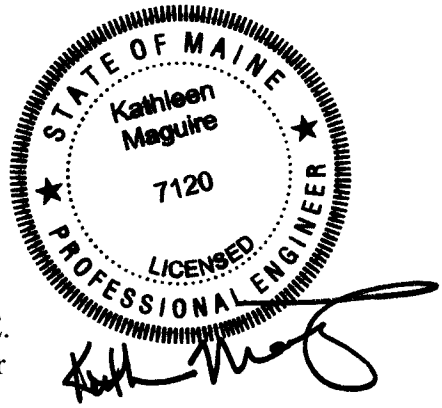
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CNR CROSSING BRIDGE
US ROUTE 202/STATE ROUTES 4 AND 100
OVER CANADIAN NATIONAL RAILWAY
AUBURN, MAINE



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GEOTECHNICAL DESIGN SUMMARY

The purpose of this design report is to make geotechnical recommendations for the replacement of the CNR Crossing Bridge on US Route 202/State Routes 4 and 100 over the Canadian National Railways line in Auburn, Maine. The proposed replacement bridge will consist of a three-span precast concrete superstructure on H-pile supported integral abutments and pipe pile pier bents for intermediate structure support. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-piles - The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 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. The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles and the geotechnical resistance of the pile. 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 and overall stability of the pile group. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading. 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.

Pile Bent Piers – Pipe pile pier bents were selected for intermediate structure support. Piles for pile bent piers shall consist of concrete filled pipe piles driven to bedrock. Pipe pile diameters ranging from 24 to 30 inches and wall thicknesses of 1/2 and 5/8 inch are recommended. Pipe pile should be fabricated in accordance with ATM A252, Grade 3, with minimum yield strength of 45 ksi. Pipe piles can be driven open-ended or closed-ended and shall be fitted with a cutting shoe constructed from Grade ASTM A148/60 steel. Pipe pile should be end bearing, driven to the required resistance on or within the bedrock. The designer shall design the pipe piles at the strength limit state considering the structural resistance of the piles and the geotechnical resistance of the pile. The structural resistance check should include checking axial, lateral, and flexural resistance. The design of the pipe piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading. The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each pier. The first pile driven at each pier 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 per LRFD Article 3.6.5.2. Piers located within a distance of 50 feet to the centerline of a railway track shall be designed for an equivalent static force of 400 kips which is assumed to act in any direction in a horizontal plane at a set distance of 4.0 feet above ground. Per MaineDOT Bridge Design Guide Section 5.5.1.5B piers located within 25 feet of the centerline of railroad tracks will require collision walls extending 6 feet above the top of rail elevation.

Stub Abutments - Integral stub abutments and wingwalls 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. Since the abutments will be pile supported, design for resistance against sliding and overturning is not required. In designing for passive earth pressure associated with integral abutments, the Rankine state is recommended. All abutment designs 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.

Settlement - Evaluation of the potential settlement due to the widening of the roadway resulted in approximately ½ to 1.0 inch of consolidation settlement. Additionally, approximately ½ inch of elastic settlement will occur in the coarse-grained soils during construction. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag forces on piles. This settlement is anticipated to occur over a long period of time (on the order of 5 to 20 years).

Downdrag – The magnitude of downdrag has been estimated to range between 70 and 85 kips depending upon pile size. It is recommended that a load factor, γ_p=1.0, be applied to the downdrag load applied to abutment piles for the strength limit state.

Frost Protection - Any foundation placed on granular subgrade soils should be founded a minimum of 5.3 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 -The CNR Crossing Bridge on US Route 202/State Routes 4 and 100 is on the National Highway System (NHS) and is therefore considered to be functionally important. Consequently, a detailed seismic analysis is required. The minimum seismic analysis requirements are defined in LFRD Article 4.7.4.3. The designer shall determine the specific analysis method using LRFD Tables 4.7.4.3.1-1 and 4.7.4.3.1-2. Seismic design requirements for Seismic Zone 2 are found in LRFD Article 3.10.9.3.

1.0 INTRODUCTION

A subsurface investigation for the replacement of the Canadian National Railway (CNR) Crossing Bridge on US Route 202/State Routes 4 and 100 over the Canadian National Railway line in Auburn, Androscoggin 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 1961 and consists of a three span, 138 foot long, steel girder superstructure with a concrete deck supported on steel H-pile supported abutments and two steel H-pile supported piers. The existing superstructure has a deck width of 35 feet. Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in "poor" (rating of 4) condition and superstructure is in "satisfactory" (rating of 6) condition. Inspection notes state that the deck has extensive cracking with excessive delamination and large spalling areas. Year 2006 MaineDOT Bridge Maintenance inspection reports indicate a Bridge Sufficiency Rating of 57.7. Bridge Inspection records assign the substructures a rating of 6, or "satisfactory". Maintenance reports indicate the substructure piers and abutments have moderate cracking, staining and delamination.

The project Preliminary Design Report (PDR) was prepared by CLD Consulting Engineers of York, Maine. The PDR investigated replacement of the bridge with both a three span structure and a single span structure. The single span structure proposes moving the abutment locations in toward the railway and will use lightweight fill to minimize settlements and any effects on the existing railway. The three-span structure proposes slight changes to the existing span arrangement in order to avoid the existing piles. The proposed replacement structure will have a new centerline approximately 2.5 feet north of the existing bridge centerline.

The selected PDR alternative is the three-span, precast/prestressed concrete voided slab superstructure founded on integral H-pile supported abutments and two pipe pile pier bents with crash walls. The new structure will have a span arrangement of 31.8 ft – 51 ft – 44.9 ft and will be skewed 8.92 degrees ahead on the right. The bridge will have two 12 foot lanes with 8 foot shoulders. The existing roadway profile will be lowered by approximately 1 to 1.5 feet in the replacement. In order to minimize impacts due to slopes 1H to 1V slopes reinforced with geocell will be utilized.

2.0 GEOLOGIC SETTING

The CNR Crossing Bridge in Auburn crosses the Canadian National Railway line approximately 1 mile north of the Auburn town line on US Route 202/State Routes 4 and 100 as shown on *Sheet 1 - Location Map* found at the end of this report.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of glaciomarine deposits.

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

According to the Surficial Bedrock Map of Maine, published by the Maine Geological Survey (1985), the bedrock at the site is identified as Carboniferous muscovite-biotite granite with abundant metasedimentary inclusions. This intrusive plutonic rock is identified as the Sebago Pluton.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions were explored by drilling five (5) test borings at the site. Test boring BB-ACNR-101 was drilled behind the location of Abutment No. 1. Test borings BB-ACNR-102 and BB-ACNR-102A were drilled at the location of Pier No. 1. Test boring BB-ACNR-103 was drilled at the location of Pier No. 2. Test boring BB-ACNR-104 was drilled behind the location of Abutment No. 2. The exploration locations and an interpretive subsurface profile depicting the site stratigraphy are shown on *Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile* found at the end of this report.

Borings BB-ACNR-101, BB-ACNR-102 and BB-ACNR-102A were drilled between May 19 and 29, 2008 by Northern Test Boring of Gorham, Maine. Borings BB-ACNR-103 and BB-ACNR-104 were drilled between May 19 and May 28, 2008 by the Maine Department of Transportation (MaineDOT) drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A - Boring Logs and on *Sheets 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. Both the MaineDOT drill rig and the Northern Test Boring drill rigs are equipped with automatic hammers to drive the split spoon. Both hammers were calibrated in 2007. The MaineDOT hammer was found to deliver approximately 30 percent more energy during driving than the standard rope and cathead system. The Northern Test Boring hammer was found to deliver approximately 6 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 MaineDOT field N-values and an average energy transfer factor of 0.633 to the raw Northern Test Boring field N-values. These hammer efficiency factors (0.77 and 0.633) and both the raw field N-values and the corrected N-values are shown on the boring logs.

Undisturbed tube samples were obtained in the soft soil deposits where possible. In-situ vane shear tests were made at regular intervals in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in the borings using an NQ 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 of samples obtained in the borings consisted of thirteen (13) standard grain size analyses, forty (40) grain size analysis with hydrometer, thirty-one (31) Atterberg Limits tests, fifteen (15) consolidation tests, and eighteen (18) standard tube openings with laboratory vanes. Laboratory test results are provided in Appendix B - Laboratory Data found 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 and piers consisted of granular fill, sand, interbedded silt, clayey silt, and silty clay, and sand all underlain by pegmatite granite. 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. A layer of fill was encountered in all of the borings. The layer was found to range from approximately 12 feet thick in boring BB-ACNR-102 and approximately 18 feet thick in boring BB-ACNR-104. The fill generally consisted of brown, light brown, or yellow, dry to damp, fine to coarse sand with trace silt, trace gravel and trace clay. Corrected SPT N-values in the fill ranged from 1 to 38 blows per foot (bpf) indicating that the fill is very loose to dense in consistency. One corrected N-value of >50 bpf was recorded in the fill but it is believed that this value was influenced by the presence of cobbles. Water contents from eight (8) samples obtained within this layer range from approximately 4 to 29%. Eight (8) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-3, A-4, A-2-4, or A-1-b by the AASHTO Classification System and a SP-SM, SC-SM, ML or SM by the Unified Soil Classification System.

