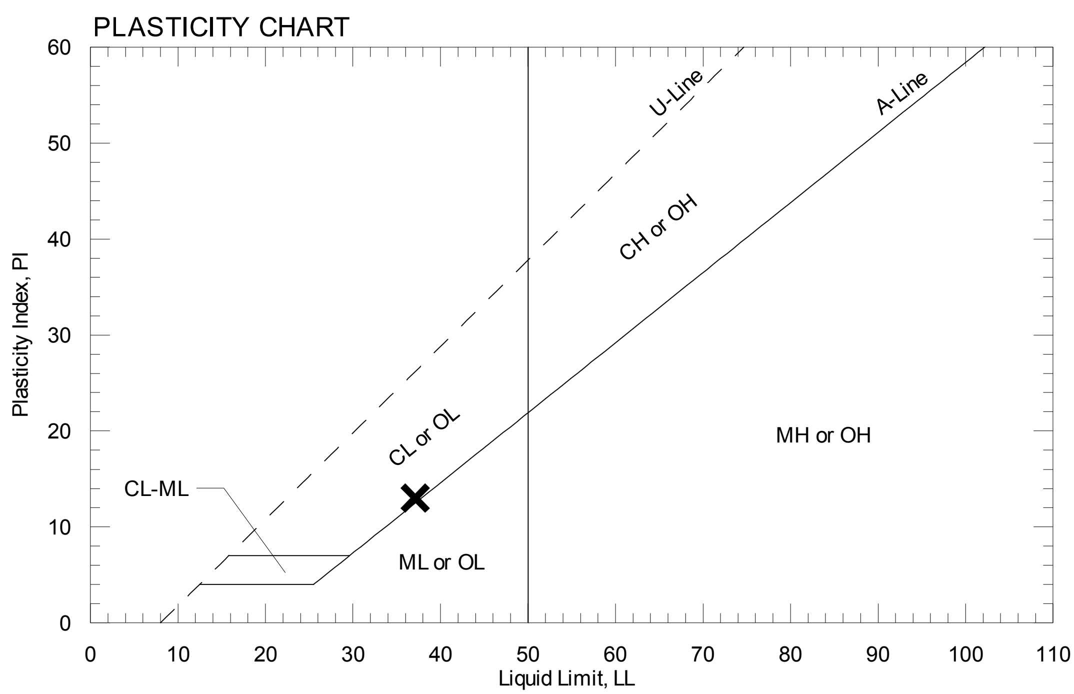
TOWN	South Berwick	Reference No.	236849
PIN	016749.00	Water Content, %	42.7
Sampled	11/4/2009	Plastic Limit	23
Boring No./Sample No.	BB-SBGW-103/7D	Liquid Limit	37
Station	15+72.7	Plasticity Index	14
Depth	35.0-37.0	Tested By	BBURR





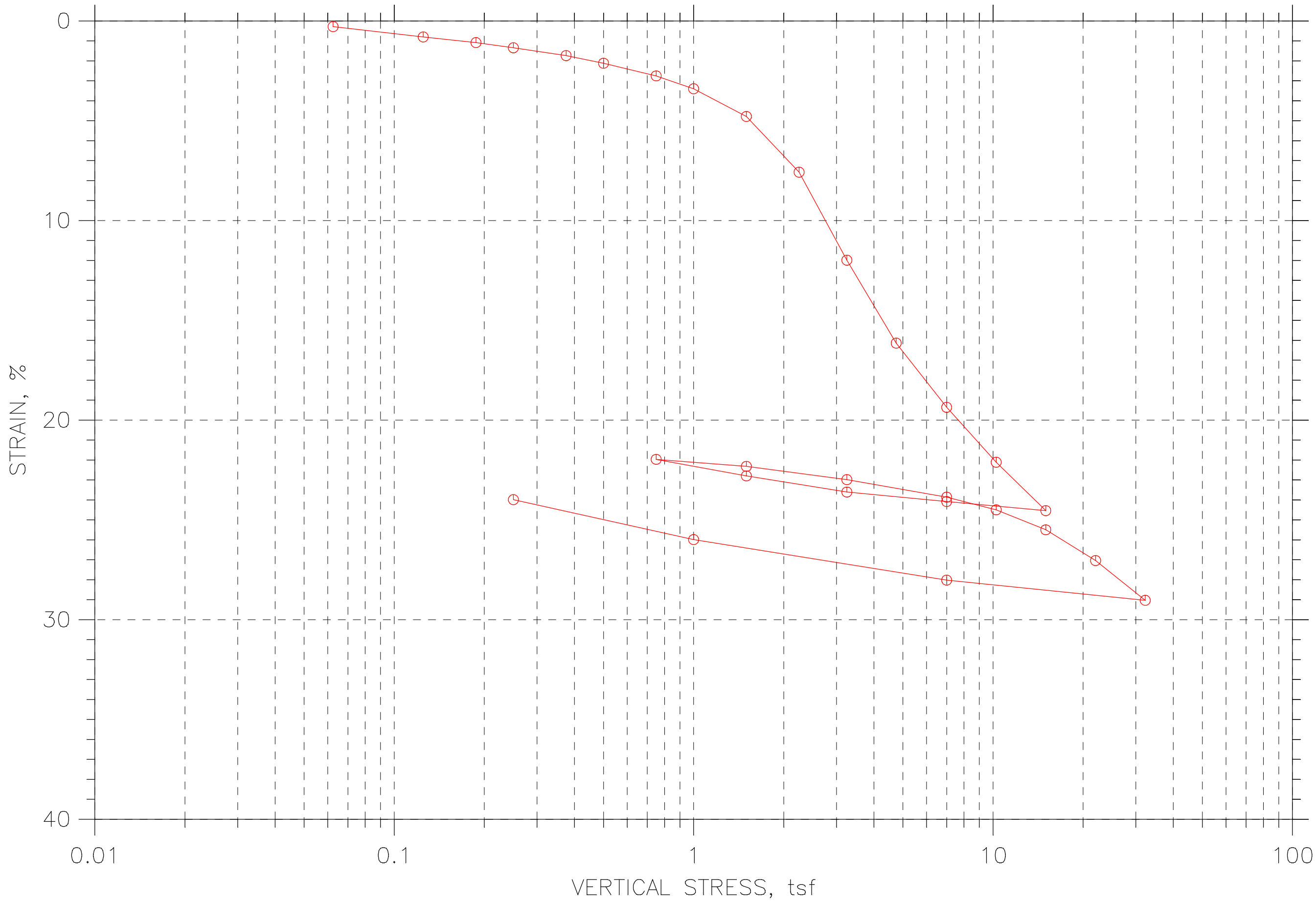
TOWN	South Berwick	Reference No.	236850
PIN	016749.00	Water Content, %	43
Sampled	11/4/2009	Plastic Limit	24
Boring No./Sample No.	BB-SBGW-103/8D	Liquid Limit	37
Station	15+72.7	Plasticity Index	13
Depth	40.0-42.0	Tested By	BBURR

