

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

**GEOTECHNICAL DESIGN REPORT**

*For the Replacement of:*

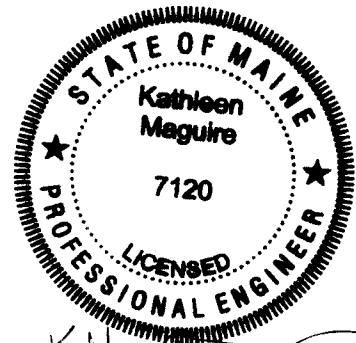
**BOURNE AVENUE BRIDGE  
OVER STEVENS BROOK  
WELLS, MAINE**

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A handwritten signature in black ink, appearing to read "Kathleen Maguire", written over the bottom portion of the professional seal.

York County  
PIN 15611.00

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## GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of the Bourne Avenue Bridge over Stevens Brook in Wells, Maine. The proposed replacement bridge will consist of a single span galvanized steel or precast concrete superstructure founded on H-pile supported integral abutments with 90 degree return wings. The bridge will be widened to the north resulting in a width of 25 feet. 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 should be fitted with driving points to protect the tips and improve penetration. 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. 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. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading. For the strength limit state and the service and extreme limit states the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances. Therefore, the axial drivability resistance governs the design. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The maximum factored axial pile load should be shown on the plans.

**Downdrag** – Settlement analyses indicate that approximately ½ inch of settlement will occur at the site due to the placement 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 70 and 80 kips depending upon pile size. It is recommended that a load factor,  $\gamma_p=1.0$ , be applied to downdrag forces in cohesive and cohesionless downdrag zones.

**Stub Abutments** - Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations specified in AASHTO LRFD Bridge Design Specifications 4<sup>th</sup> Edition (LRFD) Articles 3.4.1 and 11.5.5. In designing for passive earth pressure associated with integral abutments, the Rankine state is recommended. Abutment design shall include a drainage system to intercept any water. To avoid water intrusion behind the abutment, the approach slab should connect directly to the abutment.

**Independent Return Wings/Retaining Walls** - The return wings for both abutments will be 90 degree return wings and will be independent of the stub abutment. The independent return wings will be Precast Concrete Modular Gravity (PCMG) walls supported on spread footings. The PCMG return wings will retain approach fills, provide lateral support to the corners of the pile group and minimize slope impacts on the Rachel Carson National Wildlife Refuge property. The PCMG walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The design of independent return wings founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding, modular unit pullout and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood. The independent return wings shall be designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure. The PCMG Wall shall be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface. Return wing designs shall include a drainage system to intercept any water. The PCMG wall shall consist of Class "LP" concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor. The high water elevation shall be indicated on the retaining wall plans

**Bearing Resistance** - It is anticipated that the project independent return wingwalls will be founded on the native soils at the site. These elements will need to be designed to provide stability against bearing capacity failure with applicable permanent and transient loads. Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 12 ksf. A factored bearing resistance of 4 ksf may be used when analyzing the service limit state and for preliminary sizing of footings.

**Scour and Riprap**- The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments and independent return wings. For scour protection, any footings for independent return wings, which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Riprap shall be 3 feet thick.

**Settlement** - Evaluation of the potential settlement due to the placement of fill resulted in approximately ½ inch of settlement. Approximately half of this settlement is consolidation settlement within the compressible silty 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 16 years) and may require attention by a maintenance crew.

**Frost Protection** - Any foundation placed on granular subgrade 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** - In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively. The Bourne Avenue Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important, and since the bridge construction costs should not exceed \$10 million the bridge is not classified as a major structure. In conformance with the MaineDOT Bridge Design Guide, these criteria eliminate the requirement to design the bridge substructures for seismic earth loads.

## 1.0 INTRODUCTION

A subsurface investigation for the replacement of the Bourne Avenue Bridge over Stevens Brook (a tributary to the Ogunquit River) in Wells, York County, Maine has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing bridge was constructed in 1942 and consists of a three span, steel girder superstructure with a concrete deck supported on timber pile supported abutments and two timber pile bent piers. The existing bridge is made up of non-continuous three 27 foot spans. The existing superstructure has a curb-to-curb width of 23 feet. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck and superstructure are in “poor” (rating of 4) condition due to excessive deck cracking, corroding bottom flanges of steel girders, and apparent damage to timber piling and pile caps due to age, marine attack, and ice. Year 2007 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 47.8. Year 2007 Bridge Inspection records assign the substructures a rating of 5, or “fair”. Maintenance reports indicate that the timber pile bents show ice damage and splintering. Cracking of the timber pile caps is noted. The salt water marsh surrounding the bridge is designated as a wildlife refuge and is owned by the Rachel Carson Wildlife Refuge. The existing bridge will be closed to traffic during construction.

The proposed bridge will consist of a single span galvanized steel or precast concrete superstructure founded on H-pile supported integral abutments with 90 degree return wings. The proposed bridge will have a span of 86 feet. The bridge will be widened to the north resulting in a width of 25 feet curb to curb to match the approach width. It is understood that the existing bridge will be removed in its entirety. The existing timber pile pier bents within the brook shall be removed to 1 foot below streambed.

The horizontal alignment of the proposed bridge will be offset by 1 foot and 3 inches to the north to maximize shoulder width for pedestrian access and safety. In order to remain within the existing right-of-way the north side of Abutment No. 2 will require a retaining wall. The vertical alignment of the proposed bridge will be raised 7 inches at the west approach to improve drainage conditions and provide a crest vertical curve to the bridge.

## 2.0 GEOLOGIC SETTING

The Bourne Avenue Bridge in Wells crosses Stevens Brook approximately 0.5 miles east of US Route 1 as shown on *Sheet 1 - Location Map* found at the end of this report. Stevens Brook flows in a southerly direction to the Atlantic Ocean.

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 swamp, marsh and bog deposits. Soils in the site area are generally comprised of peat, muck, clay, silt and sand. The unit generally is deposited in areas where the topography is flat. These soils are

generally formed by accumulation of sediments and organic material in depressions and other poorly drained areas. Additional geologic units mapped nearby the site to the east are beach deposits which are generally comprised of sand and gravel.

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Silurian-Precambrian, 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-WSB-101 was drilled behind the location of Abutment No. 1 (west). Test boring BB-WSB-102 was drilled at the location of a possible pier. Test boring BB-WSB-103 was drilled behind the location of Abutment No. 2 (east). The exploration locations and an interpretive subsurface profile depicting the site stratigraphy are shown on *Sheet 2 - Boring Location and Interpretive Subsurface Profile* both found at the end of this report. The borings were drilled on between March 11 and April 3, 2008 using the Maine Department of Transportation (MaineDOT) drill rig. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on *Sheets 3 and 4 - Boring Logs* found end of this report.

The borings were drilled using driven cased wash boring and solid stem auger techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. The MaineDOT drill rig is newly equipped with a CME automatic hammer to drive the split spoon. The hammer was calibrated by MaineDOT in August of 2007 and was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.77 to the raw field N-values. This hammer efficiency factor (0.77) and both the raw field N-value and the corrected N-value are shown on the boring logs.

In-situ vane shear tests were made where possible in soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ 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, identified field and laboratory testing requirements and logged the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

## 4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of ten (10) standard grain size analyses, twenty (20) grain size analysis with hydrometer, sixteen (16) Atterberg Limits test, six (6) consolidation tests and six (6) standard tube openings. Laboratory test results are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on *Sheets 3 and 4 - Boring Logs* found at the end of this report.

## 5.0 SUBSURFACE CONDITIONS

The general soil stratigraphy encountered at the abutments consisted of fill, overlying sand, overlying silty clay overlying glacial till underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is show on *Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile* found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in detail:

**Fill.** Beneath the pavement, a layer of fill materials was encountered behind the abutments. This layer was found to be brown, damp to wet, fine to coarse sand, with little to trace silt and some gravel. The thickness of the fill layer ranged from approximately 9.0 feet in boring BB-WSB-101 to approximately 7.5 feet in boring BB-WSB-103. Corrected SPT N-values in the fill layer ranged from 8 to 19 blows per foot (bpf) indicating that the soil is loose to medium dense in consistency. Water contents from two (2) samples obtained within this layer range from approximately 5% to 7%. Two (2) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-1-b by the AASHTO Classification System and a SW-SM by the Unified Soil Classification System.

**Sand.** Beneath the fill layer a layer of sand was encountered. This layer was found to be brown to grey, wet, fine, fine to medium and fine to coarse sand, with some to trace silt and little to trace gravel. One sample from the unit contained traces of wood and roots. The thickness of the sand layer ranged from approximately 17.5 feet in boring BB-WSB-102 to approximately 25.0 feet in boring BB-WSB-103. Corrected SPT N-values in the sand layer ranged from 4 to 36 bpf indicating that the soil is loose to dense in consistency. Water contents from eight (8) samples obtained within this layer range from approximately 17% to 33%. Nine (9) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-2-4 or A-3 by the AASHTO Classification System and a SM, SP, SM-SP, or SP-SC by the Unified Soil Classification System.

**Silty Clay.** Beneath the sand a layer of silty clay was encountered in all of the borings. This layer was found to be grey, wet, silty clay, with little trace sand in layers and trace gravel. The thickness of the silty clay layer ranged from approximately 30.0 feet in boring BB-WSB-102 to approximately 34.1 feet in boring BB-WSB-103. Vane shear testing conducted within the silty clay layer showed undrained shear strengths ranging from approximately 402 pounds per square foot (psf) to 824 psf while the remolded shear strengths ranged from approximately 60 psf to 192 psf. These shear strength values indicate that the undisturbed silty clay is soft to medium stiff in consistency. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the clayey silt was determined to have sensitivities



ranging from approximately 3.3 to 7.8 and is classified as moderately sensitive to sensitive. Water contents from fifteen (15) samples obtained within the silty clay layer range from approximately 30% to 43%. Sixteen (16) grain size analyses with hydrometer conducted on samples from this layer indicate that the soil is classified as an A-6, A-4, or A-7-6 by the AASHTO Classification System and a CL or CL-ML by the Unified Soil Classification System.

The following table summarizes the results of the Atterberg Limits tests made from samples of the silty clay:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-WSB-101 7D/A	Silty Clay	29.9	30	20	10	0.99
BB-WSB-101 8D	Silty Clay	34.3	30	21	9	1.48
BB-WSB-101 1U	Silty Clay	41.8	34	21	13	1.60
BB-WSB-101 9D	Silty Clay	33.5	34	22	12	0.96
BB-WSB-101 2U	Silty Clay	41.7	38	22	16	1.23
BB-WSB-102 5D	Silty Clay	29.9	30	18	12	0.99
BB-WSB-102 7D	Silty Clay	33.1	34	22	12	0.93
BB-WSB-102 8D	Silty Clay	35.4	36	22	14	0.96
BB-WSB-102 10D	Silty Clay	36.9	34	22	12	1.24
BB-WSB-103 1U	Silty Clay	41.5	33	22	11	1.77
BB-WSB-103 7D	Silty Clay	35.2	33	22	11	1.20
BB-WSB-103 2U	Silty Clay	42.8	32	20	12	1.90
BB-WSB-103 8D	Silty Clay	38.3	40	22	18	0.91
BB-WSB-103 3U	Silty Clay	40.5	34	22	12	1.54
BB-WSB-103 9D	Silty Clay	34.0	37	21	16	0.81
BB-WSB-103 4U	Silt	21.5	Non Plastic			

Interpretation of these results indicates that the silty clay ranges from being on the verge of becoming a viscous liquid to slightly over-consolidated. For eight (8) samples the natural water content is equal to or exceeds the liquid limit and the liquidity index exceeds 1, indicating that the silty clay is on the verge of becoming a viscous liquid. These soils have a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as “quick”.

Six (6) one-dimensional consolidation tests were conducted on tube samples taken from various depths within the silty clay layer. The results of these tests were used to calculate the anticipate settlements at the site and are included in Appendix B – Laboratory Data.

**Glacial Till.** Beneath the silty clay layer a layer of glacial till was encountered. This layer was found to be grey, wet, fine and fine to coarse sand, with some to trace gravel, some to trace silt, and little to trace clay. The thickness of the glacial till layer ranged from

approximately 2.4 feet in boring BB-WSB-101 to approximately 10.6 feet in boring BB-WSB-103. Corrected SPT N-values in the glacial till layer ranged from 5 to 50 bpf indicating that the soil is loose to dense in consistency. Water contents from four (4) samples obtained within this layer range from approximately 12% to 22%. Four (4) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-2-4 or A-4 by the AASHTO Classification System and a SM, SP-SM, or SC-SM by the Unified Soil Classification System.

**Bedrock.** Bedrock was encountered and cored in all of the borings. The following table presents the bedrock findings:

Boring Number/ Location	Depth to Bedrock	Bedrock Elevation	RQD
BB-WSB-101/ Abutment No. 1	66.5 feet	-57.3 feet	39 - 60%
BB-WSB-102/ Pier	56.5 feet	-58.7 feet	50 - 72%
BB-WSB-103/ Abutment No. 2	77.2 feet	-68.0 feet	45 - 46%

The bedrock at the site can be identified as grey and white, fine-grained, sandstone. A diorite pluton was encountered in boring BB-WSB-101. The bedrock is a part of the Kittery Formation. The bedrock generally shows very little weathering and is hard. The upper portion of the bedrock has more joints that are not healed. The RQD of the bedrock ranged from 39 to 72% indicating a rock of poor to fair quality.

## 6.0 FOUNDATION ALTERNATIVES

The subsurface conditions encountered at the site indicate that the bridge location is underlain by a significant compressible silty clay layer. Due to the soft nature and depth of the soils, shallow foundations were not considered for use at the site. The following foundation alternatives are considered viable:

- Driven H-pile supported integral abutments
- Drilled shafts

The Preliminary Design Report (PDR) prepared for the project considers both one span and two span structures with both pre-cast concrete and galvanized steel superstructures. All of the alternatives discussed in the PDR include pile supported substructures. Drilled shafts are likely more expensive and have not been pursued. The recommended alternative chosen in the PDR is to replace the bridge with a single span structure founded on driven H-pile supported integral abutments.

## 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 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. 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 the following data:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Rock Quality Designation	Estimated Pile Length
Abutment #1 BB-WSB-101	1.8 feet	66.5 feet	-57.3 feet	39 - 60%	60 feet
Abutment #2 BB-WSB-103	1.8 feet	77.2 feet	-68.0 feet	45 - 46%	70 feet

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 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. The design flood scour is defined in AASHTO LRFD Bridge Design Specifications 4<sup>th</sup> 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.

#### 7.1.1 Strength Limit State

The nominal structural compressive resistance ( $P_n$ ) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The H-piles are fully embedded and  $\lambda$  shall be taken as 0. The factored structural axial compressive resistances of the four proposed H-pile sections were calculated using a resistance factor,  $\phi_c$ , of 0.60.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods and the FHWA computer program Driven. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor,  $\phi_{stat}$ , of 0.45 for both end bearing and skin friction.

The drivability of the four proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is  $\phi_{dyn} = 0.65$ . Table 10.5.5.2.3-3 requires that no less than three to four dynamic tests be conducted for sites with low to medium variability. Per LRFD Article 10.5.5.2.3 the resistance factor 0.65 is reduced by 20% since it is applied to a nonredundant pile group, i.e., there are less than 5 piles in a group. This results in a resistance factor,  $\phi_{dyn}$ , of 0.52.

The calculated factored axial compressive structural, geotechnical and drivability resistances of the four proposed H-pile sections for each abutment are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this document.

**Factored Axial Resistances for Abutment Piles at the Strength Limit State**

Pile Section	Factored Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability	Governing Resistance
HP 12 x 53	465	322	218	218
HP 14 x 73	642	431	338	338
HP 14 x 89	783	440	369	369
HP 14 x 117	1032	457	425	425

The factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances and therefore, the factored axial drivability resistance governs the design.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor  $\phi_c = 0.7$  and the flexural resistance factor  $\phi_f = 1.0$  shall be applied to the combined axial and flexural resistance of the pile in the interaction equation.

For the strength limit state, the combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2. The structural designer should evaluate the capacity of the pile in combined axial load and flexure when the loads and moments are calculated.

### 7.1.2 Service/Extreme Limit States

For the service and extreme limit states resistance factors of 1.0 are recommended for structural, geotechnical and drivability pile resistances.

The calculated factored axial structural, geotechnical and drivability resistances of the four proposed H-pile sections for each abutment are summarized in the table below. Supporting calculations are included in Appendix C- Calculations found at the end of this document.

#### Factored Axial Resistances for Abutment Piles at the Service/Extreme Limit States

Pile Section	Factored Resistance (kips)			
	Structural Resistance	Geotechnical Resistance	Drivability	Governing Resistance
HP 12 x 53	775	715	420	420
HP 14 x 73	1070	957	650	650
HP 14 x 89	1305	978	710	710
HP 14 x 117	1720	1015	817	817

The factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances and therefore, the factored axial drivability resistance governs the design.

### 7.1.3 Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.52. The maximum factored pile load should be shown on the plans. If three to four piles are dynamically tested and if there are a minimum of five piles per group, the resistance factor may be increased by 20 percent to 0.65. Calculations for the pile resistance required by a drivability wave equation analysis are included the Appendix C- Calculations.

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

## 7.2 Downdrag

Settlement analyses discussed later in this report indicate that approximately ½ inch of settlement will occur at the site due to the placement of a maximum of 7 inches of fill at the site. 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  $\beta$  factors obtained from full scale tests. The calculated downdrag values are:

Pile Section	Strength Limit State Unfactored Downdrag Load (DD) (Kips)
HP 12 x 53	70
HP 14 x 73	80
HP 14 x 89	80
HP 14 x 117	80

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

## 7.3 Stub Abutments and Return Wings

Integral stub abutments and independent return wings 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 abutments at the strength limit state shall consider the pile group and structural failure. The design of independent return wings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood.

The independent return wings shall be designed as unrestrained meaning that they are free to rotate at the top in an active state of earth pressure. Earth loads shall be calculated using as active earth pressure coefficient,  $K_a$ , calculated using Rankine Theory for cantilever return wings and Coulomb Theory for gravity shaped structures. See *Sheet 5 - Rankine and Coulomb Active Earth Pressure Coefficients* at the end of this report for guidance in calculating these values. Additional lateral earth pressure due to construction surcharge or live load surcharge is required per section 3.6.8 of the MaineDOT BDG for the independent return wings if an approach slab is not specified. Use of an approach slab may be required per the MaineDOT BDG Sections 5.4.2.10 and 5.4.4.

The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ( $h_{eq}$ ) taken from the table 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 $\geq$ 1 foot
5	5.0	2.0
10	3.5	2.0
$\geq 20$	2.0	2.0

Additional horizontal loads due to impact on flexible post and beam barriers should be distributed to the upper portions of independent return wings.

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows:  $\phi = 32$  degrees,  $\gamma = 125$  pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface. A sliding resistance factor of  $\phi_{\tau}=0.9$  shall be applied to the nominal sliding resistance of precast independent return wings found on spread footings on sand.

Integral abutments should be designed to withstand a passive earth pressure state. In designing for passive earth pressure associated with integral abutments, the Rankine state is recommended. Experience in designing integral abutments has shown that the use of the Coulomb passive earth pressure,  $K_p=6.89$ , may result in uneconomical abutment sections. For this reason, consideration may be given to using a Rankine passive earth pressure,  $K_p=3.25$ , when designing integral abutments.

The return wings for both abutments will be 90 degree return wings and will be independent of the stub abutment. The independent return wings will be supported on spread footings. The design of independent return wings founded on spread footings at the strength limit state shall consider nominal bearing resistance, overturning, lateral sliding and structural failure. Strength limit state design shall also consider foundation resistance after scour due to the design flood. The independent return wings shall be designed as unrestrained, meaning that they are free to rotate at the top in an active state of earth pressure. The Rankine active earth pressure coefficient of  $K_a = 0.307$  is recommended.

All abutment and independent return wing designs shall include a drainage system to intercept any water. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage, of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments with weep holes will provide adequate drainage. To avoid water intrusion behind the abutment, the approach slab should connect directly 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.

## 7.4 Bearing Resistance

It is anticipated that the project independent return wingwalls will be founded on the native soils at the site. These elements will need to be designed to provide stability against bearing capacity failure. Applicable permanent and transient loads are specified in LFRD Articles 3.4.1 and 11.5.5.

Bearing resistance for any structure founded on the native soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 12 ksf. The bearing resistance factor,  $\phi_b$ , for spread footings on soil is 0.45 based on bearing resistance evaluation using semi-empirical methods. A factored bearing resistance of 4 ksf may be used when analyzing the service limit state and for preliminary sizing of footings assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for spread footings shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, footings 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.

In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete, which is taken as  $0.3f'_c$ . No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material. Any organic material encountered shall be removed to the full depth and replaced with compacted Granular Borrow, MaineDOT 703.19.

## 7.5 Scour and Riprap

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at the strength and service limit states. These changes in foundation conditions shall be investigated at the abutments and independent return wings. For scour protection, any footings for independent return wings, which are constructed on granular deposits, should be embedded a minimum of 3 feet below the design scour depth and armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to item number 703.26 of the Standard Specification shall be placed at the toes of abutments and return wings. Riprap shall be 3 feet thick. In front of the independent return wings, the bottom of the riprap section shall be constructed 5.0 feet above the bottom of the structures for frost protection. The riprap shall extend 1.5 feet horizontally in front of the wall before sloping at a maximum 1.75H:1V slope to the existing ground surface. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification.



## 7.6 Settlement

The vertical alignment of the proposed bridge will be raised 7 inches at the west approach to improve drainage conditions and provide a crest vertical curve to the bridge. One dimensional consolidation tests performed on undisturbed tube samples indicate that the silty clay deposits at the site are generally under consolidated. This indicates that the soils are highly compressible and that they are susceptible to consolidation if the in-situ stresses are increased above the current levels (i.e., consolidation will occur if fill is placed, or if structures are supported on clay). Evaluation of the potential settlement due to the placement of this 7 inches of fill resulted in approximately ½ inch of settlement. Approximately half of this settlement is consolidation settlement within the compressible silty 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 16 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 frost depth maps for the State of Maine (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1100 F-degree days. This correlates to a frost depth of 5.0 feet. Therefore, 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 per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

## 7.8 Retaining Wall

The return wings for both abutments will be 90 degree return wings and will be independent of the stub abutment. The independent return wings will be Precast Concrete Modular Gravity (PCMG) walls be supported on spread footings. The PCMG return wings will retain approach fills, provide lateral support to the corners of the pile group and minimize slope impacts on the Rachel Carson National Wildlife Refuge property. The PCMG walls shall be designed by a Professional Engineer subcontracted by the Contractor as a design-build item. The PCMG Wall shall be founded on the granular soils at the site. The PCMG Wall shall be designed considering a traffic surcharge equal to 2 feet of fill placed on the backfill surface.

Bearing resistance for the PCMG wall founded on granular soils shall be investigated at the strength limit state using factored loads and a factored bearing resistance of 12 ksf. The bearing resistance factor,  $\phi_b$ , for spread footings on sand is 0.45. A factored bearing resistance of 4 ksf may be used when analyzing the service limit state assuming a resistance factor of 1.0. See Appendix C - Calculations for supporting documentation.

The bearing resistance for PCMG wall footings shall be checked for the extreme limit state with a resistance factor of 1.0. Furthermore, PCMG wall footings 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 PCMG wall shall consist of Class “LP” concrete and epoxy coated rebar. The precast concrete units shall contain a minimum of 5.5 gallons per cubic yard of calcium nitrate solution or equivalent corrosion inhibitor.

The high water elevation shall be indicated on the retaining wall plans per the design requirements for hydrostatic conditions in Special Provision 635 - Prefabricated Bin Type Retaining Wall (Prefabricated Concrete Modular Gravity Wall). The Special Provision reads: Hydrostatic forces - Unless specified otherwise, when a design high water surface is shown on the plans, the design stresses calculated from that elevation to the bottom of wall must include a 3 foot minimum differential head of flow able fill or saturated backfill. In addition, the buoyant weight of saturated soil shall be used in the calculation of pullout resistance.

## **7.9 Seismic Design Considerations**

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

- Peak Ground Acceleration coefficient (PGA) = 0.096g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.186g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.045g

Per LRFD Article 3.10.3.1 the site is assigned to Site Class E due to the presence of more than 10 feet of soft clay at the site. Per LRFD Article 3.10.6 the site is assigned to Seismic Zone 2 based on a calculated  $S_{D1}$  of 0.157 (LRFD Eq. 3.10.4.2-6).

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

According to Figure 2-2 of the BDG, the Bourne Avenue Bridge is not on the National Highway System (NHS) and is therefore not considered to be functionally important, and since the bridge construction costs should not exceed \$10 million the bridge is not classified as a major structure. Consequentially, no detailed seismic analysis is required other than connection design and support length requirements.

## **7.10 Construction Considerations**

Organic material was encountered in boring BB-WSB-101. Organic material may be encountered in excavations for the PCMG wall units and leveling slab. Any organic material encountered shall be removed to the full depth and replaced with compacted Granular Borrow.

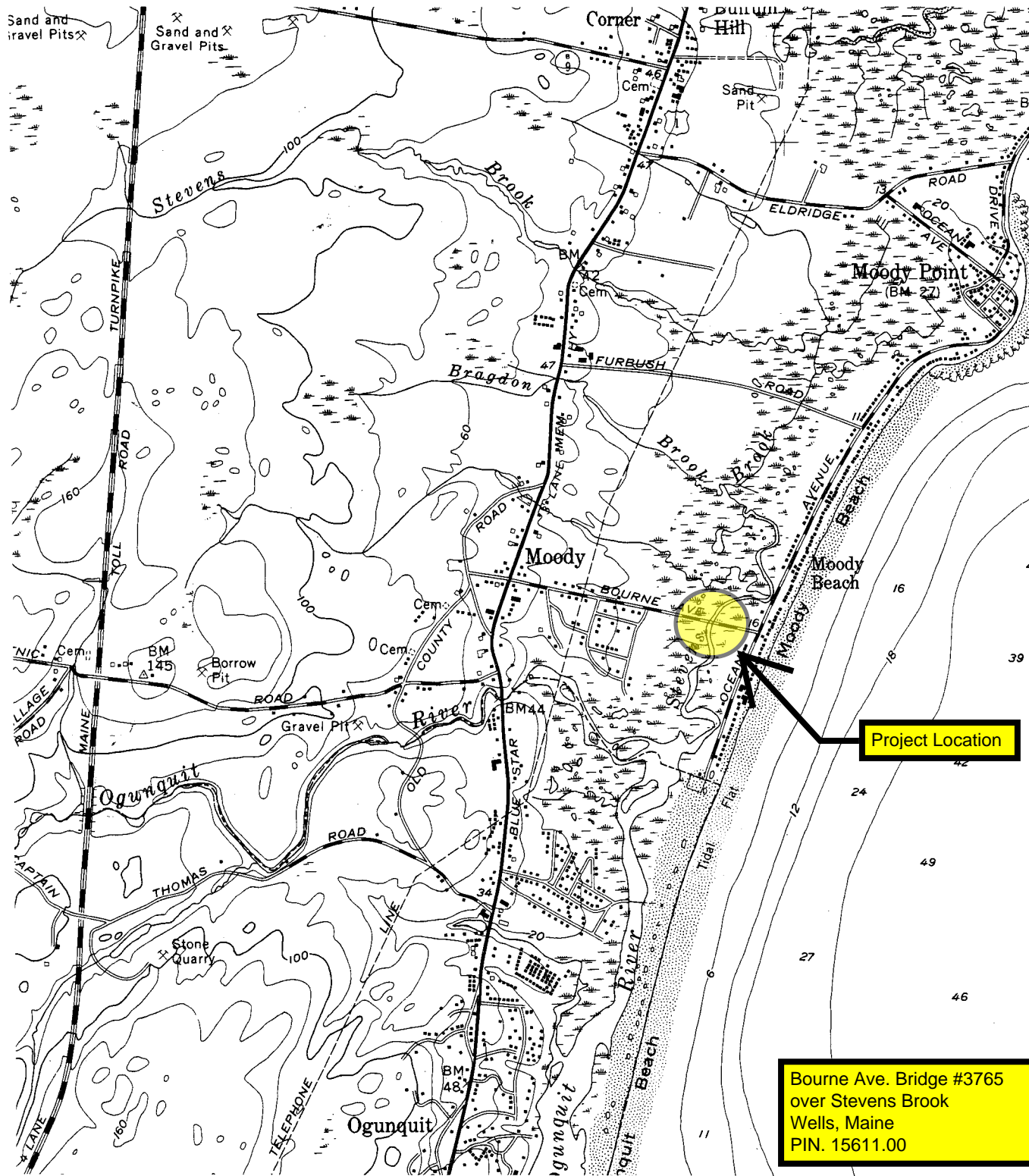
There is a potential for the existing abutment piles to interfere with the installation of the integral abutment piles. If the piles are encountered during pile installation they shall be removed by the Contractor to the resident's satisfaction. This condition should be noted on the plans and the work should be considered incidental to pile installation.

## **8.0 CLOSURE**

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Bourne Avenue Bridge in Wells, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use 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**

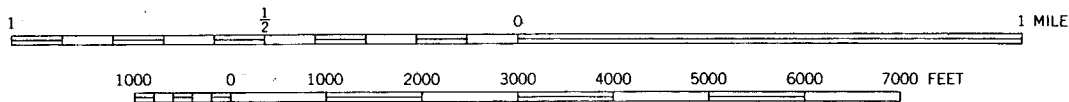


Project Location

Bourne Ave. Bridge #3765  
over Stevens Brook  
Wells, Maine  
PIN. 15611.00

WELLS QUADRANGLE  
 MAINE—YORK CO.  
 7.5 MINUTE SERIES (TOPOGRAPHIC)  
 SE/4 KENNEBUNK 15' QUADRANGLE

SCALE 1:24000



CONTOUR INTERVAL 20 FEET

Date: 7/18/2008

Username: terry.white

Division: GEOTECH

Filename: ... \geotech\msta\005\_BLP\SP1.dgn

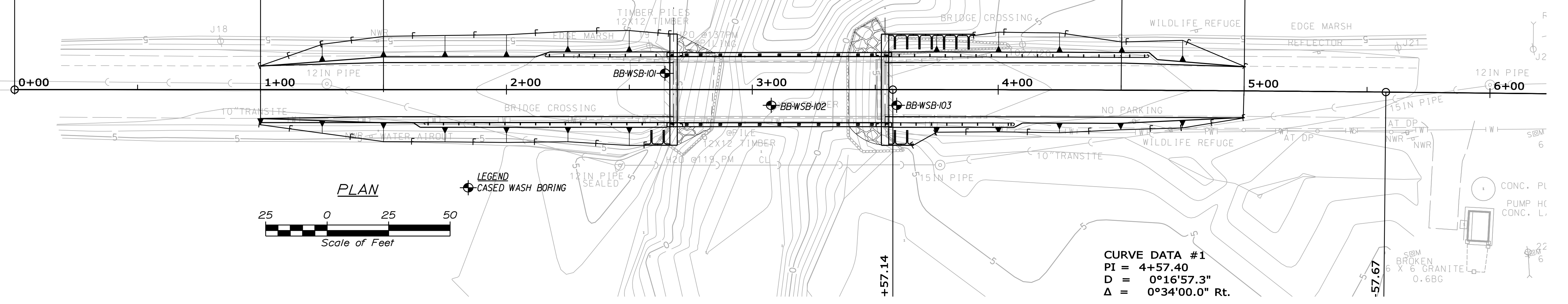
POB = STA. 0+00.00



50'-0" Transition  
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Match Existing  
Sta. 1+50.00 Begin Project  
End Transition

50'-0" Transition  
Sta. 4+50.00 End Transition  
Match Existing  
Sta. 5+00.00 End Transition

Brig., Abut. No. 1  
Sta. 2+68.00 86' Span  
Brig., Abut. No. 2  
Sta. 3+54.00

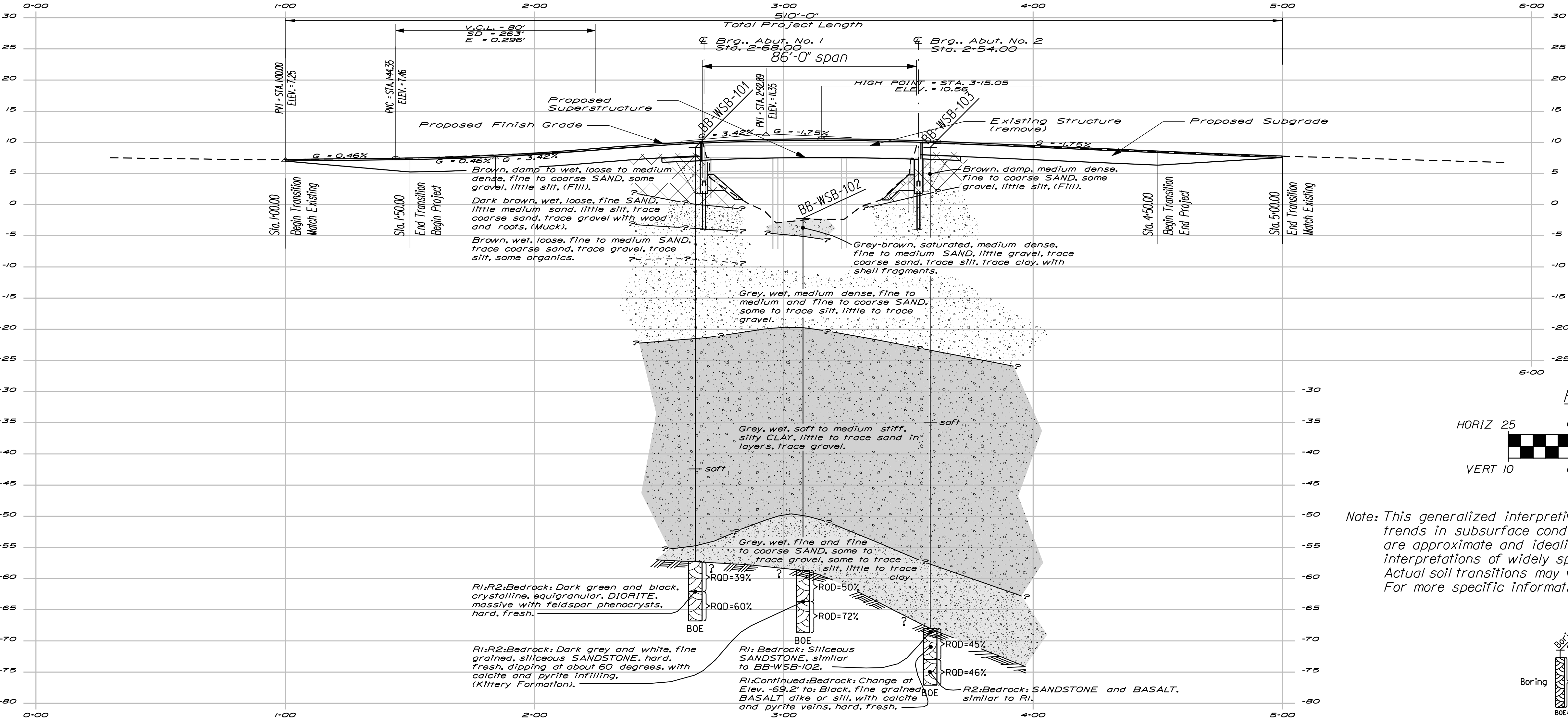


PLAN

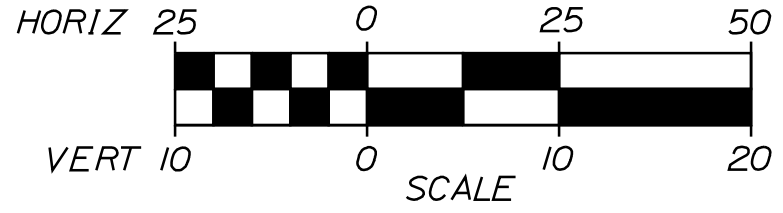


LEGEND  
CASED WASH BORING

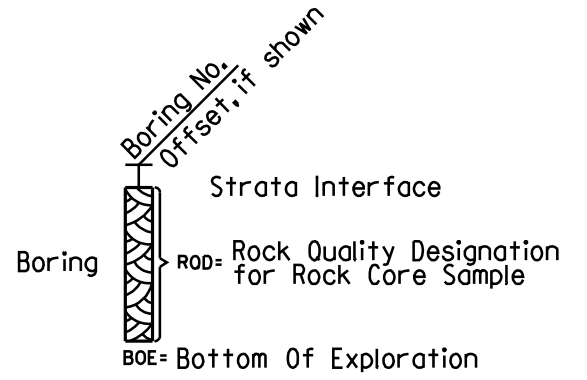
CURVE DATA #1  
PI = 4+57.40  
D = 0°16'57.3"  
Δ = 0°34'00.0" Rt.



PROFILE



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-1561(100)X  
BRIDGE NO. 3785  
PIN 15611.00  
BRIDGE PLANS

Table with columns for SIGNATURE, P.E. NUMBER, and DATE.

Table with columns for PROJ. MANAGER, DATE, BY, CHECKED, DESIGNED, REVISIONS, and FIELD CHANGES.