Native Sand. A layer of native sand was encountered beneath the fill in boring BB-ACNR-101. This sand layer was found to be light brown to dark brown, wet, fine sand, with trace silt, trace medium sand with iron staining. The thickness of the layer was approximately 5.8 feet. One corrected SPT N-value obtained in the layer was 14 bpf indicating that the soil is medium dense in consistency. One (1) water content from a sample obtained within this layer was approximately 20%. One (1) grain size analysis conducted on a sample from this layer indicates that the soil is classified as an A-2-4 by the AASHTO Classification System and a SM by the Unified Soil Classification System.

Upper Silt. Underlying the fill and native sand soils, a layer of stiff silt was encountered in three of the five borings. The layer was not observed in boring BB-ACNR-103 and boring BB-ACNR-102A did not go deep enough to encounter the layer. This upper silt layer was determined to be what is commonly known as the desiccated "upper crust" of the Presumpscot Formation typically found in this area. This silt layer was found to be olive brown to brown, wet, silt with trace fine sand in layers. The thickness of the layer ranges from approximately 3 feet in boring BB-ACNR-102 to approximately 16.7 feet in boring BB-ACNR-101. Corrected SPT N-values obtained in the layer ranged from weight of hammer (WOH) to 12 bpf indicating that the soil is very soft to stiff in consistency. Vane shear testing conducted within the silt showed measured undrained shear strengths ranging from approximately 1045 to 1317 psf while the remolded shear strength ranged from approximately 134 to 943 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the clayey silt was determined to have sensitivity ranging from approximately 1.1 to 8.3 and is classified as insensitive to very sensitive. Water contents from eight (8) samples obtained within this layer range from approximately 21% to 33%. Eight (8) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a ML and CL-ML by the Unified Soil Classification System.

The following table summarizes the results of the Atterberg Limits test made from two (2) samples of the silt:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ACNR-102 3D A	Silt	32.9	27	22	5	2.18
BB-ACNR-104 7D	Silt	30.7	25	22	3	2.90

Interpretation of these results indicates that the silt is on the verge of being a viscous liquid as the natural water content exceeds the liquid limit. This indicates that the 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".

Interbedded Silt, Clayey Silt and Silty Clay. A layer of interbedded silt, clayey silt and silty clay was encountered beneath the upper silt and fill in all of the borings. This layer was found to be grey, wet, silt, clayey silt and silty clay, with trace gravel, trace sand in layers. The thickness of the layer ranges from approximately 20.6 feet in boring BB-ACNR-104 to approximately 50.5 feet in boring BB-ACNR-101.

Silt. Vane shear testing conducted on silt samples showed measured undrained shear strengths ranging from approximately 156 to 1099 psf while the remolded shear strength ranged from approximately 54 to 247 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silt was determined to have sensitivity ranging from approximately 1.8 to 11.4 and is classified as insensitive to very sensitive. Water contents from twelve (12) samples of the silt range from approximately 26% to 41%. Twelve (12)

grain size analyses conducted on silt samples indicate that the silt is classified as an A-4 or A-6 by the AASHTO Classification System and a ML, CL-ML or CL by the Unified Soil Classification System.

The following table summarizes the results of Atterberg Limits testing on the silt samples:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ACNR-101 2U	Silt	30.6	22	19	3	3.87
BB-ACNR-101 5U	Silt	26.4	22	20	2	3.20
BB-ACNR-102 3D B	Silt	33.5	25	20	5	2.70
BB-ACNR-102 1U	Silt	30.2	Non Plastic			
BB-ACNR-102 4D	Silt	31.5	27	20	7	1.64
BB-ACNR-102 2U	Silt	29.8	34	23	11	0.62
BB-ACNR-103 1U	Silt	34.3	28	21	7	1.90
BB-ACNR-103 4D	Silt	33.2	36	22	14	0.80
BB-ACNR-103 2U	Silt	29.0	27	23	4	1.50
BB-ACNR-103 3U	Silt	34.3	35	21	14	0.95
BB-ACNR-103 4U	Silt	40.8	37	27	10	1.38
BB-ACNR-104 3U	Silt	29.5	25	20	5	1.90

Interpretation of these results indicates the silt is generally on the verge of becoming a viscous liquid if disturbed. For eight (8) of the samples the natural water content exceeds the liquid limit. This indicates that the silt has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as “quick”. Two (2) of the samples have liquidity index values less than 1 indicating soils which are over consolidated. One (1) of the samples has a liquidity index of approximately 1 indicating a soil which is normally consolidated.

One-dimensional (1-D) consolidation testing was conducted on seven (7) tube samples taken from the silt. The results of these tests were used to calculate the anticipated settlements at the site and are included in Appendix B - Laboratory Data.

Clayey Silt. Vane shear testing conducted on clayey silt samples showed measured undrained shear strengths ranging from approximately 491 to 1473 psf while the remolded shear strength ranged from approximately 22 to 223 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the clayey silt was determined to have sensitivity ranging from approximately 4.0 to 30.6 and is classified as moderately sensitive to slightly quick. Water contents from fourteen (14) samples of the clayey silt range from approximately 31% to 40%. Fourteen (14) grain size analyses conducted on clayey silt samples indicate that the clayey silt is classified as an A-4 or A-6 by the AASHTO Classification System and a ML, CL-ML or CL by the Unified Soil Classification System.

The following table summarizes the results of Atterberg Limits testing on the clayey silt samples:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ACNR-101 9D	Clayey Silt	32.4	28	22	6	1.73
BB-ACNR-101 3U	Clayey Silt	32.8	30	19	11	1.25
BB-ACNR-101 10D	Clayey Silt	30.9	30	22	8	1.11
BB-ACNR-101 4U	Clayey Silt	38.0	35	24	11	1.27
BB-ACNR-101 11D	Clayey Silt	34.4	31	12	19	1.18
BB-ACNR-101 6U	Clayey Silt	35.2	31	22	9	1.47
BB-ACNR-102 6D	Clayey Silt	35.4	35	27	8	1.05
BB-ACNR-102 4U	Clayey Silt	30.7	26	19	7	1.67
BB-ACNR-102 8D	Clayey Silt	30.6	29	20	9	1.18
BB-ACNR-102 5U	Clayey Silt	36.8	33	22	11	1.35
BB-ACNR-103 6D	Clayey Silt	36.8	36	22	14	1.06
BB-ACNR-103 5U	Clayey Silt	40.2	35	23	12	1.43
BB-ACNR-104 1U	Clayey Silt	36.7	30	22	8	1.84
BB-ACNR-104 8D	Clayey Silt	31.9	31	19	12	1.08

Interpretation of these results indicates that the clayey silt is generally on the verge of becoming a viscous liquid if disturbed. For all of the clayey silt samples the natural water content exceeds the liquid limit. This indicates that the clayey silt has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as “quick”. Three (3) of the samples have a liquidity index of approximately 1 indicating a soils which are normally consolidated.

One-dimensional (1-D) consolidation testing was conducted on six (6) tube samples taken from the clayey silt. The results of these tests were used to calculate the anticipated settlements at the site and are included in Appendix B - Laboratory Data.

Silty Clay. Vane shear testing conducted on silty clay samples showed measured undrained shear strengths ranging from approximately 670 to 1161 psf while the remolded shear strength ranged from approximately 67 to 268 psf. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the silty clay was determined to have sensitivity ranging from approximately 4.3 to 10.0 and is classified as moderately sensitive to very sensitive. Water contents from three (3) samples of the silty clay range from approximately 37% to 41%. Three (3) grain size analyses conducted on silty clay samples indicate that the silty clay is classified as an A-4 or A-6 by the AASHTO Classification System and a CL by the Unified Soil Classification System.

The following table summarizes the results of Atterberg Limits testing on the silty clay samples:

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-ACNR-102 3U	Silty Clay	38.4	27	19	8	2.43
BB-ACNR-104 2U	Silty Clay	36.9	36	24	12	1.08
BB-ACNR-104 9D	Silty Clay	40.9	39	25	14	1.14

Interpretation of these results indicates that silty clays is generally on the verge of becoming a viscous liquid if disturbed. For all of the silty clay samples the natural water content exceeds the liquid limit. This indicates that the silty clay has a high liquefaction potential. It can be inferred that overburden pressure and interparticle cementation are providing stability for these soils. Under these conditions the slightest disturbance causing remolding has the potential to convert this type of deposit into a viscous liquid. Liquidity index values greater than or equal to 1 are indicative of soils that are unconsolidated and have a high liquefaction potentially commonly referred to as "quick".

One-dimensional (1-D) consolidation testing was conducted on two (2) tube samples taken from the silty clay layer. The results of these tests were used to calculate the anticipated settlements at the site and are included in Appendix B - Laboratory Data.