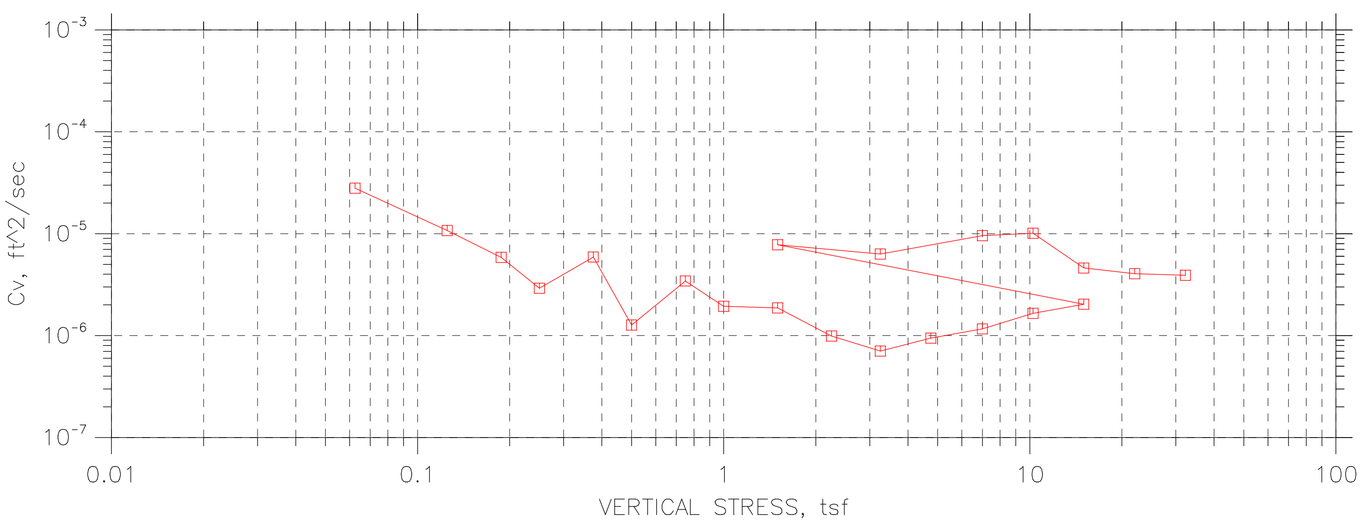




CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: GREAT HILL BRIDGE	Location: SOUTH BERWICK	Project No.: 016749.00
	Boring No.: BB-SBGW-102	Tested By: Brian Fogg	Checked By:
	Sample No.: 1U	Test Date: 1/20/2010	Depth: 30-32 FT
	Test No.: 236841	Sample Type: Shelby Tube	Elevation: ---
	Description: CLAY		
	Remarks:		

Tue, 03-AUG-2010 14:28:30

CONSOLIDATION TEST DATA

Project: GREAT HILL BRIDGE
Boring No.: BB-SBGW-102
Sample No.: 1U
Test No.: 236841

Location: SOUTH BERWICK
Tested By: Brian Fogg
Test Date: 1/20/2010
Sample Type: Shelby Tube

Project No.: 016749.00
Checked By:
Depth: 30-32 FT
Elevation: ---

Soil Description: CLAY
Remarks:

Measured Specific Gravity: 2.68
Initial Void Ratio: 1.37
Final Void Ratio: 0.80

Liquid Limit: 37
Plastic Limit: 23
Plasticity Index: 14

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	44	RING	RING	52
Wt. Container + Wet Soil, gm	190.76	400.73	382.2	184.5
Wt. Container + Dry Soil, gm	147.72	354.48	354.48	156.82
Wt. Container, gm	53.64	262.13	262.13	64.59
Wt. Dry Soil, gm	94.08	92.353	92.353	92.23
Water Content, %	45.75	50.08	30.01	30.01
Void Ratio	---	1.37	0.80	---
Degree of Saturation, %	---	97.73	100.04	---
Dry Unit Weight, pcf	---	70.497	92.742	---

CONSOLIDATION TEST DATA

Project: GREAT HILL BRIDGE
Boring No.: BB-SBGW-102
Sample No.: 1U
Test No.: 236841

Location: SOUTH BERWICK
Tested By: Brian Fogg
Test Date: 1/20/2010
Sample Type: Shelby Tube

Project No.: 016749.00
Checked By:
Depth: 30-32 FT
Elevation: ---

Soil Description: CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.002943	1.366	0.29	0.2	0.0	2.79e-005	0.00e+000	2.79e-005
2	0.125	0.008239	1.354	0.80	0.7	0.4	8.28e-006	1.53e-005	1.07e-005
3	0.188	0.01116	1.348	1.08	1.0	0.0	5.86e-006	0.00e+000	5.86e-006
4	0.25	0.01384	1.341	1.34	2.2	1.9	2.73e-006	3.12e-006	2.92e-006
5	0.375	0.01783	1.332	1.73	1.1	0.9	5.26e-006	6.73e-006	5.90e-006
6	0.5	0.02179	1.323	2.12	4.6	0.0	1.27e-006	0.00e+000	1.27e-006
7	0.75	0.02831	1.308	2.75	1.7	1.6	3.31e-006	3.57e-006	3.44e-006
8	1	0.03498	1.293	3.40	3.2	2.6	1.75e-006	2.18e-006	1.94e-006
9	1.5	0.04923	1.260	4.78	3.4	2.5	1.63e-006	2.20e-006	1.87e-006
10	2.25	0.07797	1.193	7.58	6.9	3.8	7.66e-007	1.41e-006	9.91e-007
11	3.25	0.1233	1.089	11.98	6.9	7.1	7.17e-007	6.94e-007	7.05e-007
12	4.75	0.1661	0.990	16.14	4.8	4.7	9.29e-007	9.59e-007	9.44e-007
13	7	0.1992	0.914	19.36	3.4	3.6	1.19e-006	1.14e-006	1.16e-006
14	10.3	0.2275	0.848	22.11	2.1	2.5	1.85e-006	1.50e-006	1.65e-006
15	15	0.2525	0.791	24.54	1.6	1.9	2.26e-006	1.84e-006	2.03e-006
16	7	0.2478	0.802	24.08	0.0	0.0	9.61e-005	0.00e+000	9.61e-005
17	3.25	0.2429	0.813	23.60	0.2	0.0	1.69e-005	0.00e+000	1.69e-005
18	1.5	0.2346	0.832	22.80	1.4	0.0	2.59e-006	0.00e+000	2.59e-006
19	0.75	0.226	0.852	21.97	2.1	3.1	1.74e-006	1.17e-006	1.40e-006
20	1.5	0.2297	0.844	22.32	0.5	0.0	7.79e-006	0.00e+000	7.79e-006
21	3.25	0.2365	0.828	22.98	0.5	0.7	7.35e-006	5.54e-006	6.32e-006
22	7	0.2455	0.807	23.86	0.5	0.3	7.56e-006	1.30e-005	9.56e-006
23	10.3	0.252	0.792	24.49	0.5	0.2	7.50e-006	1.55e-005	1.01e-005
24	15	0.2624	0.768	25.50	0.9	0.6	3.75e-006	5.97e-006	4.60e-006
25	22	0.2782	0.732	27.04	0.9	0.7	3.59e-006	4.66e-006	4.05e-006
26	32.3	0.2987	0.684	29.03	0.9	0.7	3.47e-006	4.47e-006	3.91e-006
27	7	0.2883	0.708	28.02	0.0	0.0	1.32e-004	0.00e+000	1.32e-004
28	1	0.2674	0.756	25.99	1.5	1.6	2.19e-006	2.04e-006	2.12e-006
29	0.25	0.2468	0.804	23.99	7.2	8.5	4.75e-007	3.98e-007	4.33e-007

<u>Appendix C</u>

Calculations

Definition of Units:

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $ksf := \frac{kip}{ft^2}$ $tsf := g \cdot \left(\frac{ton}{ft^2}\right)$ $kip := 1000 \cdot lbf$

LIQUIDITY INDEX (LI):