BOURNE AVENUE BRIDGE  
STEVENS BROOK  
YORK COUNTY  
WELLS  
BORING LOCATION PLAN &  
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 5

Maine Department of Transportation Soil/Bore Exploration Log		Project: Bourne Ave. Bridge #3165 over Stevens Brook Location: Wells, Maine		Boring No.: BB-WSB-101 PIN: 15611.00	
Driller: MoinDOT	Elevation (ft.): 9.2	Auger ID/OD: 5" Solid Stem Auger	Sampler: Standard Split Spoon		
Operator: E. Giguere/C. Giles	Date: NAVD 88	Sampler: Standard Split Spoon	Core Barrels: ND-2"		
Logged By: B. Wilder/A. Maguire	Rig Type: CME 45C	Core Barrels: ND-2"	Water Level: Tidal		
Date Start/Finish: 3/11/08, 3/13/08, 3/26/08	Drilling Method: Cased Wash Boring	Core Barrels: ND-2"	Water Level: Tidal		
Boring Location: 2464.5, 6.7 Lt.	Casing ID/OD: HW	Water Level: Tidal	Home Efficiency Factor: 0.77		
<p>Home Efficiency Factor: 0.77</p> <p>Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p> <p>Drill Bit: 2 1/2" Dia. Flat Top Shear Strength Test            3" = Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Sampler: 5" Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Water Level: Tidal            Home Efficiency Factor: 0.77            Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p>					
<p>Visual Description and Remarks</p> <p>8.50 Pavement</p> <p>10 Brown, damp, medium dense, gravelly, fine to coarse SAND, (fill).</p> <p>15 Brown, wet, loose, fine to coarse SAND, some gravel, little silt (fill).</p> <p>20 Dark brown, wet, loose, fine SAND, little medium sand, little silt, trace coarse sand, trace gravel, with wood and roots, (back).</p> <p>25 Brown, wet, loose, fine to medium SAND, trace coarse sand, trace gravel, trace silt, some organics.</p> <p>30 Grey, wet, dense, fine SAND, trace medium to coarse sand, trace silt, trace gravel.</p> <p>35 Failed hydraulic push tube attempt, would not push. Took sample 80. Similar to 20A.</p> <p>40 17D1 30.0-30.5' bgs. Grey, wet, loose, fine SAND, little medium to coarse sand, trace clay, trace silt, trace gravel. 17D/A1 30.5-32.0' bgs. Grey, wet, medium stiff, silty CLAY, little sand in layers, trace gravel.</p> <p>45 Failed hydraulic push tube attempt, would not push. Took sample 80. Similar to 20A.</p> <p>50 65x130 mm vane row torque readings: V1: 12.0/3.0 ft-lbs V2: 9.0/1.5 ft-lbs</p> <p>55 Grey, wet, soft to medium stiff, silty CLAY, trace sand.</p> <p>60 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>65 Grey, wet, soft to medium stiff, silty CLAY, trace sand, trace gravel.</p> <p>70 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>75 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>80 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>85 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>90 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>95 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p> <p>100 65x130 mm vane row torque readings: V1: 15.0/3.0 ft-lbs V2: 23.5/5.5 ft-lbs</p>					

Maine Department of Transportation Soil/Bore Exploration Log		Project: Bourne Ave. Bridge #3165 over Stevens Brook Location: Wells, Maine		Boring No.: BB-WSB-101 PIN: 15611.00	
Driller: MoinDOT	Elevation (ft.): 9.2	Auger ID/OD: 5" Solid Stem Auger	Sampler: Standard Split Spoon		
Operator: E. Giguere/C. Giles	Date: NAVD 88	Sampler: Standard Split Spoon	Core Barrels: ND-2"		
Logged By: B. Wilder/A. Maguire	Rig Type: CME 45C	Core Barrels: ND-2"	Water Level: Tidal		
Date Start/Finish: 3/11/08, 3/13/08, 3/26/08	Drilling Method: Cased Wash Boring	Core Barrels: ND-2"	Water Level: Tidal		
Boring Location: 2464.5, 6.7 Lt.	Casing ID/OD: HW	Water Level: Tidal	Home Efficiency Factor: 0.77		
<p>Home Efficiency Factor: 0.77</p> <p>Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p> <p>Drill Bit: 2 1/2" Dia. Flat Top Shear Strength Test            3" = Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Sampler: 5" Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Water Level: Tidal            Home Efficiency Factor: 0.77            Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p>					
<p>Visual Description and Remarks</p> <p>20 24/24 50.00 - 52.00 Hydraulic Push</p> <p>25 52.64 - 53.00 Sum44/134 psf</p> <p>26 53.64 - 54.00 Sum56/134 psf</p> <p>30 24/24 55.00 - 57.00</p> <p>35 24/24 60.00 - 62.00</p> <p>40 24/24 66.00 - 71.30 R00 = 33%</p> <p>45 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>50 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p> <p>55 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>60 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p> <p>65 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>70 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p> <p>75 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>80 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p> <p>85 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>90 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p> <p>95 56.47 - 56.47 71.30 - 76.00 R00 = 60%</p> <p>100 57.63 - 57.63 71.30 - 76.00 R00 = 33%</p>					

Maine Department of Transportation Soil/Bore Exploration Log		Project: Bourne Ave. Bridge #3165 over Stevens Brook Location: Wells, Maine		Boring No.: BB-WSB-102 PIN: 15611.00	
Driller: MoinDOT	Elevation (ft.): -2.2	Auger ID/OD: 5" Solid Stem Auger	Sampler: Standard Split Spoon		
Operator: E. Giguere/C. Giles	Date: NAVD 88	Sampler: Standard Split Spoon	Core Barrels: ND-2"		
Logged By: B. Wilder/A. Maguire	Rig Type: CME 45C	Core Barrels: ND-2"	Water Level: Tidal		
Date Start/Finish: 4/1/08-4/3/08	Drilling Method: Cased Wash Boring	Core Barrels: ND-2"	Water Level: Tidal		
Boring Location: 3407.1, 6.5 Rt.	Casing ID/OD: HW	Water Level: Tidal	Home Efficiency Factor: 0.77		
<p>Home Efficiency Factor: 0.77</p> <p>Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p> <p>Drill Bit: 2 1/2" Dia. Flat Top Shear Strength Test            3" = Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Sampler: 5" Solid Stem Auger            4" = Standard Split Spoon Sampler            5" = Thin Wall Tube Sampler            6" = Unconsolidated Thin Wall Tube Sampler            8" = In Situ Shear Test            9" = Unconsolidated Thin Wall Tube Sampler            10" = Unconsolidated Thin Wall Tube Sampler            12" = Unconsolidated Thin Wall Tube Sampler</p> <p>Water Level: Tidal            Home Efficiency Factor: 0.77            Home Type: Automatic <input type="checkbox"/> Hydraulic <input checked="" type="checkbox"/> Rope &amp; Cathead <input type="checkbox"/></p>					
<p>Visual Description and Remarks</p> <p>10 24/13 0.00 - 2.00 NDH7/3/3 10 13 4</p> <p>20 24/12 3.00 - 5.00 3/3/4/8 7 9 48</p> <p>30 24/16 8.00 - 10.00 10/9/13/18 22 28 45</p> <p>40 24/18 13.00 - 15.00 10/13/12/11 25 32 47</p> <p>50 24/24 18.50 - 20.50 WDR/WDR/WDR --- 101</p> <p>60 24/24 23.50 - 25.50 WDR/WDR/WDR --- 104</p> <p>70 24/24 28.50 - 30.50 WDR/WDR/WDR --- 104</p> <p>80 24/24 33.50 - 35.50 WDR/WDR/WDR --- 99</p> <p>90 24/20 38.50 - 40.50 WDR/WDR/WDR --- 106</p> <p>100 24/14 43.50 - 45.50 WDR/WDR/WDR --- 101</p> <p>110 24/22 48.50 - 50.50 WDR/WDR/4/10 4 5 118</p> <p>120 24/19 53.00 - 55.00 6/17/22/19 39 50 85</p> <p>130 56.50 - 58.50 R00 = 50%</p> <p>140 60/60 61.50 - 66.50 R00 = 72%</p> <p>150 66.50 - 68.50 R00 = 50%</p> <p>160 68.50 - 70.50 R00 = 50%</p> <p>170 70.50 - 72.50 R00 = 50%</p> <p>180 72.50 - 74.50 R00 = 50%</p> <p>190 74.50 - 76.50 R00 = 50%</p> <p>200 76.50 - 78.50 R00 = 50%</p>					

**STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION**

**BORNE AVENUE BRIDGE  
STEVENS BROOK  
YORK COUNTY**

**BR-1561(100)X**

**BORING LOGS**

**WELLS**

BRIDGE NO. 3765  
PIN 15611.00

PROJ. MANAGER	DATE	BY
J. WENTWORTH	JUNE 2008	T. WHITE
DESIGN-DETAILED	CHECKED-REVIEWED	DESIGNED-DETAILED
K. MAGUIRE		
DESIGN-REVIEWED	DESIGNED-REVIEWED	REVISIONS
		1
		2
		3
		4
SIGNATURE		P.E. NUMBER
DATE		FIELD CHANGES

**SHEET NUMBER**

**3**

OF 5

Maine Department of Transportation Soil/Rock Exploration Log US. CUSTOMARY UNITS				Project: Bourne Ave. Bridge #3165 over Stevens Brook Location: Wells, Maine				Boring No.: BB-WSB-103 PIN: 15611.00			
Driller:	MoinDOT	Elevation (ft.):	9.2	Auger ID/OD:	5" Solid Stem Auger						
Operator:	E. Clague/B. Winder	Datum:	NAVD 88	Sampler:	Standard Split Spoon						
Logged By:	B. Winder/K. Maguire	Rig Type:	DM 45C	Hammer Wt./Fall:	140W/30"						
Date Start/Finish:	3/27/08-4/1/08-4/2/08	Drilling Method:	Cased Wash Boring	Core Barrels:	MD-2"						
Boring Location:	3+58.7, S.E. Rt.	Casing ID/OD:	6"	Water Level:	Tidal						
Sampler Efficiency Factor: 0.71	Hammer: Automatic 22 Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/> Definitions: S <sub>u</sub> = Mohr Field Vane Shear Strength (psf) S <sub>u</sub> ( <sub>max</sub> ) = Lab Vane Shear Strength (psf) B = Split Spoon Sample SA = Split Stem Auger W <sub>c</sub> = water content, percent W <sub>p</sub> = Plasticity Limit (Liquid Limit) U <sub>c</sub> = Unconfined Compressive Strength (ksf) U <sub>c</sub> = Liquid Limit S = Thin Wall Tube Sample S <sub>1</sub> = Roller Core W <sub>u</sub> = Water Content W <sub>u</sub> = uncorrected Thin Wall Tube Sample attempt W <sub>u</sub> = weight of 100g. hammer V = Thin Vane Shear Test, PP = Pocket Penetration Test W = weight of ram W <sub>p</sub> = Plasticity Index W <sub>p</sub> = uncorrected correction for hammer efficiency C = grain size analysis W <sub>p</sub> = uncorrected Thin Wall Tube Sample attempt W <sub>p</sub> = weight of 100g. hammer W <sub>p</sub> = uncorrected correction for hammer efficiency C = grain size analysis										
Depth (ft.)	Sample No.	Pen./Rec. (ft.)	Sample Information	Laboratory Testing Results (Unified Class)							
0			Pavement	-							
5	10	24/13	5.00 - 3.00 6/8/7/8 15 19 23	G0210076 A-1-U, SC-54 WC=5.1%							
10	20	24/16	10.00 - 12.00 5/6/8/7 14 18 25	G0210077 A-3, SP WC=2.7%							
15	30	24/12	15.00 - 17.00 WDR/1/2/4 3 4 24	Similar to 20, but looser.							
20	40	24/14	20.00 - 22.00 2/4/8/8 12 15 74	G0210078 A-3, SP WC=3.4%							
25	50	24/16	25.00 - 27.00 3/7/12/15 19 24 89	Grey, wet, medium dense, fine to medium SAND, trace gravel, trace coarse sand, trace silt.							
30	60	24/9	30.00 - 32.00 3/3/5/2 8 10 132	Grey, wet, loose, fine to medium SAND, trace coarse sand, trace gravel, trace silt.							
35	70	24/24	35.00 - 37.00 WDR/WDR	G0210088 A-6, CL WC=1.5% LL=33 PI=11							
40	80	24/24	40.50 - 42.50 Push thru vane	G0210079 A-6, CL WC=3.7% LL=33 PI=11							
45	90	24/22	45.00 - 47.00 WDR/WDR	G0210087 A-6, CL WC=2.8% LL=33 PI=12							
50	100	24/22	47.57 - 49.00 Su=453/60 psf	65x130 mm vane row torque readings: V5: 16.5/2.2 ft-lbs V6: 17.0/2.2 ft-lbs							

Maine Department of Transportation Soil/Rock Exploration Log US. CUSTOMARY UNITS				Project: Bourne Ave. Bridge #3165 over Stevens Brook Location: Wells, Maine				Boring No.: BB-WSB-103 PIN: 15611.00			
Driller:	MoinDOT	Elevation (ft.):	9.2	Auger ID/OD:	5" Solid Stem Auger						
Operator:	E. Clague/B. Winder	Datum:	NAVD 88	Sampler:	Standard Split Spoon						
Logged By:	B. Winder/K. Maguire	Rig Type:	DM 45C	Hammer Wt./Fall:	140W/30"						
Date Start/Finish:	3/27/08-4/1/08-4/2/08	Drilling Method:	Cased Wash Boring	Core Barrels:	MD-2"						
Boring Location:	3+58.7, S.E. Rt.	Casing ID/OD:	6"	Water Level:	Tidal						
Sampler Efficiency Factor: 0.71	Hammer: Automatic 22 Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/> Definitions: S <sub>u</sub> = Mohr Field Vane Shear Strength (psf) S <sub>u</sub> ( <sub>max</sub> ) = Lab Vane Shear Strength (psf) B = Split Spoon Sample SA = Split Stem Auger W <sub>c</sub> = water content, percent W <sub>p</sub> = Plasticity Limit (Liquid Limit) U <sub>c</sub> = Unconfined Compressive Strength (ksf) U <sub>c</sub> = Liquid Limit S = Thin Wall Tube Sample S <sub>1</sub> = Roller Core W <sub>u</sub> = Water Content W <sub>u</sub> = uncorrected Thin Wall Tube Sample attempt W <sub>u</sub> = weight of 100g. hammer V = Thin Vane Shear Test, PP = Pocket Penetration Test W = weight of ram W <sub>p</sub> = Plasticity Index W <sub>p</sub> = uncorrected correction for hammer efficiency C = grain size analysis W <sub>p</sub> = uncorrected Thin Wall Tube Sample attempt W <sub>p</sub> = weight of 100g. hammer W <sub>p</sub> = uncorrected correction for hammer efficiency C = grain size analysis										
Depth (ft.)	Sample No.	Pen./Rec. (ft.)	Sample Information	Laboratory Testing Results (Unified Class)							
0			Pavement	-							
5	80	24/24	51.00 - 53.00 WDR/WDR/WDR	G0210080 A-7-6, CL WC=8.3% LL=40 PI=18							
10	90	24/24	55.00 - 57.00 WDR/WDR	G0210089 A-6, CL WC=4.5% LL=34 PI=12							
15	100	24/24	60.00 - 62.00 WDR/WDR/WDR	G0210081 A-3, SP WC=3.0% LL=37 PI=6							
20	110	24/16	70.00 - 72.00 9/3/8/10 17 22 99	Grey, wet, medium dense, fine SAND, trace medium to coarse sand, trace silt, little gravel.							
25	120	24/12	75.00 - 77.00 7/7/6/4 13 17 144	Grey, wet, medium dense, silty fine to coarse SAND, some gravel, trace clay. (T111).							
30	130	24/12	82.00 - 84.00 ROD = 45%	Roller Cored ahead from 77.0-77.2' bgs.							
35	140	24/24	85.00 - 87.00 WDR/WDR	Top of Bedrock at Elev. -88.0'. Bedrock: Dark grey and white, fine grained, Siliceous SANDSTONE, hard, fresh, dipping at about 60 degrees, with calcite and pyrite inclusions. Heavy formation. Change of Elev. -89.2' to Block, fine grained, BASALT dikes or sills, with calcite and pyrite veins, hard, fresh. Rock Quality = Poor. R1 Core Times (min:sec) 300-500 psi down pressure: 77-2-78-2' (2:35) 78-2-79-2' (3:04) 79-2-80-2' (2:43) 80-2-81-2' (2:30) 81-2-82-2' (2:45) 100% Recovery R2 SANDSTONE and BASALT similar to R1, Rock Quality = Poor. R2 Core Times (min:sec) 82-2-83-2' (3:45) 83-2-84-2' (4:15) 84-2-85-2' (4:22) 85-2-86-2' (3:45) 86-2-86-3' (1:30) 98% Recovery Core blocked on 86.3' bgs. Bottom of Exploration at 86.30 feet below ground surface.							

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
BR-1561(100)X  
PIN 15611.00  
BRIDGE NO. 3765  
BRIDGE PLANS

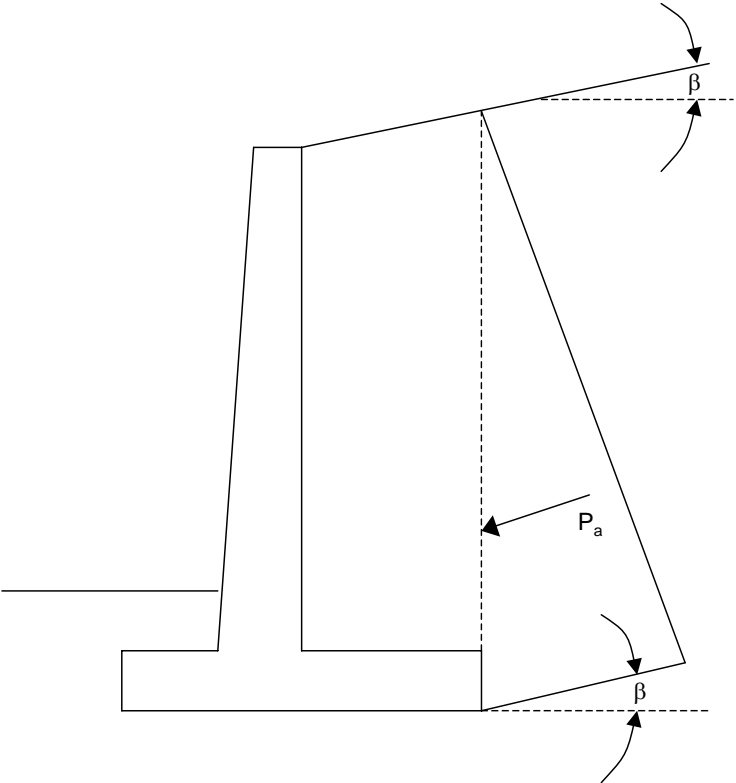
BOURNE AVENUE BRIDGE  
STEVENS BROOK  
WELLS YORK COUNTY  
BORING LOGS

SHEET NUMBER 4 OF 5

DATE	BY	REVISIONS
JUNE 2008	T. WHITE	DESIGN-DETAILED CHECKED-REVIEWED DESIGNS DETAILED DESIGNS DETAILED REVISIONS 1 REVISIONS 2 REVISIONS 3 REVISIONS 4 FIELD CHANGES

SIGNATURE  
P.E. NUMBER  
DATE





For cases where interface friction between the backfill and wall are 0 or not considered, use Rankine.

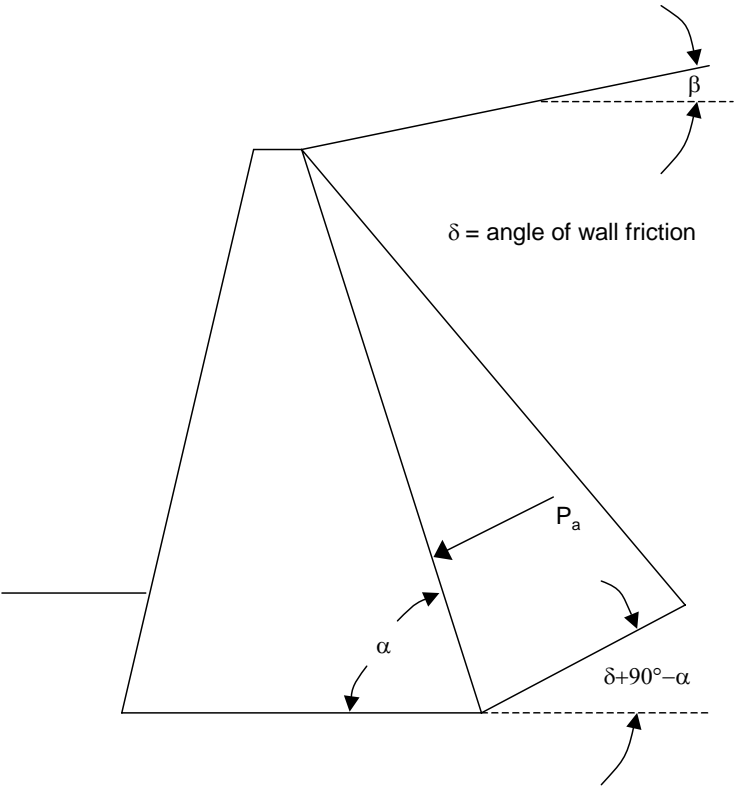
For a horizontal backfill surface,  $\beta = 0^\circ$ :

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

For a sloped backfill surface,  $\beta > 0^\circ$ :

$$K_a = \cos \beta * \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}$$

$P_a$  is oriented at  $\beta$



For cases where interface friction is considered, use Coulomb.

For horizontal or sloped backfill surfaces:

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

$P_a$  is oriented at  $\delta + 90^\circ - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients

## **Appendix A**

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																							
MAJOR DIVISIONS		GROUP SYMBOLS		TYPICAL NAMES																							
COARSE-GRAINED SOILS  (more than half of material is larger than No. 200 sieve size)	GRAVELS  (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p><b>Coarse-grained soils</b> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.</p> <p style="text-align: center;">Modified Burmister System</p> <table border="0"> <tr> <td style="text-align: center;"><u>Descriptive Term</u></td> <td style="text-align: center;"><u>Portion of Total</u></td> </tr> <tr> <td>trace</td> <td>0% - 10%</td> </tr> <tr> <td>little</td> <td>11% - 20%</td> </tr> <tr> <td>some</td> <td>21% - 35%</td> </tr> <tr> <td>adjective (e.g. sandy, clayey)</td> <td>36% - 50%</td> </tr> </table> <table border="0"> <tr> <td style="text-align: center;"><u>Density of Cohesionless Soils</u></td> <td style="text-align: center;"><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>&gt; 50</td> </tr> </table>	<u>Descriptive Term</u>	<u>Portion of Total</u>	trace	0% - 10%	little	11% - 20%	some	21% - 35%	adjective (e.g. sandy, clayey)	36% - 50%	<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50
		<u>Descriptive Term</u>	<u>Portion of Total</u>																								
		trace	0% - 10%																								
		little	11% - 20%																								
	some	21% - 35%																									
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Very loose	0 - 4																										
Loose	5 - 10																										
Medium Dense	11 - 30																										
Dense	31 - 50																										
Very Dense	> 50																										
(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.																									
<p><b>Desired Soil Observations: (in this order)</b></p> <p>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</p>				<p><b>Desired Rock Observations: (in this order)</b></p> <p>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 - &lt;5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide &gt;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</p>																							
<p><b>Maine Department of Transportation</b>  <b>Geotechnical Section</b>  <b>Key to Soil and Rock Descriptions and Terms</b>  Field Identification Information</p>				<p><b>Sample Container Labeling Requirements:</b></p> <table border="0"> <tr> <td>PIN</td> <td>Blow Counts</td> </tr> <tr> <td>Bridge Name / Town</td> <td>Sample Recovery</td> </tr> <tr> <td>Boring Number</td> <td>Date</td> </tr> <tr> <td>Sample Number</td> <td>Personnel Initials</td> </tr> <tr> <td>Sample Depth</td> <td></td> </tr> </table>		PIN	Blow Counts	Bridge Name / Town	Sample Recovery	Boring Number	Date	Sample Number	Personnel Initials	Sample Depth													
PIN	Blow Counts																										
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Sample Depth																											

Driller: MaineDOT	Elevation (ft.): 9.2	Auger ID/OD: 5" Solid Stem Auger
Operator: E. Giguere/C. 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: 3/11/08, 3/13/08, 3/26/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 2+64.5, 6.7 Lt.	Casing ID/OD: HW	Water Level*: Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic  Hydraulic  Rope & Cathead   
 Definitions: R = Rock Core Sample      S<sub>u</sub> = Insitu Field Vane Shear Strength (psf)      S<sub>u(lab)</sub> = Lab Vane Shear Strength (psf)  
 D = Split Spoon Sample      SSA = Solid Stem Auger      T<sub>v</sub> = Pocket Torvane Shear Strength (psf)      WC = water content, percent  
 MD = Unsuccessful Split Spoon Sample attempt      HSA = Hollow Stem Auger      q<sub>p</sub> = Unconfined Compressive Strength (ksf)  
 U = Thin Wall Tube Sample      RC = Roller Cone      N-uncorrected = Raw field SPT N-value      LL = Liquid Limit  
 MU = Unsuccessful Thin Wall Tube Sample attempt      WOH = weight of 140lb. hammer      Hammer Efficiency Factor = Annual Calibration Value      PL = Plastic Limit  
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer      WOR = weight of rods      N<sub>60</sub> = SPT N-uncorrected corrected for hammer efficiency      G = Grain Size Analysis  
 MV = Unsuccessful Insitu Vane Shear Test attempt      WO1P = Weight of one person      N<sub>60</sub> = (Hammer Efficiency Factor/60%)\*N-uncorrected      C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows					
0							SSA	8.50	[Cross-hatched pattern]	Pavement		
	1D	24/15	1.50 - 3.50	4/5/6/6	11	14					Brown, damp, medium dense, gravelly, fine to coarse SAND, (Fill).	
5									[Vertical line pattern]			
	2D	24/11	5.00 - 7.00	4/4/2/6	6	8	11				Brown, wet, loose, fine to coarse SAND, some gravel, little silt (Fill).	G#209933 A-1-b, SW-SM WC=7.5%
10								0.20	[Vertical line pattern]			
	3D	24/10	10.00 - 12.00	1/2/2/2	4	5	18				Dark brown, wet, loose, fine SAND, little medium sand, little silt, trace coarse sand, trace gravel, with wood and roots, (muck).	
15									[Vertical line pattern]			
	4D	24/19	15.00 - 17.00	2/2/5/8	7	9	76				Brown, wet, loose, fine to medium SAND, trace coarse sand, trace gravel, trace silt, some organics.	G#209935 A-3, SP WC=32.6%
20									[Vertical line pattern]			
	5D	24/16	20.00 - 22.00	7/10/15/30	25	32	140				Grey, wet, dense, fine SAND, trace medium to coarse sand, trace silt, trace gravel.	
25												

**Remarks:**  
When casing got to refusal, crew thought there was 61.5' of casing in hole, when there was really 66.5' of casing. Samples 10D and 3U are suspect.

Driller: MaineDOT	Elevation (ft.): 9.2	Auger ID/OD: 5" Solid Stem Auger
Operator: E. Giguere/C. 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: 3/11/08, 3/13/08, 3/26/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 2+64.5, 6.7 Lt.	Casing ID/OD: HW	Water Level*: Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic     Hydraulic     Rope & Cathead

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows					
25	6D	24/14	25.00 - 27.00	6/11/17/23	28	36	215			Grey, wet, dense, fine SAND, trace medium to coarse sand, trace silt, trace gravel.	G#209936 A-3, SP-SM WC=21.4%	
							218					
							246					
							256					
							269					
30	7D/A	24/20	30.00 - 32.00	6/4/1/1	5	6	160	-21.30		(7D) 30.0-30.5' bgs. Grey, wet, loose, fine SAND, little medium to coarse sand, trace clay, trace silt, trace gravel.	G#209937 A-2-4, SM WC=18.5% G#209938 A-6, CL WC=29.9% LL=30 PL=20 PI=10	
							156			(7D/A) 30.5-32.0' bgs.		
							164			Grey, wet, medium stiff, Silty CLAY, little sand in layers, trace gravel.		
							150					
							158					
35	MU/8D	24/24	35.00 - 37.00	WOR/WOR/WOR/WOR	---		192			Failed hydraulic push tube attempt, would not push. Took sample 8D, Similar to 7D/A.	G#209939 A-4, CL WC=34.3% LL=30 PL=21 PI=9	
							167					
	V1		37.63 - 37.99	Su=536/134 psf			167			55x110 mm vane raw torque readings: V1: 12.0/3.0 ft-lbs		
	V2		38.63 - 38.99	Su=402/67 psf			153			V2: 9.0/1.5 ft-lbs		
							152					
40	1U	24/24	40.00 - 42.00	Hydraulic Push			179			Grey, wet, soft to medium stiff, Silty CLAY, trace sand.	G,C#210084 A-6, CL WC=41.8% LL=34 PL=21 PI=13	
							133					
	V3		42.57 - 43.00	Su=467/110 psf			127			65x130 mm vane raw torque readings: V3: 17.0/4.0 ft-lbs		
	V4		43.57 - 44.00	Su=645/151 psf			121			V4: 23.5/5.5 ft-lbs		
							117					
45	9D	24/24	45.00 - 47.00	WOR/WOR/WOR/WOR	---		159			Grey, wet, soft to medium stiff, Silty CLAY, trace sand, trace gravel.	G#209940 A-6, CL WC=33.5% LL=34 PL=22 PI=12	
							156					
	V5		47.57 - 48.00	Su=412/82 psf			132			65x130 mm vane raw torque readings: V5: 15.0/3.0 ft-lbs		
	V6		48.57 - 49.00	Su=494/82 psf			123			V6: 18.0/3.0 ft-lbs		
							130			Bent 65x130 mm vane on bottom of casing.		

**Remarks:**

When casing got to refusal, crew thought there was 61.5' of casing in hole, when there was really 66.5' of casing. Samples 10D and 3U are suspect.

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> 9.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/C. Giles	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder/K. Maguire	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 3/11/08, 3/13/08, 3/26/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 2+64.5, 6.7 Lt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal
<b>Hammer Efficiency Factor:</b> 0.77	<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

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 = weight of rods WO1P = Weight of one person	Su = Insitu Field Vane Shear Strength (psf) Tv = Pocket Torvane Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw field SPT N-value Hammer Efficiency Factor = Annual Calibration Value N60 = SPT N-uncorrected corrected for hammer efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected
		Su(lab) = Lab Vane Shear Strength (psf) WC = water content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in. Shear Strength (psf) or RQD (%))	N-uncorrected	N60	Casing Blows					
50	2U	24/24	50.00 - 52.00	Hydraulic Push			a			Grey, wet, soft, Silty CLAY, trace sand. aCasing blows not recorded.	G,C#210085 A-6, CL WC=41.7% LL=38 PL=22 PI=16	
							a					
	V7		52.64 - 53.00	Su=446/134 psf			a			55x110 mm vane raw torque readings: V7: 10.0/3.0 ft-lbs V8: 12.0/3.0 ft-lbs		
	V8		53.64 - 54.00	Su=536/134 psf			a					
55	MV 10D	24/24	55.50 - 57.50	WOR/WOR/WOR/1	---		188			Failed 65x130 mm vane attempt, could not push. Grey, wet, soft to medium stiff, Silty CLAY, trace sand, trace gravel.	G#209941 A-6, CL WC=35.0%	
							184					
							170					
							162					
							269					
60	b3U	24/24	60.00 - 62.00	WOR/WOR/WOR/ WOR			189			Similar to 10D. bDo not test tube, tube was likely taken with-in casing, see Remarks.		
							178					
							166					
							168					
							264	-54.90		Till? by roller cone at 64.1' bgs.	64.10	
65							240					
	R1	57.6/53	66.50 - 71.30	RQD = 39%			NQ	-57.30		Top of Bedrock at Elev. -57.3'. Bedrock: Dark green and black, crystalline, equigranular, DIORITE, massive with feldspar phenocrysts, hard, fresh. Rock Quality = Poor. R1:Core Times (min:sec) 400 psi down pressure 66.5-67.5' (3:04) 67.5-68.5' (3:44) 68.5-69.5' (3:28) 69.5-70.5' (2:54) 70.5-71.3' (2:18) Core Blocked 92% Recovery R2 similar to R1, Rock Quality = Fair. R2:Core Times (min:sec) 71.3-72.3' (4:01) 72.3-73.3' (4:00) 73.3-74.3' (3:59) 74.3-75.3' (3:27) 75.3-76.0' (3:40) 100% Recovery	66.50	
							CORE					
70												
	R2	56.4/56.4	71.30 - 76.00	RQD = 60%								
75												


**Remarks:**  
 When casing got to refusal, crew though there was 61.5' of casing in hole, when there was really 66.5' of casing. Samples 10D and 3U are suspect.

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS	<b>Project:</b> Bourne Ave. Bridge over Stevens Brook  <b>Location:</b> Wells, Maine	<b>Boring No.:</b> BB-WSB-101  <b>PIN:</b> 15611.00
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<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> 9.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/C. Giles	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder/K. Maguire	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 3/11/08, 3/13/08, 3/26/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 2+64.5, 6.7 Lt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic     Hydraulic     Rope & Cathead

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	$N_{60}$	Casing Blows						
75									-66.80		76.00	<b>Bottom of Exploration at 76.00 feet below ground surface.</b>	
80													
85													
90													
95													
100													

**Remarks:**  
 When casing got to refusal, crew though there was 61.5' of casing in hole, when there was really 66.5' of casing. Samples 10D and 3U are suspect.

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> -2.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/C. Giles	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 4/1/08-4/3/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 3+07.7, 6.5 Rt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal

**Hammer Efficiency Factor:** 0.77 **Hammer Type:** Automatic  Hydraulic  Rope & Cathead

**Definitions:**

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows					
0	1D	24/13	0.00 - 2.00	WOH/7/3/3	10	13	4		-5.20	[Graphic Log: 0 to 2 ft]	Grey-brown, saturated, medium dense, fine to medium SAND, little gravel, trace coarse sand, trace silt, trace clay, with shell fragments.	G#209942 A-2-4, SP-SC WC=23.4%
							7					
5	2D	24/12	3.00 - 5.00	3/3/4/8	7	9	48		-19.70	[Graphic Log: 3 to 5 ft]	Grey, wet, loose, fine SAND, trace medium to coarse sand, trace silt.	G#209943 A-3, SP WC=20.9%
							41					
							38					
							39					
10	3D	24/16	8.00 - 10.00	10/9/13/18	22	28	45		-17.50	[Graphic Log: 8 to 10 ft]	Grey, wet, medium dense, fine SAND, trace medium to coarse sand, trace silt.	G#209944 A-2-4, SM WC=17.5%
							47					
							49					
							52					
15	4D	24/18	13.00 - 15.00	10/13/12/11	25	32	47		-17.50	[Graphic Log: 13 to 15 ft]	Grey, wet, medium dense, fine to coarse SAND, some silt, trace gravel.	G#209944 A-2-4, SM WC=17.5%
							51					
							56					
							57					
							99					
20	5D	24/24	18.50 - 20.50	WOR/WOR/WOR/WOR	---		101		-17.50	[Graphic Log: 18.5 to 20.5 ft]	Grey, wet, soft to medium stiff, Silty CLAY, trace fine sand.  65x130 mm vane raw torque readings: V1: 17.0/3.0 ft-lbs V2: 20.5/3.0 ft-lbs	G#209945 A-6, CL WC=29.9% LL=30 PL=18 PI=12
	V1		19.07 - 19.50	Su=467/82 psf			101					
	V2		20.07 - 20.50	Su=563/82 psf			104					
							105					
							105					
25	6D	24/24	23.50 - 25.50	WOR/WOR/WOR/WOR	---		104		-17.50	[Graphic Log: 23.5 to 25.5 ft]	Grey, wet, medium stiff, Silty CLAY, trace fine sand.  65x130 mm vane raw torque readings: V3: 20.0/4.0 ft-lbs	
	V3		24.07 - 24.50	Su=549/110 psf			101					

**Remarks:**  
17.5' from Bridge Deck to Ground.



<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS	<b>Project:</b> Bourne Ave. Bridge #3765 over Stevens Brook	<b>Boring No.:</b> BB-WSB-102
	<b>Location:</b> Wells, Maine	<b>PIN:</b> 15611.00

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> -2.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/C. Giles	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 4/1/08-4/3/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 3+07.7, 6.5 Rt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal
<b>Hammer Efficiency Factor:</b> 0.77	<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

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 = weight of rods  
WO1P = Weight of one person

S<sub>u</sub> = Insitu Field Vane Shear Strength (psf)  
T<sub>y</sub> = Pocket Torvane Shear Strength (psf)  
q<sub>p</sub> = Unconfined Compressive Strength (ksf)  
N-uncorrected = Raw field SPT N-value  
Hammer Efficiency Factor = Annual Calibration Value  
N<sub>60</sub> = SPT N-uncorrected corrected for hammer efficiency  
N<sub>60</sub> = (Hammer Efficiency Factor/60%)\*N-uncorrected

S<sub>u(lab)</sub> = 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 Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows					
25	V4		25.07 - 25.50	Su=522/110 psf			99			V4: 19.0/4.0 ft-lbs		
							106					
							104					
	7D	24/24	28.50 - 30.50	WOR/WOR/WOR/WOR	---		104			Grey, wet, soft to medium stiff, Silty CLAY, trace fine sand.	G#209946	
	V5		29.07 - 29.50	Su=522/110 psf			102			65x130 mm vane raw torque readings: V5: 19.0/4.0 ft-lbs	A-6, CL	
30	V6		30.07 - 30.50	Su=494/82 psf			101			V6: 18.0/3.0 ft-lbs	WC=33.1%	
							101				LL=34	
							100				PL=22	
	8D	24/24	33.50 - 35.50	WOR/WOR/WOR/WOR	---		99			Grey, wet, medium stiff, Silty CLAY, trace fine sand.	G#209947	
	V7		34.07 - 34.50	Su=549/96 psf			100			65x130 mm vane raw torque readings: V7: 20.0/3.5 ft-lbs	A-6, CL	
35	V8		35.07 - 35.50	Su=522/96 psf			104			V8: 19.0/3.5 ft-lbs	WC=35.4%	
							108				LL=36	
							107				PL=22	
	9D	24/20	38.50 - 40.50	WOR/WOR/WOR/WOR	---		106			Similar to 8D.	G#209948	
	V9		39.07 - 39.50	Su=549/137 psf			101			65x130 mm vane raw torque readings: V9: 20.0/5.0 ft-lbs	A-6, CL	
40	V10		40.07 - 40.50	Su=549/137 psf			103			V10: 20.0/5.0 ft-lbs	WC=36.9%	
							104				LL=34	
							103				PL=22	
	10D	24/14	43.50 - 45.50	WOR/WOR/WOR/WOR	---		101			Grey, wet, medium stiff, Silty CLAY, trace fine sand, trace gravel.	G#209948	
	V11		44.07 - 44.50	Su=769/165 psf			99			65x130 mm vane raw torque readings: V11: 28.0/6.0 ft-lbs	A-6, CL	
45	V12		45.07 - 45.50	Su=824/165 psf			104			V12: 30.0/6.0 ft-lbs	WC=36.9%	
							109				LL=34	
							113				PL=22	
	11D	24/22	48.50 - 50.50	WOR/WOH/4/10	4	5	118			Grey, wet, loose, fine SAND, trace medium to coarse sand, trace gravel, trace silt.	G#209949	
50	MV						117			Failed 65x130 mm vane attempt.	A-2-4, SP-SM	
								-49.70			WC=19.1%	

**Remarks:**  
17.5' from Bridge Deck to Ground.

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> -2.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/C. Giles	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 4/1/08-4/3/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 3+07.7, 6.5 Rt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic  Hydraulic  Rope & Cathead   

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

Sample Information										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows	Elevation (ft.)	Elevation (ft.)			
50							79				Grey, wet, dense, fine to coarse SAND, some silt, trace gravel.  450 blows for 0.1'. Roller Coned ahead from 56.1-56.5' bgs.  Top of Bedrock at Elev. -58.7'. Bedrock: Dark grey and white, fine grained, Siliceous SANDSTONE, hard, fresh, dipping at about 60 degrees, with calcite and pyrite infilling. Kittery Formation. Rock Quality = Poor. R1: Core Times (min:sec) 400-600 psi down pressure 56.5-57.5' (3:54) 57.5-58.5' (3:17) 58.5-59.5' (3:04) 59.5-60.5' (2:50) 60.5-61.5' (3:02) 98% Recovery R2 similar to R1. Rock Quality = Fair. R2: Core Times (min:sec) 61.5-62.5' (3:11) 62.5-63.5' (4:03) 63.5-64.5' (3:45) 64.5-65.5' (3:37) 65.5-66.5' (3:28) 100% Recovery	G#209950 A-2-4, SM WC=16.1%
							119					
							165					
	12D	24/19	53.00 - 55.00	6/17/22/19	39	50	85					
							88					
55							104					
	R1	60/59	56.50 - 61.50	RQD = 50%			a50 NQ CORE	-58.70				
							-58.70					
							-68.70					
60												
	R2	60/60	61.50 - 66.50	RQD = 72%								
65												
70												
75												

**Remarks:**  
17.5' from Bridge Deck to Ground.

Driller: MaineDOT	Elevation (ft.): 9.2	Auger ID/OD: 5" Solid Stem Auger
Operator: E. Giguere/B. Wilder	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: 3/27/08,4/1/08-4/2/08	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 3+58.7, 6.4 Rt.	Casing ID/OD: HW	Water Level*: Tidal

Hammer Efficiency Factor: 0.77      Hammer Type: Automatic  Hydraulic  Rope & Cathead

Definitions: R = Rock Core Sample      S<sub>u</sub> = Insitu Field Vane Shear Strength (psf)      S<sub>u(lab)</sub> = Lab Vane Shear Strength (psf)  
D = Split Spoon Sample      SSA = Solid Stem Auger      T<sub>v</sub> = Pocket Torvane Shear Strength (psf)      WC = water content, percent  
MD = Unsuccessful Split Spoon Sample attempt      HSA = Hollow Stem Auger      q<sub>p</sub> = Unconfined Compressive Strength (ksf)  
U = Thin Wall Tube Sample      RC = Roller Cone      N-uncorrected = Raw field SPT N-value  
MU = Unsuccessful Thin Wall Tube Sample attempt      WOH = weight of 140lb. hammer      Hammer Efficiency Factor = Annual Calibration Value  
V = Insitu Vane Shear Test, PP = Pocket Penetrometer      WOR = weight of rods      N<sub>60</sub> = SPT N-uncorrected corrected for hammer efficiency  
MV = Unsuccessful Insitu Vane Shear Test attempt      WO1P = Weight of one person      N<sub>60</sub> = (Hammer Efficiency Factor/60%)\*N-uncorrected  
C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows					
0							SSA	8.60		Pavement		
5	1D	24/13	5.00 - 7.00	6/8/7/8	15	19	23	1.70		Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt, (Fill).	G#210076 A-1-b, SW-SM WC=5.1%	
10	2D	24/16	10.00 - 12.00	5/6/8/7	14	18	25			Grey, wet, medium dense, fine SAND, trace medium to coarse sand, trace silt, trace gravel.	G#210077 A-3, SP WC=22.7%	
15	3D	24/12	15.00 - 17.00	WOR/1/2/4	3	4	24			Similar to 2D, but loose.		
20	4D	24/14	20.00 - 22.00	2/4/8/8	12	15	74			Grey, wet, medium dense, fine to medium SAND, trace silt, trace coarse sand, trace gravel, with iron staining.	G#210078 A-3, SP WC=23.4%	
25							209			Wash water very black with organics from 22.5-23.0' bgs.		