Sand. A layer of sand was encountered beneath the interbedded silt, clayey silt and silty clay in all of the borings with the exception of boring BB-ACNR-104. This layer was found to be grey and brown, wet, fine to coarse sand, with trace to some gravel, trace to little silt, and trace clay. Boulders were encountered within the sand layer in borings BB-ACNR-101 and BB-ACNR-102. The thickness of the sand layer ranged from approximately 10.5 feet in boring BB-ACNR-101 to approximately 30.0 feet in boring BB-ACNR-102. Corrected SPT N-values in the sand layer ranged from 13 to 105 bpf indicating that the soil is medium dense to very dense in consistency. Water contents from seven (7) samples obtained within the sand layer range from approximately 10% to 23%. Seven (7) grain size analyses conducted on samples from this layer indicate that the soil is classified as an A-2-4 or A-1-b by the AASHTO Classification System and a SC-SM, SW-SM, SP-SM or 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-ACNR-101 Abutment No. 1	100.5 feet	137.7 feet	77%
BB-ACNR-102 Pier No. 1	94.0 feet	123.8 feet	65%
BB-ACNR-103 Pier No. 2	73.3 feet	151.1 feet	45 - 65%
BB-ACNR-104 Abutment No. 2	56.1 feet	185.0 feet	28 - 53%

The bedrock at the site can be identified as white, green and grey, coarse-grained, pegmatite GRANITE, with garnet and mica, hard, slightly weathered. Black, white and grey GNEISS interbedded with pegmatite granite intrusions was encountered in boring BB-ACNR-102. The bedrock is a part of the Sebago Pluton. The RQD of the bedrock was determined to range from 28 to 77% indicating a rock mass quality of poor to good.

6.0 FOUNDATION ALTERNATIVES

The subsurface investigation indicates the presence of a significant compressible clay layer underlying the bridge site. Due to the soft nature and depth of the soils, deep foundations are recommended. The following alternatives, with varying levels of risk, may be considered for the bridge replacement:

- A three-span structure utilizing cast-in-place concrete or precast concrete integral abutments supported on driven H-piles and concrete column piers on driven H-pile supported distribution slabs
- A three-span structure utilizing cast-in-place concrete or precast concrete integral abutments supported on driven H-piles and pipe pile pier bents with crash walls
- A single span structure utilizing full height concrete abutments supported on traditional driven H-pile groups with crash walls and lightweight fill behind the location of the existing piers to minimize settlements and negative impact to the existing railway line
- A single span structure utilizing pile supported integral abutments which are supported laterally by Mechanically Stabilized Earth (MSE) wrapped embankments with crash walls and lightweight fill behind the location of the existing piers to minimize bridge and approach settlements and railway line settlement/damage

It is preferred that the new bridge alignment closely match the existing bridge alignment. A three span structure with an alignment closely matching the existing bridge alignment is the geotechnically preferred option. Additionally, if accelerated construction is desired this alternative will have the shortest construction schedule due to no need to preload, wick drain or construct costly lightweight fill approaches.

A single span structure with an alignment closely matching the existing bridge alignment is also a viable alternative. This alternative would require the use of lightweight fill behind the location of the existing piers to minimize settlements and damage to the existing railway line

The selected PDR alternative is the three-span, precast/prestressed concrete voided slab superstructure founded on integral H-pile supported abutments and two pipe pile pier bents with crash walls. The new structure will have a span arrangement of 31.8 ft – 51 ft – 44.9 ft and will be skewed 8.92 degrees ahead on the right. The bridge will have two 12 foot lanes with 8 foot shoulders. In order to minimize impacts due to slopes 1H to 1V slopes reinforced with geocell will be utilized.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for the stub abutments founded on a single row of integral H-piles driven to bedrock and pipe pile pier bents driven to bedrock with crash walls which are identified as the optimal substructure types at 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 table below:

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-ACNR-101	229.2 feet	100.5 feet	137.7 feet	77%	95 feet
Abutment #2 BB-ACNR-104	231.8 feet	56.1 feet	185.0 feet	28 - 53%	50 feet

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

The designer shall design the H-piles at the strength limit state considering the structural resistance of the piles and the geotechnical resistance of the pile. 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 and overall stability of the pile group. Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and flexure as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2.

7.1.1 Strength Limit State Design

The nominal compressive structural resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. For preliminary analysis, the H-piles can be assumed fully embedded and λ can be taken as 0. It is the responsibility of

the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural pile resistances. The factored structural axial compressive resistances of the four proposed H-pile sections presented in this report were calculated using a resistance factor, ϕ_c, of 0.60 and a λ of 0.

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

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 ϕ_{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 10.5.5.2.3, the resistance factor of 0.65 is reduced by 20% since it is applied to a nonredundant pile group. This results in a resistance factor, ϕ_{dyn}, of 0.52.

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

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	354	217	354
HP 14 x 73	642	446	358	446
HP 14 x 89	783	542	424	542
HP 14 x 117	1032	710	533	710

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

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the factored geotechnical resistance in the table above.

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

(LFRD Eq. 6.9.2.2-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.12.2.

7.1.2 Service and Extreme Limit State Design

For the service and extreme limit states resistance factors, ϕ, of 1.0 are recommended for structural, geotechnical and drivability pile resistances. For preliminary analysis, the H-piles can be assumed fully embedded and λ can be taken as 0. It is the responsibility of the structural engineer to recalculate the column slenderness factor (λ) for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific L-Pile® analyses and determine structural 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 report.

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

Pile Section	Factored Resistance (kips)			
	Structural Resistance*	Geotechnical Resistance	Drivability	Governing Resistance
HP 12 x 53	775	786	418	775
HP 14 x 73	1070	991	688	991
HP 14 x 89	1305	1204	815	1204
HP 14 x 117	1720	1578	1025	1578

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

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the resistances shown in the last column of the table above. It should be noted that the structural resistance governs for the HP 12x53 pile section while the remaining pile sections are governed by the geotechnical resistance.

7.1.3 Pile Resistance and Pile Quality Control

Based on the anticipated depth to bedrock at the site, pile splices will be required. The location and number of pile splices shall be in conformance with MaineDOT Standard Specification 501 and be subject to the approval of the Resident. The splices shall be the Champion HP-30000, or approved equivalent, mechanical splicer. Evaluation of equivalent products will be based on the submission of data demonstrating the capability of transferring the full pile strength in compression and tension and developing the bending moment capacity of the pile in both the x-x and y-y axes. The splicers shall be installed and welded as recommended by the manufacturer. Welding shall not be done when the temperature in

the immediate vicinity of the weld is below 0°F; when the surfaces are damp or exposed to rain, snow, or high wind; or when the welders or welding operators are exposed to inclement conditions. The pile shall be preheated to and maintained at 150°F minimum within 6 inches from the weld during welding. Formal welding procedures are not required. Welders shall be prequalified in accordance with Section 504 - Structural Steel.

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 is a minimum of five (5) 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 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.2 Pipe Pile Pier Bents

Pipe pile pier bents were selected for intermediate structure support. Piles for pile bent piers shall consist of concrete filled pipe piles driven to bedrock. Pipe pile diameters ranging from 24 to 30 inches and wall thicknesses of 1/2 and 5/8 inch are recommended. Pipe pile should be fabricated in accordance with ATM A252, Grade 3, with minimum yield strength of 45 ksi. Pipe piles can be driven open-ended or closed ended and shall be fitted with a cutting shoe constructed from Grade ASTM A148/60 steel. Pipe pile should be end bearing, driven to the required resistance on or within the bedrock.

Pile lengths at the proposed pier locations may be estimated based on the table below:

Location	Estimated Pile Cap Bottom Elevation	Depth to Bedrock From Ground Surface	Top of Rock Elevation	Rock Quality Designation	Estimated Pile Length
Pier #1 BB-ACNR-102	232.0 feet	94.0 feet	123.8 feet	65%	110 feet
Pier #2 BB-ACNR-103	233.0 feet	73.3 feet	151.1 feet	45 - 65%	85 feet

These pile lengths do not take into account the additional eight (8) feet of pile required for dynamic testing instrumentation or any additional pile length needed to accommodate the Contractor's leads and driving equipment.

The designer shall design the pipe piles at the strength limit state considering the structural resistance of the piles and the geotechnical resistance of the pile. 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 pipe piles at the service limit state shall consider tolerable horizontal movement of the piles and overall stability of the pile group. Since the pier piles will be subjected to lateral loading and have a substantial unbraced length, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2.

Per LRFD Article 3.6.5.2 piers located within a distance of 50 feet to the centerline of a railway track shall be designed for an equivalent static force of 400 kips which is assumed to act in any direction in a horizontal plane at a set distance of 4.0 feet above ground. Per MaineDOT Bridge Design Guide (BDG) Section 5.5.1.5B piers located within 25 feet of the centerline of railroad tracks will require collision walls extending 6 feet above the top of rail elevation.

7.2.1 Strength Limit State Design

The nominal compressive structural resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.5.1. The pipe piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9.5.1.

For the strength limit state the factored axial compressive structural resistance of the pile (P_r) shall be calculated using the resistance factor (ϕ_c) of 0.70 as specified in LRFD Article 6.5.4.2. The proposed piles at Pier No. 1 will potentially have the longest unsupported pile length, approximately 20 feet, and will therefore govern the structural resistance of piles at the piers.

Per LRFD Article 6.5.4.2, at the strength limit state, for pipe piles in compression and bending, the axial resistance factor ϕ_c=0.8 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined nominal axial and flexural resistance of the pile in the interaction equation, (LRFD Eq. 6.9.2.2-1 or-2) with flexural resistance determined as specified in LRFD Article 6.12. The factored structural resistance for pipe pile sections in combined axial compression and flexure are not provided in this report as these analyses are considered part of the structural design and the responsibility of the structural designer.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the eight proposed pipe pile sections were calculated using a resistance factor, ϕ_{stat}, of 0.45 for end bearing pile on bedrock.