Liquidity Index = (natural water content - Plastic Limit) / (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	LI	
BB-SBGW-101/4D	33.5	34	22	12	0.96	Normally consolidated
BB-SBGW-101/1U	44.9	37	23	14	1.56	Viscous liquid when remolded
BB-SBGW-101/5D	43.9	35	22	13	1.68	Viscous liquid when remolded
BB-SBGW-102/5D	40.5	36	23	13	1.35	Viscous liquid when remolded
BB-SBGW-102/6D	45.0	37	24	13	1.62	Viscous liquid when remolded
BB-SBGW-102/1U	41.8	37	23	14	1.34	Viscous liquid when remolded
BB-SBGW-103/3D	37.0	34	23	11	1.27	Viscous liquid when remolded
BB-SBGW-103/4D	39.5	34	23	11	1.50	Viscous liquid when remolded
BB-SBGW-103/5D	38.7	37	24	13	1.13	Viscous liquid when remolded
BB-SBGW-103/6D	42.2	37	23	14	1.37	Viscous liquid when remolded
BB-SBGW-103/7D	42.7	37	23	14	1.41	Viscous liquid when remolded
BB-SBGW-103/8D	43.0	37	24	13	1.46	Viscous liquid when remolded

CONSOLIDATION TEST RESULTS

BB-SBGW-102 Sample 1U

Determine in-situ over burden stress:

Sample depth = 31.0 ft below ground surface

Groundwater table at 12.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.37$

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

$$\sigma'_{vo} := 11 \cdot ft \cdot 125 \cdot pcf + 1.0 \cdot ft \cdot (125) \cdot pcf + 6.5 \cdot ft \cdot (125 - 62.4) \cdot pcf + 12.5 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$\sigma'_{vo} = 2564 \cdot psf$ or $\sigma'_{vo} = 1.282 \cdot tsf$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.9 \cdot tsf$

Determine OCR: $OCR := \dfrac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.4818$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 1.193$ $p_2 := 4.75 \cdot tsf$ $e_2 := 0.990$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.6256$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \dfrac{7.58}{100}$ $\varepsilon_2 := \dfrac{16.14}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.2638$$

or: $C'c := \dfrac{C_c}{1 + e_0}$ $C'c = 0.2639$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.844$ $p_2 := 7 \cdot tsf$ $e_2 := 0.807$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0553$$

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

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

Note: All matrices set up in this order

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

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

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

Where λ=normalized column slenderness factor

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

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

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

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

STRENGTH LIMIT STATE:

Factored Resistance:

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 $P_f := \phi_c \cdot P_n$ $P_f = \begin{pmatrix} 465 \\ 654 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot kip$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

Where λ=normalized column slenderness factor

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

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

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

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

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

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

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

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

Service/Extreme Limit States

Geotechnical Resistance

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

Bedrock Type:

Sandstone RQD ranges from 0 to 50%.

Use RQD = 25% and ϕ = 27 to 34 deg (LRFD Table C10.4.6.4-1)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

Calculate pile box area:

$A_{box} := \overrightarrow{(d \cdot b)}$ $\quad A_{box} = \begin{pmatrix} 141.8901 \\ 148.1679 \\ 198.5018 \\ 203.2318 \\ 211.5159 \end{pmatrix} \cdot in^2$

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

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

q_u for sandstone compressive strength ranges from 9700 to 25000 psi

use $\quad \sigma_{cS} := 20000 \cdot psi$

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

Spacing of discontinuities: $c := 36 \cdot in$ 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 14 x 73
HP 14 x 89
HP 14 x 117

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

$$K_{sp} = \begin{pmatrix} 0.5633 \\ 0.5594 \\ 0.5144 \\ 0.5126 \\ 0.5097 \end{pmatrix}$$

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

$$q_a := \sigma_{cS} \cdot K_{sp} \cdot d_f$$

$$q_a = \begin{pmatrix} 1622 \\ 1611 \\ 1481 \\ 1476 \\ 1468 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

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

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)}$$

$$R_p = \begin{pmatrix} 524 \\ 732 \\ 660 \\ 803 \\ 1052 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

$R_f := \phi_{stat} \cdot R_p$

$$R_f = \begin{pmatrix} 236 \\ 329 \\ 297 \\ 361 \\ 473 \end{pmatrix} \cdot kip$$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

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

$\phi := 1.0$

$R_{fse} := \phi \cdot R_p$

$$R_{fse} = \begin{pmatrix} 524 \\ 732 \\ 660 \\ 803 \\ 1052 \end{pmatrix} \cdot kip$$

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

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

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

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

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

Pile Size = 12 x 53

Assume Contractor will use a Delmag D 19-42 hammer to install 12 x 53 piles

State of Maine Dept. Of Transportation 30-Mar-2010
South Berwick Great Hill Drivability GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
530.0	44.44	4.50	14.5	9.87	22.36
531.0	44.46	4.52	14.5	9.88	22.40
532.0	44.48	4.51	14.7	9.89	22.40
533.0	44.53	4.50	14.9	9.89	22.40
534.0	44.57	4.49	15.1	9.90	22.40
535.0	44.58	4.51	15.2	9.91	22.44
536.0	44.63	4.50	15.4	9.92	22.44
537.0	44.63	4.51	15.4	9.92	22.48
538.0	44.69	4.52	15.5	9.94	22.51
539.0	44.71	4.52	15.7	9.94	22.51

Limit blow count to 15 blows per inch

$R_{dr\_12x53} := 534 \cdot kip$

Strength Limit State:

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

$R_{dr\_12x53\_strength} = 347 \cdot kip$

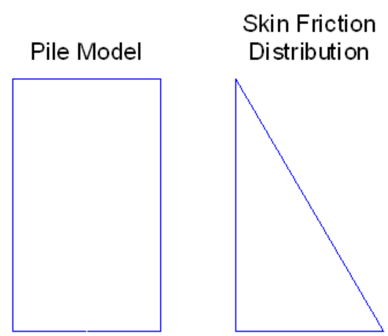
Service and Extreme Limit States: $\phi := 1.0$

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

$R_{dr\_12x53\_servext} = 534 \cdot kip$

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	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	45.00	ft
Pile Top Area	15.50	in2



Res. Shaft = 15 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag D 19-42 hammer to install 12 x 74 piles

State of Maine Dept. Of Transportation — 30-Mar-2010
South Berwick Great Hill Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
590.0	37.28	4.57	14.8	9.73	21.06
591.0	37.31	4.58	14.9	9.73	21.09
592.0	37.33	4.58	14.9	9.74	21.11
593.0	37.34	4.59	15.0	9.74	21.13
594.0	37.37	4.56	15.2	9.75	21.09
595.0	37.39	4.58	15.3	9.75	21.12
596.0	37.40	4.58	15.4	9.76	21.14
597.0	37.42	4.60	15.5	9.76	21.17
598.0	37.45	4.60	15.6	9.77	21.19
599.0	37.45	4.59	15.8	9.77	21.16

Limit blow count to 15 blows per inch

$R_{dr\_12x74} := 593 \cdot kip$

Strength Limit State:

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

$R_{dr\_12x74\_strength} = 385 \cdot kip$

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

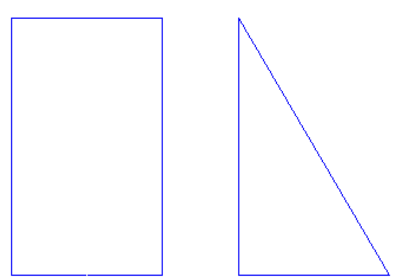
$R_{dr\_12x74\_servext} := R_{dr\_12x74} \cdot \phi$