**Remarks:**

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

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> 9.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/B. Wilder	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder/K. Maguire	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 3/27/08,4/1/08-4/2/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 3+58.7, 6.4 Rt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic     Hydraulic     Rope & Cathead

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

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows						
25	5D	24/16	25.00 - 27.00	3/7/12/15	19	24	89		-23.30	Grey, wet, medium dense, fine to medium SAND, trace gravel, trace coarse sand, trace silt.			
										160			
										200			
										237			
										239			
30	6D	24/9	30.00 - 32.00	3/3/5/2	8	10	132					Grey, wet, loose, fine to medium SAND, trace coarse sand, trace gravel, trace silt.	
										165			
										134			
										141			
										141			
35	1U	24/24	35.00 - 37.00	WOR/WOR			110			Grey, wet, soft to medium stiff, Silty CLAY, trace fine sand.	G,C#210088 A-6, CL WC=41.5% LL=33 PL=22 PI=11		
							121						
	V1		37.57 - 38.00	Su=522/96 psf			125			65x130 mm vane raw torque readings: V1: 19.0/3.5 ft-lbs V2: 18.0/4.0 ft-lbs			
	V2		38.57 - 39.00	Su=494/110 psf			116						
							119						
40	7D	24/24	40.50 - 42.50	Push thru vane			112			Grey, wet, soft to medium stiff, Silty CLAY, trace fine sand.		G#210079 A-6, CL WC=35.2% LL=33 PL=22 PI=11	
	V3		41.07 - 41.50	Su=492/82 psf			126			65x130 mm vane raw torque readings: V3: 17.9/3.0 ft-lbs V4: 18.5/3.5 ft-lbs			
	V4		42.07 - 42.50	Su=508/96 psf			124						
							122						
							122						
							122						
45	2U	24/22	45.00 - 47.00	WOR/WOR			130			Grey, wet, soft, Silty CLAY, trace fine sand.	G,C#210087 A-6, CL WC=42.8% LL=32 PL=20 PI=12		
							122						
	V5		47.57 - 48.00	Su=453/60 psf			124			65x130 mm vane raw torque readings: V5: 16.5/2.2 ft-lbs V6: 17.0/2.2 ft-lbs			
	V6		48.57 - 49.00	Su=467/60 psf			110						
50							117						

**Remarks:**

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

<b>Driller:</b> MaineDOT	<b>Elevation (ft.):</b> 9.2	<b>Auger ID/OD:</b> 5" Solid Stem Auger
<b>Operator:</b> E. Giguere/B. Wilder	<b>Datum:</b> NAVD 88	<b>Sampler:</b> Standard Split Spoon
<b>Logged By:</b> B. Wilder/K. Maguire	<b>Rig Type:</b> CME 45C	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 3/27/08, 4/1/08-4/2/08	<b>Drilling Method:</b> Cased Wash Boring	<b>Core Barrel:</b> NQ-2"
<b>Boring Location:</b> 3+58.7, 6.4 Rt.	<b>Casing ID/OD:</b> HW	<b>Water Level*:</b> Tidal

**Hammer Efficiency Factor:** 0.77      **Hammer Type:** Automatic  Hydraulic  Rope & Cathead

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

Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows	Elevation (ft.)			
50							108			Grey, wet, soft to medium stiff, Silty CLAY, trace sand. 65x130 mm vane raw torque readings: V7: 18.0/3.5 ft-lbs V8: 21.0/4.0 ft-lbs	G#210080 A-7-6, CL WC=38.3% LL=40 PL=22 PI=18
	8D V7	24/24	51.00 - 53.00	WOR/WOR/WOR/ WOR	---		119				
	V8		52.57 - 53.00	Su=494/96 psf Su=577/110 psf				105			
								102			
55							99			Grey, wet, medium stiff, Silty CLAY, trace sand, trace gravel.  65x130 mm vane raw torque readings: V9: 23.0/6.0 ft-lbs V10: 26.0/6.5 ft-lbs	G,C#210089 A-6, CL WC=40.5% LL=34 PL=22 PI=12
	3U	24/24	55.00 - 57.00	WOR/WOR			113				
	V9		57.57 - 58.00	Su=632/165 psf				104			
	V10		58.57 - 59.00	Su=714/179 psf				103			
60							101			Grey, wet, medium stiff, Silty CLAY, trace sand, trace gravel. 65x130 mm vane raw torque readings: V11: 25.0/7.0 ft-lbs V12: 25.0/7.0 ft-lbs	G#210081 A-6, CL WC=34.0% LL=37 PL=21 PI=16
	9D V11	24/24	60.00 - 62.00	WOR/WOR/WOR/ WOR	---		99				
	V12		61.57 - 62.00	Su=687/192 psf Su=687/192 psf				100			
								105			
65							102			bHydraulic Push Grey, wet, medium stiff, SILT, some sand, some clay, trace gravel. Sand in bottom of tube. Bent tube at 66.6' bgs.	G,C#210090 A-4, CL-ML WC=21.5% Non-Plastic
	4U	19.2/16	65.00 - 66.60	WOR/bHP-0.58'			115				
								113			
								116			
70							94			Grey, wet, loose, fine SAND, trace medium to coarse sand, little silt, little clay, trace gravel.	G#210082 A-2-4, SC-SM WC=21.7%
	10D	24/20	67.00 - 69.00	WOR/2/3/7	5	6	116				
								94			
								102			
75							166			Grey, wet, medium dense, fine SAND, trace medium to coarse sand, trace silt, little gravel.	
	11D	24/16	70.00 - 72.00	9/9/8/10	17	22	99				
								166			
								165			
							154				
							133				

**Remarks:**

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

<b>Maine Department of Transportation</b>		<b>Project:</b> Bourne Ave. Bridge over Stevens Brook	<b>Boring No.:</b> BB-WSB-103
Soil/Rock Exploration Log US CUSTOMARY UNITS		<b>Location:</b> Wells, Maine	<b>PIN:</b> 15611.00
<b>Driller:</b>	MaineDOT	<b>Elevation (ft.):</b>	9.2
<b>Operator:</b>	E. Giguere/B. Wilder	<b>Datum:</b>	NAVD 88
<b>Logged By:</b>	B. Wilder/K. Maguire	<b>Rig Type:</b>	CME 45C
<b>Date Start/Finish:</b>	3/27/08, 4/1/08-4/2/08	<b>Drilling Method:</b>	Cased Wash Boring
<b>Boring Location:</b>	3+58.7, 6.4 Rt.	<b>Casing ID/OD:</b>	HW
<b>Hammer Efficiency Factor:</b> 0.77		<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
<small>           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 = weight of rods            WO1P = Weight of one person            S<sub>u</sub> = Insitu Field Vane Shear Strength (psf)            T<sub>v</sub> = Pocket Torvane Shear Strength (psf)            q<sub>p</sub> = Unconfined Compressive Strength (ksf)            N-uncorrected = Raw field SPT N-value            Hammer Efficiency Factor = Annual Calibration Value            N<sub>60</sub> = SPT N-uncorrected corrected for hammer efficiency            N<sub>60</sub> = (Hammer Efficiency Factor/60%) * N-uncorrected            S<sub>u(lab)</sub> = 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         </small>			

Sample Information											Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows	Elevation (ft.)						
75	12D	24/12	75.00 - 77.00	7/7/6/4	13	17	144				-68.00	Grey, wet, medium dense, Silty fine to coarse SAND, some gravel, trace clay, (Till).  Roller Coned ahead from 77.0-77.2' bgs.	G#210083 A-4, SC-SM WC=11.9%	
	R1	60/60	77.20 - 82.20	RQD = 45%			NO CORE							-77.10
	R2	49.2/49	82.20 - 86.30	RQD = 46%								<b>Bottom of Exploration at 86.30 feet below ground surface.</b>		
80														
85														
90														
95														
100														

**Remarks:**

## **Appendix B**

Laboratory Data

State of Maine - Department of Transportation  
**Laboratory Testing Summary Sheet**

**Town(s): Wells**

**Project Number: 15611.00**

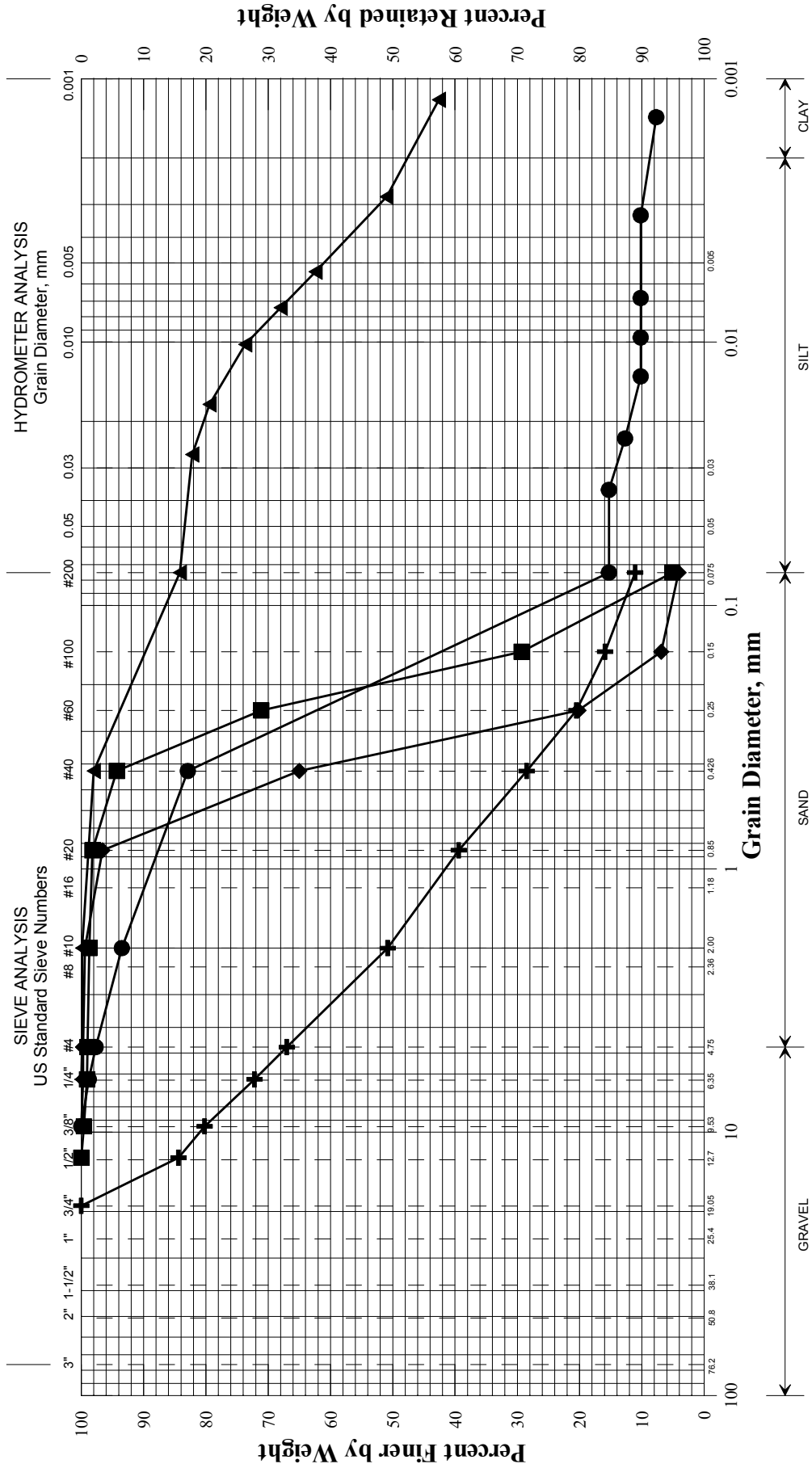
Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C.	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-WSB-101, 2D	2+64.5	6.7 Lt.	5.0-7.0	209933	1	7.5			SW-SM	A-1-b	0
BB-WSB-101, 4D	2+64.5	6.7 Lt.	15.0-17.0	209935	1	32.6			SP	A-3	0
BB-WSB-101, 6D	2+64.5	6.7 Lt.	25.0-27.0	209936	1	21.4			SP-SM	A-3	0
BB-WSB-101, 7D	2+64.5	6.7 Lt.	30.0-30.5	209937	1	18.5			SM	A-2-4	II
BB-WSB-101, 7D/A	2+64.5	6.7 Lt.	30.5-32.0	209938	1	29.9	30	10	CL	A-6	IV
BB-WSB-101, 8D	2+64.5	6.7 Lt.	35.0-37.0	209939	---	34.3	30	9	CL	A-4	IV
BB-WSB-101, 1U	2+64.5	6.7 Lt.	40.0-42.0	210084	2	41.8	34	13	CL	A-6	III
BB-WSB-101, 9D	2+64.5	6.7 Lt.	45.0-47.0	209940	2	33.5	34	12	CL	A-6	III
BB-WSB-101, 2U	2+64.5	6.7 Lt.	50.0-52.0	210085	2	41.7	38	16	CL	A-6	III
BB-WSB-101, 10D	2+64.5	6.7 Lt.	55.5-57.5	209941	2	35.0			CL	A-6	III
BB-WSB-102, 1D	3+07.7	6.5 Rt.	0.0-2.0	209942	3	23.4			SP-SC	A-2-4	II
BB-WSB-102, 3D	3+07.7	6.5 Rt.	8.0-10.0	209943	3	20.9			SP	A-3	0
BB-WSB-102, 4D	3+07.7	6.5 Rt.	13.0-15.0	209944	3	17.5			SM	A-2-4	II
BB-WSB-102, 5D	3+07.7	6.5 Rt.	18.5-20.5	209945	3	29.9	30	12	CL	A-6	III
BB-WSB-102, 7D	3+07.7	6.5 Rt.	28.5-30.5	209946	3	33.1	34	12	CL	A-6	III
BB-WSB-102, 8D	3+07.7	6.5 Rt.	33.5-35.5	209947	4	35.4	36	14	CL	A-6	III
BB-WSB-102, 10D	3+07.7	6.5 Rt.	43.5-45.5	209948	4	36.9	34	12	CL	A-6	III
BB-WSB-102, 11D	3+07.7	6.5 Rt.	48.5-50.5	209949	4	19.1			SP-SM	A-2-4	II
BB-WSB-102, 12D	3+07.7	6.5 Rt.	53.0-55.0	209950	4	16.1			SM	A-2-4	II
BB-WSB-103, 1D	3+58.7	6.4 Rt.	5.0-7.0	210076	5	5.1			SW-SM	A-1-b	0
BB-WSB-103, 2D	3+58.7	6.4 Rt.	10.0-12.0	210077	5	22.7			SP	A-3	0
BB-WSB-103, 4D	3+58.7	6.4 Rt.	20.0-22.0	210078	5	23.4			SP	A-3	0
BB-WSB-103, 1U	3+58.7	6.4 Rt.	35.0-37.0	210088	5	41.5	33	11	CL	A-6	IV
BB-WSB-103, 7D	3+58.7	6.4 Rt.	40.5-42.5	210079	5	35.2	33	11	CL	A-6	IV
BB-WSB-103, 2U	3+58.7	6.4 Rt.	45.0-47.0	210087	5	42.8	32	12	CL	A-6	III
BB-WSB-103, 8D	3+58.7	6.4 Rt.	51.0-53.0	210080	6	38.3	40	18	CL	A-7-6	III
BB-WSB-103, 3U	3+58.7	6.4 Rt.	55.0-57.0	210089	6	40.5	34	12	CL	A-6	III
BB-WSB-103, 9D	3+58.7	6.4 Rt.	60.0-62.0	210081	6	34.0	37	16	CL	A-6	III
BB-WSB-103, 4U	3+58.7	6.4 Rt.	65.0-66.6	210090	6	21.5	-N	P-	CL-ML	A-4	IV
BB-WSB-103, 10D	3+58.7	6.4 Rt.	67.0-69.0	210082	6	21.7			SC-SM	A-2-4	III
BB-WSB-103, 12D	3+58.7	6.4 Rt.	75.0-77.0	210083	6	11.9			SC-SM	A-4	III

**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 MDOT 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

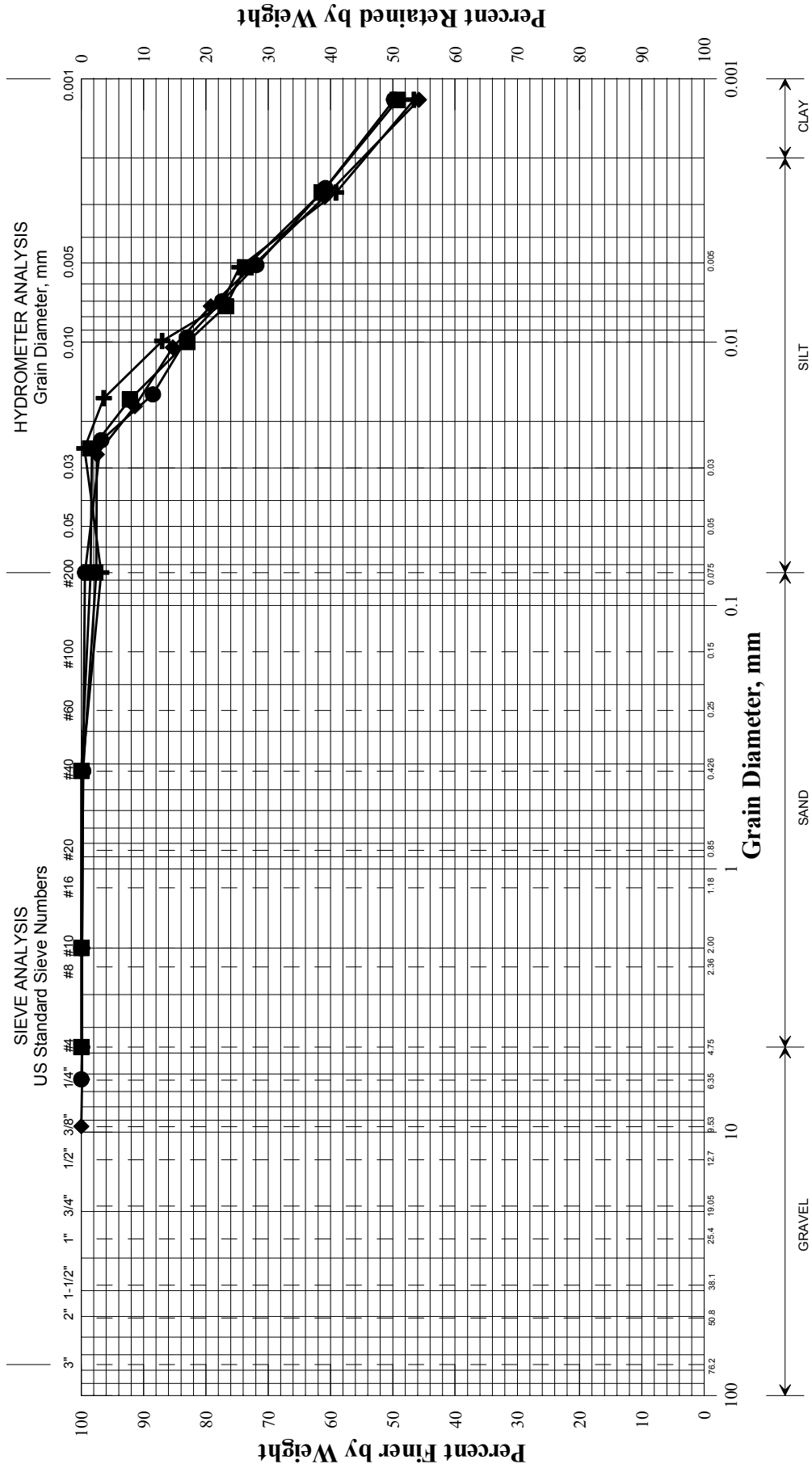


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	2+64.5	6.7 LT	5.0-7.0	SAND, some gravel, little silt.	7.5			
◆	2+64.5	6.7 LT	15.0-17.0	SAND, trace silt, trace gravel.	32.6			
■	2+64.5	6.7 LT	25.0-27.0	SAND, trace silt, trace gravel.	21.4			
●	2+64.5	6.7 LT	30.0-30.5	SAND, trace clay, trace silt, trace gravel.	18.5			
▲	2+64.5	6.7 LT	30.5-32.0	Silty CLAY, little sand, trace gravel.	29.9	30	20	10

015611.00	PIN
Wells	Town
WHITE, TERRY A	Reported by/Date
	7/17/2008

*State of Maine Department of Transportation*  
**GRAIN SIZE DISTRIBUTION CURVE**

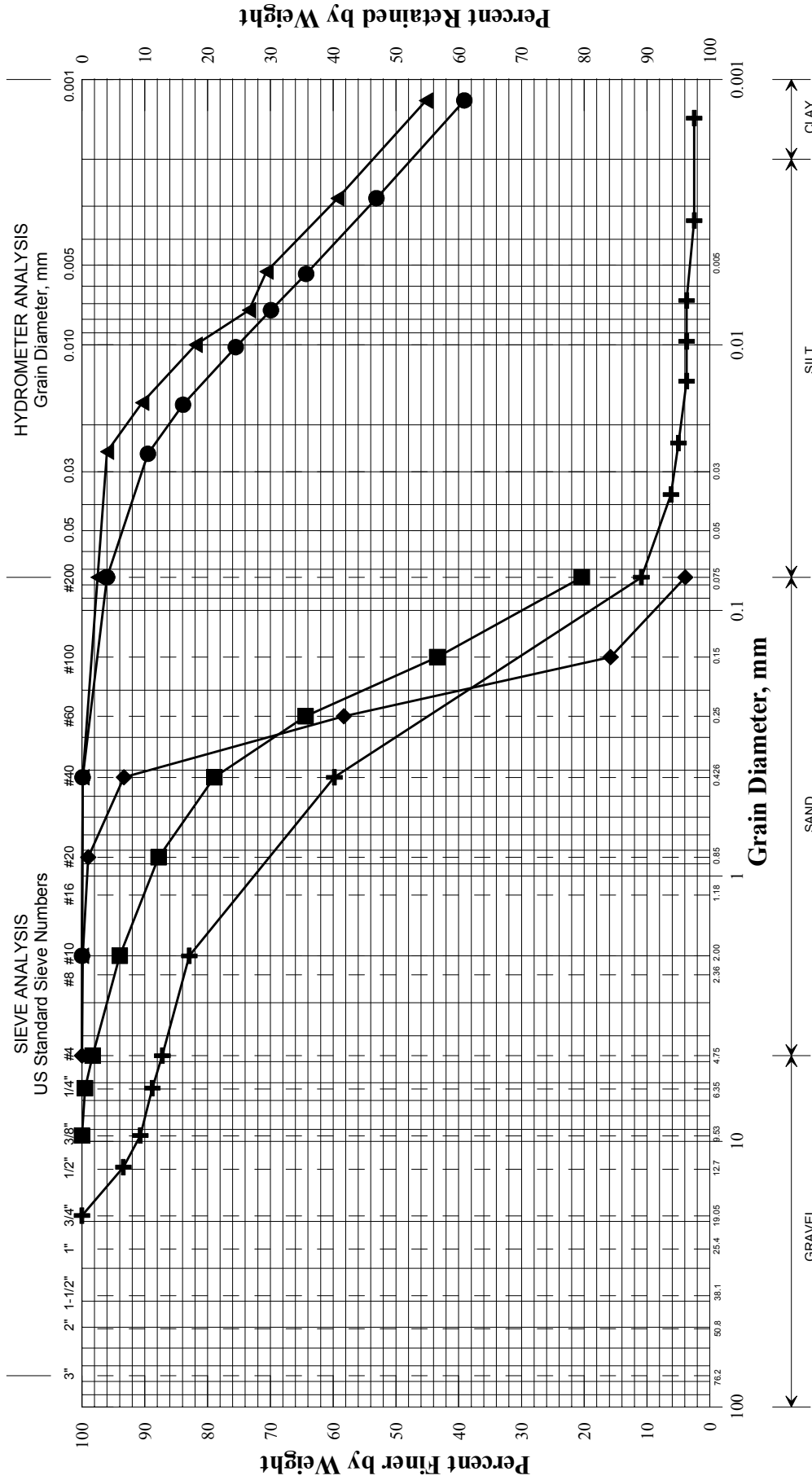


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	2+64.5	6.7 LT	40.0-42.0	Silty CLAY, trace sand.	41.8	34	21	13
◆	2+64.5	6.7 LT	45.0-47.0	Silty CLAY, trace sand, trace gravel.	33.5	34	22	12
■	2+64.5	6.7 LT	50.0-52.0	Silty CLAY, trace sand.	41.7	38	22	16
●	2+64.5	6.7 LT	55.5-57.5	Silty CLAY, trace sand, trace gravel.	35.0			
▲								
×								

015611.00	PIN
Wells	Town
WHITE, TERRY A	Reported by/Date
	5/14/2008

*State of Maine Department of Transportation*  
**GRAIN SIZE DISTRIBUTION CURVE**

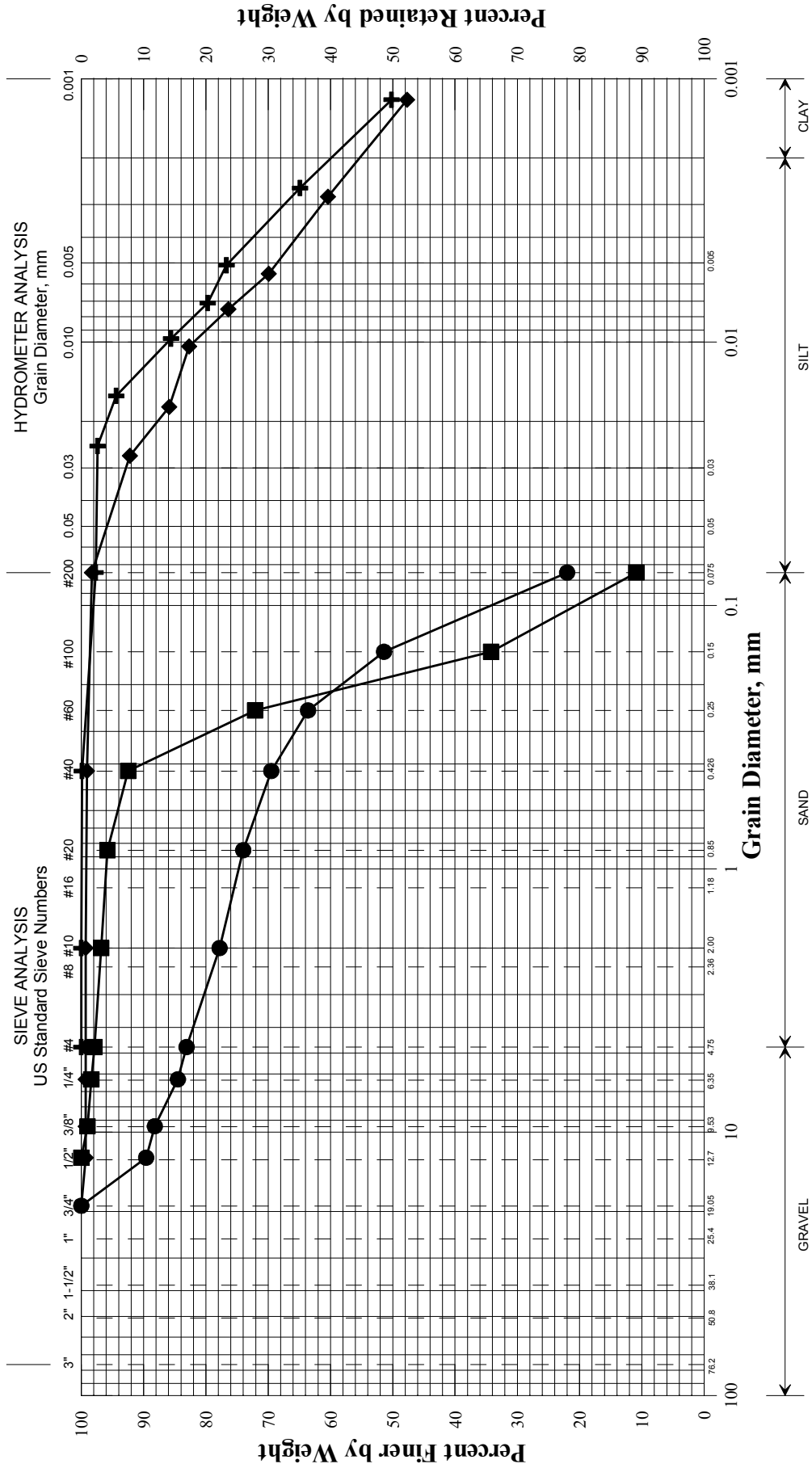


**UNIFIED CLASSIFICATION**

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	3+07.7	6.5 RT	0.0-2.0	SAND, little gravel, trace silt, trace clay.	23.4			
◆	3+07.7	6.5 RT	8.0-10.0	SAND, trace silt.	20.9			
■	3+07.7	6.5 RT	13.0-15.0	SAND, some silt, trace gravel.	17.5			
●	3+07.7	6.5 RT	18.5-20.5	Silty CLAY, trace sand.	29.9	30	18	12
▲	3+07.7	6.5 RT	28.5-30.5	Silty CLAY, trace sand.	33.1	34	22	12

015611.00	PIN
Wells	Town
WHITE, TERRY A	Reported by/Date
	5/14/2008

*State of Maine Department of Transportation*  
GRAIN SIZE DISTRIBUTION CURVE

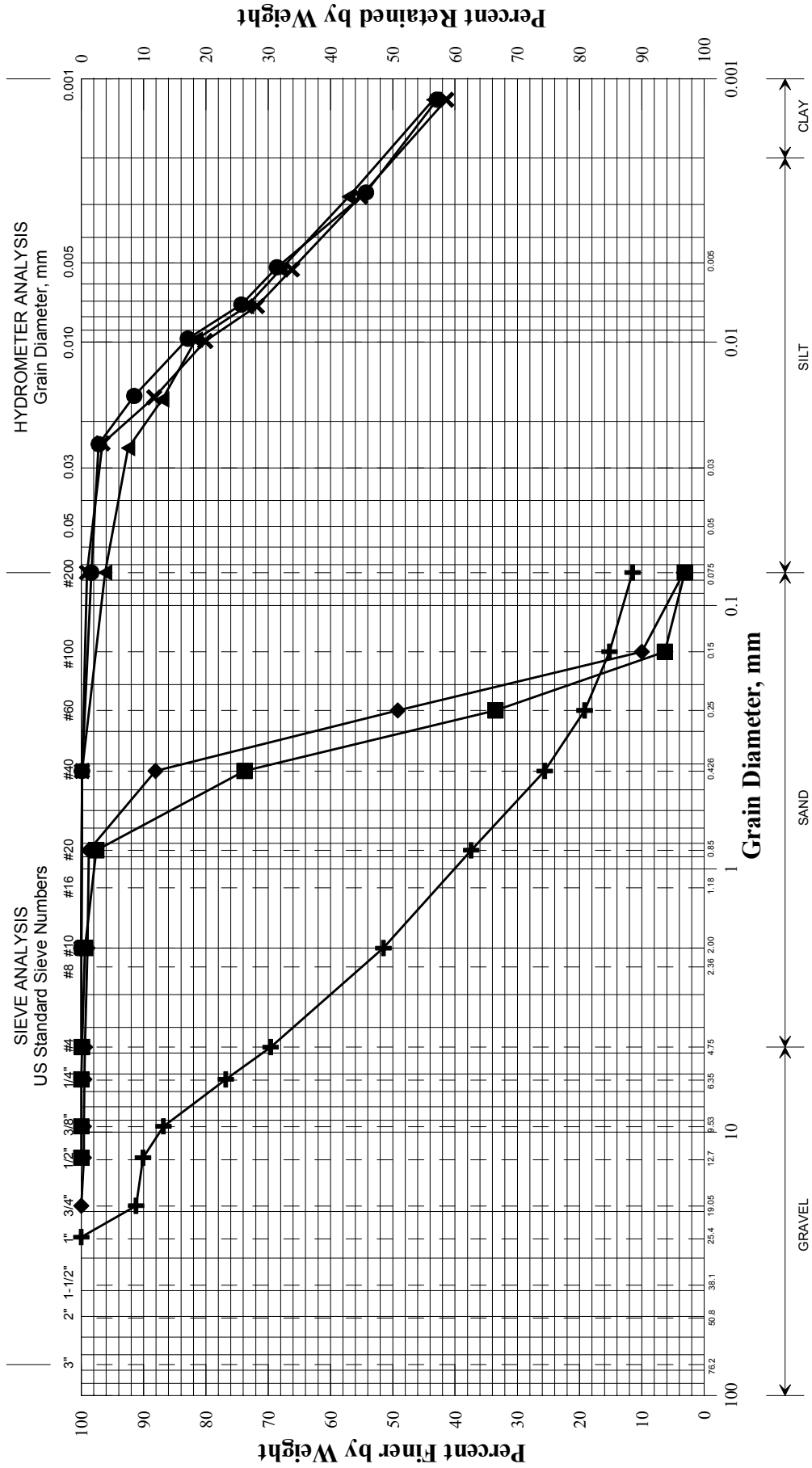


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	3+07.7	6.5 RT	33.5-35.5	Silty CLAY, trace sand.	35.4	36	22	14
◆	3+07.7	6.5 RT	43.5-45.5	Silty CLAY, trace sand, trace gravel.	36.9	34	22	12
■	3+07.7	6.5 RT	48.5-50.5	SAND, little silt, trace gravel.	19.1			
●	3+07.7	6.5 RT	53.0-55.0	SAND, some silt, little gravel.	16.1			
▲								
×								