The drivability of the eight proposed pipe pile sections was considered. The maximum driving stresses in the pile, assuming the use of 45 ksi steel, shall be less than 40 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance that must be achieved was conduced. The resistance factor for a single pile in axial compression when a dynamic test is done given in LRFD Table 10.5.5.2.3-1 is ϕ_{dyn}= 0.65. LRFD 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 LFRD 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 five (5) piles in a group. This results in a resistance factor, ϕ_{dyn}, of 0.52.

Factored axial compressive structural resistances, factored geotechnical resistances and drivability resistances in the lower portion of the eight piles sections analyzed are summarized in the table below. Supporting calculations are included in Appendix C-Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles at the Strength Limit State

Pipe Pile		Factored Resistance (kips)			
Diameter	Wall thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
24-in	1/2–in	957	507	476	507
26-in	1/2–in	1057	540	497	540
28-in	1/2–in	1157	572	530	572
30-in	1/2–in	1256	605	557	605
24-in	5/8-in	1181	631	575	631
26-in	5/8-in	1306	671	598	671
28-in	5/8-in	1431	712	619	712
30-in	5/8-in	1555	753	640	753

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the geotechnical resistance shown in the table above.

7.2.2 Service and Extreme Limit State Design

Per LRFD Article 10.5.5.1 the ability of the pier bents to meet defection criteria at the service limit state shall be investigated using a resistance factor of 1.0. Per LRFD Article 10.5.5.3.3 the design of pier bents at the extreme limit state shall be investigated using a resistance factor of 1.0.

The axial structural resistance of eight pipe pile sections was investigated using a resistance factor of 1.0. The pipe piles have an unbraced length and require calculation of the λ factor as specified in LRFD Article 6.9.5.1. The axial geotechnical compressive resistance of eight pipe pile sections was calculated using Canadian Foundation Engineering Manual methods

and a resistance factor of 1.0. The drivability of the eight proposed pipe pile sections was considered. The maximum driving stresses in the pile, assuming the use of 45 ksi steel, shall be less than 40 ksi. The resistance factor for a single pile in axial compression for the service and extreme limit states of 1.0 was used.

Factored axial structural, geotechnical and drivability resistances of eight pipe pile sections were calculated for the service and extreme limit states and are summarized below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Factored Axial Resistances for Pipe Piles at the Service and Extreme Limit States

Pipe Pile		Factored Resistance (kips)			
Diameter	Wall thickness	Structural Resistance	Geotechnical Resistance	Drivability Resistance	Governing Resistance
24-in	1/2–in	1367	1127	916	1127
26-in	1/2–in	1510	1199	955	1199
28-in	1/2–in	1652	1272	1020	1272
30-in	1/2–in	1794	1344	1110	1344
24-in	5/8-in	1688	1401	1106	1401
26-in	5/8-in	1866	1492	1150	1492
28-in	5/8-in	2044	1582	1191	1582
30-in	5/8-in	2221	1673	1230	1673

Although the factored axial drivability resistance is less than both the factored axial structural and geotechnical resistances, LRFD Article 10.7.8 states that for routine pile installation applications where significant local experience can be applied to keep the risk of pile installation problems low, a project specific drivability analysis using the wave equation may be waived. In light of this, it is recommended that the governing resistance used in design be the geotechnical resistances shown in the table above.

7.2.3 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 pier. The first pile driven at each pier 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 per LRFD Article 3.6.5.2. If three to four piles are dynamically tested, and if there is a minimum of five (5) 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 40 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 15 blows per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

7.3 Stub Abutments

Integral stub abutments and wingwalls 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. Since the abutments will be pile supported, design for resistance against sliding and overturning is not required.

A resistance factor of ϕ= 1.0 shall be used to assess abutment design at the service limit state including: settlement, horizontal movement and overall stability. Extreme limit state design checks for abutment shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure and overall stability. A resistance factor of ϕ=1.0 shall be used for the extreme limit state.

Conventional wingwalls 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 wingwalls 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 wingwalls and abutments if an approach slab is not specified. In the situation a structural approach slab is specified, reduction of the surcharge loads is permitted per LRFD Article 3.11.6.2. 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 form the table below:

Equivalent Height of Soil for Vehicular Loading

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

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf. Sliding computations for resistance to lateral loads shall assume a maximum allowable frictional coefficient of 0.45 at the soil-concrete interface.

Integral abutments and wingwall sections that are integral with the abutment should be designed to withstand a passive earth pressure state. In designing for passive earth pressure

associated with integral abutments, the Coulomb state is recommended. Experience in designing wingwalls for integral abutments has shown that the use of the Coulomb passive earth pressure K_p=6.89 may result in uneconomical wall sections. For this reason, consideration may be given to using a Rankine passive earth pressure, K_p=3.25 when designing integral abutments and integral wingwall extensions.

All abutment designs shall include a drainage system behind the abutments 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 and wingwalls 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 Settlement

In order to accommodate the proposed widened bridge superstructure, the roadway will be widened behind each abutment on the western side by approximately 8 feet. Due to the presence of soft compressible soils underlying the site, traditional fill (soil) placed in the widened area will result in differential settlement between the existing roadway and the widened area.

One dimensional consolidation tests performed on undisturbed tube samples indicate that the soft compressible silt, silty clay and clayey silt deposits at the site are generally over consolidated. This indicates that the soils are compressible and that they are susceptible to consolidation if the in-situ stresses are increased above the maximum past pressures (i.e., consolidation will occur if fill is placed, or if structures are supported on clay). Evaluation of the potential settlement due to the widening of the roadway resulted in approximately ½ to 1.0 inch of consolidation settlement. Additionally, approximately ½ inch of elastic settlement will occur in the coarse-grained soils during construction. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag forces on piles. This settlement is anticipated to occur over a long period of time (on the order of 5 to 20 years).

7.5 Downdrag

Settlement analyses indicate that approximately ½ to 1.0 inch of settlement will occur in the widened embankment areas due to the placement of a maximum of 4.3 feet of fill along the western side of the roadway. Studies indicate that settlements in excess of 0.4 inches in soils where driven piles are present will result in downdrag (negative skin friction) forces on piles. The magnitude of downdrag has been estimated based on the effective vertical stress and empirical β factors obtained from full scale tests.

The calculated downdrag values are:

Pile Section	Strength Limit State Unfactored Downdrag Load (DD) (Kips)
HP 12 x 53	70
HP 14 x 73	82
HP 14 x 89	83
HP 14 x 117	85

Calculations for the pile downdrag loads are included the Appendix C- Calculations. Based on past practice, it is recommended that a load factor, γ_p=1.0, is applied to the downdrag load applied to abutment piles for the strength limit state.

The effects of downdrag can be reduced by coating the pile with soft bitumen. Bitumen coating should only be applied to the portion of the pile which will be embedded in the negative shaft resistance zone. Care should be taken during pile installation to protect the coating. The use of an oversized collar around the pile below the bitumen coating can open an oversized hole in the soil during driving which is adequate to permit passage of the coated pile through the site soils. If the design team chooses to use the bitumen coating a Special Provision will be provided for the Contract Documents.

7.6 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the Modberg Software by the US Army Cold Regions Research and Engineering Laboratory the site has an air design-freezing index of approximately 1224 F-degree days. In a granular soil with a water content of approximately 10%, this air design-freezing index correlates to a frost depth of approximately 5.3 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.3 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.

7.7 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) = 0.088g
- Short-term (0.2-second period) spectral acceleration coefficient = 0.177g
- Long-term (1.0-second period) spectral acceleration coefficient = 0.047g

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 0.163 (LRFD Eq. 3.10.4.2-6).

According to Figure 2-2 of the MaineDOT BDG, the CNR Crossing Bridge on US Route 202/State Routes 4 and 100 is on the National Highway System (NHS) and is therefore considered to be functionally important. Consequently, a detailed seismic analysis is required. The minimum seismic analysis requirements are defined in LFRD Article 4.7.4.3. The designer shall determine the specific analysis method using LRFD Tables 4.7.4.3.1-1 and 4.7.4.3.1-2. Seismic design requirements for Seismic Zone 2 are found in LRFD Article 3.10.9.3.

7.8 Construction Considerations

There is a potential for the existing abutment and pier piles to interfere with the installation of the proposed 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.