$R_{dr\_12x74\_servext} = 593 \cdot kip$

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	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	45.00	ft
Pile Top Area	21.80	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D 19-42 hammer to install 14 x 73 piles

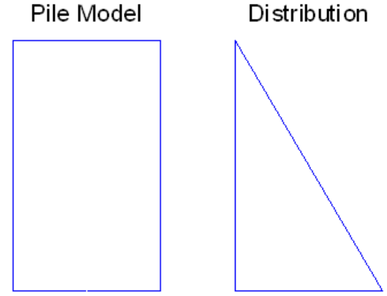
State of Maine Dept. Of Transportation — 30-Mar-2010
South Berwick Great Hill Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
585.0	37.46	4.65	14.5	9.74	21.12
586.0	37.49	4.67	14.6	9.75	21.14
587.0	37.51	4.65	14.8	9.75	21.11
588.0	37.53	4.66	14.9	9.76	21.13
589.0	37.55	4.67	15.0	9.76	21.16
590.0	37.57	4.68	15.0	9.77	21.18
591.0	37.58	4.70	15.1	9.77	21.21
592.0	37.61	4.68	15.3	9.78	21.17
593.0	37.63	4.68	15.4	9.78	21.20
594.0	37.65	4.70	15.5	9.79	21.23

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	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	45.00	ft
Pile Top Area	21.40	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Limit blow count to 15 blows per inch

$R_{dr\_14x73} := 590 \cdot kip$

Strength Limit State:

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

$R_{dr\_14x73\_strength} = 384 \cdot kip$

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

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

$R_{dr\_14x73\_servext} = 590 \cdot kip$

Pile Size = 14 x 89

Assume Contractor will use a Delmag D 36-32 hammer to install 14 x 89 piles

State of Maine Dept. Of Transportation 30-Mar-2010
South Berwick Great Hill Drivability GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
680.0	44.81	2.66	4.3	9.10	43.34
681.0	44.85	2.71	4.3	9.11	43.49
682.0	44.88	2.72	4.3	9.11	43.48
683.0	44.90	2.72	4.3	9.12	43.48
684.0	44.92	2.73	4.3	9.12	43.47
685.0	44.98	2.74	4.3	9.14	43.56
686.0	45.01	2.75	4.4	9.14	43.56
687.0	45.03	2.75	4.4	9.15	43.55
688.0	45.05	2.76	4.4	9.15	43.54
689.0	45.09	2.81	4.4	9.16	43.70

Limit stress to 45 ksi

$$R_{dr\_14x89} := 686 \cdot kip$$

Strength Limit State:

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

$$R_{dr\_14x89\_strength} = 446 \cdot kip$$

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

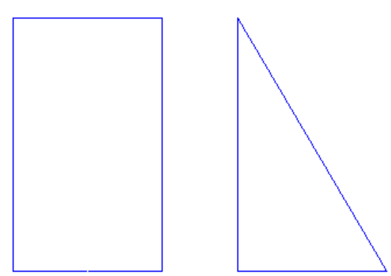
$$R_{dr\_14x89\_servext} := R_{dr\_14x89} \cdot \phi$$

$$R_{dr\_14x89\_servext} = 686 \cdot kip$$

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	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	45.00	ft
Pile Penetration	45.00	ft
Pile Top Area	26.10	in2

Pile Model Skin Friction Distribution



Res. Shaft = 15 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag D 36-32 hammer to install 14 x 117 piles

State of Maine Dept. Of Transportation — 30-Mar-2010
South Berwick Great Hill Drivability — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
885.0	44.92	3.46	6.8	9.86	44.05
886.0	44.97	3.46	6.8	9.88	44.12
887.0	44.95	3.45	6.9	9.88	44.07
888.0	44.99	3.47	6.9	9.89	44.17
889.0	44.98	3.48	6.9	9.89	44.16
890.0	45.03	3.48	6.9	9.89	44.12
891.0	45.00	3.50	6.9	9.90	44.23
892.0	45.08	3.48	7.0	9.90	44.17
893.0	45.09	3.51	7.0	9.91	44.30
894.0	45.10	3.52	7.0	9.91	44.29

Limit stress to 45 ksi

$R_{dr\_14x117} := 891 \cdot kip$

Strength Limit State:

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

$R_{dr\_14x117\_strength} = 579 \cdot kip$

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

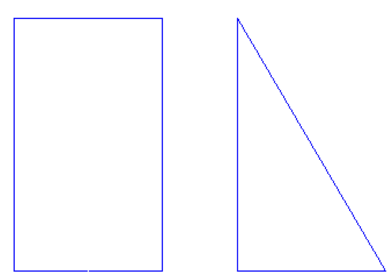
$R_{dr\_14x117\_servext} := R_{dr\_14x117} \cdot \phi$

$R_{dr\_14x117\_servext} = 891 \cdot kip$

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	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	45.00 ft
Pile Penetration	45.00 ft
Pile Top Area	34.40 in2

Pile Model — Skin Friction Distribution



Res. Shaft = 15 %
(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.

Rankine Theory - Active Earth Pressure from Maine DOT Bridge Design Guide Section 3.6.5.2 pg 3-7

For a horizontal backfill surface:

$\phi := 32 \cdot deg$

$$K_a := \tan\left(45 \cdot deg - \frac{\phi}{2}\right)^2$$

$K_a = 0.307$

Bearing Resistance - Native Soils:

Bearing resistance reported here for us in desiging any retaining walls above Q1.1 associated with the bridge replacement. The use of spread footings to support the bridge is not recommended.

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - spread footing on fill soils

Presumptive Bearing Resistance for Service Limit State ONLY

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

Type of Bearing Material: Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)

Based on corrected N-values ranging from 3 to 22 - Soils are loose to medium dense

Consistency In Place: loose

Bearing Resistance: Ordinary Range (ksf) 2 to 6

Recommended Value of Use: 3 ksf

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

Recommended Value: $3 \cdot ksf = 1.5 \cdot tsf$

Therefore: $q_{nom} := 1.5 \cdot tsf$

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

$q_{factored\_bc} := 1.5 \cdot tsf$ or $q_{factored\_bc} = 3 \cdot ksf$

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - spread footing on native soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. Footings will be embedded 5.0 feet for frost protection. $D_f := 5.0 \cdot ft$
2. Assumed parameters for fill soils: (Ref: Bowles 5th Ed Table 3-4)

   Saturated unit weight: $\gamma_s := 125 \cdot pcf$

   Dry unit weight: $\gamma_d := 120 \cdot pcf$

   Internal friction angle: $\phi_{ns} := 30 \cdot deg$

   Undrained shear strength: $c_{ns} := 0 \cdot psf$
3. Use Terzaghi strip equations as L>B
4. Effective stress analysis footing on ϕ-c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth to Groundwater table: $D_w := 5 \cdot ft$ Based on boring logs

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

Look at several footing widths

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

Terzaghi Shape factors from Table 4-1

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

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

For ϕ=28 deg

$N_c := 30.13$ $N_q := 18.4$ $N_\gamma := 15.7$

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

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

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

$$q_{nominal} = \begin{pmatrix} 4.1 \\ 4.8 \\ 5.3 \\ 5.8 \\ 6.6 \end{pmatrix} \cdot tsf$$

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

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

$$q_{factored} = \begin{pmatrix} 1.8 \\ 2.2 \\ 2.4 \\ 2.6 \\ 3 \end{pmatrix} \cdot tsf$$

Based on these footing widths

$$q_{factored} = \begin{pmatrix} 3.7 \\ 4.4 \\ 4.8 \\ 5.2 \\ 5.9 \end{pmatrix} \cdot ksf \qquad B := \begin{pmatrix} 5 \\ 8 \\ 10 \\ 12 \\ 15 \end{pmatrix} \cdot ft$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 4 ksf for walls less than 8 feet wide.
Recommend a limiting factored bearing resistance of 5 ksf for walls between 8.5 and 12 feet wide.