PIN	015611.00
Town	Wells
Reported by/Date	WHITE, TERRY A 5/14/2008

*State of Maine Department of Transportation*  
GRAIN SIZE DISTRIBUTION CURVE

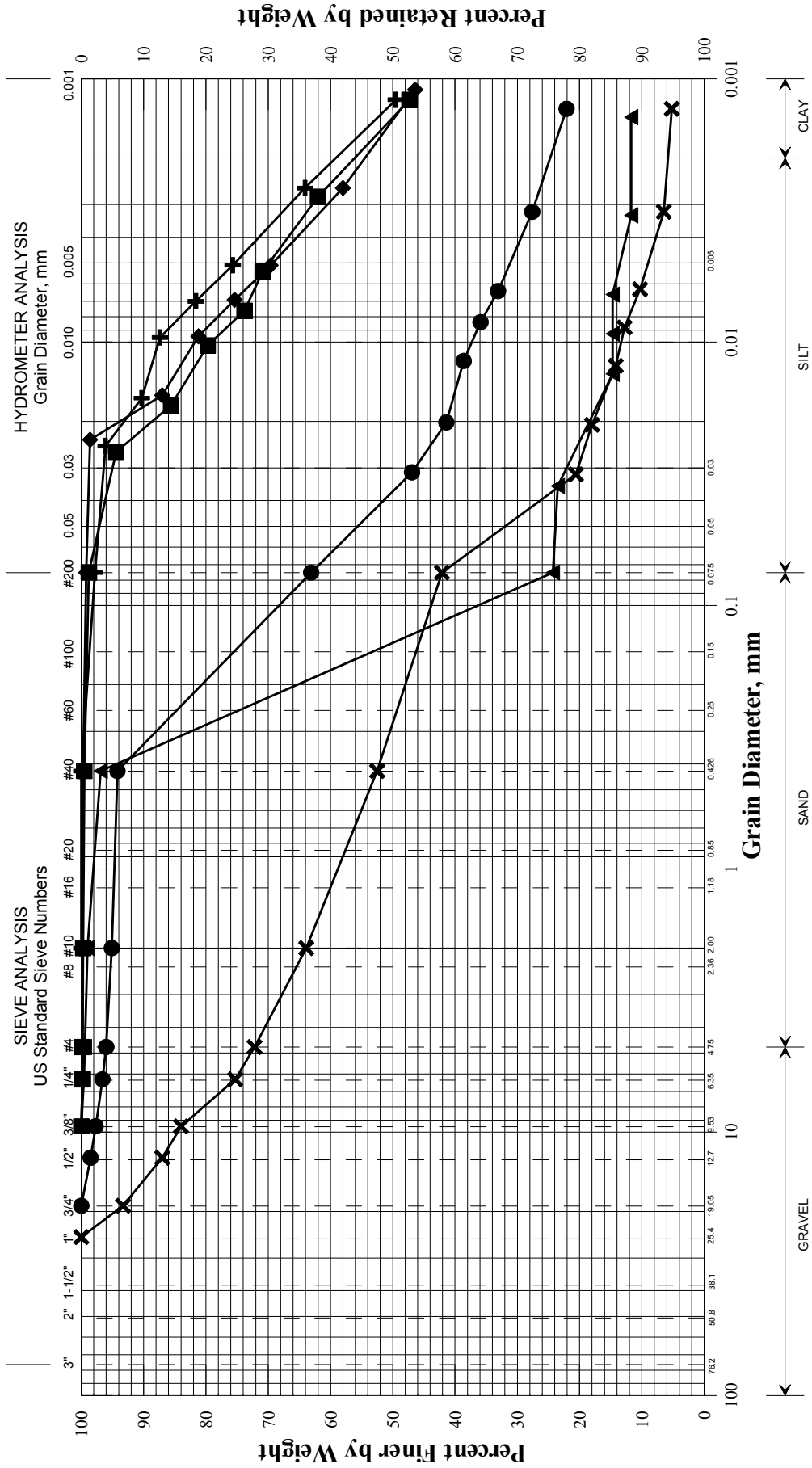


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+ BB-WSB-103/1D	3+58.7	6.4 RT	5.0-7.0	SAND, some gravel, little silt.	5.1			
◆ BB-WSB-103/2D	3+58.7	6.4 RT	10.0-12.0	SAND, trace silt, trace gravel.	22.7			
■ BB-WSB-103/4D	3+58.7	6.4 RT	20.0-22.0	SAND, trace silt, trace gravel.	23.4			
● BB-WSB-103/1U	3+58.7	6.4 RT	35.0-37.0	Silty CLAY, trace sand.	41.5	33	22	11
▲ BB-WSB-103/7D	3+58.7	6.4 RT	40.5-42.5	Silty CLAY, trace sand.	35.2	33	22	11
× BB-WSB-103/2U	3+58.7	6.4 RT	45.0-47.0	Silty CLAY, trace sand.	42.8	32	20	12

015611.00	PIN
Wells	Town
WHITE, TERRY A	Reported by/Date
5/14/2008	

*State of Maine Department of Transportation*  
GRAIN SIZE DISTRIBUTION CURVE



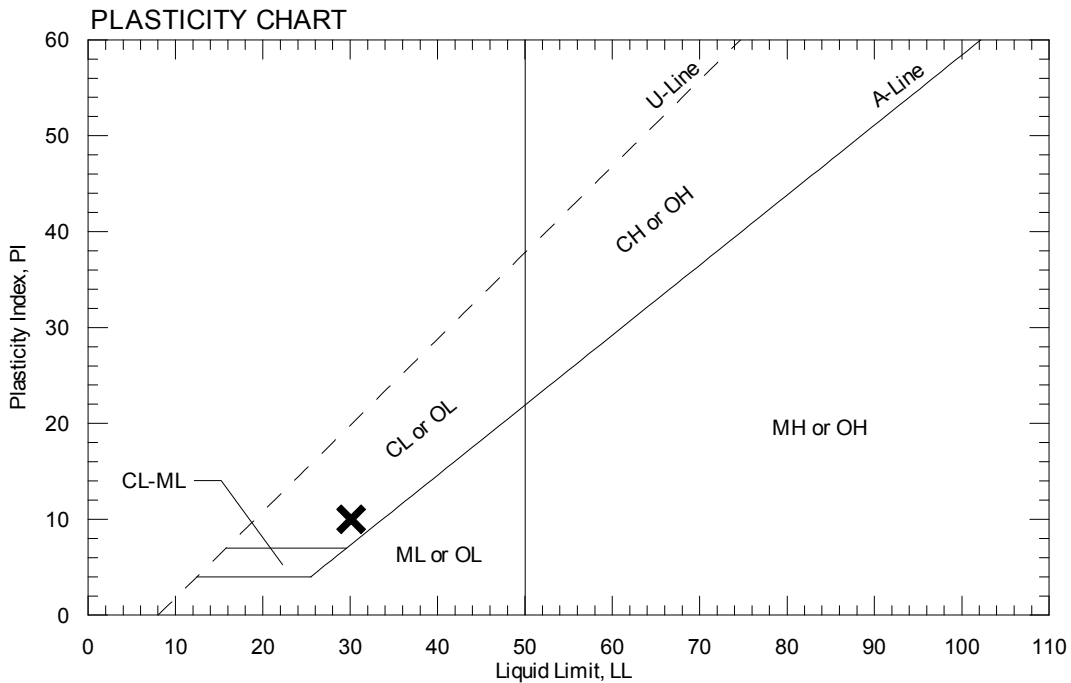
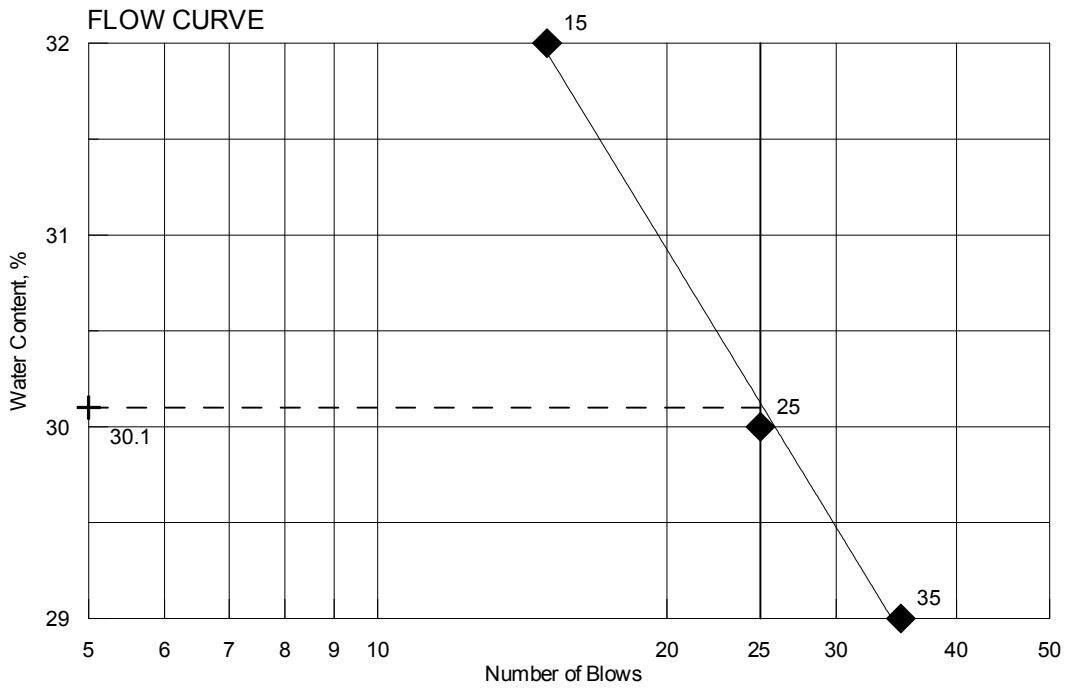
UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+ BB-WSB-103/8D	3+58.7	6.4 RT	51.0-53.0	Silty CLAY, trace sand.	38.3	40	22	18
◆ BB-WSB-103/3U	3+58.7	6.4 RT	55.0-57.0	Silty CLAY, trace sand, trace gravel.	40.5	34	22	12
■ BB-WSB-103/9D	3+58.7	6.4 RT	60.0-62.0	Silty CLAY, trace sand, trace gravel.	34.0	37	21	16
● BB-WSB-103/4U	3+58.7	6.4 RT	65.0-66.6	SILT, some sand, some clay, trace gravel.	21.5			NP
▲ BB-WSB-103/10D	3+58.7	6.4 RT	67.0-69.0	SAND, little silt, little clay, trace gravel.	21.7			
× BB-WSB-103/12D	3+58.7	6.4 RT	75.0-77.0	SILT, some sand, some gravel, trace clay.	11.9			

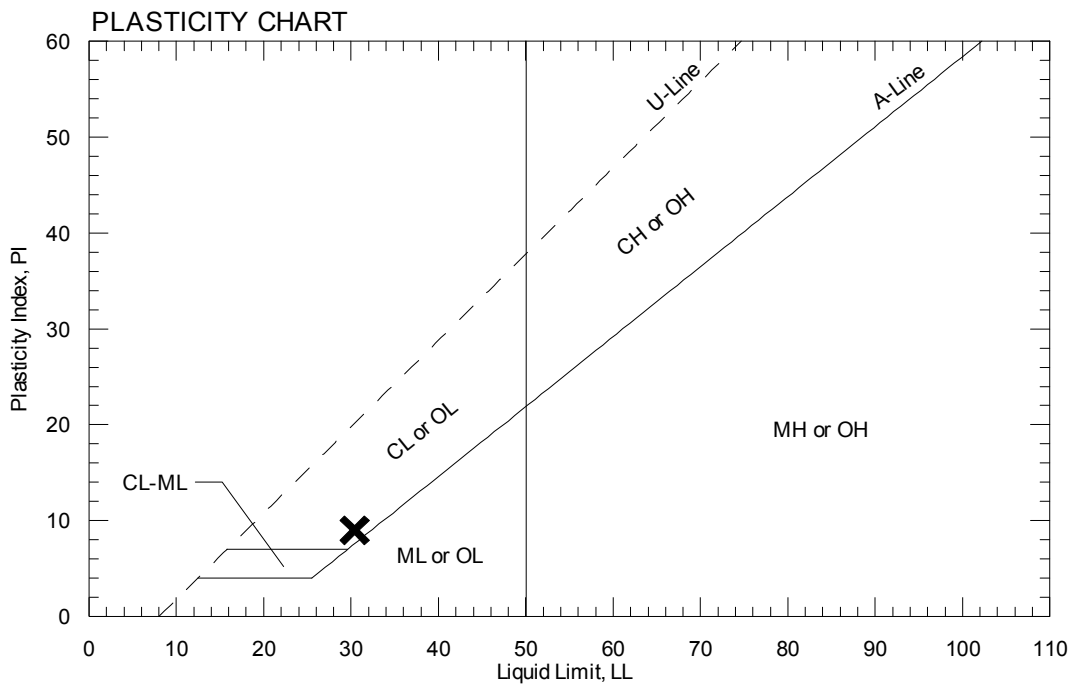
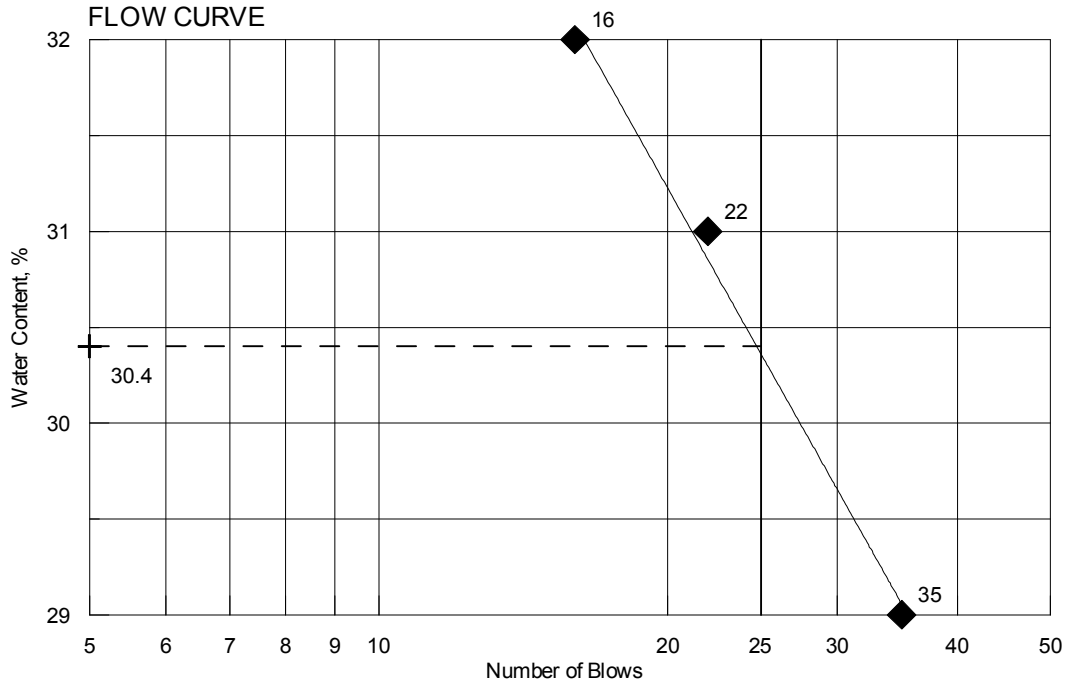
PIN	015611.00
Town	Wells
Reported by/Date	WHITE, TERRY A 5/14/2008

# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209938
PIN	015611.00	Water Content, %	29.9
Sampled	3/11/2008	Plastic Limit	20
Boring No./Sample No.	BB-WSB-101/7D(A)	Liquid Limit	30
Station	2+64.5	Plasticity Index	10
Depth	30.5-32.0	Tested By	BBURR



TOWN	Wells	Reference No.	209939
PIN	015611.00	Water Content, %	34.3
Sampled	3/13/2008	Plastic Limit	21
Boring No./Sample No.	BB-WSB-101/8D	Liquid Limit	30
Station	2+64.5	Plasticity Index	9
Depth	35.0-37.0	Tested By	BBURR



**A U T H O R I Z A T I O N   A N D   D I S T R I B U T I O N**

Reported by: **FOGG, BRIAN**

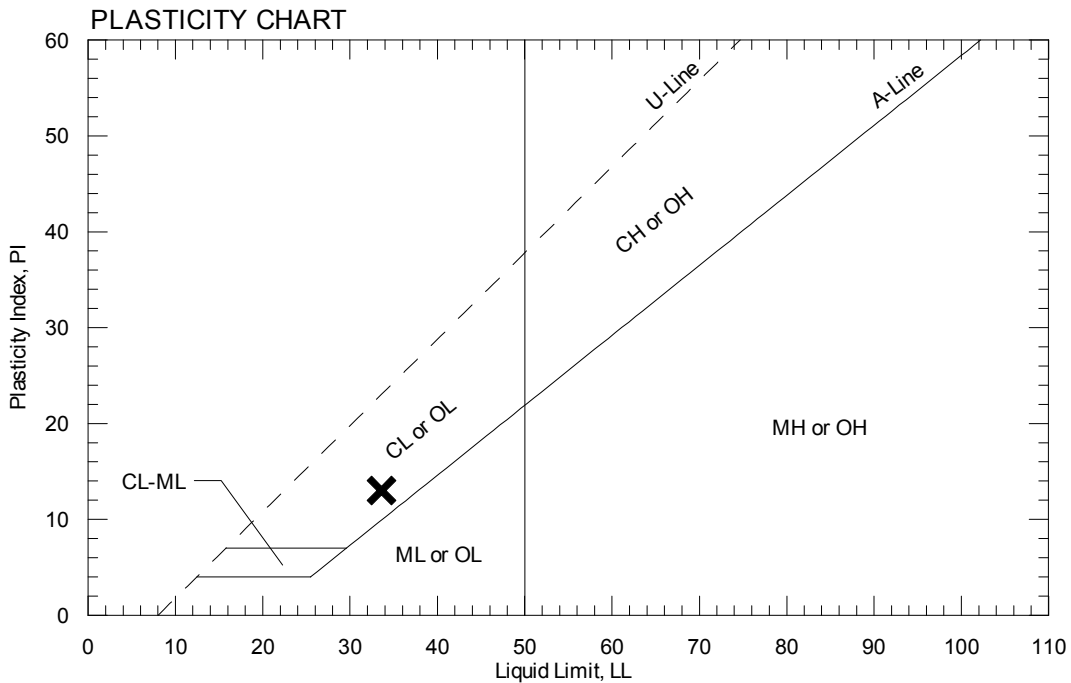
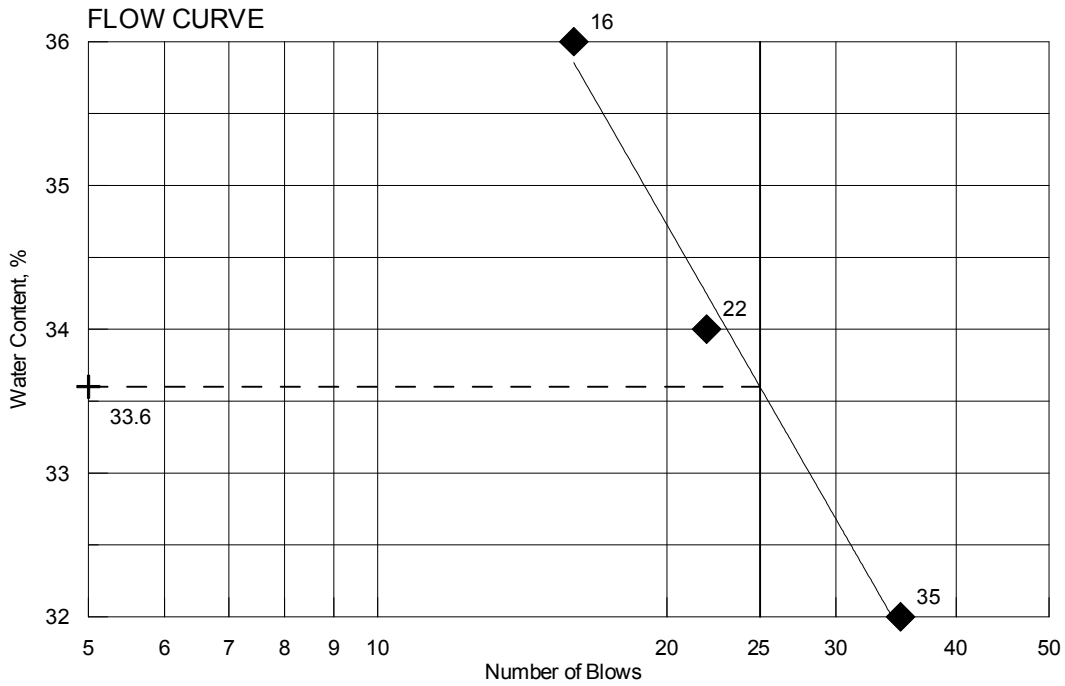
Date Reported: **4/29/2008**

Paper Copy: Lab File; Project File; Geotech File



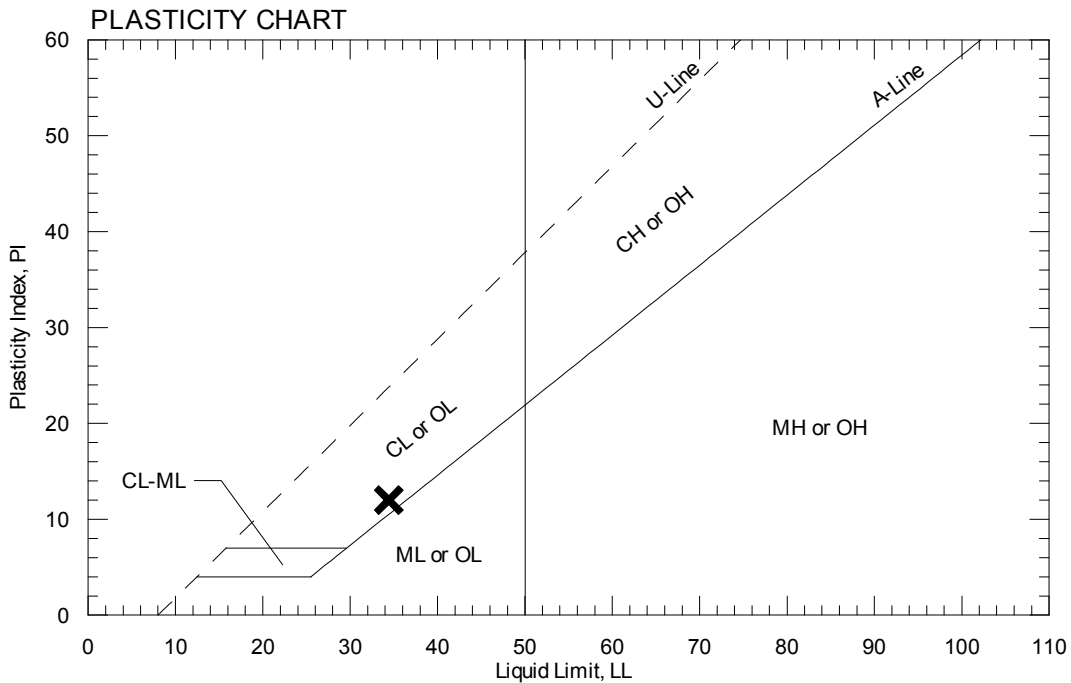
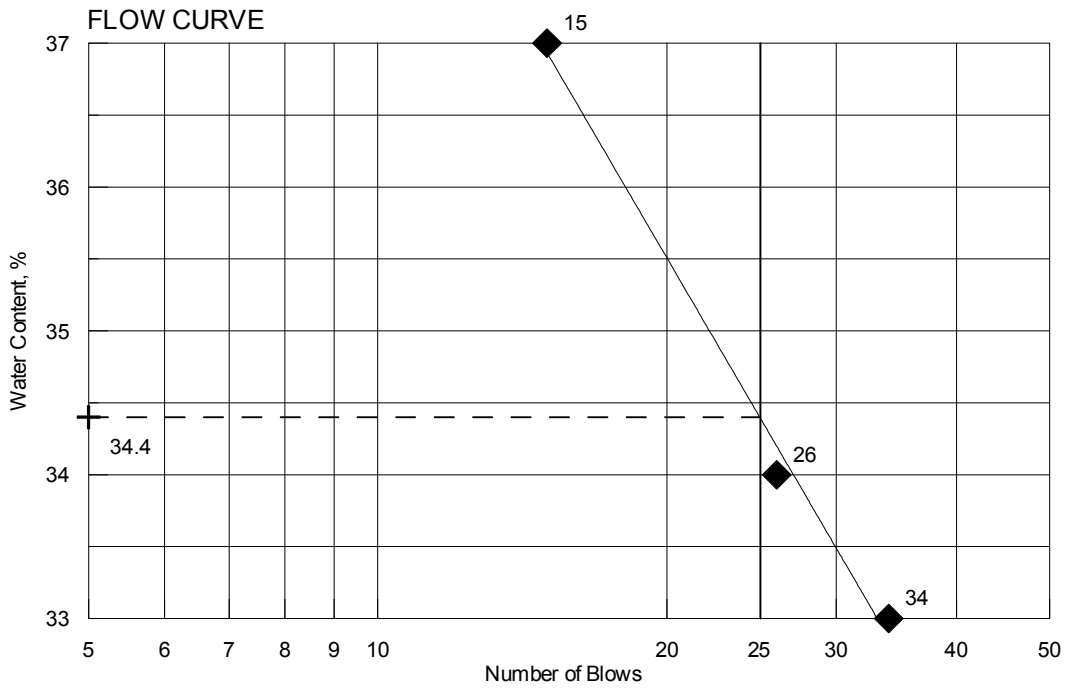
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210084
PIN	015611.00	Water Content, %	41.8
Sampled	3/13/2008	Plastic Limit	21
Boring No./Sample No.	BB-WSB-101/1U	Liquid Limit	34
Station	2+64.5	Plasticity Index	13
Depth	40.0-42.0	Tested By	BBURR



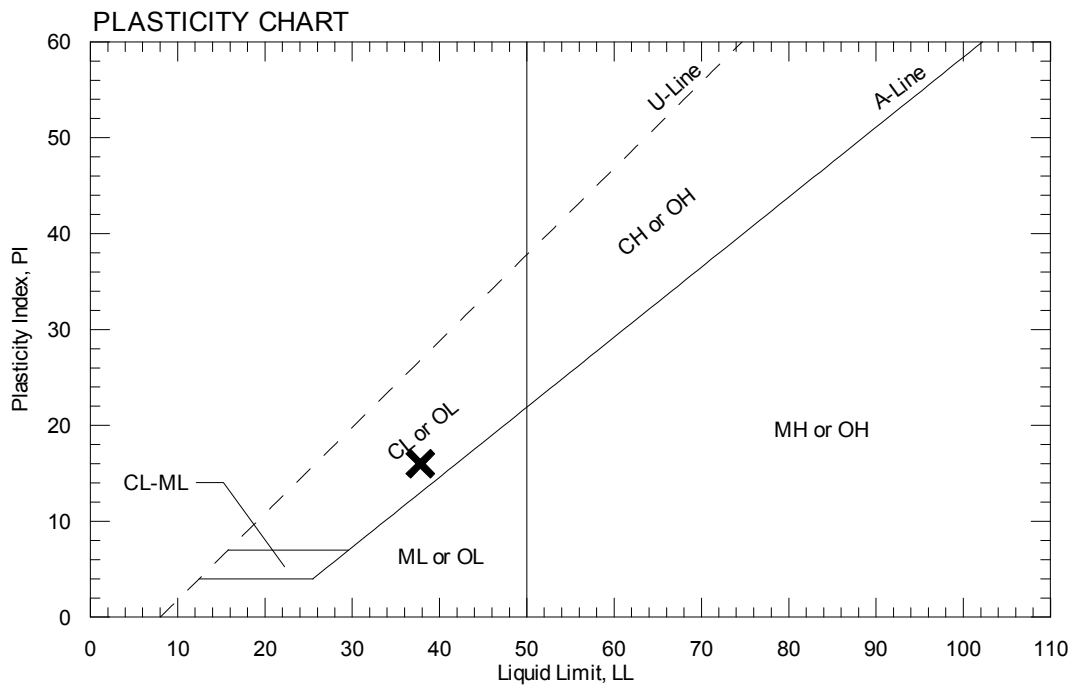
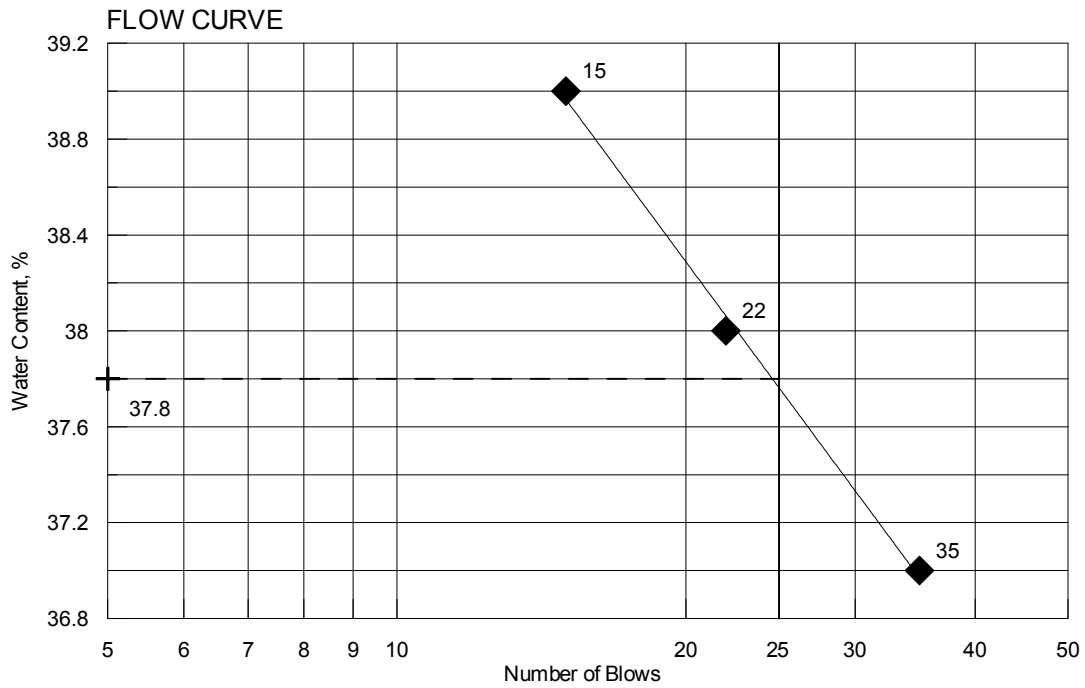
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209940
PIN	015611.00	Water Content, %	33.5
Sampled	3/13/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-101/9D	Liquid Limit	34
Station	2+64.5	Plasticity Index	12
Depth	45.0-47.0	Tested By	BBURR



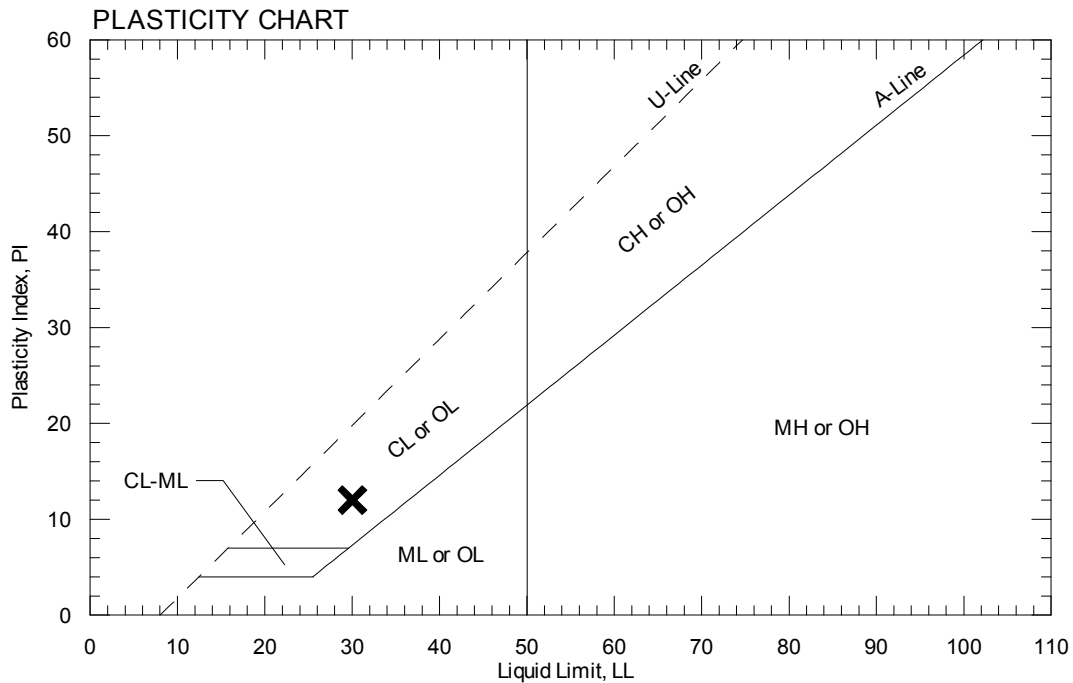
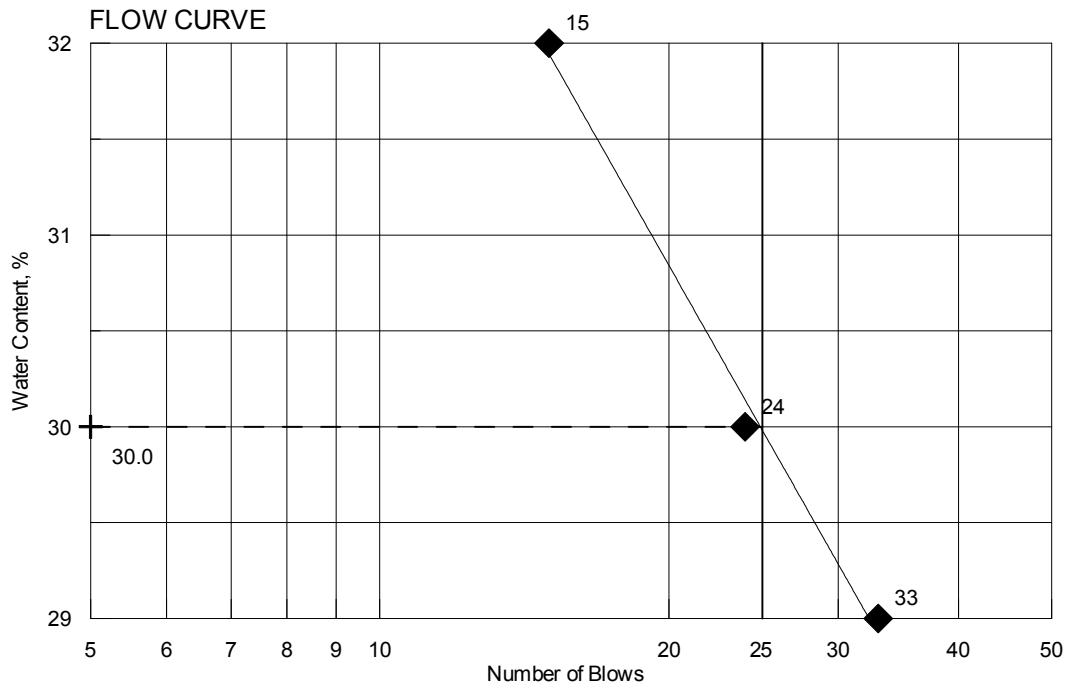
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210085
PIN	015611.00	Water Content, %	41.7
Sampled	3/18/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-101/2U	Liquid Limit	38
Station	2+64.5	Plasticity Index	16
Depth	50.0-52.0	Tested By	BBURR