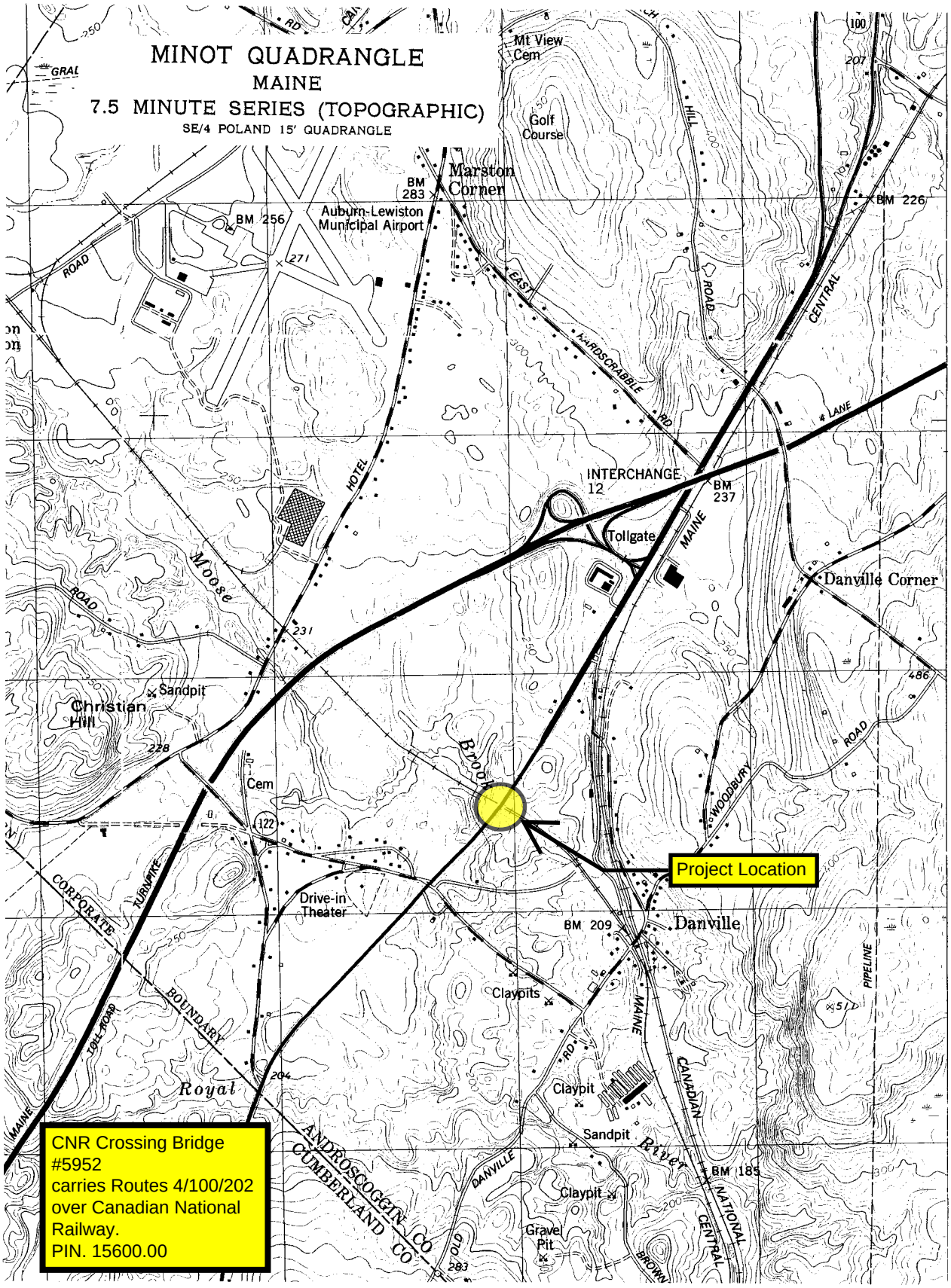
Boulders and cobbles were encountered within the sand layer above the bedrock in borings BB-ACNR-101 and BB-ACNR-102. There is potential for these obstructions to impact the pile installation operations. These impacts include, but are not limited to, driving the piles and cleaning out pipe piles. Obstruction may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program and CLD Consulting Engineers for specific application to the proposed replacement of the CNR Crossing Bridge in Auburn, 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


MINOT QUADRANGLE
MAINE
7.5 MINUTE SERIES (TOPOGRAPHIC)
SE/4 POLAND 15' QUADRANGLE
Mt View Cem
Golf Course
Marston Corner
BM 283
BM 256
Auburn-Lewiston Municipal Airport
BM 226
INTERCHANGE 12
BM 237
Tollgate
Danville Corner
Moose
Christian Hill
Sandpit
Cem
Drive-in Theater
Project Location
Danville
BM 209
Claypits
Claypit
Sandpit
Gravel Pit
BM 185
Royal
CNR Crossing Bridge #5952
carries Routes 4/100/202
over Canadian National
Railway.
PIN. 15600.00
SCALE 1:24 000
1 MILE
7000 FEET
1 KILOMETER
CONTOUR INTERVAL 10 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

L = 441.01'
T = 221.05'
E = 4.88'

FULL DEPTH CONSTRUCTION
-66.00

MAG. 19KX

℄ BRG ABUT 1
STA. 4+53.74

℄ PIER 1
STA. 4+91.86

8°53' 30"SKEW

℄ PIER 2
STA. 5+42.86

℄ BRG ABUT 2
STA. 5+87.74

END FULL DEPTH CONSTRUCTION
CONTINUE SHOULDER WORK
STA. 6+50.00

END PROJECT
END GUARDRAIL
STA. 8+41.00

3+00 4+00 5+00 6+00 7+00 8+00

BB-ACNR-101 BB-ACNR-102 BB-ACNR-102A BB-ACNR-103 BB-ACNR-104

BEGIN PROJECT
BEGIN SHOULDER WORK
STA. 2+42.00

PT = ST

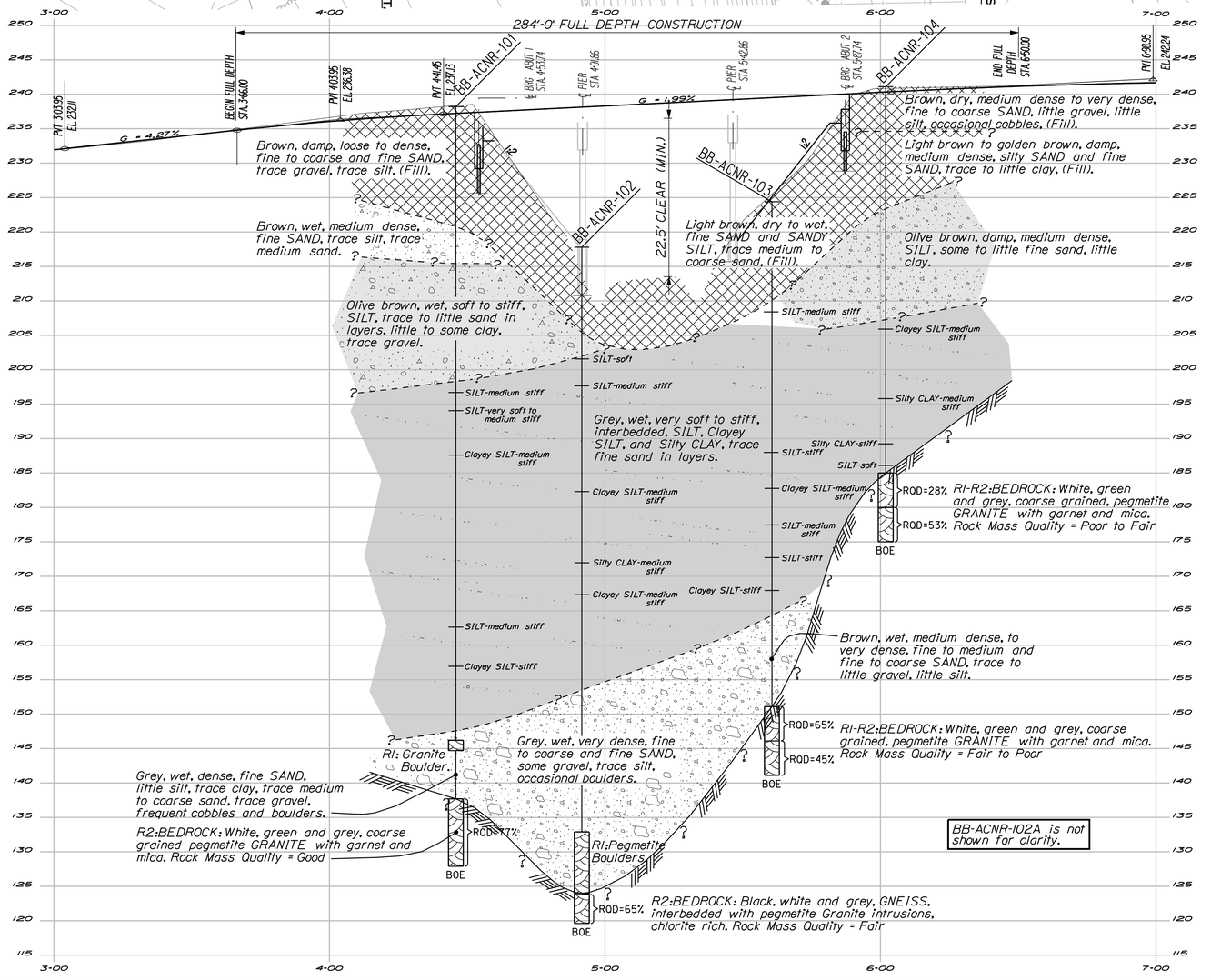
PC = S

END SHOULDER WORK
CONTINUE GUARDRAIL WORK
STA. 7+80

LEGEND
CASED WASH BORING

PLAN

25 0 25 50
Scale of Feet



Boring No.
Offset, if shown
Boring
Strata Interface
ROD= Rock Quality Designation for Rock Core Sample
BOE= Bottom Of Exploration

PROFILE

HORIZ 25 0 25 50
VERT 10 0 10 20
SCALE

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-A560(000)X

PIN 15600.00

BRIDGE PLANS

BRIDGE NO. 5952

	BY	DATE
PROJ. MANAGER	-----	-----
DESIGN-DETAILED	K. MAGUIRE	SEPT 2008
CHECKED-REVIEWED	-----	-----
DESIGN2-DETAILED2	-----	-----
DESIGN3-DETAILED3	-----	-----
REVISIONS 1	-----	-----
REVISIONS 2	-----	-----
REVISIONS 3	-----	-----
REVISIONS 4	-----	-----
FIELD CHANGES	-----	-----