Settlement

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

In order to straighten out the roadway alignment, fills will be required behind both of the abutments.
Look at a simplified soil profile based on BB-SBGW-102 with greatest amount of fill.

Finished Grade

Proposed Fill - Look at 12.0 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Existing Grade

Existing Fill - fine to coarse sand

$H_1 := 11.0 \cdot ft \qquad \gamma_{sand} := 125 \cdot pcf \qquad N_{sand1} := 12$

Groundwater at top of silt

Silt/Clay - Su=350 to 800 psf (soft to medium stiff)
Total Layer height: H = 24.0 ft - divide into 6 layers

$H_2 := 24.0 \cdot ft \qquad \gamma_w := 62.4pcf$

$H_{2silt1} := 4.0 \cdot ft \qquad \gamma_{silt} := 115 \cdot pcf \qquad N_{silt1} := 6$

$H_{2silt2} := 4.0 \cdot ft \qquad N_{silt2} := 1$

$H_{2silt3} := 4.0 \cdot ft \qquad N_{silt3} := 1$

$H_{2silt4} := 4.0 \cdot ft \qquad N_{silt4} := 3$

$H_{2silt5} := 4.0 \cdot ft \qquad N_{silt5} := 4$

$H_{2silt6} := 4.0 \cdot ft \qquad N_{silt6} := 7$

$e_0 = 1.37$

$C_c = 0.6256$

$C_r = 0.0553$

Maximum Past Pressure: $\sigma'_p := 1.9 \cdot tsf$

Native Sand - fine to coarse sand, medium dense

$H_3 := 12.0 \cdot ft \qquad \gamma_{sand} := 125 \cdot pcf \qquad N_{sand2} := 25$

Bedrock - Sandstone

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name:	Great Hill Bridge	Client:	South Berwick
Project Number:	16749.00	Project Manager:	KCummings
Date:	06/16/10	Computed by:	km

Embank. slope a = 41.00(ft) Embank. width b = 55.00(ft)
p load/unit area = 1500.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 41.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	1500.00
1.00	1488.24
2.00	1475.82
3.00	1462.16
4.00	1446.81
5.00	1429.51
6.00	1410.17
7.00	1388.87
8.00	1365.80
9.00	1341.22
10.00	1315.46
11.00	1288.82
12.00	1261.63
13.00	1234.15
14.00	1206.62
15.00	1179.25
16.00	1152.19
17.00	1125.57
18.00	1099.49
19.00	1074.02
20.00	1049.22
21.00	1025.11
22.00	1001.71
23.00	979.04
24.00	957.09
25.00	935.85
26.00	915.31
27.00	895.46
28.00	876.28
29.00	857.74
30.00	839.83
31.00	822.53
32.00	805.81
33.00	789.66
34.00	774.04
35.00	758.94
36.00	744.34
37.00	730.21
38.00	716.55
39.00	703.32
40.00	690.52
41.00	678.12
42.00	666.11
43.00	654.47
44.00	643.19
45.00	632.26
46.00	621.65
47.00	611.36
48.00	601.37
49.00	591.67

at 5.5 ft
$\Delta\sigma_{zsand1} := 1419.84 \cdot psf$

at 13.0 ft
$\Delta\sigma_{zsilt1} := 1234.15 \cdot psf$

at 17.0 ft
$\Delta\sigma_{zsilt2} := 1125.57 \cdot psf$

at 21.0 ft
$\Delta\sigma_{zsilt3} := 1025.11 \cdot psf$

at 25.0 ft
$\Delta\sigma_{zsilt4} := 935.85 \cdot psf$

at 29.0 ft
$\Delta\sigma_{zsilt5} := 857.74 \cdot psf$

at 33.0 ft
$\Delta\sigma_{zsilt6} := 789.66 \cdot psf$

at 41.0 ft
$\Delta\sigma_{zsand2} := 678.12 \cdot psf$

Existing Fill $tsf := psf \cdot 1000$

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress: $\sigma_{sand1o} := \frac{H_1}{2} \cdot (\gamma_{sand})$ $\sigma_{sand1o} = 0.687 \cdot tsf$ at mid-point

Corrected Average SPT N_{60}-value (bpf) from borings $N_{sand1} = 12$

At P_o = 0.687 tsf $C_{Nsand1} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{sand1o}}\right)$ $C_{Nsand1} = 1.3589$

Corrected N-value normalized for overburden $N1_{60}$: $N1_{sand1} := C_{Nsand1} \cdot N_{sand1}$ $N1_{sand1} = 16$
Eq 10.4.6.2.4-1 LRFD

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

Bearing Capacity Index: $C1 := 62$

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

$$\Delta\sigma_{zsand1} = 1419.84 \cdot psf$$

Silt/Clay - 6 layers

Silt/Clay Layer 1:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt1o} := \left[\frac{H_{2silt1}}{2} \cdot (\gamma_{silt} - \gamma_w)\right] + H_1 \cdot (\gamma_{sand}) \qquad \sigma_{silt1o} = 1.4802 \cdot tsf$$

at mid-point

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

$$\Delta\sigma_{zsilt1} = 1234.15 \cdot psf$$

Silt/Clay Layer 2:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt2o} := \left[\frac{H_{2silt2}}{2} \cdot (\gamma_{silt} - \gamma_w)\right] + H_{2silt1} \cdot (\gamma_{silt} - \gamma_w) + H_1 \cdot (\gamma_{sand}) \qquad \sigma_{silt2o} = 1.6906 \cdot tsf$$

at mid-point

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

$$\Delta\sigma_{zsilt2} = 1125.57 \cdot psf$$

Silt/Clay Layer 3:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt3o} := \left[\frac{H_{2silt3}}{2} \cdot \left(\gamma_{silt} - \gamma_w\right)\right] + \left(H_{2silt2} + H_{2silt1}\right) \cdot \left(\gamma_{silt} - \gamma_w\right) + H_1 \cdot \left(\gamma_{sand}\right)$$

$\sigma_{silt3o} = 1.901 \cdot tsf$ at mid-point

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

$$\Delta\sigma_{zsilt3} = 1025.11 \cdot psf$$

Silt/Clay Layer 4:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt4o} := \left[\frac{H_{2silt4}}{2} \cdot \left(\gamma_{silt} - \gamma_w\right)\right] + \left(H_{2silt3} + H_{2silt2} + H_{2silt1}\right) \cdot \left(\gamma_{silt} - \gamma_w\right) + H_1 \cdot \left(\gamma_{sand}\right)$$

$\sigma_{silt4o} = 2.1114 \cdot tsf$ at mid-point

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

$$\Delta\sigma_{zsilt4} = 935.85 \cdot psf$$

Silt/Clay Layer 5:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt5o} := \left[\frac{H_{2silt5}}{2} \cdot \left(\gamma_{silt} - \gamma_w\right)\right] + \left(H_{2silt4} + H_{2silt3} + H_{2silt2} + H_{2silt1}\right) \cdot \left(\gamma_{silt} - \gamma_w\right) + H_1 \cdot \left(\gamma_{sand}\right)$$

$\sigma_{silt5o} = 2.3218 \cdot tsf$ at mid-point

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

$$\Delta\sigma_{zsilt5} = 857.74 \cdot psf$$

Silt/Clay Layer 6:

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$$\sigma_{silt6o} := \left[\frac{H_{2silt6}}{2} \cdot \left(\gamma_{silt} - \gamma_w\right)\right] + \left(H_{2silt5} + H_{2silt4} + H_{2silt3} + H_{2silt2} + H_{2silt1}\right) \cdot \left(\gamma_{silt} - \gamma_w\right) + H_1 \cdot \left(\gamma_{sand}\right)$$

$\sigma_{silt6o} = 2.5322 \cdot tsf$ at mid-point

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

$$\Delta\sigma_{zsilt6} = 789.66 \cdot psf$$

Native Sand

Determine corrected N-value normalized for overburden N1:

Calculate vertical stress:

$\sigma_{sand2o} := \frac{H_3}{2}(\gamma_{sand} - \gamma_w) + H_2 \cdot (\gamma_{silt} - \gamma_w) + H_1 \cdot (\gamma_{sand})$ $\sigma_{sand2o} = 3.013 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_{sand2} = 25$

AT P_o = 3.0 tsf $C_{Nsand2} := 0.77 \cdot \log\left(\frac{40 \cdot ksf}{\sigma_{sand2o}}\right)$ $C_{Nsand2} = 0.8648$

Corrected N-value normalized for overburden $N1_{60}$:
Eq 10.4.6.2.4-1 LRFD

$N1_{60} := C_{Nsand2} \cdot N_{sand2}$ $N1_{60} = 22$

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

Bearing Capacity Index: $C3_{sand2} := 73$

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

$\Delta\sigma_{zsand2} = 678.12 \cdot psf$

Calculate Settlement:

Fill/Sand: $\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{sand1o} + \Delta\sigma_{zsand1}}{\sigma_{sand1o}}\right)$ $\Delta H_1 = 1.0357 \cdot in$

Silt/Clay Layer 1: $\Delta H_{2silt1} := H_{2silt1} \cdot \frac{C_r}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt1o} + \Delta\sigma_{zsilt1}}{\sigma_{silt1o}}\right)$ $\Delta H_{2silt1} = 0.295 \cdot in$

Silt/Clay Layer 2: $\Delta H_{2silt2} := H_{2silt2} \cdot \frac{C_r}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt2o} + \Delta\sigma_{zsilt2}}{\sigma_{silt2o}}\right)$ $\Delta H_{2silt2} = 0.2482 \cdot in$

Silt/Clay Layer 3: $\Delta H_{2silt3} := H_{2silt3} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt3o} + \Delta\sigma_{zsilt3}}{\sigma_{silt3o}}\right)$ $\Delta H_{2silt3} = 2.3731 \cdot in$

Silt/Clay Layer 4: $\Delta H_{2silt4} := H_{2silt4} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt4o} + \Delta\sigma_{zsilt4}}{\sigma_{silt4o}}\right)$ $\Delta H_{2silt4} = 2.0187 \cdot in$

Silt/Clay Layer 5: $\Delta H_{2silt5} := H_{2silt5} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt5o} + \Delta\sigma_{zsilt5}}{\sigma_{silt5o}}\right)$ $\Delta H_{2silt5} = 1.7299 \cdot in$

Silt/Clay Layer 6: $\Delta H_{2silt6} := H_{2silt6} \cdot \frac{C_c}{1 + e_0} \cdot \log\left(\frac{\sigma_{silt6o} + \Delta\sigma_{zsilt6}}{\sigma_{silt6o}}\right)$ $\Delta H_{2silt6} = 1.4935 \cdot in$

Native Sand: $\Delta H_3 := H_3 \cdot \frac{1}{C3_{sand2}} \cdot \log\left(\frac{\sigma_{sand2o} + \Delta\sigma_{zsand2}}{\sigma_{sand2o}}\right)$ $\Delta H_3 = 0.1739 \cdot in$

Total Settlement =

$\Delta H_T := \Delta H_1 + \Delta H_{2silt1} + \Delta H_{2silt2} + \Delta H_{2silt3} + \Delta H_{2silt4} + \Delta H_{2silt5} + \Delta H_{2silt6} + \Delta H_3$

$\Delta H_T = 9.4 \cdot in$

Elastic Settlement = $\Delta H_1 + \Delta H_3 = 1.2 \cdot in$

Plastic Settlement = $\Delta H_{2silt1} + \Delta H_{2silt2} + \Delta H_{2silt3} + \Delta H_{2silt4} + \Delta H_{2silt5} + \Delta H_{2silt6} = 8.2 \cdot in$

With 8.2 inches of settlment in the clay downdrag forces will be fully developed.

Time Rate of Settlement:

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

Thickness of the silt/clay layer = $H_{siltclay} := 24.0 \cdot ft$

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

$H_{scv} := 12 \cdot ft$

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

Coefficient of consolidation from lab data: $C_v := 7.05 \cdot 10^{-7} \cdot \frac{ft^2}{sec}$ $C_v = 0.0609 \cdot \frac{ft^2}{day}$

Time rate of settlement to achieve 90% Primary Settlement

$t_{90} := \frac{T_v \cdot H_{scv}^2}{C_v}$ $t_{90} = 2004.7281 \cdot day$ $year := 365 \cdot day$

$t_{90} = 5.4924 \cdot year$

Determination of Downdrag:

Use beta method to determine downdrag

Granular soil (NavFac 7.2) $\beta_{gr} := 0.3$

Silt/Clay (Dixon & Sandford), Presumpscot formation $\beta_{clay} := 0.13$

Assumed values

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

Groundwater table at top of silt/clay layer

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

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

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

Stress from overburden material. Overburden consists of approximately 12 feet of fill on 11 feet of existing sand fill on 24 feet of marine silt/clay. Watertable is at the top of the silt/clay layer.

Additional Overburden Stress due to fill =

$\sigma_{v\_ob} := 12 \cdot ft \cdot \gamma_{sand}$ $\sigma_{v\_ob} = 1500 \cdot psf$

Effective vertical stress in middle of each layer

Total thickness of each stratum

$D_{sand} := 11 \cdot ft$ $D_{siltclay} := 24 \cdot ft$

$$\sigma'_{v\_sand} := \sigma_{v\_ob} + \frac{D_{sand}}{2} \cdot \gamma_{sand} \qquad \sigma'_{v\_sand} = 2187.5 \cdot psf$$

$$\sigma'_{v\_siltclay} := \sigma_{v\_ob} + D_{sand} \cdot \gamma_{sand} + \frac{D_{siltclay}}{2} \cdot \left(\gamma_{siltclay} - \gamma_w\right) \qquad \sigma'_{v\_siltclay} = 3506.2 \cdot psf$$

Pile parameters:

Look at these piles:

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

Note: All matrices set up in this order

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

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

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

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

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

Magnitude of maximum downdrag, considered over entire clay thickness

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

$$Q_{dd} = \begin{pmatrix} 72 \\ 74 \\ 85 \\ 86 \\ 88 \end{pmatrix} \cdot kip$$

Based on past practice in the estimation of downdrag forces in Maine, a downdrag load factor of 1.0 is recommended

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:
South Berwick, Maine
DFI = 1100 degree-days

From the lab testing: fill soils are coarse grained assume a water content = ~20%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1100 frost penetration = 5738 inches

$\text{Frost\_depth} := 57.8\text{in} \qquad \text{Frost\_depth} = 4.8167 \cdot \text{ft}$

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Sanford

--- ModBerg Results ---

Project Location: Sanford 2 NNW, Maine

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

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	60.0	20.0	125.0	34	46	3.8	1.9	3,600
2-Fine	3.3	30.0	115.0	37	54	2.0	1.3	4,968

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

Total Depth of Frost Penetration = 5.27 ft = 63.3 in.