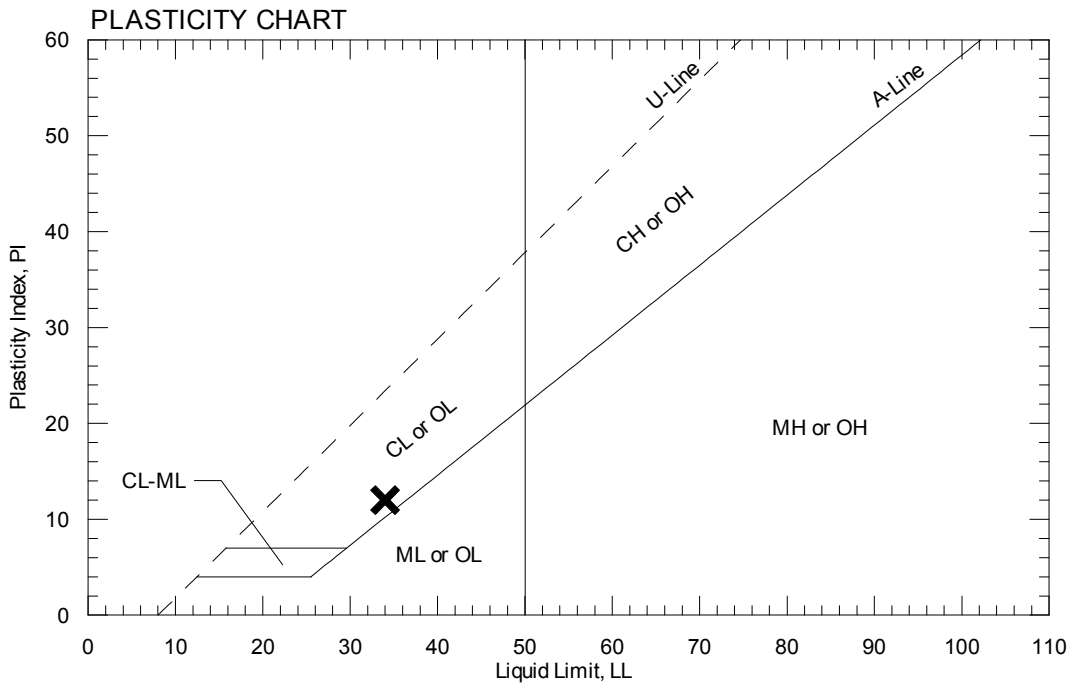
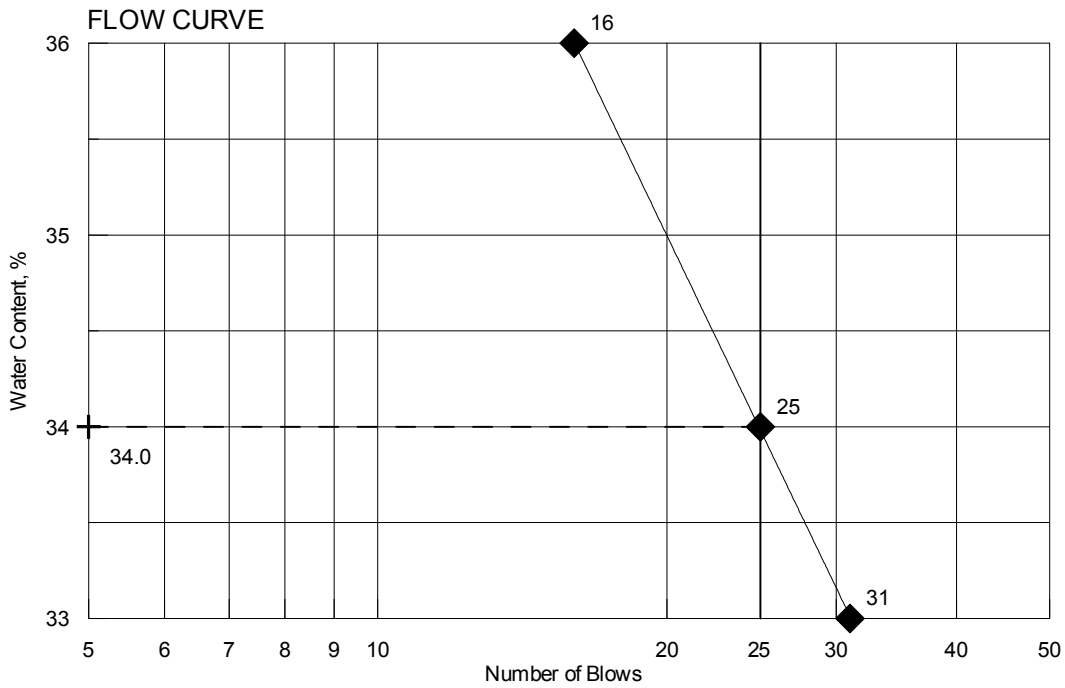
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209945
PIN	015611.00	Water Content, %	29.9
Sampled	4/2/2008	Plastic Limit	18
Boring No./Sample No.	BB-WSB-102/5D	Liquid Limit	30
Station	3+07.7	Plasticity Index	12
Depth	18.5-20.5	Tested By	BBURR



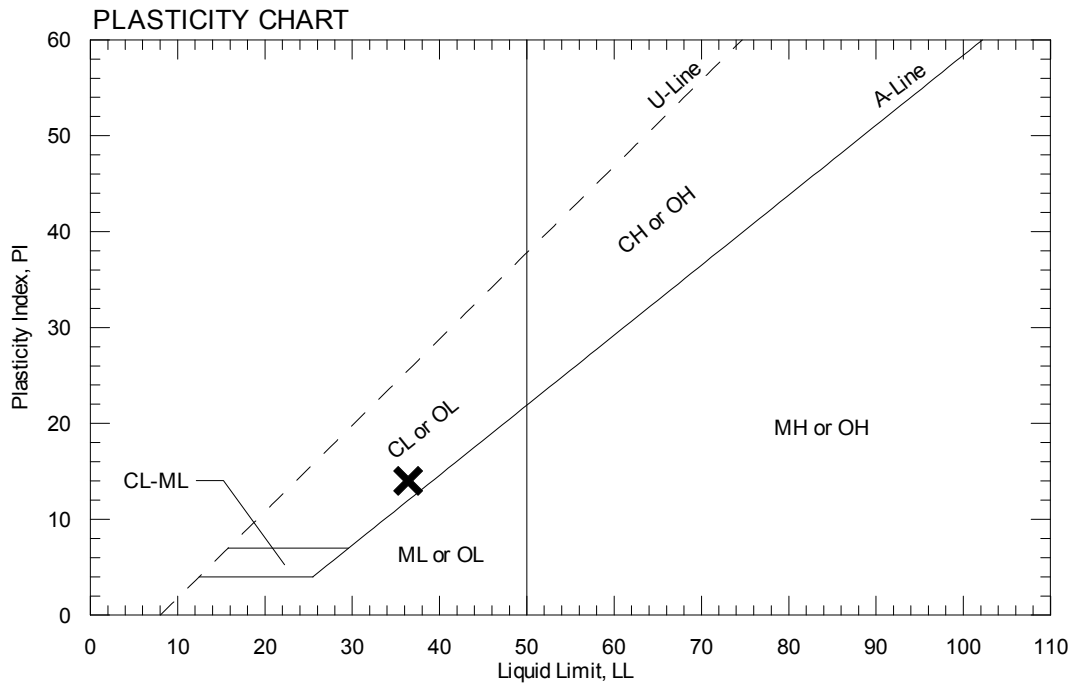
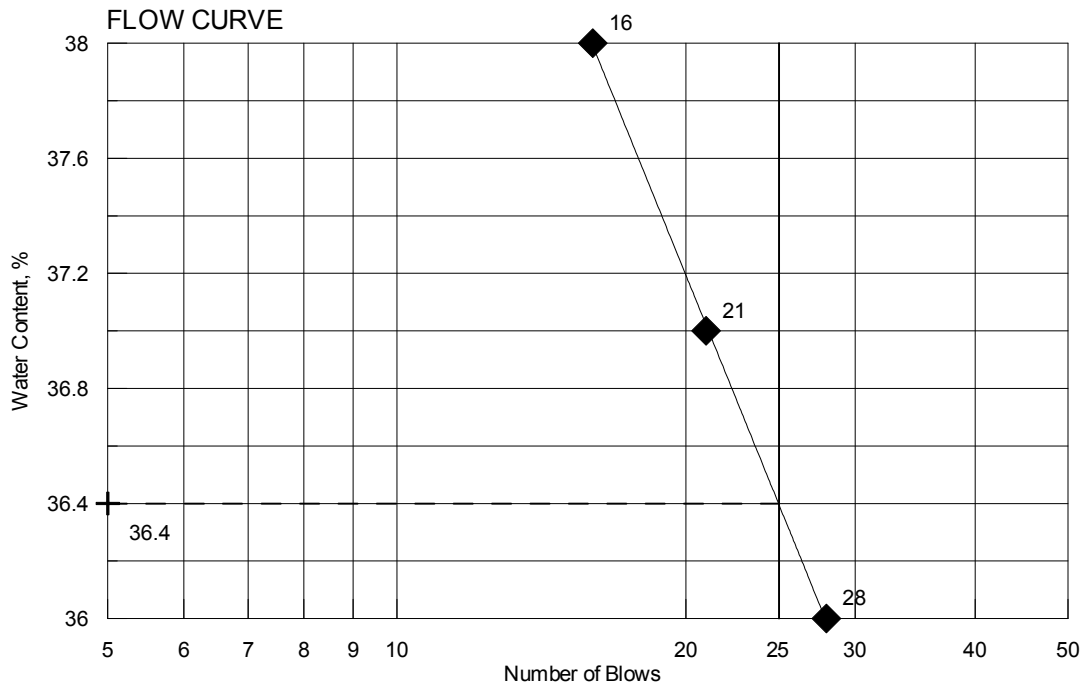
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209946
PIN	015611.00	Water Content, %	33.1
Sampled	4/3/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-102/7D	Liquid Limit	34
Station	3+07.7	Plasticity Index	12
Depth	28.5-30.5	Tested By	BBURR



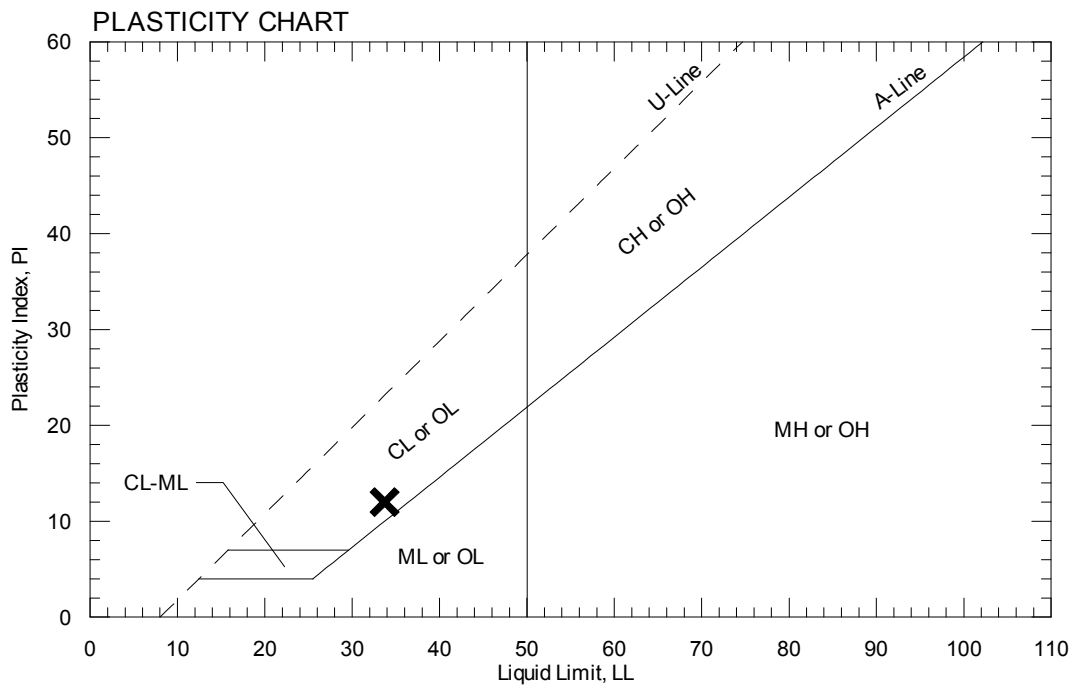
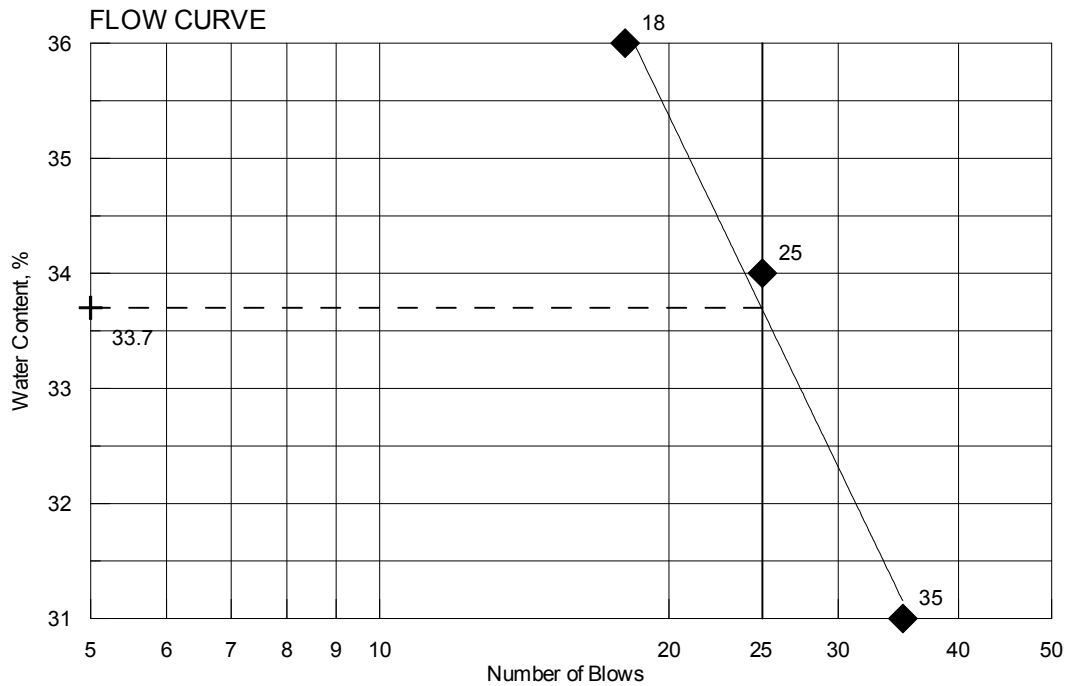
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209947
PIN	015611.00	Water Content, %	35.4
Sampled	4/3/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-102/8D	Liquid Limit	36
Station	3+07.7	Plasticity Index	14
Depth	33.5-35.5	Tested By	BBURR



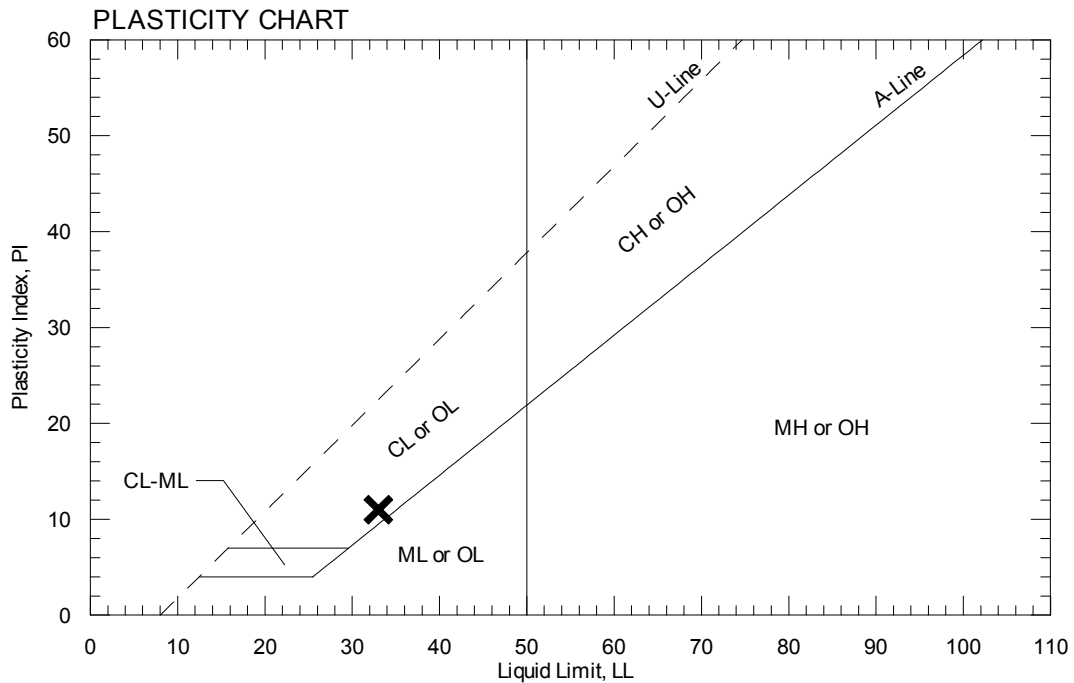
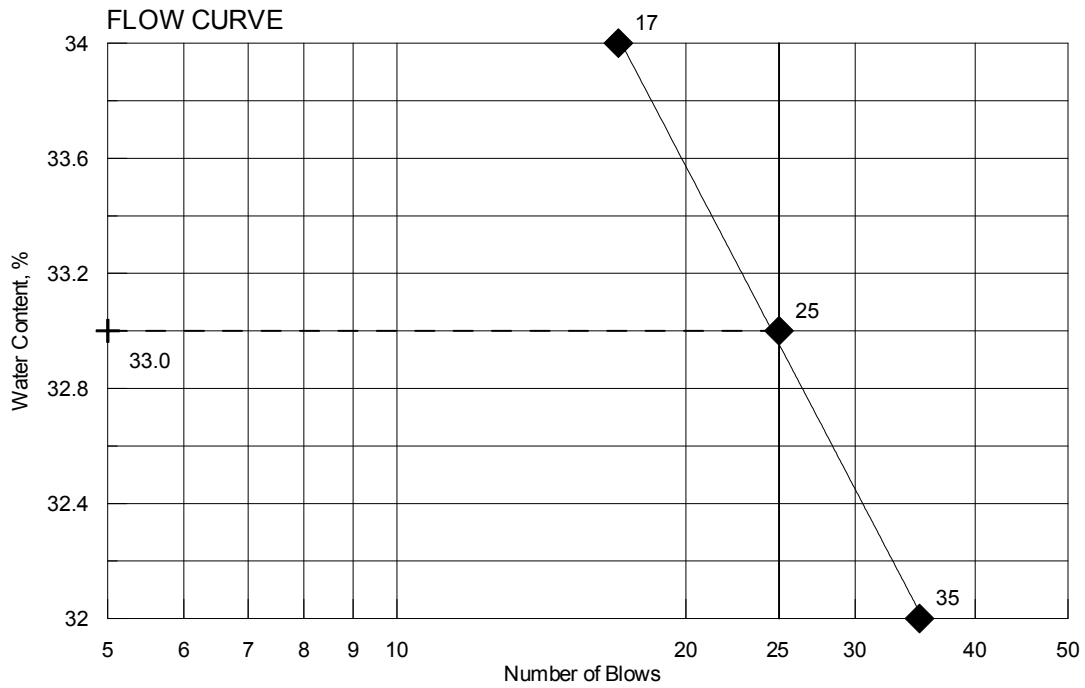
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	209948
PIN	015611.00	Water Content, %	36.9
Sampled	4/3/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-102/10D	Liquid Limit	34
Station	3+07.7	Plasticity Index	12
Depth	43.5-45.5	Tested By	BBURR



# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

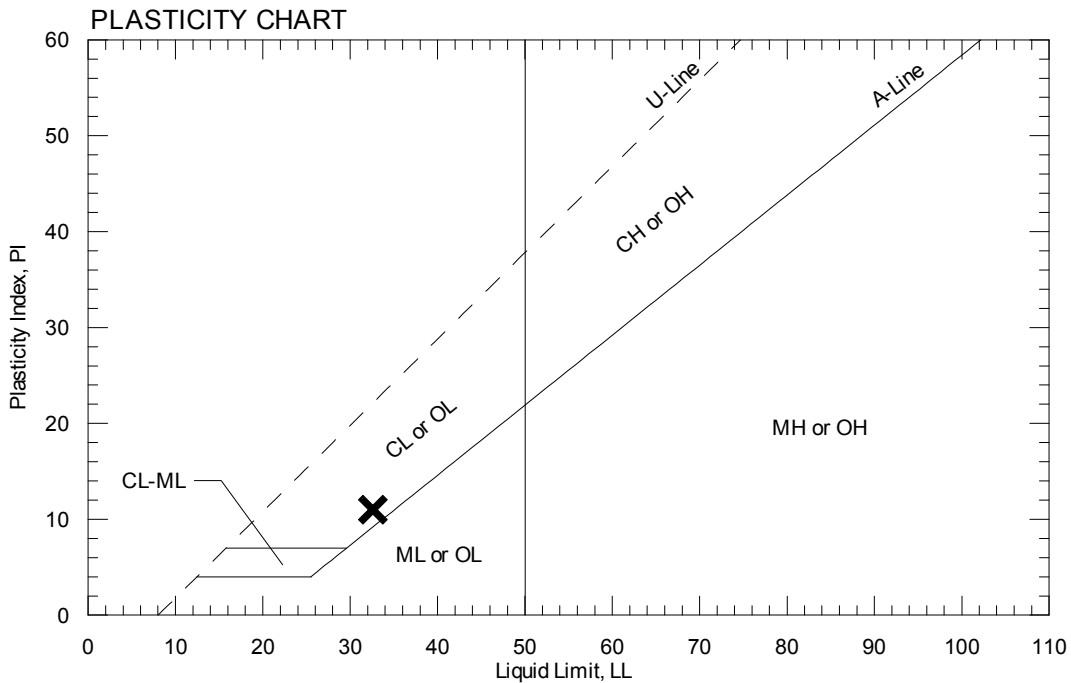
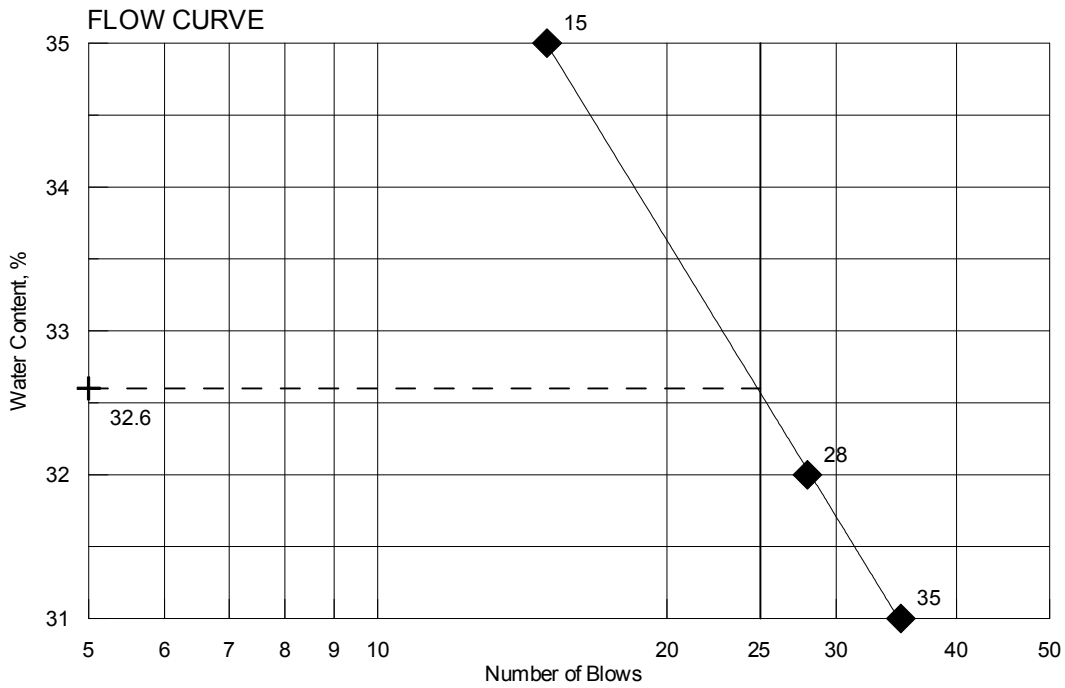
TOWN	Wells	Reference No.	210088
PIN	015611.00	Water Content, %	41.5
Sampled	3/27/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-103/1U	Liquid Limit	33
Station	3+58.7	Plasticity Index	11
Depth	35.0-37.0	Tested By	BBURR





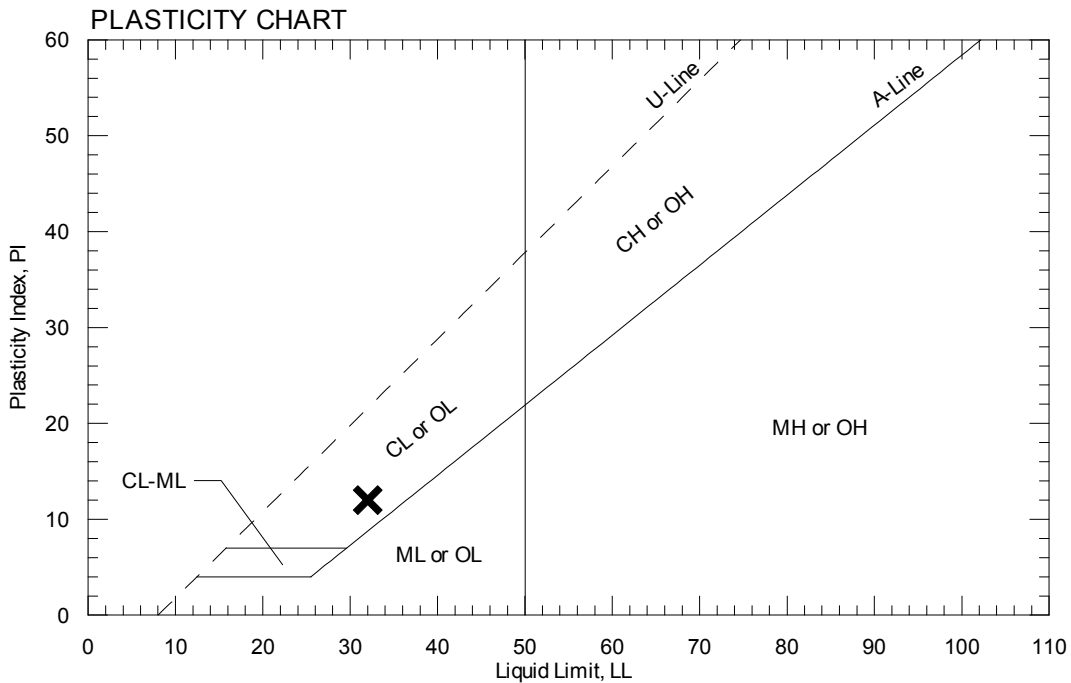
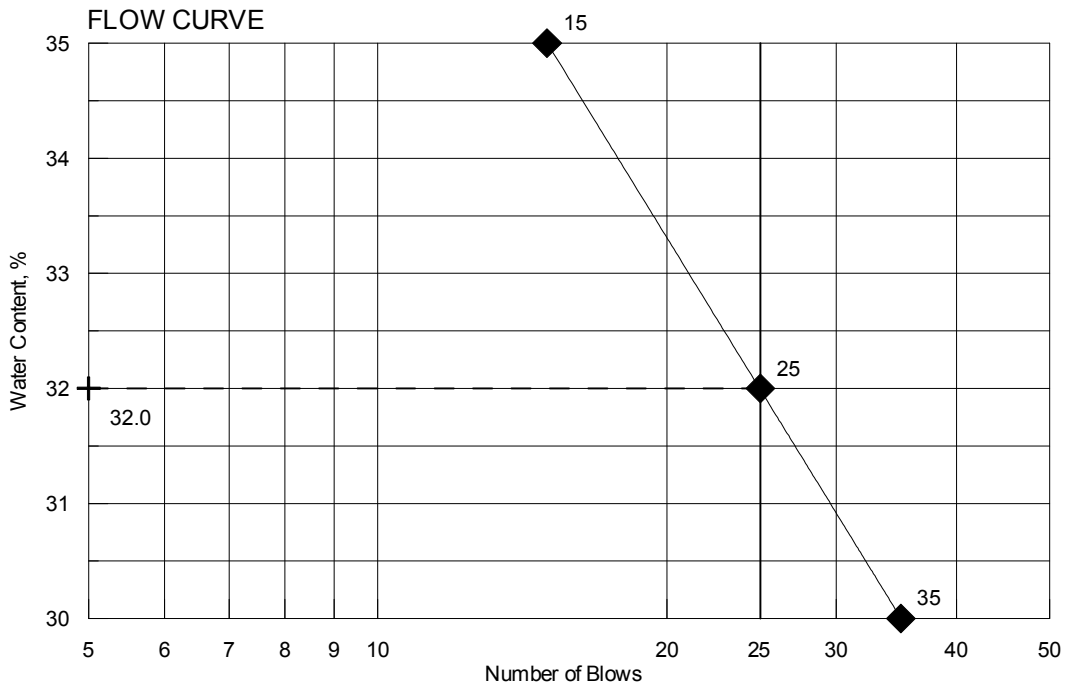
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210079
PIN	015611.00	Water Content, %	35.2
Sampled	4/2/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-103/7D	Liquid Limit	33
Station	3+58.7	Plasticity Index	11
Depth	40.5-42.5	Tested By	BBURR



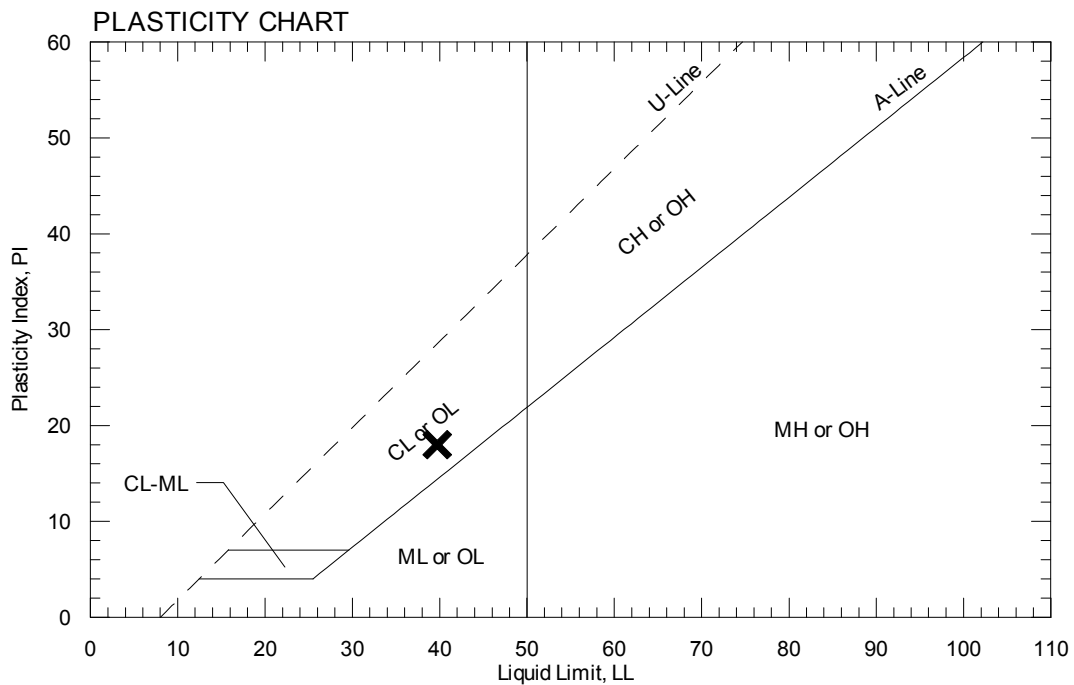
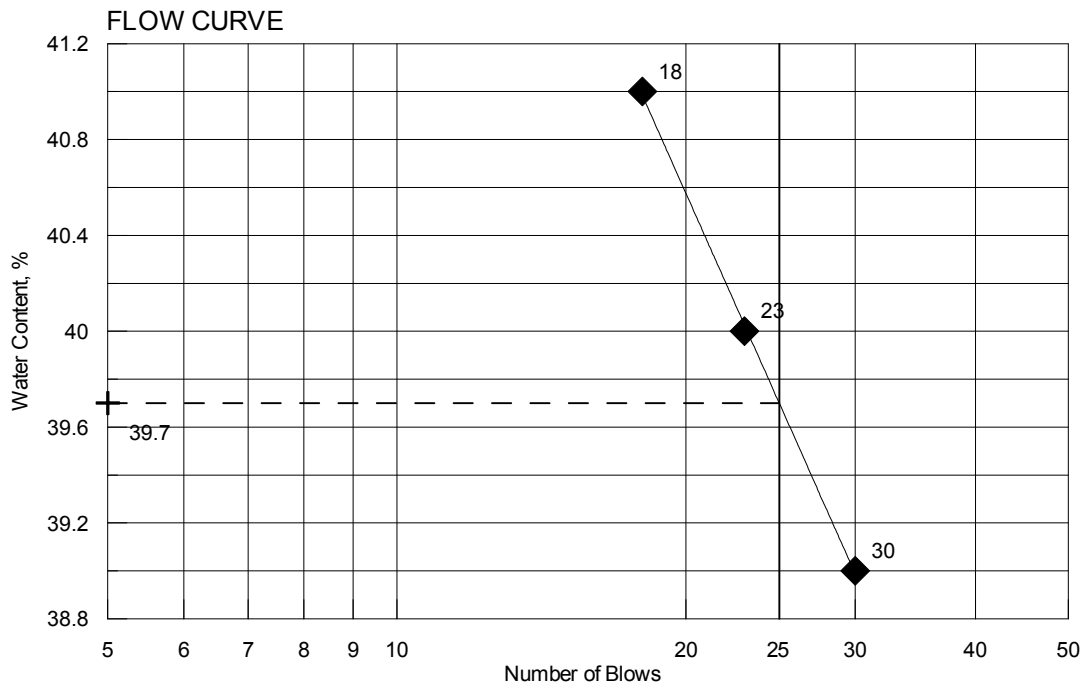
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210087
PIN	015611.00	Water Content, %	42.8
Sampled	3/27/2008	Plastic Limit	20
Boring No./Sample No.	BB-WSB-103 /2U	Liquid Limit	32
Station	3+58.7	Plasticity Index	12
Depth	45.0-47.0	Tested By	BBURR



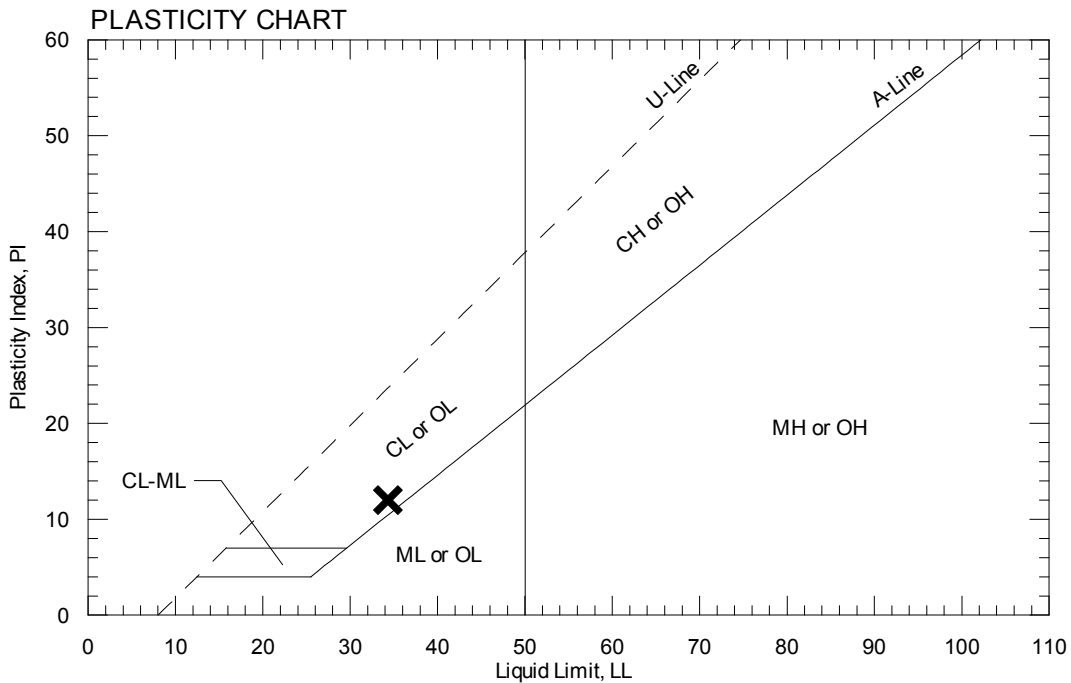
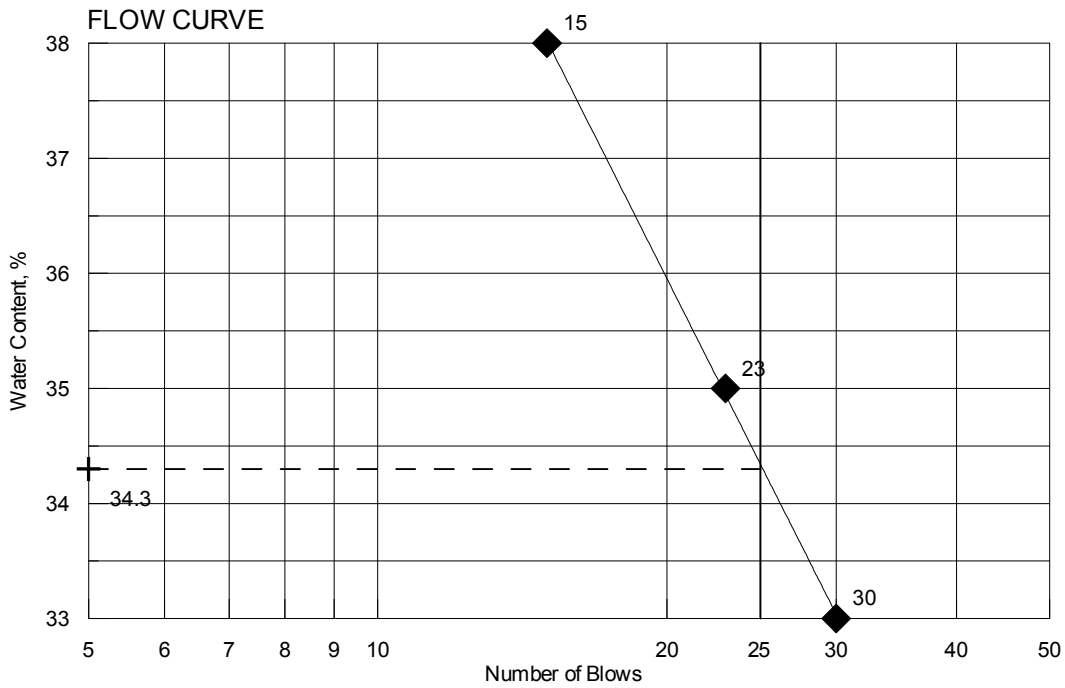
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210080
PIN	015611.00	Water Content, %	38.3
Sampled	4/1/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-103/8D	Liquid Limit	40
Station	3+58.7	Plasticity Index	18
Depth	51.0-53.0	Tested By	BFOGG