SIGNATURE

P.E. NUMBER

DATE

CNR CROSSING BRIDGE
CANADIAN NATIONAL RAILWAY
AUBURN ANDROSCOGGIN COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

2

OF 5

Filename: ...\msto\008_BORING LOGS2.dgn Division: GEOTECH Username: terry.white Date:10/15/2008

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

Project: CNR Railroad Crossing. Routes 4/100/202
Location: Auburn, Maine
Boring No.: BB-ACNR-102A
PIN: 15600.00

Driller:	Northern Test Boring	Elevation (ft.)	216.6	Auger ID/OD:	N/A
Operator:	Mike/Nick	Datum	NAVD 88	Sampler:	2" Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Diedrick D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/22/08 13:00-14:00	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	4+93.9, 14.3 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.633	Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample — SSA = Solid Stem Auger — S_u = Vane Shear Strength (psf) — $S_{u(rem)}$ = Lab Vane Shear Strength (psf)
ST = Thin Wall Tube Sample — HSA = Hollow Stem Auger — q_u = Unconfined Compressive Strength (ksf) — w = water content, percent
MD = Unconfined Thin Wall Tube Sample attempt — R = Roller Cone — qp = unconfined Compressive Strength (ksf) — LL = Liquid Limit
N = Indiv. Vane Shear Test. — PP = Pocket Penetrometer — WOR = Weight of Rods or casing — N_{60} = SPT N-uncorrected corrected for hammer efficiency — PL = Plasticity Index
MV = Unconfined Indiv. Vane Shear Test attempt — WOH = Weight of one person — N_{60} = (Hammer Efficiency Factor/60%)•N-uncorrected — C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#) or Strength (tsf) or RQD (%)	Uncorrected	N_{60}	Casing	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/6	0.00 - 2.00	1/0/1/1	1	1	6HP			Grass and brush at surface.	GM210275 A-2-4, SP-SM WC=9.6%
							13			Light brown/yellow, damp, loose, fine SAND, little silt, trace gravel, trace medium to coarse sand. (Fill).	
							13			Hydraulic Push	
							17				
							28				
5							45			Casing refusal on top of Pier Pile Cap, abandon hole and moved 2.5' South to BB-ACNR-102.	
								210.60		Bottom of Exploration at 6.00 feet below ground surface.	
10											
15											
20											
25											

Remarks:
Hammer #283
0.7' Concrete Deck thickness.
23.2' from top of Bridge Deck to Ground Surface.

Stratification lines represent approximate boundaries between soil typest transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-ACNR-102A

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

Project: CNR Railroad Crossing. Routes 4/100/202
Location: Auburn, Maine
Boring No.: BB-ACNR-103
PIN: 15600.00

Driller:	MaineDOT	Elevation (ft.)	224.4	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum	NAVD 88	Sampler:	2" Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample — SSA = Solid Stem Auger — S_u = Indiv Field Vane Shear Strength (tsf) — $S_{u(rem)}$ = Lab Vane Shear Strength (psf)
ST = Thin Wall Tube Sample — HSA = Hollow Stem Auger — S_{ur} = Pocket Torvane Shear Strength (tsf) — w = water content, percent
MD = Unsuccessful Split Spoon Sample attempt — R = Roller Cone — q_u = Unconfined Compressive Strength (ksf) — LL = Liquid Limit
MD = Unsuccessful Thin Wall Tube Sample attempt — WOH = Weight of 140lb. hammer — q_p = Unconfined Compressive Strength Incorrected w Pocket Pen (tsf) — PL = Plastic Limit
N = Indiv Vane Shear Test. — PP = Pocket Penetrometer — WOR = Weight of Rods or casing — Hammer Efficiency Factor = Annual Calibration Value — PI = Plasticity Index
MV = Unsuccessful Indiv Vane Shear Test attempt — WOH = Weight of one person — N_{60} = SPT N-uncorrected corrected for hammer efficiency — C = Grain Size Analysis
N_{60} = (Hammer Efficiency Factor/60%)•N-uncorrected — C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#) or Strength (tsf) or RQD (%)	Uncorrected	N_{60}	Casing	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/24	0.00 - 2.00	1/1/1/1	2	3	6			Light brown, dry, very loose, fine SAND with roots and wood. (Fill).	
							7				
							5				
							3				
5							5			Light brown, wet, loose- fine Sandy SILT, some silt, trace medium to coarse sand, trace gravel, trace clay. (Fill),	GM210603 A-4, ML WC=25.8%
	2D	24/12	5.00 - 7.00	1/2/2/1	4	5	8				
							2				
							2				
							3				
10							2				
	3D	24/14	10.00 - 12.00	WOR/WOH/WOH/WOR	---		3				
							7				
							10				
							6				
							3	210.40			
15										14.00	
	1U	24/24	15.00 - 17.00	WOR/WOR			WOC			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G:CM210627 A-4, CL-ML WC=34.3% LL=28 PL=21 PI=7
							WOC				
	V1		17.52 - 17.95	Su=632/110 psf			WOH			65x130 mm vane raw torque readings: V1: 23.0/4.0 ft-lbs	
	V2		18.52 - 18.95	Su=632/82 psf			WOH			V2: 23.0/3.0 ft-lbs	
20							WOH				
	4D	24/24	20.50 - 22.50	push thru vane	---		WOC			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	GM210604 A-6, CL WC=33.2% LL=36 PL=22 PI=14
	V3		21.07 - 21.50	Su=659/110 psf			WOC			65x130 mm vane raw torque readings: V3: 24.0/4.0 ft-lbs	
	V4		22.07 - 22.50	Su=522/82 psf			WOH			V4: 19.0/3.0 ft-lbs	
							WOH				
25							WOH				
	2U	24/24	25.00 - 27.00	WOR/WOR			WOH			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G:CM210628 A-4, ML WC=29.0% LL=27 PL=23 PI=4
							WOH				
							14				
	V5		28.57 - 29.00	Su=522/110 psf			22			65x130 mm vane raw torque readings: V5: 19.0/4.0 ft-lbs	
30	V6		29.57 - 30.00	Su=577/137 psf			19			V6: 21.0/5.0 ft-lbs	
	5D	24/18	30.50 - 32.50	push thru vane	---		14			Similar to above.	
	V7		31.07 - 31.50	Su=838/165 psf			19			65x130 mm vane raw torque readings: V7: 30.5/6.0 ft-lbs	
	V8		32.07 - 32.50	Su=879/192 psf			21			V8: 32.0/7.0 ft-lbs	
							21				
							22				
35	3U	24/24	35.00 - 37.00	WOR/HydraulicPush			14			Grey, wet, stiff, SILT, some clay, trace fine sand.	G:CM210629 A-6, CL WC=34.3% LL=35 PL=21 PI=14
							21				
	V9		37.57 - 38.00	Su=1044/220 psf			27			65x130 mm vane raw torque readings: V9: 38.0/8.0 ft-lbs	
	V10		38.57 - 39.00	Su=1099/247 psf			29			V10: 40.0/9.0 ft-lbs	
							30				
40	6D	24/24	40.50 - 42.50	push thru vane	---		12			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	GM210605 A-6, CL WC=36.8% LL=36 PL=22 PI=14
	V11		41.13 - 41.50	Su=893/156 psf			22			65x110 mm vane raw torque readings: V11: 20.0/3.5 ft-lbs	
	V12		42.13 - 42.50	Su=893/156 psf			26			V12: 20.0/3.5 ft-lbs	
							26				
							24				
45	4U	24/24	45.00 - 47.00	WOR/WOR			13			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G:CM210630 A-6, ML WC=40.8% LL=37 PL=27 PI=10
							18				
	V13		47.63 - 48.00	Su=871/134 psf			20			65x110 mm vane raw torque readings: V13: 19.5/3.0 ft-lbs	
	V14		48.63 - 49.00	Su=982/223 psf			19			V14: 22.0/5.0 ft-lbs	
50							21				

Remarks:
0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

Stratification lines represent approximate boundaries between soil typest transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 2
Boring No.: BB-ACNR-103

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

Project: CNR Railroad Crossing. Routes 4/100/202
Location: Auburn, Maine
Boring No.: BB-ACNR-103
PIN: 15600.00