Use Modberg Frost Depth = 5.0 feet for design

Seismic:

South Berwick Great Hill Bridge 16749.00
Date and Time: 4/1/2010 1:40:54 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 03908
Zip Code Latitude = 43.233800
Zip Code Longitude = -070.791400
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.101	PGA - Site Class B
0.2	0.192	Ss - Site Class B
1.0	0.045	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 03908
Zip Code Latitude = 43.233800
Zip Code Longitude = -070.791400
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class E - Fpga = 2.49, Fa = 2.50, Fv = 3.50
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.251	As - Site Class E
0.2	0.481	SDs - Site Class E
1.0	0.159	SD1 - Site Class E

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

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

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

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

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

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

Appendix D

Special Provisions

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

Add the following paragraph to Section 610.02:

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

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

Add the following paragraph to Section 610.032.a.

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

Add the following paragraph to Section 610.032.b:

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

Add the following to Section 610.032:

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

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

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

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

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

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

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces
Subrounded: Particles have nearly plane sides but have well-rounded corners and edges
Rounded: Particles have smoothly curved sides and no edges

SPECIAL PROVISION
SECTION 635
PREFABRICATED CONCRETE MODULAR GRAVITY WALL

The following replaces Section 635 in the Standard Specifications in its entirety:

635.01 Description. This work shall consist of the construction of a prefabricated modular reinforced concrete gravity wall in accordance with these specifications and in reasonably close conformance with the lines and grades shown on the plans, or established by the Resident.

Included in the scope of the Prefabricated Concrete Modular Gravity Wall construction are**:** all grading necessary for wall construction, excavation, compaction of the wall foundation, backfill, construction of leveling pads, placement of geotextile, segmental unit erection, and all incidentals necessity to complete the work.

The Prefabricated Concrete Modular Gravity Wall design shall follow the general dimensions of the wall envelope shown in the contract plans. The top of the leveling pad shall be located at or below the theoretical leveling pad elevation. The minimum wall embedment shall be at or below the elevation shown on the plans. The top of the face panels shall be at or above the top of the panel elevation shown on the plans.

The Contractor shall require the design-supplier to supply an on-site, qualified experienced technical representative to advise the Contractor concerning proper installation procedures. The technical representative shall be on-site during initial stages of installation and thereafter shall remain available for consultation as necessary for the Contractor or as required by the Resident. The work done by this representative is incidental.

635.02 Materials. Materials shall meet the requirements of the following subsections of Division 700 - Materials:

Gravel Borrow	703.20
Preformed Expansion Joint Material	705.01
Reinforcing Steel	709.01
Structural Pre-cast Concrete Units	712.061
Drainage Geotextile	722.02

The Contractor is cautioned that all of the materials listed are not required for every Prefabricated Concrete Modular Gravity Wall. The Contractor shall furnish the Resident a Certificate of Compliance certifying that the applicable materials comply with this section of the specifications. Materials shall meet the following additional requirements:

Concrete Units:

Tolerances. In addition to meeting the requirements of 712.061, all prefabricated units shall be manufactured with the following tolerances. All units not meeting the listed tolerances will be rejected.

1. All dimensions shall be within (edge to edge of concrete) ±3/16 inch.

2. Squareness. The length differences between the two diagonals shall not exceed 5/16 inch.
3. Surface Tolerances. For steel formed surfaces, and other formed surface, any surface defects in excess of 0.08 inch in 4 feet will be rejected. For textured surfaces, any surface defects in excess of 5/16 inch in 5 feet shall be rejected.

Joint Filler. (where applicable) Joints shall be filled with material approved by the Resident and supplied by the approved Prefabricated Concrete Modular Gravity Wall supplier. 4 inch wide, by 0.5 inch thick preformed expansion joint filler shall be placed in all horizontal joints between facing units. In all vertical joints, a space of 0.25 inch shall be provided. All Preformed Expansion Joint Material shall meet the requirements of subsection 502.03.

Woven Drainage Geotextile. Woven drainage geotextile 12 inches wide shall be bonded with an approved adhesive compound to the back face, covering all joints between units, including joints abutting concrete structures. Geotextile seam laps shall be 6 inches minimum. The fabric shall be secured to the concrete with an adhesive satisfactory to the Resident. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Concrete Shear Keys. (where applicable) Shear keys shall have a thickness at least equal to the pre-cast concrete stem.

Concrete Leveling Pad. Cast-in-place concrete shall be Fill Concrete conforming to the requirements of Section 502 Structural Concrete. The horizontal tolerance on the surface of the pad shall be 0.25 inch in 10 feet. Dimensions may be modified per the wall supplier's recommendations, with written approval of the Resident.

Backfill and Bedding Material. Bedding and backfill material placed behind and within the reinforced concrete modules shall be gravel borrow conforming to the requirements of Subsection 703.20. The backfill materials shall conform to the following additional requirements: the plasticity index (PI) as determined by AASHTO T90 shall not exceed 6. Compliance with the gradation and plasticity requirements shall be the responsibility of the Contractor, who shall furnish a copy of the backfill test results prior to construction.

The backfilling of the interior of the wall units and behind the wall shall progress simultaneously. The material shall be placed in layers not over 8 inches in depth, loose measure, and thoroughly compacted by mechanical or vibratory compactors. Puddling for compaction will not be allowed.

Materials Certificate Letter. The Contractor, or the supplier as his agent, shall furnish the Resident a Materials Certificate Letter for the above materials, including the backfill material, in accordance with Section 700 of the Standard Specifications. A copy of all test results performed by the Contractor or his supplier necessary to assure contract compliance shall also be furnished to the Resident. Acceptance will be based upon the materials Certificate Letter, accompanying test reports, and visual inspection by the Resident.

635.03 Design Requirements. The Prefabricated Concrete Modular Gravity Wall shall be designed and sealed by a licensed Professional Engineer registered in accordance with the laws of the State of Maine. The design to be performed by the wall system supplier shall be in accordance with AASHTO LRFD Bridge Design Specifications, current edition, except as required herein. Design shall consider Strength and Extreme Limit States. Thirty days prior to beginning construction of the wall, the design computations shall be submitted to the Resident for review by the Department. Design calculations that consist of computer generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. Design calculations shall provide thorough documentation of the sources of equations used and material properties. The design by the wall system supplier shall consider the stability of the wall as outlined below:

A. Stability Analysis:
   1. Overturning: Location of the resultant of the reaction forces shall be within the middle one-half of the base width.
   2. Sliding: $R_R \geq \gamma_{p(max)} \cdot (EH+ES)$
      Where: R_R = Factored Sliding Resistance
      $\gamma_{p(max)}$ = Maximum Load Factor
      EH = Horizontal Earth Pressure
      ES = Earth Surcharge (as applicable)
   4. Bearing Pressure: $q_R \geq$ Factored Bearing Pressure
      Where: q_R = Factored Bearing Resistance, as shown on the plans
      Factored Bearing Pressure = Determined considering the applicable loads and load factors which result in the maximum calculated bearing pressure.
   5. Pullout Resistance: Pullout resistance shall be determined using nominal resistances and forces. The ratio of the sum of the nominal resistances to the sum of the nominal forces shall be greater than, or equal to, 1.5.

   Traffic impact loads transmitted to the wall through guardrail posts shall be calculated and applied in compliance with LRFD Section 11, where Article 11.10.10.2 is modified such that the upper 3.5 feet of concrete modular units shall be designed for an additional horizontal load of $\gamma P_{H1,}$ where γP_{H1}=300 lbs per linear foot of wall.