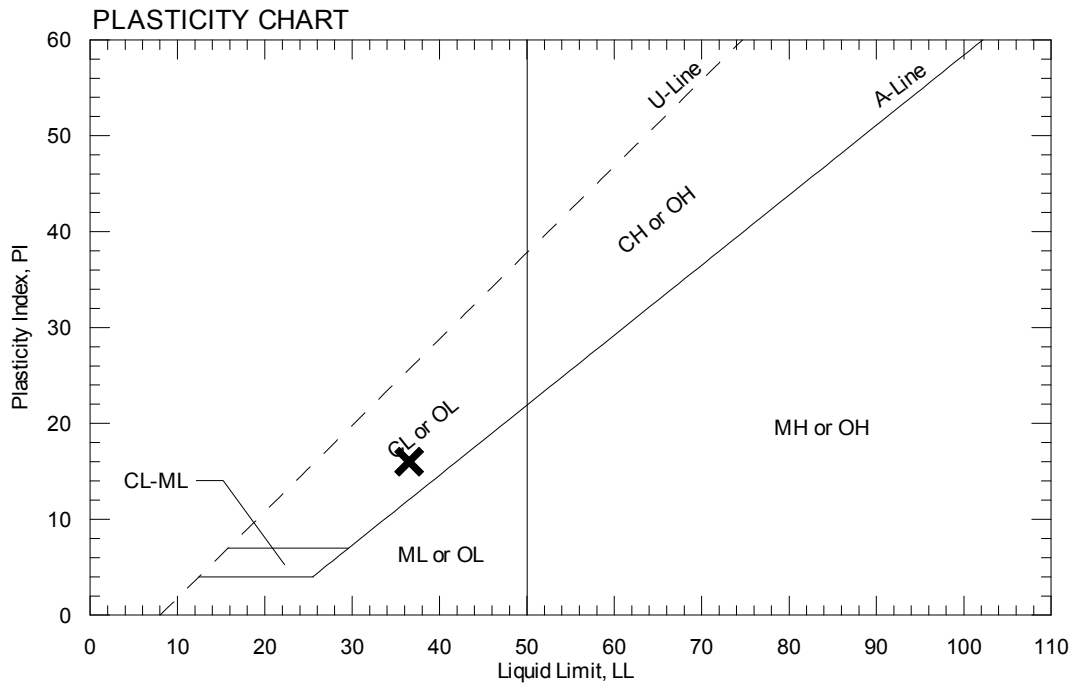
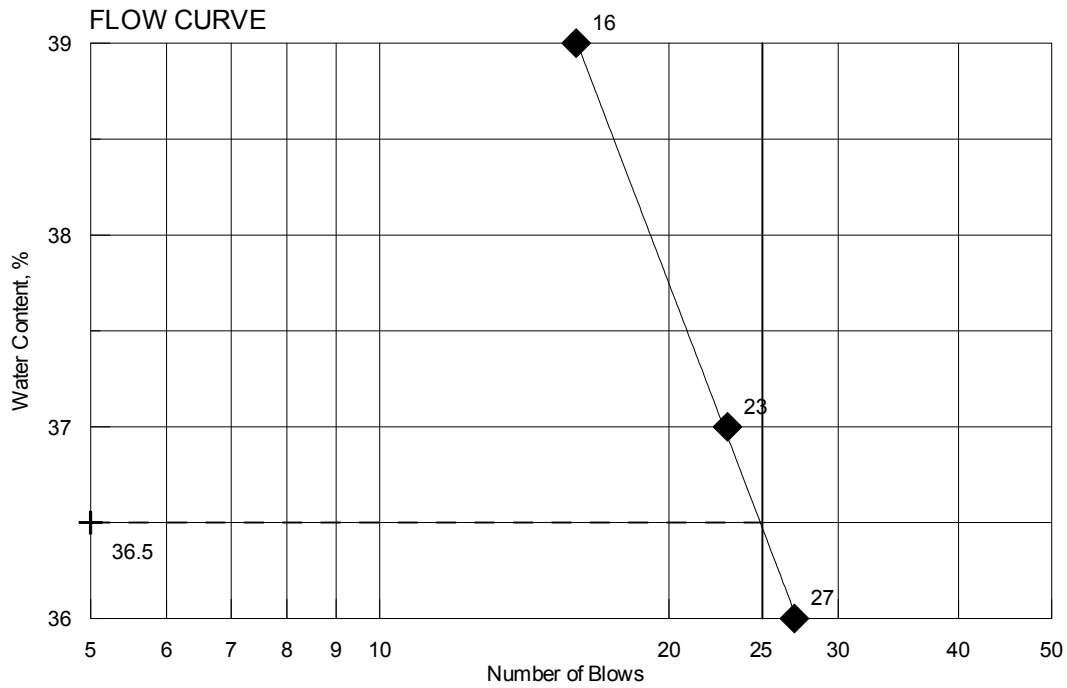
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210089
PIN	015611.00	Water Content, %	40.5
Sampled	4/1/2008	Plastic Limit	22
Boring No./Sample No.	BB-WSB-103/3U	Liquid Limit	34
Station	3+58.7	Plasticity Index	12
Depth	55.0-57.0	Tested By	BBURR



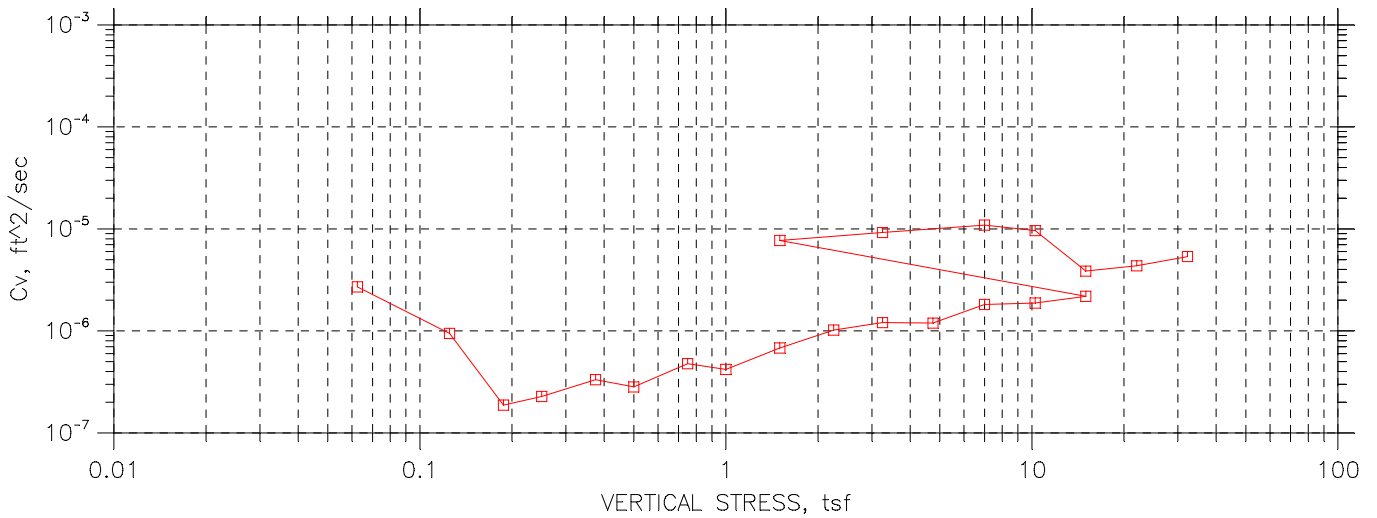
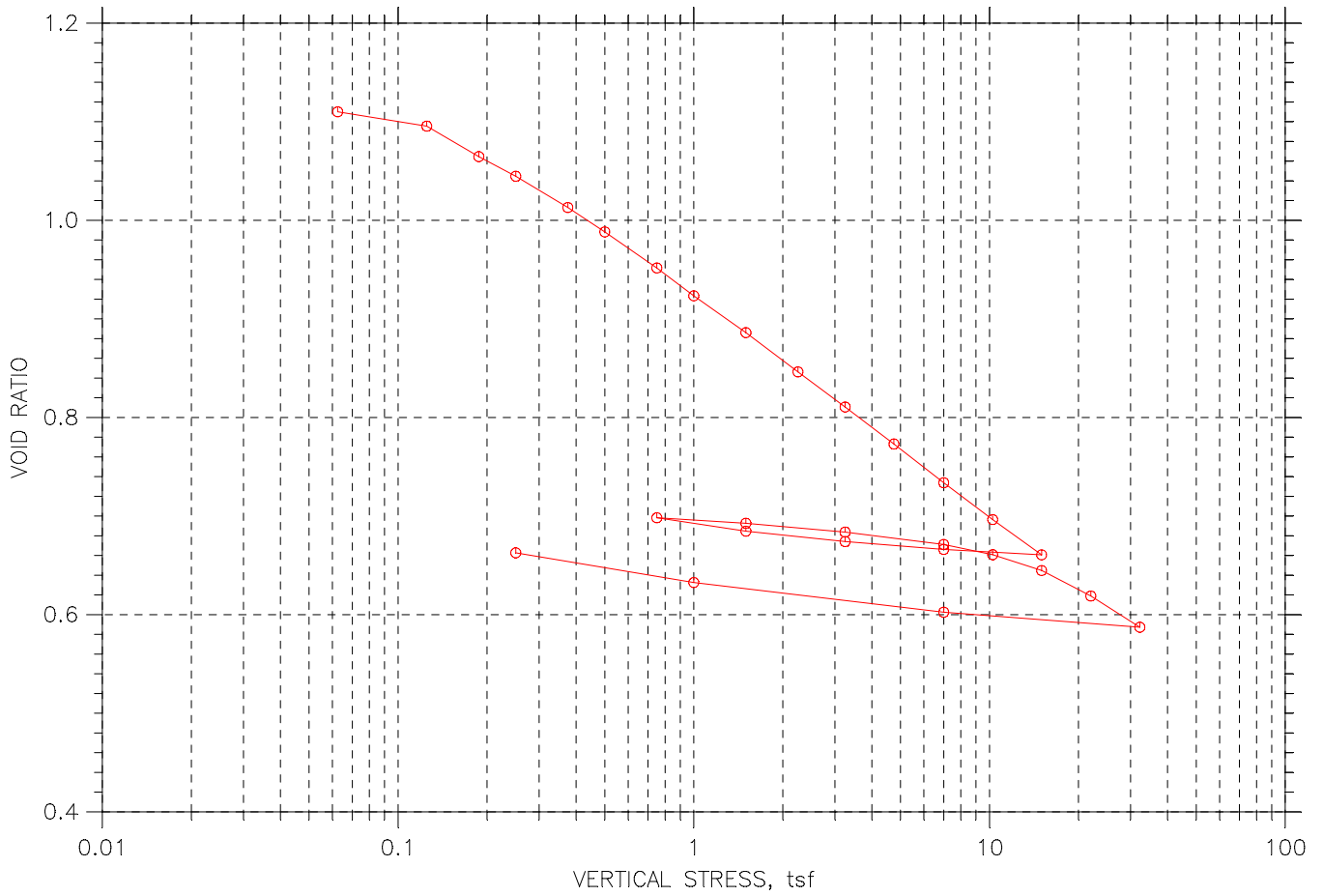
# State of Maine-Department of Transportation Atterberg Limits Test Summary Sheet

TOWN	Wells	Reference No.	210081
PIN	015611.00	Water Content, %	34
Sampled	4/1/2008	Plastic Limit	21
Boring No./Sample No.	BB-WSB-103/9D	Liquid Limit	37
Station	3+58.7	Plasticity Index	16
Depth	60.0-62.0	Tested By	BFOGG



# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-101	Tested By: Brian Fogg	Checked By:
Sample No.: 1U	Test Date: 5/1/08	Depth: 40-42 FT
Test No.: 210084	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-101  
 Sample No.: 1U  
 Test No.: 210084

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/1/08  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 40-42 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.73  
 Initial Void Ratio: 1.14  
 Final Void Ratio: 0.66

Liquid Limit: 34  
 Plastic Limit: 21  
 Plasticity Index: 13

Initial Height: 1.01 in  
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	216	RING	RING	200
Wt. Container + Wet Soil, gm	201.38	406.77	389.38	192.23
Wt. Container + Dry Soil, gm	161.95	364.49	364.49	167.39
Wt. Container, gm	62.3	262.05	262.05	65.16
Wt. Dry Soil, gm	99.65	102.44	102.44	102.23
Water Content, %	39.57	41.27	24.30	24.30
Void Ratio	---	1.14	0.66	---
Degree of Saturation, %	---	98.72	100.09	---
Dry Unit Weight, pcf	---	79.588	102.5	---

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-101  
 Sample No.: 1U  
 Test No.: 210084

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/1/08  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 40-42 FT  
 Elevation: ---

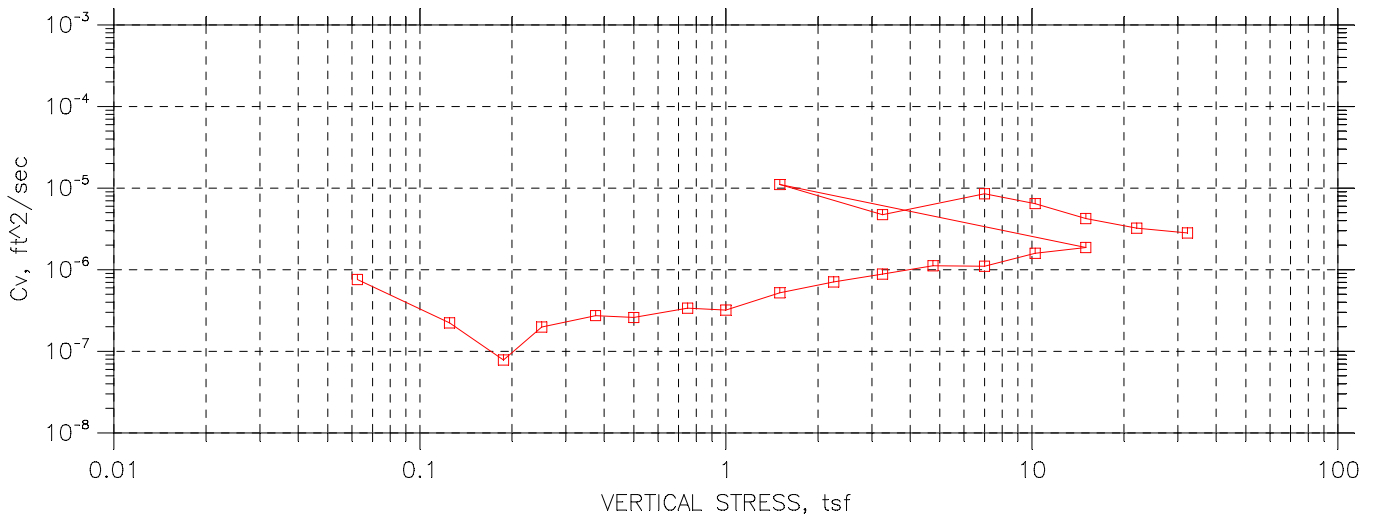
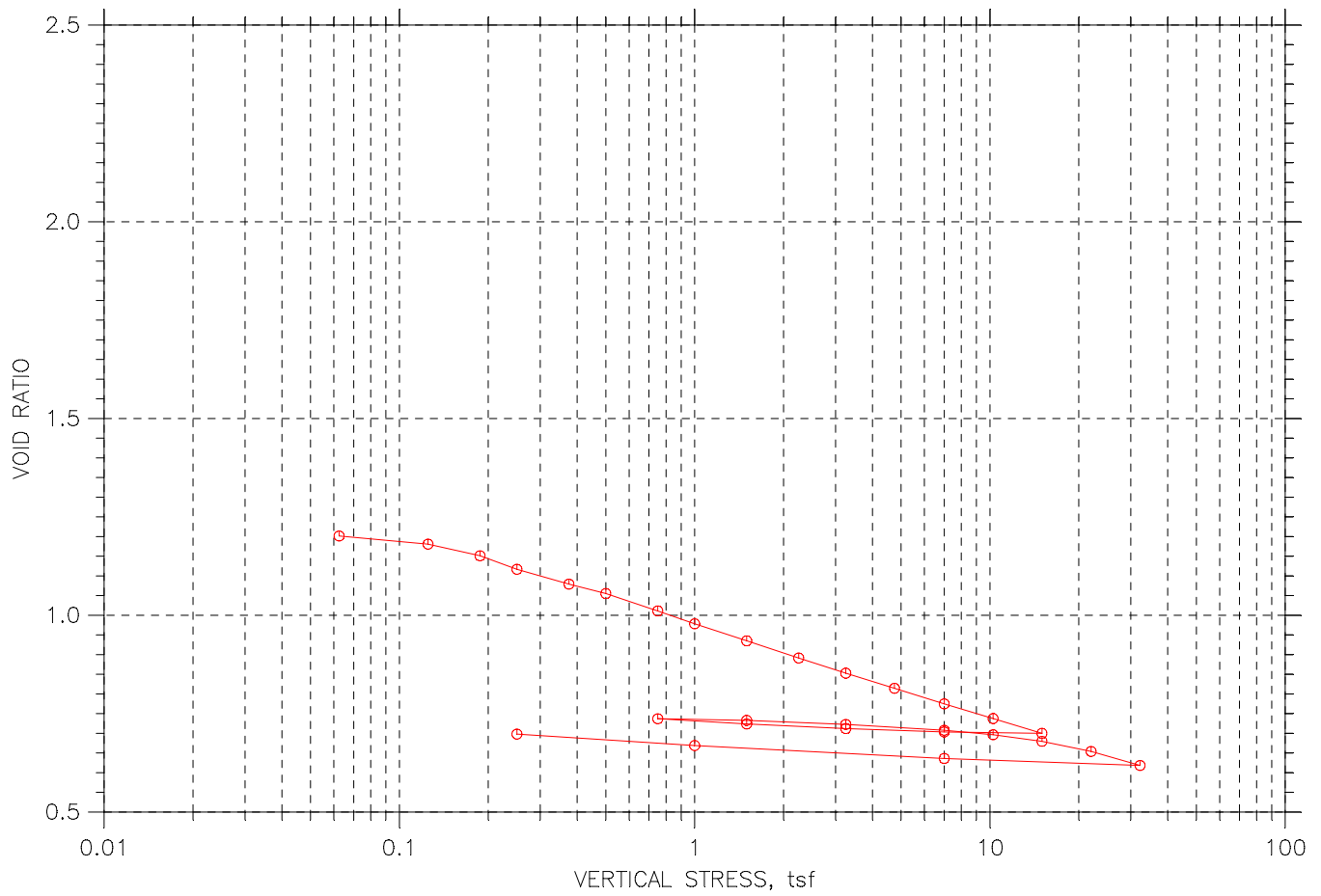
Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec	Ave. ft <sup>2</sup> /sec
1	0.0625	0.01476	1.110	1.46	1.8	2.5	3.19e-006	2.34e-006	2.70e-006
2	0.125	0.02167	1.095	2.14	7.2	4.8	7.84e-007	1.18e-006	9.43e-007
3	0.188	0.03626	1.065	3.59	29.4	0.0	1.87e-007	0.00e+000	1.87e-007
4	0.25	0.04568	1.045	4.52	23.6	0.0	2.27e-007	0.00e+000	2.27e-007
5	0.375	0.06062	1.013	6.00	15.7	0.0	3.33e-007	0.00e+000	3.33e-007
6	0.5	0.07229	0.988	7.15	18.0	0.0	2.82e-007	0.00e+000	2.82e-007
7	0.75	0.08956	0.952	8.86	11.1	9.6	4.46e-007	5.13e-007	4.77e-007
8	1	0.1029	0.923	10.18	11.4	0.0	4.19e-007	0.00e+000	4.19e-007
9	1.5	0.1205	0.886	11.92	6.8	6.8	6.77e-007	6.81e-007	6.79e-007
10	2.25	0.1393	0.846	13.78	4.6	4.1	9.71e-007	1.07e-006	1.02e-006
11	3.25	0.1562	0.811	15.45	3.4	3.7	1.25e-006	1.16e-006	1.21e-006
12	4.75	0.1739	0.773	17.20	3.5	3.4	1.18e-006	1.20e-006	1.19e-006
13	7	0.1924	0.734	19.03	2.1	2.2	1.85e-006	1.78e-006	1.82e-006
14	10.3	0.21	0.696	20.78	1.8	2.2	2.13e-006	1.68e-006	1.88e-006
15	15	0.227	0.661	22.45	1.5	1.7	2.32e-006	2.06e-006	2.18e-006
16	7	0.2243	0.666	22.19	0.0	0.0	1.06e-004	0.00e+000	1.06e-004
17	3.25	0.2206	0.674	21.82	0.1	0.0	2.96e-005	0.00e+000	2.96e-005
18	1.5	0.2156	0.685	21.33	0.7	0.0	4.90e-006	0.00e+000	4.90e-006
19	0.75	0.2092	0.698	20.69	2.1	2.1	1.75e-006	1.73e-006	1.74e-006
20	1.5	0.2118	0.693	20.95	0.5	0.0	7.70e-006	0.00e+000	7.70e-006
21	3.25	0.2161	0.684	21.37	0.5	0.3	7.05e-006	1.33e-005	9.21e-006
22	7	0.222	0.671	21.96	0.5	0.2	7.55e-006	1.95e-005	1.09e-005
23	10.3	0.2269	0.661	22.44	0.5	0.2	6.88e-006	1.60e-005	9.62e-006
24	15	0.2345	0.645	23.19	1.0	0.8	3.56e-006	4.22e-006	3.86e-006
25	22	0.2467	0.619	24.40	0.9	0.6	3.68e-006	5.31e-006	4.35e-006
26	32.3	0.2616	0.587	25.88	0.9	0.3	3.63e-006	1.02e-005	5.36e-006
27	7	0.2544	0.603	25.16	0.0	0.0	1.76e-004	0.00e+000	1.76e-004
28	1	0.2402	0.633	23.76	1.2	0.0	2.72e-006	0.00e+000	2.72e-006
29	0.25	0.226	0.663	22.35	7.2	6.3	4.80e-007	5.53e-007	5.14e-007



# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-101	Tested By: Brian Fogg	Checked By:
Sample No.: 2U	Test Date: 4/9/2008	Depth: 50-52 FT
Test No.: 210085	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-101  
 Sample No.: 2U  
 Test No.: 210085

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 4/9/2008  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 50-52 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.76  
 Initial Void Ratio: 1.24  
 Final Void Ratio: 0.70

Liquid Limit: 38  
 Plastic Limit: 22  
 Plasticity Index: 16

Initial Height: 1.01 in  
 Specimen Diameter: 2.48 in

Container ID	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
	20	RING	RING	30
Wt. Container + Wet Soil, gm	206.26	404.44	385.74	166.49
Wt. Container + Dry Soil, gm	159.81	360.76	360.76	141.55
Wt. Container, gm	52.93	262.09	262.09	43.03
Wt. Dry Soil, gm	106.88	98.672	98.672	98.52
Water Content, %	43.46	44.27	25.31	25.31
Void Ratio	---	1.24	0.70	---
Degree of Saturation, %	---	98.72	100.10	---
Dry Unit Weight, pcf	---	77.004	101.47	---

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-101  
 Sample No.: 2U  
 Test No.: 210085

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 4/9/2008  
 Sample Type: Shelby Tube

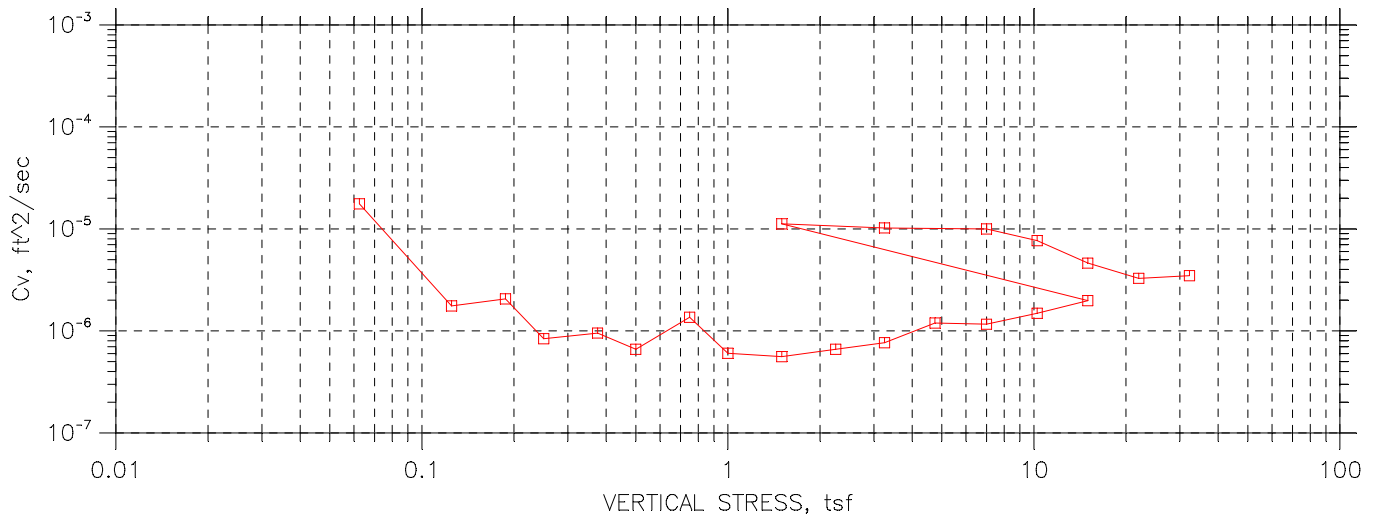
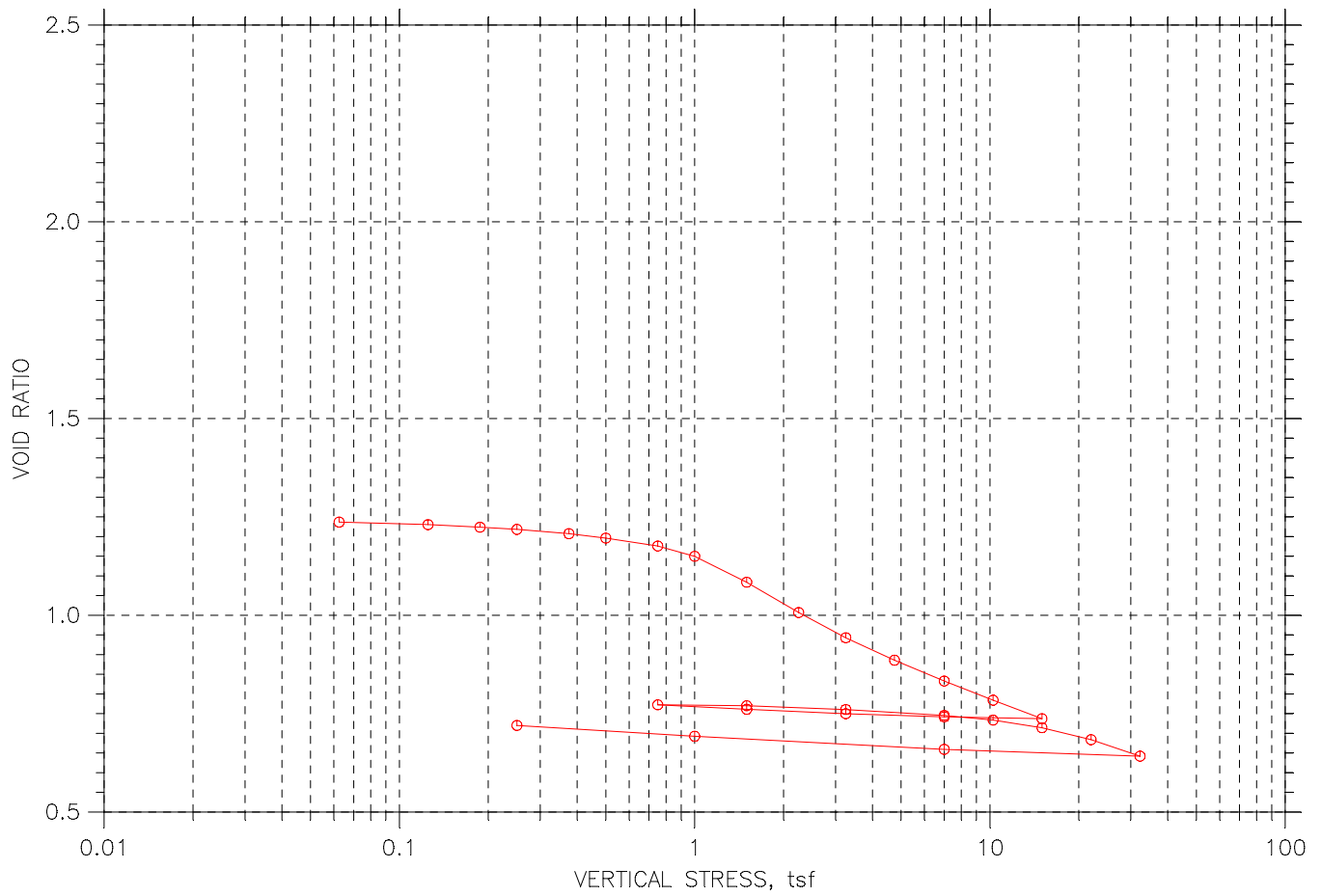
Project No.: 15611.00  
 Checked By:  
 Depth: 50-52 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec	Ave. ft <sup>2</sup> /sec
1	0.0625	0.01623	1.201	1.61	7.5	0.0	7.57e-007	0.00e+000	7.57e-007
2	0.125	0.02563	1.181	2.55	24.8	0.0	2.23e-007	0.00e+000	2.23e-007
3	0.188	0.03889	1.151	3.86	69.3	0.0	7.82e-008	0.00e+000	7.82e-008
4	0.25	0.0543	1.117	5.39	26.5	0.0	1.98e-007	0.00e+000	1.98e-007
5	0.375	0.07122	1.079	7.08	17.9	19.3	2.84e-007	2.64e-007	2.74e-007
6	0.5	0.08202	1.055	8.15	19.0	0.0	2.60e-007	0.00e+000	2.60e-007
7	0.75	0.1019	1.011	10.12	14.1	0.0	3.38e-007	0.00e+000	3.38e-007
8	1	0.1167	0.978	11.59	14.4	0.0	3.20e-007	0.00e+000	3.20e-007
9	1.5	0.1361	0.935	13.52	9.3	7.7	4.78e-007	5.73e-007	5.21e-007
10	2.25	0.1557	0.891	15.47	7.1	4.8	5.95e-007	8.75e-007	7.08e-007
11	3.25	0.1731	0.853	17.20	4.7	4.5	8.63e-007	9.06e-007	8.84e-007
12	4.75	0.1903	0.814	18.91	3.4	3.5	1.14e-006	1.10e-006	1.12e-006
13	7	0.2081	0.775	20.68	3.7	3.0	1.00e-006	1.22e-006	1.10e-006
14	10.3	0.2249	0.737	22.35	2.0	2.5	1.77e-006	1.44e-006	1.59e-006
15	15	0.2419	0.700	24.03	1.6	2.0	2.09e-006	1.70e-006	1.87e-006
16	7	0.2403	0.703	23.87	0.0	0.0	2.06e-004	0.00e+000	2.06e-004
17	3.25	0.2364	0.712	23.48	0.2	0.0	1.45e-005	0.00e+000	1.45e-005
18	1.5	0.2311	0.724	22.96	1.0	0.0	3.54e-006	0.00e+000	3.54e-006
19	0.75	0.2252	0.737	22.37	2.3	3.2	1.48e-006	1.08e-006	1.25e-006
20	1.5	0.227	0.733	22.56	0.5	0.1	7.14e-006	2.46e-005	1.11e-005
21	3.25	0.2317	0.722	23.02	0.7	0.7	4.70e-006	4.76e-006	4.73e-006
22	7	0.2383	0.708	23.68	0.5	0.3	6.80e-006	1.14e-005	8.53e-006
23	10.3	0.2435	0.696	24.19	0.7	0.3	4.70e-006	1.03e-005	6.46e-006
24	15	0.2509	0.680	24.93	0.9	0.6	3.57e-006	5.22e-006	4.24e-006
25	22	0.2626	0.654	26.09	0.9	1.0	3.41e-006	3.06e-006	3.23e-006
26	32.3	0.2785	0.618	27.67	0.9	1.2	3.26e-006	2.49e-006	2.82e-006
27	7	0.2705	0.636	26.88	0.0	0.0	8.28e-005	0.00e+000	8.28e-005
28	1	0.2558	0.669	25.42	1.4	1.8	2.22e-006	1.73e-006	1.94e-006
29	0.25	0.2427	0.698	24.11	7.3	7.0	4.49e-007	4.68e-007	4.59e-007

# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-103	Tested By: Brian Fogg	Checked By:
Sample No.: 1U	Test Date: 5/2/08	Depth: 35-37 FT
Test No.: 210088	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 1U  
 Test No.: 210088

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/2/08  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 35-37 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.75  
 Initial Void Ratio: 1.26  
 Final Void Ratio: 0.72

Liquid Limit: 33  
 Plastic Limit: 22  
 Plasticity Index: 11

Initial Height: 1.01 in  
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	123	RING	RING	128
Wt. Container + Wet Soil, gm	202.81	403.97	384.98	183.9
Wt. Container + Dry Soil, gm	167.74	359.47	359.47	158.44
Wt. Container, gm	69.7	262.03	262.03	61.21
Wt. Dry Soil, gm	98.04	97.436	97.436	97.23
Water Content, %	35.77	45.68	26.19	26.19
Void Ratio	---	1.26	0.72	---
Degree of Saturation, %	---	99.60	100.00	---
Dry Unit Weight, pcf	---	75.927	99.808	---

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 1U  
 Test No.: 210088

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/2/08  
 Sample Type: Shelby Tube