Driller:	MaineDOT	Elevation (ft.)	224.4	Auger ID/OD:	N/A
Operator:	E. Giguere/C. Giles	Datum	NAVD 88	Sampler:	2" Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample — SSA = Solid Stem Auger — S_u = Indiv Field Vane Shear Strength (tsf) — $S_{u(rem)}$ = Lab Vane Shear Strength (psf)
ST = Thin Wall Tube Sample — HSA = Hollow Stem Auger — S_{ur} = Pocket Torvane Shear Strength (tsf) — w = water content, percent
MD = Unsuccessful Split Spoon Sample attempt — R = Roller Cone — q_u = Unconfined Compressive Strength (ksf) — LL = Liquid Limit
MD = Unsuccessful Thin Wall Tube Sample attempt — WOH = Weight of 140lb. hammer — q_p = Unconfined Compressive Strength Incorrected w Pocket Pen (tsf) — PL = Plastic Limit
N = Indiv Vane Shear Test. — PP = Pocket Penetrometer — WOR = Weight of Rods or casing — Hammer Efficiency Factor = Annual Calibration Value — PI = Plasticity Index
MV = Unsuccessful Indiv Vane Shear Test attempt — WOH = Weight of one person — N_{60} = SPT N-uncorrected corrected for hammer efficiency — C = Grain Size Analysis
N_{60} = (Hammer Efficiency Factor/60%)•N-uncorrected — C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#) or Strength (tsf) or RQD (%)	Uncorrected	N_{60}	Casing	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
50	T0	24/24	50.50 - 52.50	push thru vane	---		18			Grey, wet, stiff, SILT, trace fine sand.	
	V15		51.13 - 51.50	Su=1161/112 psf			21			S5x110 mm vane raw torque readings: V15: 26.0/2.5 ft-lbs	
	V16		52.13 - 52.50	Su=1250/201 psf			24			V16: 28.0/4.5 ft-lbs	
							24				
							23				
55	5U	24/24	55.00 - 57.00	WOR/WOR			21			Gret, wet, stiff, Clayey SILT, trace fine sand.	G:CM210631 A-6, CL WC=40.2% LL=35 PL=23 PI=12
							26				
	V17		57.63 - 58.00	Su=1473/179 psf			26			S5x110 mm vane raw torque readings: V17: 33.0/4.0 ft-lbs	
	V18		58.63 - 59.00	Su=1384/223 psf			26			V18: 31.0/5.0 ft-lbs	
							48	164.90			
60	8D	24/16	60.00 - 62.00	7/6/7/14	13	17	48			59.50 Brown, wet, medium dense, fine to medium SAND, little silt, trace coarse sand, trace gravel.	GM210606 A-2-4, SM WC=22.5%
							62				
							77				
							77				
65							82				
	9D	24/17	65.00 - 67.00	4/5/5/10	10	13	64				
							93				
							120				
							174				
70							246				
	10D	24/16	70.00 - 72.00	26/42/40/55	82	105	74			Brown, wet, very dense, fine to coarse SAND, little gravel, little silt.	G:CM210607 A-2-4, SM WC=13.0%
							127				
							175				
	R1	60/59	73.30 - 78.30	ROD = 65%			RUN #1 CORE	151.10		0125 blows for 0.3'. 73.30	
75										Top of Bedrock at Elev. 151.1' Bedrock: White, green and grey, coarse grained, pegmatite GRANITE, with garnet and mica. no bedding. Rock Mass Quality = Fair Rit:Core Times (minsec) 73.3-74.3' (1T41) 74.3-75.3' (1T06) 75.3-76.3' (1A35) 76.3-77.3' (1A41) 77.3-78.3' (1A52) 98% Recovery	
	R2	60/58	78.30 - 83.30	ROD = 45%						R2: Rock Quality = Poor Core Times (minsec) 78.3-79.3' (1S14) 79.3-80.3' (1S08) 80.3-81.3' (1S24) 81.3-82.3' (1S29) 82.3-83.3' (1S00) 96% Recovery	
80											
								141.10		83.30 Bottom of Exploration at 83.30 feet below ground surface.	
85											
90											
95											
100											

Remarks:
0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

Stratification lines represent approximate boundaries between soil typest transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 2 of 2
Boring No.: BB-ACNR-103

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

Project: CNR Railroad Crossing. Routes 4/100/202
Location: Auburn, Maine
Boring No.: BB-ACNR-104
PIN: 15600.00

Driller:	MaineDOT	Elevation (ft.)	241.1	Auger ID/OD:	5" Solid Stem
Operator:	E. Giguere/C. Giles	Datum	NAVD 88	Sampler:	2" Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/21/08, 5/27/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	6+01.9, 11.7 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor:	0.77	Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample — SSA = Solid Stem Auger — S_u = Indiv Field Vane Shear Strength (tsf) — $S_{u(rem)}$ = Lab Vane Shear Strength (psf)
ST = Thin Wall Tube Sample — HSA = Hollow Stem Auger — S_{ur} = Pocket Torvane Shear Strength (tsf) — w = water content, percent
MD = Unsuccessful Split Spoon Sample attempt — R = Roller Cone — q_u = Unconfined Compressive Strength (ksf) — LL = Liquid Limit
MD = Unsuccessful Thin Wall Tube Sample attempt — WOH = Weight of 140lb. hammer — q_p = Unconfined Compressive Strength Incorrected w Pocket Pen (tsf) — PL = Plastic Limit
N = Indiv Vane Shear Test. — PP = Pocket Penetrometer — WOR = Weight of Rods or casing — Hammer Efficiency Factor = Annual Calibration Value — PI = Plasticity Index
MV = Unsuccessful Indiv Vane Shear Test attempt — WOH = Weight of one person — N_{60} = SPT N-uncorrected corrected for hammer efficiency — C = Grain Size Analysis
N_{60} = (Hammer Efficiency Factor/60%)•N-uncorrected — C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (#) or Strength (tsf) or RQD (%)	Uncorrected	N_{60}	Casing	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							S6A	240.60		Pavement 0.50	
	1D	13.2/13.2	1.00 - 2.10	10/25/20(1.2")	---					Brown, dry, very dense, fine to coarse SAND, little gravel, little silt, occasional cobbles. (Fill),	
										Boulder from 2.1-3.2' bgs.	
5											
	2D/4I	24/18	5.00 - 7.00	4/6/5/4	11	14		234.60		(2D/4) 5.0-6.5'. Similar to above, medium dense.	GM210608 A-1-b, SM WC=14.1%
										6.50	GM210609
										(2D) 6.5-7.0' bgs. Light brown, damp, Sandy SILT, little clay. (Fill).	A-4, ML WC=20.9%
10								232.10		9.00	
	3D	24/19	10.00 - 12.00	15/9/9/16	18	23				Golden brown, damp, medium dense, fine SAND, trace silt, trace medium to coarse sand. (Fill),	
15											
	4D	24/20	15.00 - 17.00	4/7/9/9	16	21	143			Golden brown, damp, medium dense, fine SAND, trace silt, trace medium to coarse sand. (Fill),	GM210610 A-3, SP-SM WC=8.6%
							157				
							123	223.10		18.00	
20							19				
	5D	24/20	20.50 - 22.50	2/4/8/7	12	15	64			Olive-brown, moist, medium dense, SILT, some fine sand, little clay, trace roots.	GM210611 A-4, ML WC=21.0%
							81				
							88				
							87				
	6D	24/22	24.00 - 26.00	7/6/7/6	13	17	57				GM210612 A-4, ML WC=25.5%
25							67			Olive-brown, wet, medium dense, SILT, little sand, little clay.	
							68				
							67				
							66				
	7D/WW	24/24	29.50 - 31.50	3/WOH/WOH/1	---		53			Failed S5x110 mm vane attempt, could not push. Olive, wet, very soft, SILT, little clay, trace fine sand.	GM210613 A-4, ML WC=30.7% LL=25 PL=22 PI=3
30							59				
							61				
							55				
							54	207.70		33.40	
	1U	24/24	34.00 - 36.00	WOR/Hyd Push			46			Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand.	G:CM210632 A-4, CL WC=36.7% LL=30 PL=22 PI=8
35							55				
	V1		36.63 - 37.00	Su=513/89 psf			59			S5x110 mm vane raw torque readings: V1: 11.5/2.0 ft-lbs	
	V2		37.63 - 38.00	Su=491/89 psf			54			V2: 11.0/2.0 ft-lbs	
							59				
							54				
40	8D	24/24	40.50 - 42.50	push thru vane	---		70			Similar to above, medium stiff.	GM210614 A-6, CL WC=31.9% LL=31 PL=19 PI=12
	V3		41.13 - 41.50	Su=625/89 psf			91			S5x110 mm vane raw torque readings: V3: 14.0/2.0 ft-lbs	
	V4		42.13 - 42.50	Su=737/134 psf			79			V4: 16.5/3.0 ft-lbs	
							73				
	2U	24/24	44.00 - 46.00	WOR/HydraulicPus			62			Grey, wet, stiff, Silty CLAY, trace fine sand.	G:CM210633 A-6, CL WC=36.9% LL=36 PL=24 PI=12
45							73				
	V5		46.63 - 47.00	Su=1049/223 psf			79			S5x110 mm vane raw torque readings: V5: 23.5/5.0 ft-lbs	
	V6		47.63 - 48.00	Su=1071/223 psf			77			V6: 24.0/5.0 ft-lbs	
							73				
	9D	24/24	49.50 - 51.50	push thru vane	---		66			Similar to above, stiff.	GM210615 A-6, CL WC=40.9% LL=39 PL=25 PI=14
	V7		50.13 - 50.50	Su=1027/201 psf			67			S5x110 mm vane raw torque readings: V7: 23.0/4.5 ft-lbs	
50	V8		51.13 - 51.50	Su=1161/268 psf			67			V8: 26.0/6.0 ft-lbs	
							73				
							72	187.10		54.00	
	3U	12/12	54.00 - 55.00	Hydraulic Push			53			Grey, wet, soft, SILT, some clay, trace sand, trace gravel.	G:CM210634 A-4, CL-ML WC=29.5% LL=25 PL=20 PI=5
55							200				
	R1	60/60	56.10 - 61.10	ROD = 28%			NQ CORE	185.00		Roller Coned ahead to 56.1' bgs. 86.10 Top of Bedrock at Elev. 185.0' Bedrock: White, green and grey, coarse grained, pegmatite GRANITE, with garnet and mica. Rock Mass Quality = Poor R1: Core Times (minsec) 56.1-57.1' (1T28) 57.1-58.1' (1A42) 58.1-59.1' (1A43) 59.1-60.1' (1A19) 60.1-61.1' (1I0132) 100% Recovery	
60											
	R2	60/60	61.10 - 66.10	ROD = 53%						Rock Mass Quality = Fair R2: Core Times (minsec) 61.1-62.1' (1T08) 62.1-63.1' (1A28) 63.1-64.1' (1T48) 64.1-65.1' (1B00) 65.1-66.1' (1B15) 100% Recovery	
65								175.00			
70										66.10 Bottom of Exploration at 66.10 feet below ground surface.	