B. Backfill and Wall Unit Soil Parameters. For overturning and sliding stability calculations, earth pressure shall be assumed acting on a vertical plane rising from the back of the lowest wall stem. For overturning, the unit weight of the backfill within the wall units shall be limited to 96 pcf. For sliding analyses, the unit weight of the backfill within the wall units can be assumed to be 120 pcf. Both analyses may assume a friction angle of 34 degrees for backfill within the wall units.

   These unit weights and friction angles are based on a wall unit backfill meeting the requirements for select backfill in this specification. Backfill behind the wall units shall be assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. The friction angle of the foundation soils shall be assumed to be 30 degrees unless otherwise noted on the plans.

C. Internal Stability. Internal stability of the wall shall be demonstrated using accepted methods, such as Elias' Method, 1991. Shear keys shall not contribute to pullout resistance. Soil-to-soil frictional component along stem shall not contribute to pullout resistance. The failure plane used to determine pullout resistance shall be found by the Rankine theory only for vertical walls with level backfills. When walls are battered or with backslopes > 0 degrees are considered, the angle of the failure plane shall be per Jumikus Method. For computation of pullout force, the width of the backface of each unit shall be no greater than 4.5 feet. A unit weight of the soil inside the units shall be assumed no greater than 120 pcf when computing pullout. Coulomb theory may be used.

D. External loads which affect the internal stability such as those applied through piling, bridge footings, traffic, slope surcharge, hydrostatic and seismic loads shall be accounted for in the design.

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

F. Stability During Construction. Stability during construction shall be considered during design, and shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, Extreme Limit State.

G. Hydrostatic forces. Unless specified otherwise, when a design high water surface is shown on the plans at the face of the wall, the design stresses calculated from that elevation to the bottom of wall must include a 3 feet minimum differential head of saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

H. Design Life. The wall design life shall be a minimum of 75 years.

I. Not more than two vertically consecutive units shall have the same stem length, or the same unit depth. Walls with units with extended height curbs shall be designed for the added earth pressure. A separate computation for pullout of each unit with extended height curbs, or extended height coping, shall be prepared and submitted in the design package described above.

635.04 Submittals. The Contractor shall supply wall design computations, wall details, dimensions, quantities, and cross sections necessary to construct the wall. Thirty (30) days prior to beginning construction of the wall, the design computations and wall details shall be submitted to the Resident for review. The fully detailed plans shall be prepared in conformance with Subsection 105.7 of the Standard Specifications and shall include, but not be limited to the following items:

A. A plan and elevation sheet or sheets for each wall, containing the following: elevations at the top of leveling pads, the distance along the face of the wall to all steps in the leveling pads, the designation as to the type of prefabricated module, the distance along the face of the wall to where changes in length of the units occur, the location of the original and final ground line.

B. All details, including reinforcing bar bending details, shall be provided. Bar bending details shall be in accordance with Department standards.

C. All details for foundations and leveling pads, including details for steps in the leveling pads, as well as allowable and actual maximum bearing pressures shall be provided.

D. All prefabricated modules shall be detailed. The details shall show all dimensions necessary to construct the element, and all reinforcing steel in the element.

E. The wall plans shall be prepared and stamped by a Professional Engineer. Four sets of design drawings and detail design computations shall be submitted to the Resident.

F. Four weeks prior to the beginning of construction, the contractor shall supply the Resident with two copies of the design-supplier's Installation Manual. In addition, the Contractor shall have two copies of the Installation Manual on the project site.

635.05 Construction Requirements

Excavation. The excavation and use as fill disposal of all excavated material shall meet the requirements of Section 203 -- Excavation and Embankment, except as modified herein.

Foundation. The area upon which the modular gravity wall structure is to rest, and within the limits shown on the submitted plans, shall be graded for a width equal to, or exceeding, the length of the module. Prior to wall and leveling pad construction, this foundation material shall be compacted to at least 95 percent of maximum laboratory dry density, determined using AASHTO T180, Method C or D. Frozen soils and soils unsuitable or incapable of sustaining the required compaction, shall be removed and replaced.

A concrete leveling pad shall be constructed as indicated on the plans. The leveling pad shall be cast to the design elevations as shown on the plans, or as required by the wall supplier upon written approval of the Resident. Allowable elevation tolerances are +0.01 feet and -0.02 feet from the design elevations. Leveling pads which do not meet this requirement shall be repaired or replaced as directed by the Resident at no additional cost to the Department. Placement of wall units may begin after 24 hours curing time of the concrete leveling pad.

Method and Equipment. Prior to erection of the Prefabricated Concrete Modular Gravity Wall, the Contractor shall furnish the Resident with detailed information concerning the proposed construction method and equipment to be used. The erection procedure shall be in accordance with the manufacturer's instructions. Any pre-cast units that are damaged due to handling will be replaced at the Contractor's expense.

Installation of Wall Units. A field representative from the wall system being used shall be available, as needed, during the erection of the wall. The services of the representative shall be at no additional cost to the Department. Vertical and horizontal joint fillers shall be installed as shown on the plans.

The maximum offset in any unit joint shall be 3/4 inch. The overall vertical tolerance of the wall, plumb from top to bottom, shall not exceed 1/2 inch per 10 feet of wall height. The prefabricated wall units shall be installed to a tolerance of plus or minus 3/4 inch in 10 feet in vertical alignment and horizontal alignment.

Select Backfill Placement. Backfill placement shall closely follow the erection of each row of prefabricated wall units. The Contractor shall decrease the lift thickness if necessary to obtain the specified density. The maximum lift thickness shall be 8 inches (loose). Gravel borrow backfill shall be compacted in accordance with Subsection 203.12 except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180 Method C or D. Backfill compaction shall be accomplished without disturbance or displacement of the wall units. Sheepsfoot rollers will not be allowed. Whenever a compaction test fails, no additional backfill shall be placed over the area until the lift is recompacted and a passing test achieved.

The moisture content of the backfill material prior to and during compaction shall be uniform throughout each layer. Backfill material shall have a placement moisture content less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift. The optimum moisture content shall be determined in accordance with AASHTO T180, Method C or D. At the end of the day's operations, the Contractor shall shape the last level of backfill so as to direct runoff of rain water away from the wall face.

635.06 Method of Measurement. Prefabricated Concrete Modular Gravity Wall will be measured by the square meter of front surface not to exceed the dimensions shown on the contract plans or authorized by the Resident. Vertical and horizontal dimensions will be from the edges of the facing units. No field measurements for computations will be made unless the Resident specifies, in writing, a change in the limits indicated on the plans.

635.07 Basis of Payment. The accepted quantity of Prefabricated Concrete Modular Gravity Retaining Wall will be paid for at the contract unit price per square meter complete in place. Payment shall be full compensation for furnishing all labor, equipment and materials including excavation, foundation material, backfill material, pre-cast concrete units hardware, joint fillers, woven drainage geotextile, cast-in-place coping or traffic barrier and technical field representative. Cost of cast-in-place concrete for leveling pad will not be paid for separately, but will be considered incidental to the Prefabricated Concrete Modular Gravity Wall.

There will be no allowance for excavating and backfilling for the Prefabricated Concrete Modular Gravity Wall beyond the limits shown on the approved submitted plans, except for excavation required to remove unsuitable subsoil in preparation for the foundation, as approved by the Resident. Payment for excavating unsuitable material shall be full compensation for all costs of pumping, drainage, sheeting, bracing and incidentals for proper execution of the work.

Payment will be made under:

Pay Item	Pay Unit
635.14 Prefabricated Concrete Modular Gravity Wall	Square Foot