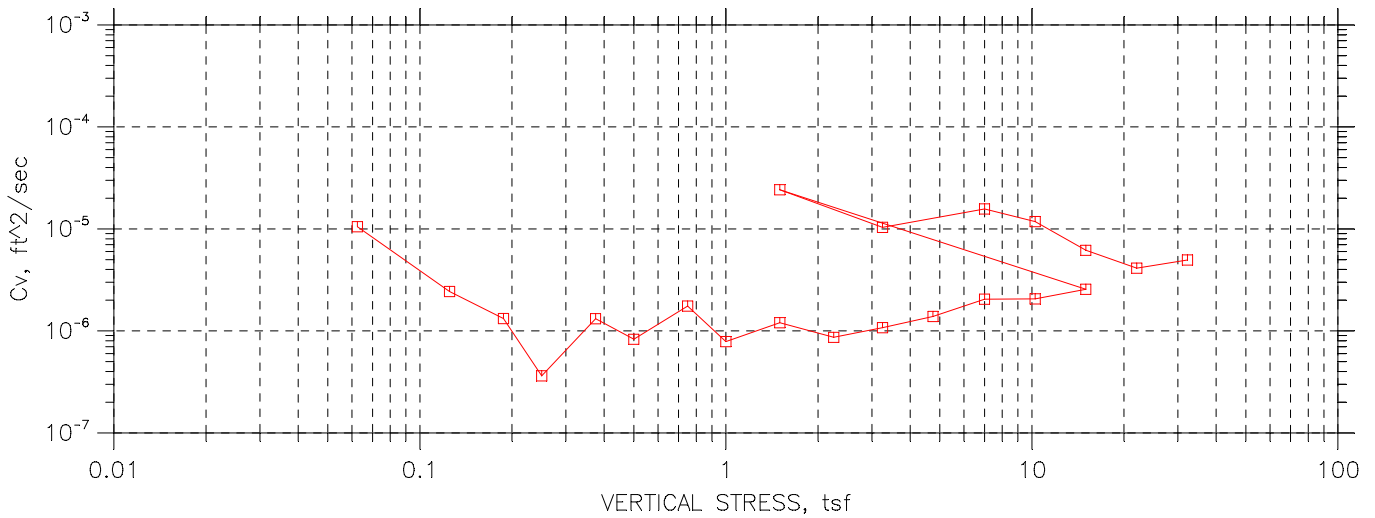
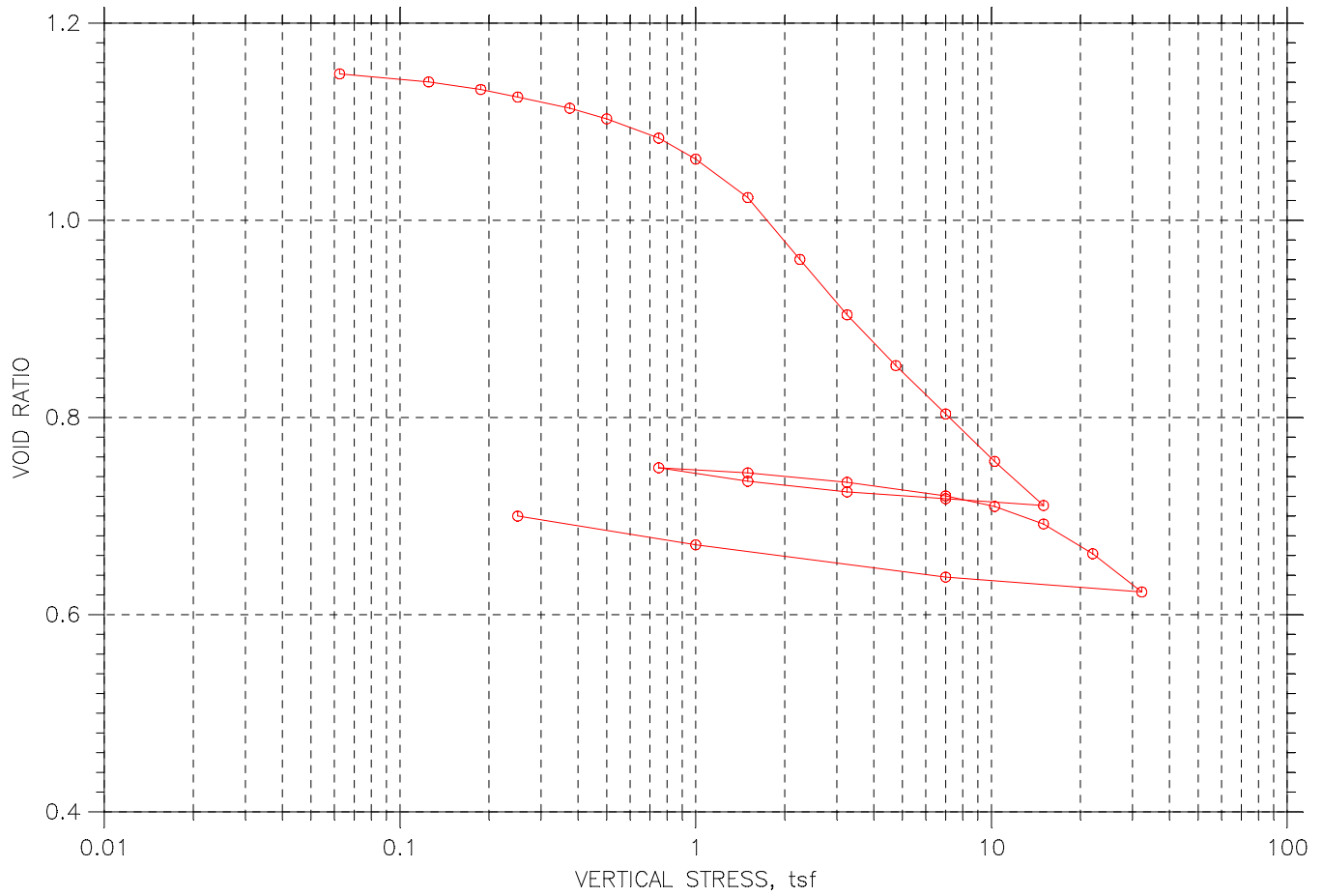
Project No.: 15611.00  
 Checked By:  
 Depth: 35-37 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec	Ave. ft <sup>2</sup> /sec
1	0.0625	0.01085	1.237	1.08	0.5	0.2	1.22e-005	3.19e-005	1.76e-005
2	0.125	0.01371	1.230	1.36	3.3	3.1	1.71e-006	1.81e-006	1.76e-006
3	0.188	0.01658	1.224	1.65	3.0	2.4	1.85e-006	2.32e-006	2.06e-006
4	0.25	0.01906	1.218	1.89	6.5	6.9	8.63e-007	8.17e-007	8.40e-007
5	0.375	0.02392	1.207	2.37	8.9	2.8	6.27e-007	1.99e-006	9.53e-007
6	0.5	0.02894	1.196	2.87	8.3	0.0	6.61e-007	0.00e+000	6.61e-007
7	0.75	0.03795	1.176	3.76	4.3	3.7	1.26e-006	1.47e-006	1.36e-006
8	1	0.04958	1.150	4.92	8.8	0.0	6.04e-007	0.00e+000	6.04e-007
9	1.5	0.07894	1.084	7.83	9.1	9.1	5.62e-007	5.59e-007	5.60e-007
10	2.25	0.1133	1.007	11.24	7.0	7.4	6.80e-007	6.44e-007	6.61e-007
11	3.25	0.1419	0.943	14.07	7.0	4.6	6.35e-007	9.67e-007	7.67e-007
12	4.75	0.1673	0.886	16.59	3.4	3.6	1.24e-006	1.15e-006	1.19e-006
13	7	0.1909	0.833	18.94	3.5	3.2	1.12e-006	1.21e-006	1.16e-006
14	10.3	0.2126	0.784	21.09	2.0	2.9	1.82e-006	1.26e-006	1.49e-006
15	15	0.2336	0.737	23.18	1.7	1.8	2.02e-006	1.94e-006	1.98e-006
16	7	0.2315	0.742	22.97	0.0	0.0	7.23e-005	0.00e+000	7.23e-005
17	3.25	0.2279	0.750	22.61	0.2	0.0	1.43e-005	0.00e+000	1.43e-005
18	1.5	0.2229	0.761	22.12	0.7	1.1	5.01e-006	3.16e-006	3.88e-006
19	0.75	0.2179	0.772	21.61	3.7	2.6	9.68e-007	1.36e-006	1.13e-006
20	1.5	0.2189	0.770	21.72	0.5	0.2	7.56e-006	2.18e-005	1.12e-005
21	3.25	0.2232	0.760	22.14	0.5	0.2	7.51e-006	1.60e-005	1.02e-005
22	7	0.2298	0.746	22.80	0.5	0.2	7.35e-006	1.56e-005	9.98e-006
23	10.3	0.2351	0.734	23.32	0.7	0.2	4.98e-006	1.68e-005	7.69e-006
24	15	0.2438	0.714	24.19	0.9	0.6	3.80e-006	5.89e-006	4.62e-006
25	22	0.2574	0.684	25.53	1.0	1.0	3.41e-006	3.15e-006	3.28e-006
26	32.3	0.2761	0.642	27.39	0.9	0.9	3.49e-006	3.48e-006	3.48e-006
27	7	0.2683	0.659	26.62	0.0	0.0	1.67e-004	1.69e-004	1.68e-004
28	1	0.2534	0.693	25.14	1.0	0.0	3.07e-006	0.00e+000	3.07e-006
29	0.25	0.2412	0.720	23.93	4.7	4.4	7.07e-007	7.52e-007	7.29e-007

# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-103	Tested By: Brian Fogg	Checked By:
Sample No.: 2U	Test Date: 5/1/08	Depth: 45-47 FT
Test No.: 210087	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 2U  
 Test No.: 210087

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/1/08  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 45-47 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.72  
 Initial Void Ratio: 1.17  
 Final Void Ratio: 0.70

Liquid Limit: 32  
 Plastic Limit: 20  
 Plasticity Index: 12

Initial Height: 1.02 in  
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	23	RING	RING	34
Wt. Container + Wet Soil, gm	211.64	407.01	390.21	180.16
Wt. Container + Dry Soil, gm	169.08	363.97	363.97	153.96
Wt. Container, gm	63.81	262	262	52.16
Wt. Dry Soil, gm	105.27	101.97	101.97	101.8
Water Content, %	40.43	42.21	25.74	25.74
Void Ratio	---	1.17	0.70	---
Degree of Saturation, %	---	98.23	100.01	---
Dry Unit Weight, pcf	---	78.292	99.886	---



CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 2U  
 Test No.: 210087

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/1/08  
 Sample Type: Shelby Tube

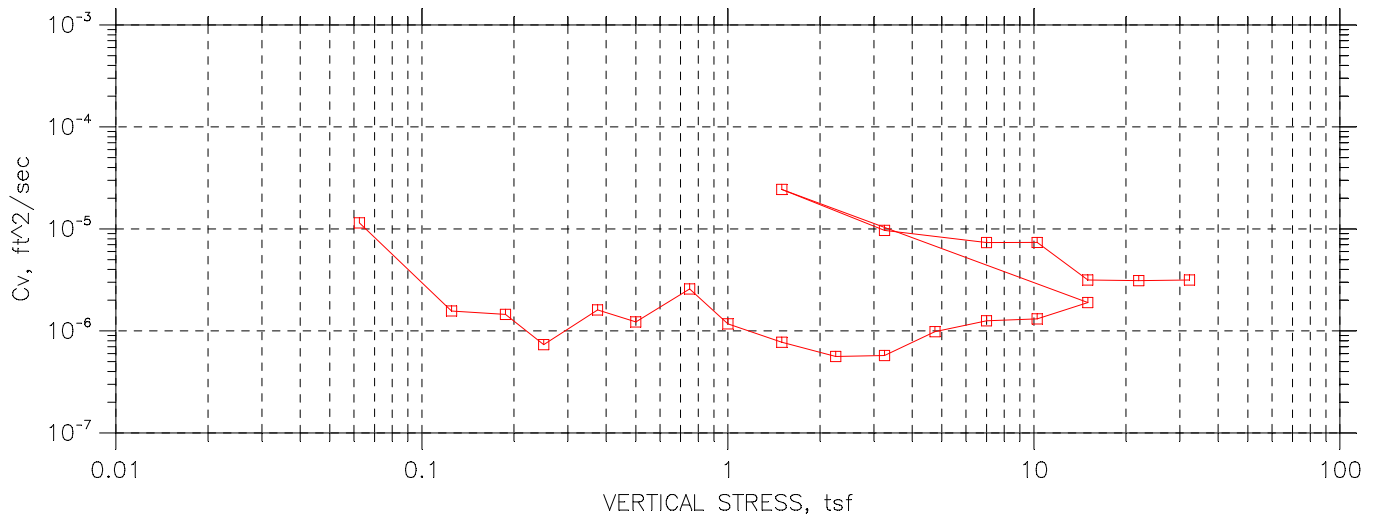
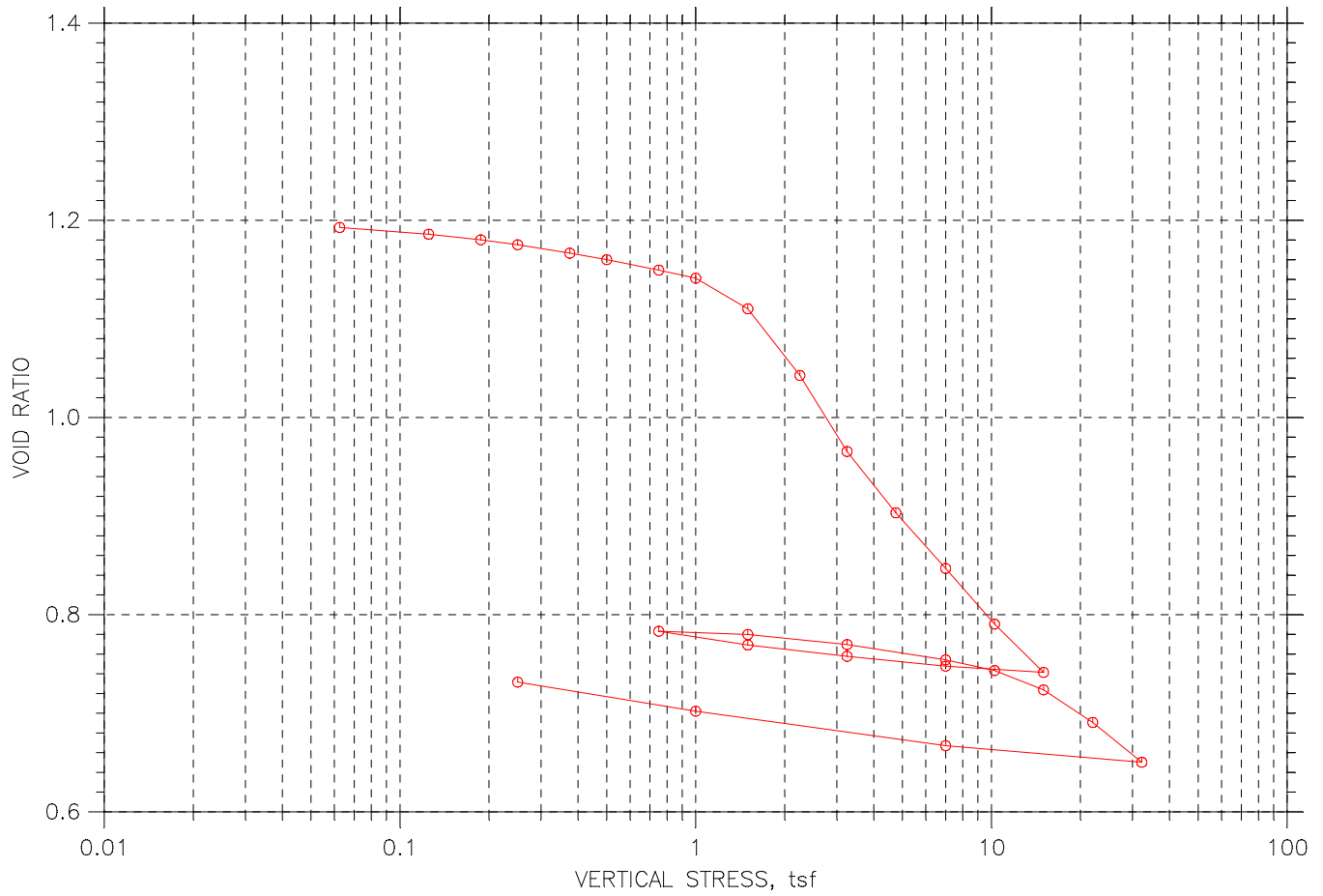
Project No.: 15611.00  
 Checked By:  
 Depth: 45-47 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	Coefficient of Consolidation		Ave. ft <sup>2</sup> /sec	
						Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec		
1	0.0625	0.009584	1.149	0.94	1.0	0.2	6.17e-006	3.56e-005	1.05e-005
2	0.125	0.01346	1.140	1.32	2.4	0.0	2.44e-006	0.00e+000	2.44e-006
3	0.188	0.01704	1.133	1.67	3.5	5.3	1.66e-006	1.10e-006	1.32e-006
4	0.25	0.02068	1.125	2.02	15.9	0.0	3.63e-007	0.00e+000	3.63e-007
5	0.375	0.026	1.114	2.54	4.6	4.1	1.25e-006	1.39e-006	1.32e-006
6	0.5	0.03112	1.103	3.04	6.8	0.0	8.28e-007	0.00e+000	8.28e-007
7	0.75	0.04033	1.083	3.94	3.3	3.1	1.69e-006	1.82e-006	1.75e-006
8	1	0.05034	1.062	4.92	7.0	0.0	7.85e-007	0.00e+000	7.85e-007
9	1.5	0.06875	1.023	6.72	4.5	4.3	1.18e-006	1.23e-006	1.20e-006
10	2.25	0.09835	0.960	9.61	6.8	4.8	7.40e-007	1.04e-006	8.65e-007
11	3.25	0.1248	0.904	12.20	4.6	4.2	1.03e-006	1.13e-006	1.08e-006
12	4.75	0.1492	0.853	14.58	3.4	3.1	1.31e-006	1.47e-006	1.38e-006
13	7	0.1723	0.804	16.84	2.0	2.1	2.10e-006	2.00e-006	2.05e-006
14	10.3	0.195	0.755	19.06	1.7	2.2	2.30e-006	1.86e-006	2.06e-006
15	15	0.2161	0.711	21.13	1.6	1.4	2.45e-006	2.69e-006	2.56e-006
16	7	0.2128	0.718	20.80	0.0	0.0	3.11e-004	0.00e+000	3.11e-004
17	3.25	0.2096	0.725	20.49	0.2	0.0	2.14e-005	0.00e+000	2.14e-005
18	1.5	0.2045	0.735	19.99	0.5	0.0	7.41e-006	0.00e+000	7.41e-006
19	0.75	0.1981	0.749	19.36	1.5	1.6	2.57e-006	2.34e-006	2.45e-006
20	1.5	0.2005	0.744	19.60	0.2	0.1	2.01e-005	3.03e-005	2.42e-005
21	3.25	0.205	0.734	20.04	0.5	0.3	8.19e-006	1.41e-005	1.04e-005
22	7	0.2115	0.720	20.68	0.2	0.2	1.61e-005	1.53e-005	1.57e-005
23	10.3	0.2165	0.710	21.17	0.5	0.1	7.45e-006	2.84e-005	1.18e-005
24	15	0.2249	0.692	21.99	0.9	0.3	4.10e-006	1.25e-005	6.17e-006
25	22	0.2392	0.662	23.38	1.1	0.6	3.20e-006	5.84e-006	4.13e-006
26	32.3	0.2575	0.623	25.17	1.0	0.3	3.28e-006	1.01e-005	4.95e-006
27	7	0.2503	0.638	24.47	0.0	0.0	2.64e-004	0.00e+000	2.64e-004
28	1	0.2349	0.671	22.96	0.7	0.0	4.82e-006	0.00e+000	4.82e-006
29	0.25	0.2212	0.700	21.62	4.6	4.3	7.82e-007	8.30e-007	8.05e-007

# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-103	Tested By: Brian Fogg	Checked By:
Sample No.: 3U	Test Date: 5/6/2008	Depth: 53-57 FT
Test No.: 210089	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 3U  
 Test No.: 210089

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/6/2008  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 53-57 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.81  
 Initial Void Ratio: 1.38  
 Final Void Ratio: 0.73

Liquid Limit: 34  
 Plastic Limit: 22  
 Plasticity Index: 12

Initial Height: 1.09 in  
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	35	RING	RING	67
Wt. Container + Wet Soil, gm	201.57	408.43	390.42	194.85
Wt. Container + Dry Soil, gm	161.04	363.89	363.89	168.4
Wt. Container, gm	65.03	262.07	262.07	66.91
Wt. Dry Soil, gm	96.01	101.82	101.82	101.49
Water Content, %	42.21	43.75	26.06	26.06
Void Ratio	---	1.38	0.73	---
Degree of Saturation, %	---	88.99	100.11	---
Dry Unit Weight, pcf	---	73.661	101.31	---

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 3U  
 Test No.: 210089

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/6/2008  
 Sample Type: Shelby Tube

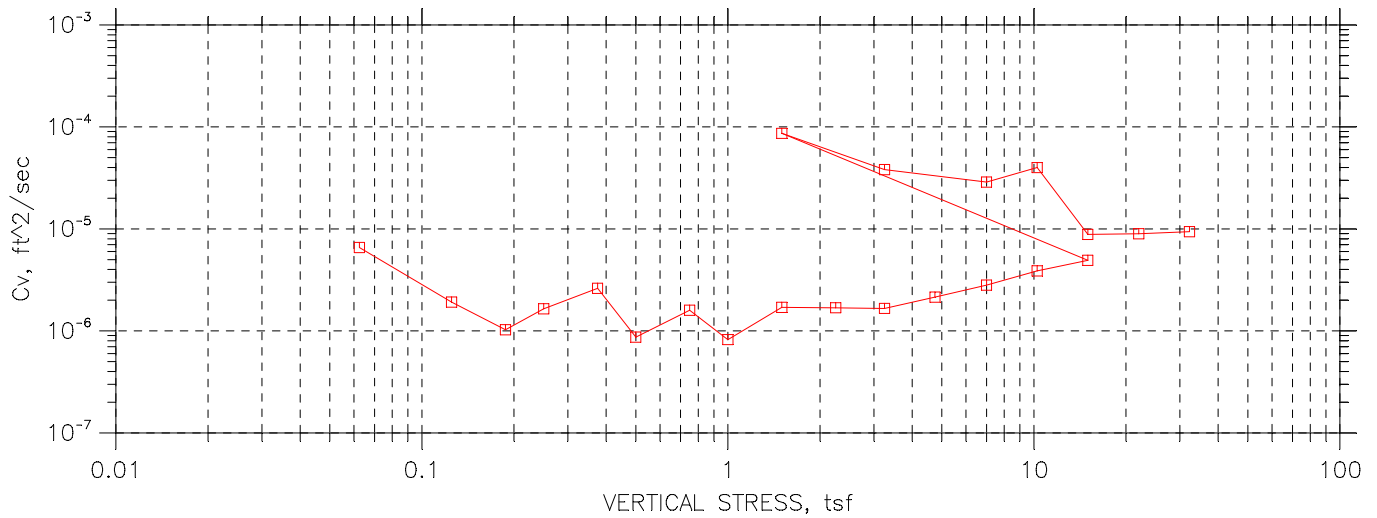
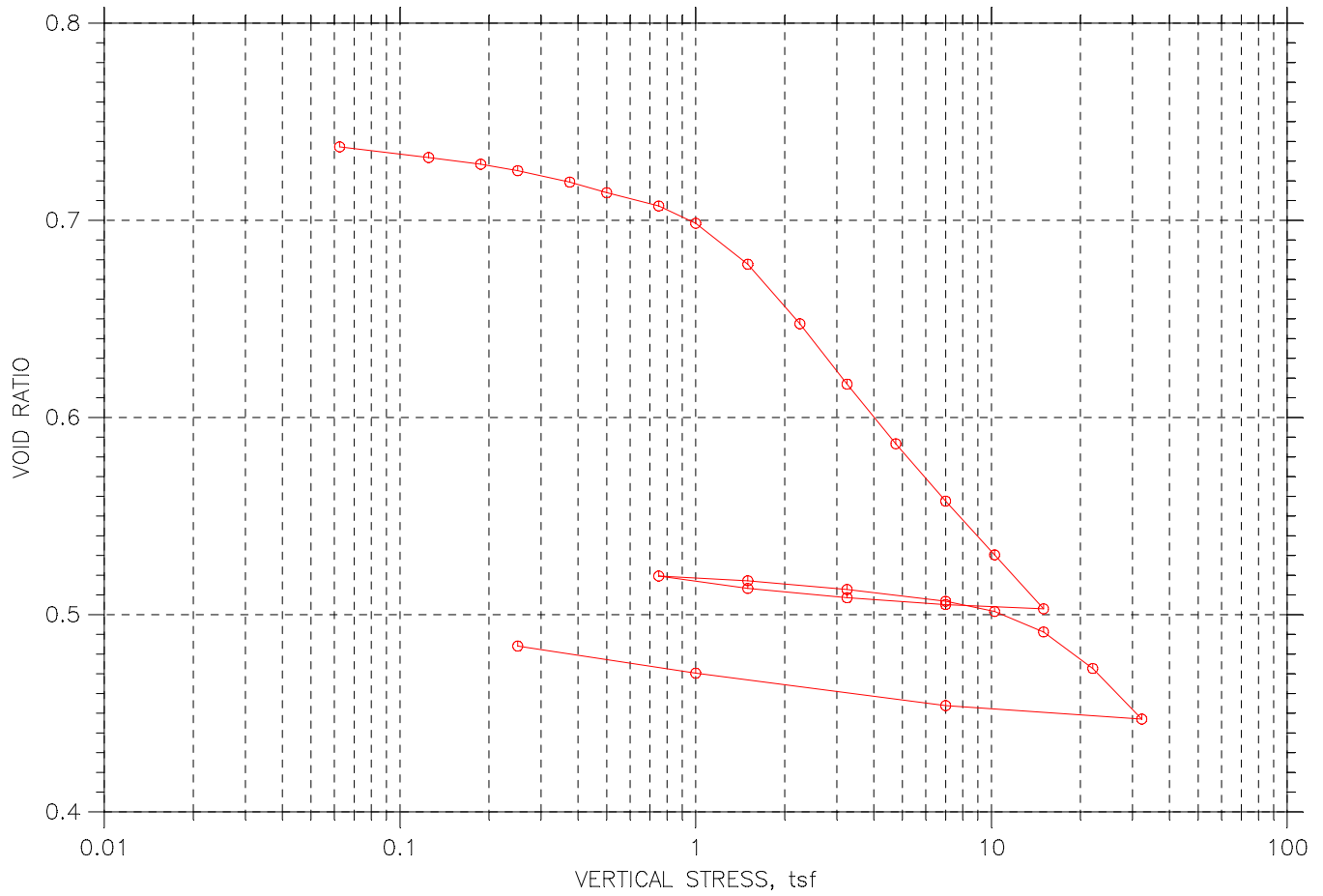
Project No.: 15611.00  
 Checked By:  
 Depth: 53-57 FT  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec	Ave. ft <sup>2</sup> /sec
1	0.0625	0.08604	1.193	7.92	0.6	0.5	1.03e-005	1.28e-005	1.15e-005
2	0.125	0.08916	1.186	8.21	3.6	3.7	1.58e-006	1.56e-006	1.57e-006
3	0.188	0.09177	1.180	8.45	4.7	3.0	1.19e-006	1.86e-006	1.45e-006
4	0.25	0.09405	1.175	8.66	7.7	0.0	7.34e-007	0.00e+000	7.34e-007
5	0.375	0.09794	1.167	9.02	3.5	3.5	1.61e-006	1.61e-006	1.61e-006
6	0.5	0.1009	1.160	9.30	4.5	0.0	1.23e-006	0.00e+000	1.23e-006
7	0.75	0.1058	1.149	9.74	2.1	0.0	2.58e-006	0.00e+000	2.58e-006
8	1	0.1095	1.141	10.09	4.7	0.0	1.17e-006	0.00e+000	1.17e-006
9	1.5	0.1237	1.110	11.39	6.9	0.0	7.74e-007	0.00e+000	7.74e-007
10	2.25	0.1545	1.043	14.23	9.1	0.0	5.61e-007	0.00e+000	5.61e-007
11	3.25	0.1896	0.966	17.47	9.0	7.6	5.27e-007	6.29e-007	5.74e-007
12	4.75	0.218	0.903	20.08	4.6	4.4	9.64e-007	1.00e-006	9.83e-007
13	7	0.2437	0.847	22.45	3.4	3.2	1.21e-006	1.30e-006	1.26e-006
14	10.3	0.2695	0.790	24.82	3.4	2.6	1.15e-006	1.52e-006	1.31e-006
15	15	0.2918	0.741	26.88	2.0	1.9	1.88e-006	1.91e-006	1.90e-006
16	7	0.2889	0.748	26.61	0.0	0.0	2.83e-004	0.00e+000	2.83e-004
17	3.25	0.2844	0.758	26.19	0.2	0.0	1.56e-005	0.00e+000	1.56e-005
18	1.5	0.2791	0.769	25.71	1.4	0.0	2.72e-006	0.00e+000	2.72e-006
19	0.75	0.2728	0.783	25.13	2.4	2.7	1.56e-006	1.38e-006	1.47e-006
20	1.5	0.2743	0.780	25.26	0.2	0.1	1.96e-005	3.24e-005	2.44e-005
21	3.25	0.2789	0.770	25.69	0.5	0.3	7.39e-006	1.40e-005	9.67e-006
22	7	0.286	0.754	26.34	0.5	0.0	7.35e-006	0.00e+000	7.35e-006
23	10.3	0.2909	0.743	26.80	0.5	0.0	7.36e-006	0.00e+000	7.36e-006
24	15	0.2999	0.724	27.62	1.6	0.6	2.18e-006	5.68e-006	3.15e-006
25	22	0.3149	0.691	29.01	1.2	1.1	3.00e-006	3.24e-006	3.11e-006
26	32.3	0.3334	0.650	30.70	0.9	1.2	3.65e-006	2.78e-006	3.16e-006
27	7	0.3257	0.667	30.00	0.0	0.0	7.47e-005	0.00e+000	7.47e-005
28	1	0.3098	0.702	28.53	1.0	0.0	3.49e-006	0.00e+000	3.49e-006
29	0.25	0.2963	0.732	27.29	5.0	6.5	7.00e-007	5.37e-007	6.08e-007

# CONSOLIDATION TEST DATA

## SUMMARY REPORT



Project: Bourne Avenue Bridge	Location: Wells	Project No.: 15611.00
Boring No.: BB-WSB-103	Tested By: Brian Fogg	Checked By:
Sample No.: 4U	Test Date: 5/6/2008	Depth: 65.0-66.7 F
Test No.: 210090	Sample Type: Shelby Tube	Elevation: ---
Description: GREY CLAY		
Remarks:		

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 4U  
 Test No.: 210090

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/6/2008  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 65.0-66.7 F  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

Measured Specific Gravity: 2.71  
 Initial Void Ratio: 0.75  
 Final Void Ratio: 0.48

Liquid Limit: 0  
 Plastic Limit: 0  
 Plasticity Index: 0

Initial Height: 1.02 in  
 Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	150	RING	RING	162
Wt. Container + Wet Soil, gm	207	421.94	410.28	214.55
Wt. Container + Dry Soil, gm	179.31	387.83	387.83	192.12
Wt. Container, gm	70.43	262.15	262.15	66.55
Wt. Dry Soil, gm	108.88	125.68	125.68	125.57
Water Content, %	25.43	27.14	17.86	17.86
Void Ratio	---	0.75	0.48	---
Degree of Saturation, %	---	97.65	100.00	---
Dry Unit Weight, pcf	---	96.5	114	---

CONSOLIDATION TEST DATA

Project: Bourne Avenue Bridge  
 Boring No.: BB-WSB-103  
 Sample No.: 4U  
 Test No.: 210090

Location: Wells  
 Tested By: Brian Fogg  
 Test Date: 5/6/2008  
 Sample Type: Shelby Tube

Project No.: 15611.00  
 Checked By:  
 Depth: 65.0-66.7 F  
 Elevation: ---

Soil Description: GREY CLAY  
 Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft <sup>2</sup> /sec	Log ft <sup>2</sup> /sec	Ave. ft <sup>2</sup> /sec
1	0.0625	0.00928	0.737	0.91	1.7	0.1	3.57e-006	4.18e-005	6.58e-006
2	0.125	0.01245	0.732	1.22	3.4	2.7	1.71e-006	2.19e-006	1.92e-006
3	0.188	0.0144	0.728	1.41	8.6	2.8	6.77e-007	2.11e-006	1.03e-006
4	0.25	0.01632	0.725	1.60	3.5	0.0	1.65e-006	0.00e+000	1.65e-006
5	0.375	0.01971	0.719	1.93	2.4	2.0	2.40e-006	2.88e-006	2.62e-006
6	0.5	0.02286	0.714	2.23	6.6	0.0	8.69e-007	0.00e+000	8.69e-007
7	0.75	0.02679	0.707	2.62	4.3	2.8	1.32e-006	2.01e-006	1.59e-006
8	1	0.03189	0.699	3.12	6.9	0.0	8.22e-007	0.00e+000	8.22e-007
9	1.5	0.04401	0.678	4.30	3.3	3.2	1.68e-006	1.72e-006	1.70e-006
10	2.25	0.06164	0.648	6.03	3.4	2.9	1.57e-006	1.82e-006	1.69e-006
11	3.25	0.07951	0.617	7.77	3.5	2.7	1.46e-006	1.93e-006	1.66e-006
12	4.75	0.09715	0.587	9.50	2.2	2.4	2.24e-006	2.05e-006	2.14e-006
13	7	0.1141	0.558	11.16	1.6	1.8	3.05e-006	2.60e-006	2.81e-006
14	10.3	0.13	0.530	12.71	1.3	1.1	3.52e-006	4.32e-006	3.88e-006
15	15	0.146	0.503	14.27	1.1	0.7	4.15e-006	6.05e-006	4.92e-006
16	7	0.1447	0.505	14.15	0.0	0.0	6.67e-004	0.00e+000	6.67e-004
17	3.25	0.1427	0.509	13.95	0.1	0.0	5.58e-005	2.24e-004	8.94e-005
18	1.5	0.14	0.513	13.68	0.5	0.2	9.50e-006	2.77e-005	1.41e-005
19	0.75	0.1363	0.520	13.32	0.9	0.0	4.87e-006	0.00e+000	4.87e-006
20	1.5	0.1377	0.517	13.46	0.1	0.0	7.40e-005	1.04e-004	8.65e-005
21	3.25	0.1403	0.513	13.71	0.1	0.1	3.37e-005	4.41e-005	3.82e-005
22	7	0.1438	0.507	14.05	0.2	0.0	2.88e-005	0.00e+000	2.88e-005
23	10.3	0.1468	0.502	14.35	0.1	0.1	3.18e-005	5.35e-005	3.99e-005
24	15	0.1528	0.491	14.94	0.8	0.1	5.14e-006	3.14e-005	8.83e-006
25	22	0.1636	0.473	16.00	0.7	0.3	6.15e-006	1.66e-005	8.97e-006
26	32.3	0.1786	0.447	17.46	0.7	0.2	6.15e-006	2.00e-005	9.41e-006
27	7	0.1747	0.454	17.07	0.0	0.0	3.23e-004	0.00e+000	3.23e-004
28	1	0.1651	0.470	16.13	0.5	0.0	9.06e-006	0.00e+000	9.06e-006
29	0.25	0.157	0.484	15.35	3.4	3.8	1.23e-006	1.12e-006	1.17e-006

## **Appendix C**

Calculations



Definition of Units:

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

**LIQUIDITY INDEX (LI):**

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

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-WSB-101/7D A	29.9	30	20	10	0.99	normally consolidated
BB-WSB-101/8D	34.3	30	21	9	1.48	viscous liquid when remolded
BB-WSB-101/1U	41.8	34	21	13	1.60	viscous liquid when remolded
BB-WSB-101/9D	33.5	34	22	12	0.96	normally consolidated
BB-WSB-101/2U	41.7	38	22	16	1.23	viscous liquid when remolded
BB-WSB-102/5D	29.9	30	18	12	0.99	normally consolidated
BB-WSB-102/7D	33.1	34	22	12	0.93	normally consolidated
BB-WSB-102/8D	35.4	36	22	14	0.96	normally consolidated
BB-WSB-102/10D	36.9	34	22	12	1.24	viscous liquid when remolded
BB-WSB-103/1U	41.5	33	22	11	1.77	viscous liquid when remolded
BB-WSB-103/7D	35.2	33	22	11	1.20	viscous liquid when remolded
BB-WSB-103/2U	42.8	32	20	12	1.90	viscous liquid when remolded
BB-WSB-103/8D	38.3	40	22	18	0.91	normally consolidated
BB-WSB-103/3U	40.5	34	22	12	1.54	viscous liquid when remolded
BB-WSB-103/9D	34.0	37	21	16	0.81	Slightly Overconsolidated

## CONSOLIDATION TEST RESULTS

### BB-WSB-101 Sample 1U

Determine in-situ over burden stress:

Sample depth = 40.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 1.14$

Clay is overlain by:

9 ft of Fill at 125 pcf

21.5 ft of sand at 120 pcf and

9.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 9 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 21.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 9.5 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf} = 2863 \cdot \text{psf} \text{ OR } \sigma'_{vo} = 1.432 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 0.195 \cdot \text{tsf}$

Determine OCR: 
$$\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}} \quad \text{OCR} = 0.1362 \quad \text{under consolidated}$$

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 0.75 \cdot \text{tsf} \quad e_1 := 0.952 \quad p_2 := 4.75 \cdot \text{tsf} \quad e_2 := 0.773$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.2233$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{8.86}{100} \quad \varepsilon_2 := \frac{17.2}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.104 \quad \text{OR:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.1043$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 0.75 \cdot \text{tsf} \quad e_1 := 0.698 \quad p_2 := 3.25 \cdot \text{tsf} \quad e_2 := 0.684$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.022$$

[BB-WSB-101 Sample 2U](#)

Determine in-situ over burden stress:

Sample depth = 50.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 1.24$

Clay is overlain by:

9 ft of Fill at 125 pcf

21.5 ft of sand at 120 pcf and

19.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 9 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 21.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 19.5 \cdot \text{ft} \cdot (115 - \sigma'_{vo}) = 3389 \cdot \text{psf} \text{ OR } \sigma'_{vo} = 1.695 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 0.18 \cdot \text{tsf}$

Determine OCR: 
$$\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}} \quad \text{OCR} = 0.1062 \quad \text{under consolidated}$$

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 0.5 \cdot \text{tsf} \quad e_1 := 1.055 \quad p_2 := 4.75 \cdot \text{tsf} \quad e_2 := 0.814$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.2465$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{8.15}{100} \quad \varepsilon_2 := \frac{18.91}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.1101 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.11$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 0.75 \cdot \text{tsf} \quad e_1 := 0.737 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.708$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0299$$

[BB-WSB-103 Sample 1U](#)

Determine in-situ over burden stress:

Sample depth = 35.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 1.26$

Clay is overlain by:

7.5 ft of Fill at 125 pcf

25 ft of sand at 120 pcf and

2.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 7.5 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 1.5 \cdot \text{ft} \cdot 120 \cdot \text{pcf} + 23.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 2.5 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$
$$\sigma'_{vo} = 2603 \cdot \text{psf} \text{ or } \sigma'_{vo} = 1.301 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 0.99 \cdot \text{tsf}$

Determine OCR:  $\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}}$        $\text{OCR} = 0.7608$       **under consolidated**

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 1.084 \quad p_2 := 3.25 \cdot \text{tsf} \quad e_2 := 0.943$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.4199$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{7.83}{100} \quad \varepsilon_2 := \frac{14.07}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.1858 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.1858$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 0.770 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.746$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0359$$

BB-WSB-103 Sample 2U

Determine in-situ over burden stress:

Sample depth = 45.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 1.17$

Clay is overlain by:

7.5 ft of Fill at 125 pcf

25 ft of sand at 120 pcf and

12.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 7.5 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 15 \cdot \text{ft} \cdot 120 \cdot \text{pcf} + 23.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 12.5 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$

$$\sigma'_{vo} = 3129 \cdot \text{psf} \quad \text{or} \quad \sigma'_{vo} = 1.564 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 1.1 \cdot \text{tsf}$

Determine OCR:  $\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}} \quad \text{OCR} = 0.7032 \quad \text{under consolidated}$

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 1.023 \quad p_2 := 3.25 \cdot \text{tsf} \quad e_2 := 0.904$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.3544$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{6.72}{100} \quad \varepsilon_2 := \frac{12.2}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.1632 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.1633$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 0.744 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.720$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0359$$

[BB-WSB-103 Sample 3U](#)

Determine in-situ over burden stress:

Sample depth = 55.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 1.38$

Clay is overlain by:

7.5 ft of Fill at 125 pcf

25 ft of sand at 120 pcf and

22.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 7.5 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 1.5 \cdot \text{ft} \cdot 120 \cdot \text{pcf} + 23.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 22.5 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$

$$\sigma'_{vo} = 3655 \cdot \text{psf} \text{ or } \sigma'_{vo} = 1.827 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 1.7 \cdot \text{tsf}$

Determine OCR: 
$$\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}} \quad \text{OCR} = 0.9303 \quad \text{under consolidated}$$

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 2.25 \cdot \text{tsf} \quad e_1 := 1.043 \quad p_2 := 4.75 \cdot \text{tsf} \quad e_2 := 0.903$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.4314$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{14.23}{100} \quad \varepsilon_2 := \frac{20.08}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.1803 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.1813$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 0.780 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.754$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0389$$

BB-WSB-103 Sample 4U

Determine in-situ over burden stress:

Sample depth = 65.0 ft below ground surface

Groundwater table at 9.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio  $e_0 := 0.75$

Clay is overlain by:

7.5 ft of Fill at 125 pcf

25 ft of sand at 120 pcf and

32.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 7.5 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 1.5 \cdot \text{ft} \cdot 120 \cdot \text{pcf} + 23.5 \cdot \text{ft} \cdot (120 - 62.4) \cdot \text{pcf} + 32.5 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$

$$\sigma'_{vo} = 4181 \cdot \text{psf} \text{ OR } \sigma'_{vo} = 2.09 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction:  $\sigma'_p := 1.1 \cdot \text{tsf}$

Determine OCR: 
$$\text{OCR} := \frac{\sigma'_p}{\sigma'_{vo}} \quad \text{OCR} = 0.5262 \quad \text{under consolidated}$$

Determine  $C_c$ :

from consolidation curve and lab results:

$$p_1 := 2.25 \cdot \text{tsf} \quad e_1 := 0.648 \quad p_2 := 7.0 \cdot \text{tsf} \quad e_2 := 0.558$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.1826$$

Determine  $C'_c$ :

from consolidation curve and lab results:

$$\epsilon_1 := \frac{6.03}{100} \quad \epsilon_2 := \frac{11.16}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\epsilon_2 - \epsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.1041 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.1043$$

Determine  $C_r$ :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 0.517 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.507$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.0149$$

## Abutment Foundations: Integral driven H-piles

### Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design  
 Specifications 4th Edition 2007

PDR Estimate based on HP 14 x 73 pile size

Look at the following piles:

HP 12 x 53

HP 14 x 73

HP 14 x 89

HP 14 x 117

Note: All matrices set up in this order

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

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

Where  $\lambda$  = normalized column slenderness factor

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

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

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

HP 12 x 53  
 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       $P_f := \phi_c \cdot P_n$

$P_f = \begin{pmatrix} 465 \\ 642 \\ 783 \\ 1032 \end{pmatrix} \cdot \text{kip}$       HP 12 x 53  
 HP 14 x 73      Strength Limit State  
 HP 14 x 89  
 HP 14 x 117



## SERVICE/EXTREME LIMIT STATES:

### Service and Extreme Limit States Axial Resistance

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

Where  $\lambda$  = normalized column slenderness factor

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

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

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

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

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

$$\phi := 1.0$$

**Factored** Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1  $P_f := \phi \cdot P_n$

$$P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

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

Service/Extreme Limit States

## Geotechnical Resistance

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

### Bedrock Type:

- Abutment No. 1 = Diorite (intrusion) RQD ranges from 39 to 60%.  
Use RQD = 50% and  $\phi = 27$  to 34 deg (Tomlinson 4th Ed. pg 139)
- Abutment No. 2 = Sandstone (host rock) RQD ranges from 45 to 46%.  
Use RQD = 45% and  $\phi = 27$  to 34 deg (Tomlinson 4th Ed. pg 139)

### Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design  
Specifications 4th Edition 2007

PDR Estimate based on HP 14 x 73 pile size

Look at these piles:

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

Note: All matrices set up in this order

Steel area:

$$A_s = \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2$$

Pile depth:

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

Pile width:

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

Calculate pile box area:

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

use 1/3 of the box area:

$$A_{\text{box}33\%} := A_{\text{box}} \cdot 0.33$$

$$A_{\text{box}33\%} = \begin{pmatrix} 46.8237 \\ 65.5056 \\ 67.0665 \\ 69.8002 \end{pmatrix} \cdot \text{in}^2$$

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

Abutment No. 1 -  $q_u$  for diorite compressive strength

(similar to gabbro) ranges from 18000 to 45000 psi

Abutment No. 2 -  $q_u$  for sandstone compressive strength ranges from 9700 to 25000 psi

use  $\sigma_{cA1} := 20000 \cdot \text{psi}$  for Abutment No. 1

$\sigma_{cA2} := 17000 \cdot \text{psi}$  for Abutment No. 2

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

Spacing of discontinuities:  $c := 12 \cdot \text{in}$  Assumed based on rock core

Aperture of discontinuities:  $\delta := \frac{1}{32} \cdot \text{in}$  joints are tight

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

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

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

$$K_{sp} = \begin{pmatrix} 0.2994 \\ 0.2864 \\ 0.286 \\ 0.2852 \end{pmatrix} \quad K_{sp} \text{ includes a factor of safety of 3}$$

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

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

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

$$q_{aA1} := \sigma_{cA1} \cdot K_{sp} \cdot d_f$$

$$q_{aA1} = \begin{pmatrix} 862 \\ 825 \\ 824 \\ 821 \end{pmatrix} \cdot \text{ksf}$$

$$q_{aA2} := \sigma_{cA2} \cdot K_{sp} \cdot d_f$$

$$q_{aA2} = \begin{pmatrix} 733 \\ 701 \\ 700 \\ 698 \end{pmatrix} \cdot \text{ksf}$$

**Nominal Geotechnical Tip Resistance,  $R_p$ :**

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

$$R_{pA1} := \overrightarrow{(3q_{aA1} \cdot A_{\text{box}33\%})}$$

$$R_{pA1} = \begin{pmatrix} 841 \\ 1126 \\ 1151 \\ 1194 \end{pmatrix} \cdot \text{kip}$$

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

for Abutment No. 1

$$R_{pA2} := \overrightarrow{(3q_{aA2} \cdot A_{\text{box}33\%})}$$

$$R_{pA2} = \begin{pmatrix} 715 \\ 957 \\ 978 \\ 1015 \end{pmatrix} \cdot \text{kip}$$

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

for Abutment No. 2

## 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,  $\phi_{stat}$        $\phi_{stat} := 0.45$       LRFD Table 10.5.5.2.3-1

$$R_{fA1} := \phi_{stat} \cdot R_{pA1}$$

$$R_{fA1} = \begin{pmatrix} 379 \\ 507 \\ 518 \\ 537 \end{pmatrix} \cdot \text{kip}$$

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

Strength Limit State

$$R_{fA2} := \phi_{stat} \cdot R_{pA2}$$

$$R_{fA2} = \begin{pmatrix} 322 \\ 431 \\ 440 \\ 457 \end{pmatrix} \cdot \text{kip}$$

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

$$\phi := 1.0$$

$$R_{fseA1} := \phi \cdot R_{pA1}$$

$$R_{fseA1} = \begin{pmatrix} 841 \\ 1126 \\ 1151 \\ 1194 \end{pmatrix} \cdot \text{kip}$$

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

Service/Extreme  
 Limit States

$$R_{fseA2} := \phi \cdot R_{pA2}$$

$$R_{fseA2} = \begin{pmatrix} 715 \\ 957 \\ 978 \\ 1015 \end{pmatrix} \cdot \text{kip}$$

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

Service/Extreme  
 Limit States

## DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension

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

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

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

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

### 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,  $\phi_{dyn}$ :

$$\phi_{dyn} := 0.65$$

Table 10.5.5.2.3-3 requires no less than 3 to 4 piles dynamically tested for a site with low to medium site variability. There will probably only be 8 to 10 piles total on the project. Only 1 or 2 piles will be tested - one per abutment will be requested. Therefore, reduce the  $\phi$  by 20%

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

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

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

Pile Size = 12 x 53

State of Maine Dept. Of Transportation		16-Jun-2008				
Wells Bourne Avenue		GRLWEAP (TM) Version 2003				
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft	
200.0	28.61	0.56	2.3	7.31	18.37	
250.0	33.20	2.00	3.3	7.72	19.04	
300.0	37.68	3.26	4.5	8.30	20.46	
350.0	41.19	3.53	6.5	8.74	21.60	
400.0	44.10	4.03	9.5	9.13	22.60	
450.0	46.37	4.30	14.0	9.47	23.50	
500.0	48.15	4.42	21.4	9.75	24.17	
550.0	49.71	4.37	36.0	9.99	24.80	
600.0	51.07	4.44	77.4	10.20	25.37	
650.0	52.49	4.56	193.2	10.45	26.00	

Limited to driving stress of 45 ksi

DELMAG D 19-42

Interpolate:

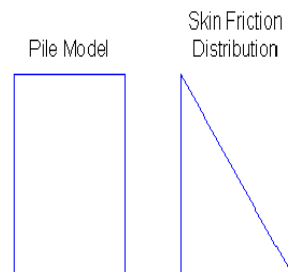
$$UC_{at\_45ksi} := \left[ \left( \frac{45 - 44.1}{46.37 - 44.1} \right) \cdot (450 \cdot \text{kip} - 400 \cdot \text{kip}) \right] + 400 \cdot \text{kip}$$

$$UC_{at\_45ksi} = 420 \cdot \text{kip}$$

$$R_{dr\_12x53\_factored} := 420 \cdot \text{kip} \cdot \phi_{dyn.reduced}$$

$$R_{dr\_12x53\_factored} = 218 \cdot \text{kip}$$

Efficiency	0.800
Helmet Hammer Cushion	1.90 kips 109972 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	70.00 ft
Pile Top Area	15.50 in <sup>2</sup>



Res. Shaft = 18 %  
 (Constant Res. Shaft)

Assume Contractor will use a ICE I-30 hammer to install 14 x 73 piles

Pile Size = 14 x 73

State of Maine Dept. Of Transportation Wells Bourne Avenue		16-Jul-2008 GRLWEAP (TM) Version 2003				
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft	
300.0	27.81	0.32	2.0	7.60	30.59	
400.0	33.94	2.15	3.0	8.29	32.47	
450.0	36.65	3.25	3.7	8.67	33.73	
500.0	39.15	3.91	4.7	9.08	34.93	
550.0	40.91	4.39	6.1	9.33	35.69	
600.0	42.70	4.80	8.2	9.65	36.73	
650.0	44.29	5.09	11.5	9.95	37.84	
700.0	45.71	5.39	17.2	10.21	38.81	
750.0	46.91	5.51	26.3	10.42	39.60	
800.0	47.91	5.63	44.6	10.59	40.20	

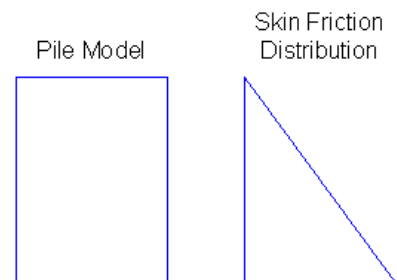
Limited to driving stress of 45 ksi

ICE I-30

$$R_{dr\_14x73\_factored} := 650 \cdot \text{kip} \cdot \phi_{dyn.reduced}$$

$$R_{dr\_14x73\_factored} = 338 \cdot \text{kip}$$

Efficiency	0.800
Helmet Hammer Cushion	3.20 kips 34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	70.00 ft
Pile Top Area	21.40 in <sup>2</sup>



Res. Shaft = 20 %  
 (Constant Res. Shaft)

**Pile Size = 14 x 89**

**Assume Contractor will use a ICE I-30 hammer to install 14 x 89 piles**

State of Maine Dept. Of Transportation		16-Jul-2008			
Wells Bourne Avenue		GRLWEAP (TM) Version 2003			
Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
300.0	26.34	0.19	2.0	7.66	29.57
400.0	30.91	1.52	3.0	8.24	30.82
450.0	33.43	2.47	3.6	8.56	31.81
500.0	35.92	2.94	4.5	8.91	32.88
550.0	37.88	3.23	5.7	9.15	33.78
600.0	39.67	3.49	7.2	9.43	34.77
650.0	41.27	3.72	9.4	9.68	35.77
700.0	42.66	3.84	12.1	9.91	36.57
750.0	43.77	3.88	15.7	10.09	37.21
800.0	44.88	4.10	20.8	10.27	37.94

ICE I-30

Limit blow count to 15 blows per inch

Interpolate:

$$UC_{at\_15blows} := \left[ \left( \frac{15 - 12.1}{15.7 - 12.1} \right) \cdot (700 \cdot \text{kip} - 750 \cdot \text{kip}) \right] + 750 \cdot \text{kip}$$

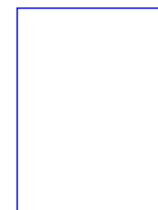
$$UC_{at\_15blows} = 710 \cdot \text{kip}$$

$$R_{dr\_14x89\_factored} := UC_{at\_15blows} \cdot \phi_{dyn.reduced}$$

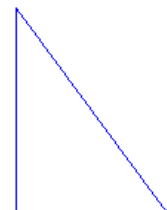
$$R_{dr\_14x89\_factored} = 369 \cdot \text{kip}$$

Efficiency	0.800
Helmet Hammer Cushion	3.20 kips 34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	70.00 ft
Pile Top Area	26.10 in <sup>2</sup>

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %  
 (Constant Res. Shaft)



## Pile Size = 14 x 117

Assume Contractor will use a ICE I-30 hammer to install 14 x 117 piles

State of Maine Dept. Of Transportation  
 Wells Bourne Avenue

16-Jul-2008  
 GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
400.0	26.21	0.88	3.2	8.25	28.91
500.0	29.37	1.82	4.5	8.75	30.53
550.0	31.36	1.96	5.4	9.02	31.35
600.0	32.83	2.29	6.5	9.14	31.89
650.0	34.33	2.72	7.9	9.33	32.50
700.0	35.73	2.93	9.4	9.50	33.21
750.0	36.90	3.01	11.3	9.66	33.79
800.0	37.97	2.96	13.9	9.81	34.30
850.0	38.90	2.87	17.1	9.95	34.77
900.0	39.74	2.82	21.6	10.08	35.25

Limit blow count to 15 blows per inch

ICE I-30

Interpolate:

$$UC_{at15blows} := \left[ \left( \frac{15 - 13.9}{17.1 - 13.9} \right) \cdot (850 \cdot \text{kip} - 800 \cdot \text{kip}) \right] + 800 \cdot \text{kip}$$

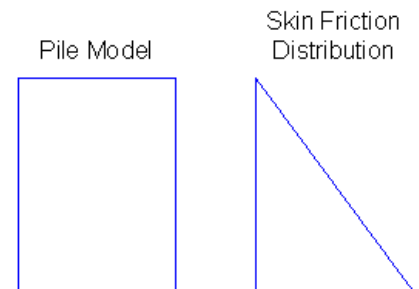
$$UC_{at15blows} = 817 \cdot \text{kip}$$

$$R_{dr\_14x117\_factored} := UC_{at15blows} \cdot \phi_{dyn.reduced}$$

$$R_{dr\_14x117\_factored} = 425 \cdot \text{kip}$$

Efficiency	0.800
Helmet Hammer Cushion	3.20 kips 34790 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	70.00 ft
Pile Penetration	70.00 ft
Pile Top Area	34.40 in <sup>2</sup>

DRIVABILITY RESISTANCE GOVERNS



Res. Shaft = 20 %  
 (Constant Res. Shaft)

## **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:  $\alpha := 90 \cdot \text{deg}$

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

Friction angle between fill and wall:

From LRFD Table 3.11.5.3-1 range from 17 to 22  $\delta := 20 \cdot \text{deg}$

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

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

$$K_p = 6.89$$

[Rankine Theory - Passive Earth Pressure](#) from Bowles 5th Edition Section 11-5 pg 602

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

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

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

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

[Rankine Theory - Active Earth Pressure](#) from Maine DOT Bridge Design Guide  
Section 3.6.5.2 pg 3-7

For a horizontal backfill surface:

$\phi := 32 \cdot \text{deg}$

$$K_a := \tan\left(45 \cdot \text{deg} - \frac{\phi}{2}\right)^2 \quad K_a = 0.307$$

## **Bearing Resistance - Fill Soils:**

### **Nominal and factored Bearing Resistance - spread footing on fill soils**

#### **Presumptive Bearing Resistance for Service Limit State ONLY**

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

<u>Type of Bearing Material:</u>	<u>Consistency In Place:</u>	<u>Bearing Resistance Ordinary Range (ksf)</u>	<u>Recommended Value of Use (ksf)</u>
Coarse to medium sand, with little gravel (SW, SP)	Very Dense	8 to 12	8
	Medium dense to dense	4 to 8	6
	Loose	2 to 6	3

Based on corrected N-values ranging from 8 to 19 - Soils are loose to medium dense

#### **Recommended Value of Use:**

$$4 \cdot \text{ksf} = 2 \cdot \text{tsf}$$

Therefore:  $q_{\text{nom}} := 2 \cdot \text{tsf}$

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

$$q_{\text{factored\_bc}} := 2 \cdot \text{tsf} \quad \text{or} \quad q_{\text{factored\_bc}} = 4 \cdot \text{ksf}$$

## Nominal and factored Bearing Resistance - spread footing on fill soils At the Strength Limit State

Assumptions:

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

Depth to Groundwater table:  $D_w := 9 \cdot \text{ft}$  Based on boring logs

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

Look at several footing widths

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

Terzaghi Shape factors from Table 4-1

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

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

For  $\phi=32 \text{ deg}$

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

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

$$q := D_w \cdot \gamma_d + (D_f - D_w) \cdot (\gamma_s - \gamma_w) \quad q = 0.4148 \cdot \text{tsf}$$

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

$$q_{\text{nominal}} = \begin{pmatrix} 11 \\ 12 \\ 13 \\ 14 \\ 15 \end{pmatrix} \cdot \text{tsf}$$

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

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

$$q_{\text{factored}} = \begin{pmatrix} 5 \\ 6 \\ 6 \\ 6 \\ 7 \end{pmatrix} \cdot \text{tsf}$$

Based on these footing widths

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

$$q_{\text{factored}} = \begin{pmatrix} 10 \\ 11 \\ 12 \\ 12 \\ 13 \end{pmatrix} \cdot \text{ksf}$$

At **Strength Limit State:**

Recommend factored bearing resistance of 6 tsf or 12 ksf for footings 12 feet wide or less on fill soils

## Settlement Analysis:

The vertical alignment of the proposed bridge will be raised 7 inches at the west approach to improve drainage conditions and provide a crest vertical curve to the bridge.

Evaluate the amount of settlement due to this fill:

Reference: FHWA Soils and Foundation Workshop Manual (FHWA HI-88-009)  
 Bazaraa 1967 pg 168

Simplified soil profile based on BB-WSB-101:

<hr/>		Finished Grade Elevation 9.8 ft
Proposed Fill Assume:	7 inches of fill N = 25 bpf (medium dense) $\gamma = 125$ pcf	
<hr/>		Elevation 9.2 ft
Existing Fill: fine to coarse sand 9.0 feet thick N = 9 bpf (loose) $\gamma = 125$ pcf	$H_1 := 9 \cdot \text{ft}$ $\gamma_{\text{fill}} := 125 \cdot \text{pcf}$ $N_{\text{fill}} := 9$	Groundwater Elevation 9.2 ft
<hr/>		Elevation 0.2 ft
Sand: fine and fine to medium sand 21.5 feet thick N = 20 bpf (medium dense) $\gamma = 125$ pcf	$H_2 := 21.5 \cdot \text{ft}$ $\gamma_{\text{sand}} := 125 \cdot \text{pcf}$ $N_{\text{sand}} := 20$	
<hr/>		Elevation -21.3 ft
Silty Clay 33.6 feet thick Su=500 psf (medium stiff) $\gamma = 115$ pcf	$H_3 := 33.6 \cdot \text{ft}$ $\gamma_{\text{siltyclay}} := 115 \cdot \text{pcf}$ $C_{c\_siltyclay} := 0.235$	$e_o := 1.19$
<hr/>		Elevation -54.9 ft
Glacial Till 2.4 feet thick N = 15 (medium dense) $\gamma = 130$ pcf	$H_4 := 2.4 \cdot \text{ft}$ $\gamma_{\text{till}} := 130 \cdot \text{pcf}$ $N_{\text{till}} := 15$	
<hr/>		Top of Bedrock Elevation -57.3 ft

LOADING ON AN INFINITE STRIP  
 VERTICAL EMBANKMENT LOADING

Project Name: Bourne Avenue Bridge Client: Wells  
 Project Number: 15611.00 Project Manager: JWentworth  
 Date: 06/17/08 Computed by: km

Embank. slope a = 7.00(ft)  
 Embank. width b = 22.00(ft)  
 p load/unit area = 80.00(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION

X = 22.00(ft)

Z	Vert. $\Delta z$	
(ft)	(psf)	
0.00	40.00	
2.00	39.98	
4.00	39.83	at 4.5 ft $\Delta\sigma_{zfill} := 78.48 \cdot psf$
6.00	39.45	multiply by 2 for full embankment
8.00	38.82	
10.00	37.94	
12.00	36.86	
14.00	35.62	
16.00	34.29	
18.00	32.92	at 19.75 ft $\Delta\sigma_{zsand} := 63.44 \cdot psf$
20.00	31.55	multiply by 2 for full embankment
22.00	30.20	
24.00	28.90	
26.00	27.65	
28.00	26.46	
30.00	25.34	
32.00	24.28	
34.00	23.29	
36.00	22.36	
38.00	21.49	
40.00	20.67	
42.00	19.90	
44.00	19.18	
46.00	18.50	at 47.3 ft $\Delta\sigma_{zsiltyclay} := 36.18 \cdot psf$
48.00	17.87	multiply by 2 for full embankment
50.00	17.27	
52.00	16.71	
54.00	16.18	
56.00	15.68	
58.00	15.21	
60.00	14.76	
62.00	14.34	
64.00	13.94	at 65.3 ft $\Delta\sigma_{ztill} := 27.39 \cdot psf$
66.00	13.56	multiply by 2 for full embankment
68.00	13.19	
70.00	12.85	

### Existing Fill

Determine corrected SPT value N':

N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{1o} := \frac{H_1}{2} \cdot (\gamma_{\text{fill}} - \gamma_w) \quad \sigma_{1o} = 281.7 \cdot \text{psf} \quad \text{at mid-point}$$

SPT N-value (bpf)  $N_{\text{fill}} := 9$

AT  $P_o = 560 \text{ psf}$   $N'/N_{\text{fill}} = r1 = 1.75$   $r1 := 1.75$

Corrected Blow Count  $N' := r1 \cdot N_{\text{fill}}$   $N' = 16$

From Figure 13 using the "well graded fine to medium silty sand" curve

Bearing Capacity Index:  $C1 := 52$

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

$$\Delta\sigma_{z\text{fill}} = 78.48 \cdot \text{psf}$$

### Sand

Determine corrected SPT value N':

N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{2o} := \left[ \frac{H_2}{2} \cdot (\gamma_{\text{sand}} - \gamma_w) \right] + H_1 \cdot (\gamma_{\text{fill}} - \gamma_w) \quad \sigma_{2o} = 1236.35 \cdot \text{psf} \quad \text{at mid-point}$$

SPT N-value (bpf)  $N_{\text{sand}} := 20$

AT  $P_o = 1900 \text{ psf}$   $N'/N_{\text{fill}} = r1 = 0.95$   $r1 := 0.95$

Corrected Blow Count  $N' := r1 \cdot N_{\text{sand}}$   $N' = 19$

From Figure 13 using the "well graded fine to medium silty sand" curve

Bearing Capacity Index:  $C2 := 57$

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

$$\Delta\sigma_{z\text{sand}} = 63.44 \cdot \text{psf}$$

### Silty Clay

Average values from lab data:

$$e_o = 1.19 \quad C_{c\_siltyclay} = 0.235$$

$$\sigma_{3o} := H_1 \cdot (\gamma_{\text{fill}} - \gamma_w) + H_2 \cdot (\gamma_{\text{sand}} - \gamma_w) + \frac{H_3}{2} \cdot (\gamma_{\text{siltyclay}} - \gamma_w) \quad \sigma_{3o} = 2792.98 \cdot \text{psf} \quad \text{at mid-point}$$

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

$$\Delta\sigma_{z\text{siltyclay}} = 36.18 \cdot \text{psf}$$



## Till

Determine corrected SPT value N':

N'/N - Ratio of Corrected blow count to SPT Value

$$\sigma_{40} := H_1 \cdot (\gamma_{\text{fill}} - \gamma_w) + H_2 \cdot (\gamma_{\text{sand}} - \gamma_w) + H_3 \cdot (\gamma_{\text{siltyclay}} - \gamma_w) + \frac{H_4}{2} \cdot (\gamma_{\text{till}} - \gamma_w)$$

$$\sigma_{40} = 3757.78 \cdot \text{psf} \quad \text{at mid-point}$$

SPT N-value (bpf)	$N_{\text{till}} := 15$		
AT $P_o = 3750$ psf	$N'/N_{\text{fill}} = r1 = 0.75$	$r1 := 0.75$	
Corrected Blow Count	$N' := r1 \cdot N_{\text{till}}$	$N' = 11$	

From Figure 13 using the "well graded fine to medium silty sand" curve

Bearing Capacity Index:  $C4 := 44$

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

$$\Delta\sigma_{\text{ztill}} = 27.39 \cdot \text{psf}$$

### Calculate Settlement:

Fill:  $\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{10} + \Delta\sigma_{\text{zfill}}}{\sigma_{10}}\right)$   $\Delta H_1 = 0.2217 \cdot \text{in}$

Sand:  $\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{20} + \Delta\sigma_{\text{zsand}}}{\sigma_{20}}\right)$   $\Delta H_2 = 0.0984 \cdot \text{in}$

Silty Clay:  $\Delta H_3 := H_3 \cdot \left(\frac{C_{\text{siltyclay}}}{1 + e_o}\right) \cdot \log\left(\frac{\sigma_{30} + \Delta\sigma_{\text{zsiltyclay}}}{\sigma_{30}}\right)$   $\Delta H_3 = 0.2418 \cdot \text{in}$

Till:  $\Delta H_4 := H_4 \cdot \frac{1}{C4} \cdot \log\left(\frac{\sigma_{40} + \Delta\sigma_{\text{ztill}}}{\sigma_{40}}\right)$   $\Delta H_4 = 0.0021 \cdot \text{in}$

Total Settlement =  $\Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 = 0.5639 \cdot \text{in}$

Check Clay settlement using Das in an Excel spreadsheet:

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Wells (15611.00) embankment settlements with 7 inches of fill												
2	0.6 ft of new fill overlying 9 ft of existing fill, 21.5 ft of sand, 33.6 ft of silty clay and 2.4 ft of till.												
3	Groundwater table at top of existing fill.												
4										Cc	0.235		
5	Unit weight of clay	115 pcf					B1	15 ft		Cr	0.026		
6	Unit Weight of sand	125 pcf											
7	Unit weight of water	62.4 pcf					B2	7 ft		e	1.19		
8							H	0.6					
9													
10	Depth	Ho	Po	ocr	Pmax	a1	a2	Dstress	settlement				
11	(ft)	(ft)	(psf)		(psf)	(rad)	(rad)	(psf)	(ft)				
12	30		1996.8	0.1211	241.812	-0.169101	-0.463648	47.51294	33.6 ft CLAY				
13	31	1	2049.4	0.1211	248.182	-0.16653	-0.450661	46.50706	0.001046				
14	32	1	2102	0.1211	254.552	-0.163951	-0.438337	45.53155	0.000999				
15	33	1	2154.6	0.1211	260.922	-0.161375	-0.426627	44.58597	0.000955				
16	34	1	2207.2	0.1211	267.292	-0.158813	-0.415492	43.66978	0.000913				
17	35	1	2259.8	0.1211	273.662	-0.156272	-0.404892	42.78234	0.000874				
18	36	1	2312.4	0.1211	280.032	-0.153758	-0.394791	41.92291	0.000837				
19	37	1	2365	0.1211	286.402	-0.151278	-0.385158	41.09074	0.000803				
20	38	1	2417.6	0.1211	292.771	-0.148835	-0.375961	40.28502	0.00077				
21	39	1	2470.2	0.1211	299.141	-0.146432	-0.367174	39.50493	0.000739				
22	40	1	2522.8	0.1211	305.511	-0.144073	-0.358771	38.74962	0.00071				
23	41	1	2575.4	0.1211	311.881	-0.141758	-0.350728	38.01826	0.000683				
24	42	1	2628	0.1211	318.251	-0.139489	-0.343024	37.31001	0.000657				
25	43	1	2680.6	0.1211	324.621	-0.137268	-0.335639	36.62406	0.000632				
26	44	1	2733.2	0.1211	330.991	-0.135094	-0.328553	35.95958	0.000609				
27	45	1	2785.8	0.1211	337.36	-0.132969	-0.321751	35.31579	0.000587				
28	46	1	2838.4	0.1211	343.73	-0.130891	-0.315215	34.69192	0.000566				
29	47	1	2891	0.1211	350.1	-0.128861	-0.308931	34.08722	0.000546				
30	48	1	2943.6	0.1211	356.47	-0.126877	-0.302885	33.50096	0.000527				
31	49	1	2996.2	0.1211	362.84	-0.124941	-0.297064	32.93244	0.000509				
32	50	1	3048.8	0.1211	369.21	-0.12305	-0.291457	32.38098	0.000492				
33	51	1	3101.4	0.1211	375.58	-0.121204	-0.286051	31.84593	0.000476				
34	52	1	3154	0.1211	381.949	-0.119403	-0.280838	31.32666	0.000461				
35	53	1	3206.6	0.1211	388.319	-0.117645	-0.275806	30.82258	0.000446				
36	54	1	3259.2	0.1211	394.689	-0.115929	-0.270947	30.33309	0.000432				
37	55	1	3311.8	0.1211	401.059	-0.114254	-0.266252	29.85765	0.000418				
38	56	1	3364.4	0.1211	407.429	-0.11262	-0.261714	29.39572	0.000405				
39	57	1	3417	0.1211	413.799	-0.111025	-0.257324	28.94679	0.000393				
40	58	1	3469.6	0.1211	420.169	-0.109469	-0.253076	28.51037	0.000381				
41	59	1	3522.2	0.1211	426.538	-0.107949	-0.248963	28.086	0.00037				
42	60	1	3574.8	0.1211	432.908	-0.106466	-0.244979	27.67322	0.000359				
43	61	1	3627.4	0.1211	439.278	-0.105018	-0.241118	27.2716	0.000349				
44	62	1	3680	0.1211	445.648	-0.103605	-0.237374	26.88075	0.000339				
45	63	1	3732.6	0.1211	452.018	-0.102224	-0.233743	26.50025	0.00033				
46	64	1	3785.2	0.1211	458.388	-0.100876	-0.23022	26.12974	0.000321				
47													
48									0.019936391				
49	Reference: Principles of Foundation Engineering Fourth Edition Braja M. Das												
50	Section 4.6 Stress Increase Under an Embankment pg 233												
													<b>total settlement (in.)</b> <b>0.24</b>

## Time Rate of Settlement:

### Look at case of most settlement: West Approach with 7 inches of fill; 0.25 inches of settlement

Determine the time for 90% consolidation for primary settlement

Reference: *FHWA Soils and Foundation Workshop Manual Second Edition page 179*

Thickness of the clay layer =  $H_c := 33.6 \cdot \text{ft}$

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

$$H_{cv} := 16.8 \cdot \text{ft}$$

Time factor from Table on page 179  $TF := 0.848$

At 90% primary consolidation

$$\text{Coefficient of consolidation from lab data: } C_v := 4.7 \cdot 10^{-7} \cdot \frac{\text{ft}^2}{\text{sec}} \quad C_v = 0.0406 \cdot \frac{\text{ft}^2}{\text{day}}$$

Time rate of settlement to achieve 90% Primary Settlement

$$t_{90} := \frac{TF \cdot H_{cv}^2}{C_v} \quad t_{90} = 5893.9007 \cdot \text{day} \quad \text{year} := 365 \cdot \text{day}$$

$$t_{90} = 16.1477 \cdot \text{year}$$

## Determination of Downdrag:

Use beta method to determine downdrag

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

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

Assumed values

Unit weight of granular soil  $\gamma_t := 125 \cdot \text{pcf}$

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

Effective unit weight of granular soil  $\gamma' := \gamma_t - \gamma_w$   $\gamma' = 62.6 \cdot \text{pcf}$

Unit weight of clay  $\gamma_{clay} := 115 \cdot \text{pcf}$

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

Stress from overburden material. Overburden consists of approximately 7 inches of fill on 9 feet of existing fill material on 21.5 feet of marine sand. Water table is at the top of the existing fill.

Additional Overburden Stress due to fill =

$$\sigma_{v\_ob} := 0.7 \cdot \text{ft} \cdot \gamma_t \quad \sigma_{v\_ob} = 87.5 \cdot \text{psf}$$

Effective vertical stress in middle of each layer, water elevation coincides with top of overburden

Total thickness of each stratum

$$D_{fill} := 9 \cdot \text{ft} \quad D_{sand} := 21.5 \cdot \text{ft} \quad D_{clay} := 33.6 \cdot \text{ft}$$

$$\sigma'_{v\_fill} := \sigma_{v\_ob} + \frac{D_{fill}}{2} \cdot \gamma' \quad \sigma'_{v\_fill} = 369.2 \cdot \text{psf}$$

$$\sigma'_{v\_sand} := \sigma_{v\_ob} + D_{fill} \cdot \gamma' + \frac{D_{sand}}{2} \cdot \gamma' \quad \sigma'_{v\_sand} = 1323.9 \cdot \text{psf}$$

$$\sigma'_{v\_clay} := \sigma_{v\_ob} + D_{fill} \cdot \gamma' + D_{sand} \cdot \gamma' + \frac{D_{clay}}{2} \cdot \gamma'_{clay} \quad \sigma'_{v\_clay} = 2880.5 \cdot \text{psf}$$

Pile parameters:

Look at piles: 12x53 14x73, 14x89 and 14x117

Pile depth:

$$d := \begin{pmatrix} 11.78 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot \text{in} \quad \begin{matrix} \text{HP 12 x 53} \\ \text{HP 14 x 73} \\ \text{HP 14 x 89} \\ \text{HP 14 x 117} \end{matrix}$$

Flange width:

$$B_f := \begin{pmatrix} 12.045 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot \text{in} \quad \begin{matrix} \text{HP 12 x 53} \\ \text{HP 14 x 73} \\ \text{HP 14 x 89} \\ \text{HP 14 x 117} \end{matrix}$$

$$\text{Box perimeter:} \quad P := 2 \cdot (d + B_f) \quad P = \begin{pmatrix} 47.65 \\ 56.39 \\ 57.05 \\ 58.19 \end{pmatrix} \cdot \text{in} \quad \begin{matrix} \text{HP 12 x 53} \\ \text{HP 14 x 73} \\ \text{HP 14 x 89} \\ \text{HP 14 x 117} \end{matrix}$$

Magnitude of maximum dowdrag, considered over entire clay thickness

$$Q_{dd} := (D_{\text{fill}} \cdot \sigma'_{v_{\text{fill}}} \cdot \beta_{\text{gr}} + D_{\text{sand}} \cdot \sigma'_{v_{\text{sand}}} \cdot \beta_{\text{gr}} + D_{\text{clay}} \cdot \sigma'_{v_{\text{clay}}} \cdot \beta_{\text{clay}}) \cdot P \quad Q_{dd} = \begin{pmatrix} 87.8253 \\ 103.9343 \\ 105.1508 \\ 107.252 \end{pmatrix} \cdot \text{kip}$$

If dowdrag is considered over entire clay stratum, what is the factor of safety.

Ultimate capacity based on 50ksi steel and area of pile

Pile area:

$$A_{\text{pile}} := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot \text{in}^2 \quad \begin{matrix} \text{HP 12 x 53} \\ \text{HP 14 x 73} \\ \text{HP 14 x 89} \\ \text{HP 14 x 117} \end{matrix}$$

$$Q_{\text{app}} := 400 \cdot \text{kip} \quad Q_{\text{ult}} := 50 \cdot \text{ksi} \cdot A_{\text{pile}} \quad Q_{\text{ult}} = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot \text{kip}$$

$$\text{FS} := \frac{Q_{\text{ult}}}{Q_{\text{app}} + Q_{\text{dd}}} \quad \text{FS} = \begin{pmatrix} 1.5887 \\ 2.1233 \\ 2.5834 \\ 3.3908 \end{pmatrix}$$

Magnitude of downdrag, considered over top 2/3 of clay stratum, realistic

$$\sigma'_{v.cl.2_3} := \sigma_{v.ob} + D_{fill} \cdot \gamma' + D_{sand} \cdot \gamma' + \frac{D_{clay} \cdot \frac{2}{3}}{2} \cdot \gamma' \quad \sigma'_{v.cl.2_3} = 2697.92 \cdot \text{psf}$$

$$Q_{dd.2_3} := \left( D_{fill} \cdot \sigma'_{v.fill} \cdot \beta_{gr} + D_{sand} \cdot \sigma'_{v.sand} \cdot \beta_{gr} + D_{clay} \cdot \frac{2}{3} \cdot \sigma'_{v.cl.2_3} \cdot \beta_{clay} \right) \cdot P$$

$$Q_{dd.2_3} = \begin{pmatrix} 69.0608 \\ 81.728 \\ 82.6845 \\ 84.3368 \end{pmatrix} \cdot \text{kip}$$

Factor of safety, downdrag over 2/3 of clay stratum

$$FS := \frac{Q_{ult}}{Q_{app} + Q_{dd.2_3}} \quad FS = \begin{pmatrix} 1.6522 \\ 2.2212 \\ 2.7036 \\ 3.5512 \end{pmatrix}$$

## USE downdrag load of 80 kips

*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:  
 Wells, Maine  
 DFI = 1100 degree-days

From the lab testing: soils are coarse grained assume a water content = ~5%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1100 frost penetration = 69.8 inches

Frost\_depth := 69.8in      Frost\_depth = 5.8167 · ft

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

**Method 2 - Check Frost Depth using Modberg Software**

Closest Station is Portland

**ModBerg Results**

Project Location: Portland Wsfo Airport, Maine

Air Design Freezing Index      =      1195 F-days  
 N-Factor                              =      0.80  
 Surface Design Freezing Index   =      956 F-days  
 Mean Annual Temperature        =      45.5 deg F  
 Design Length of Freezing Season =      118 days

Layer #:	Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-	Coarse	61.3	7.0	125.0	26	30	1.5	1.5	1,260

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 = 5.11 ft = 61.3 in.  
 \*\*\*\*\*

Use Modberg Frost Depth = 5.0 feet for design

## Seismic:

Wells Bourne Avenue Bridge		PIN 15611.00
Date and Time: 6/17/2008 9:04:26 AM		
Conterminous 48 States		
2007 AASHTO Bridge Design Guidelines		
AASHTO Spectrum for 7% PE in 75 years		
State - Maine		
Zip Code - 04090		
Zip Code Latitude = 43.329000		
Zip Code Longitude = -070.625500		
Site Class B		
Data are based on a 0.05 deg grid spacing.		
Period	Sa	
(sec)	(g)	
0.0	0.096	PGA - Site Class B
0.2	0.186	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 - 04090		
Zip Code Latitude = 43.329000		
Zip Code Longitude = -070.625500		
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1		
Site Class E - Fpga = 2.50, Fa = 2.50, Fv = 3.50		
Data are based on a 0.05 deg grid spacing.		
Period	Sa	
(sec)	(g)	
0.0	0.241	As - Site Class E
0.2	0.466	SDs - Site Class E
1.0	0.157	SD1 - Site Class E