Remarks:

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Page 1 of 1
Boring No.: BB-ACNR-104

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BR-A560(000)X

PIN 15600.00

BRIDGE NO. 5962

BRIDGE PLANS

PROJ. MANAGER			
DESIGN-DETAILED	K. MAGUIRE	BY	DATE
CHECKED-REVIEWED			
DESIGN2-DETAILED2	T. WHITE	SIGNATURE	SEPT 2008
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2		P.E. NUMBER	DATE
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

CNR CROSSING BRIDGE
CANADIAN NATIONAL RAILWAY
AUBURN ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER
4
OF 5

Filename: ...\msta\007_BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:10/15/2008

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

Boring No.: BB-ACNR-101 — PIN: 15600.00

Project:	CNR Railroad Crossing, Routes 4/100/202	Location:	Auburn, Maine		
Driller:	Northern Test Boring	Elevation (ft.)	238.2	Auger ID/OD:	5" Solid Stem
Operator:	Mike/Nick	Datum:	NAVD 88	Sampler:	24" Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Diedrick D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.633		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Columns: Depth (ft.) | Sample Information: Sample No., Pen./Rec. (in.), Sample Depth (ft.), Blows (/6 in.) Shear Strength (psf) or RQD (%), N-uncorrected, N60 | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/AASHTO and Unified Class

Elev. / Depth	Visual Description and Remarks	Laboratory Testing Results
237.30 / 0.90	Pavement	
233.20 / 5.00	Brown, damp, dense, fine to coarse SAND, trace gravel, trace silt, (Fill).	
	Brown, damp, loose, fine SAND, trace silt, trace medium sand, (Fill).	G#209920 A-3, SP-SM WC=4.4%
	Similar to 2D, medium dense.	
221.20 / 17.00	Brown, damp, dense, fine SAND, trace silt, (Fill).	G#209921 A-3, SP-SM WC=11.3%
215.40 / 22.80	Light brown to dark brown, wet, medium dense, fine SAND with iron staining, little silt, trace medium sand.	G#209922 A-2-4, SM WC=19.7%
	Olive brown, wet, stiff, SILT, trace sand in layers, trace gravel. ªWashed ahead of Casing.	G#209923 A-4, ML WC=30.1%
	24.5x50.8 mm vane raw torque readings: V1:624/24 in-lbs	
	Failed 24.5x50.8 mm vane attempt.	
	(7D/A) 30.0-31.5' bgs. Brown/olive, wet, stiff, SILT, some clay, trace sand in layers. 55x110 mm vane raw torque readings: V2: 29.5/8.0 ft-lbs	G#209924 A-4, CL-ML WC=33.1%
	(7D/B) 31.5-32.0' bgs. Brown, wet, SILT, some sand with iron staining, trace clay. Failed 55x110 mm vane attempt.	G#209925 A-4, ML WC=25.3%
198.70 / 39.50	Failed Tube sample with piston sampler, no recovery. Brown, wet, soft, SILT, little clay, little sand in layers.	G#210269 A-4, ML WC=26.9%
	Grey, wet, medium stiff, SILT, some clay, trace fine sand in layers. ªPiston sampler had sand in it, making it difficult to release. tube dropped when taken off sampler.	
	55x110 mm vane raw torque readings: V3: 12.0/2.4 ft-lbs V4: 14.2/3.5 ft-lbs	
	Grey, wet, very soft to medium stiff, SILT, some clay, trace fine sand in layers.	G,C#210617 A-4, ML WC=30.6% LL=22 PL=19 PI=3
	55x110 mm vane raw torque readings: V5: 3.5/2.0 ft-lbs V6: 16.2/3.0 ft-lbs	
	Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V7: 14.0/3.0 ft-lbs V8: 14.0/3.5 ft-lbs	G#210270 A-4, CL-ML WC=32.4% LL=28 PL=22 PI=6
	Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,C#210618 A-6, CL WC=32.8% LL=30 PL=19 PI=11
	55x110 mm vane raw torque readings: V9: 14.0/3.3 ft-lbs V10: 16.5/3.8 ft-lbs	
	Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V11: 16.5/3.8 ft-lbs V12: 19.0/4.3 ft-lbs	G#210271 A-4, CL WC=30.9% LL=30 PL=22 PI=8
	Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,C#210619 A-6, CL WC=38.0% LL=35 PL=24 PI=11
	55x110 mm vane raw torque readings: V13: 13.5/1.0 ft-lbs V14: 13.2/1.0 ft-lbs	
	Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V15: 15.1/0.5 ft-lbs V16: 12.6/0.9 ft-lbs	G#210272 A-6, CL WC=34.4% LL=31 PL=12 PI=19

Remarks: Hammer #283

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 2 — Boring No.: BB-ACNR-101

Boring No.: BB-ACNR-101 — PIN: 15600.00 (continued)

Elev. / Depth	Visual Description and Remarks	Laboratory Testing Results
	Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210620 A-4, ML WC=26.4% LL=22 PL=20 PI=2
	55x110 mm vane raw torque readings: V17: 14.0/1.5 ft-lbs V18: 13.8/1.2 ft-lbs	
	Grey, wet, stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V19: 28.8/3.1 ft-lbs V20: ₵30.0/4.3 ft-lbs	
	Grey, wet, stiff, Clayey SILT, trace fine sand. Lost bottom 6" of tube, fine sand seam.	G,C#210621 A-4, CL WC=35.2% LL=31 PL=22 PI=9
	Failed 55x110 mm vane attempt.	
148.20 / 90.00	Grey, wet, dense, fine SAND, little silt, trace clay, trace medium to coarse sand, trace gravel. Refusal at 90.35' bgs. Roller Coned ahead to 92.0' bgs.	G#210273 A-2-4, SC-SM WC=21.8%
146.20 / 92.00	R1:White, grey with garnets, Granite pegmetite BOULDER. R1:Core Times (min:sec) 92.0-93.0' (1:45) 93.0-93.5' (0:36) 53% Recovery	
144.70 / 93.50	Roller Coned ahead from 93.5-95.0' bgs.	
	Grey, wet, fine to coarse SAND, trace silt, trace gravel, broken rock in nose of spoon. Roller Coned ahead from 95.0-100.5' bgs.	G#210274 A-2-4, SP-SM WC=18.6%
	Sand with frequent cobbles and boulders. Casing shoe bent, roller cone thru shoe. 60 blows/in movement on casing.	
137.70 / 100.50	Top of Bedrock at Elev. 137.7' Bedrock: White, green and grey, coarse grained pegmetite GRANITE with garnet and mica. Rock Mass Quality = Good R2:Core Times (min:sec) 100.5-101.5' (5:01) 101.5-102.5' (5:01) 102.5-103.5' (4:39) 103.5-104.5' (5:40) 104.5-105.5' (5:06) 105.5-106.5' (6:09) 106.5-107.5' (6:20) 107.5-108.5' (4:58) 108.5-109.5' (5:07) 109.5-110.3' (4:52) 100% Recovery	
127.90 / 110.30	Bottom of Exploration at 110.30 feet below ground surface.	

Remarks: Hammer #283

Page 2 of 2 — Boring No.: BB-ACNR-101

Boring No.: BB-ACNR-102 — PIN: 15600.00

Project:	CNR Railroad Crossing, Routes 4/100/202	Location:	Auburn, Maine		
Driller:	Northern Test Boring	Elevation (ft.)	217.8	Auger ID/OD:	N/A
Operator:	Mike/Nick	Datum:	NAVD 88	Sampler:	24" Standard Split Spoon
Logged By:	K. Maguire	Rig Type:	Diedrick D50	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	5/22/08, 5/29/08	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"x10'
Boring Location:	4+91.6, 14.3 Rt.	Casing ID/OD:	HW	Water Level*:	None Observed
Hammer Efficiency Factor: 0.633		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Elev. / Depth	Visual Description and Remarks	Laboratory Testing Results
217.50 / 0.30	Grass at Ground Surface.	
	Brown, damp, loose, fine to coarse SAND, trace gravel, broken rock, 0.2' decomposed wood layer at top of spoon, (Fill).	
	Brown, damp, very loose, fine SAND, some silt, little clay in layers, trace medium to coarse sand, trace gravel, (Fill). ªHydraulic Push	G#210294 A-4, SC-SM WC=28.6%
205.80 / 12.00	Brown, wet, stiff, SILT, some clay, trace fine sand.	
202.80 / 15.00	(3D/A) 14.0-15.0' 55x110 mm vane raw torque readings: V1: 25.0/3.0 ft-lbs	G#210296 A-4, CL-ML WC=32.9% LL=27 PL=22 PI=5
	(3D/B)15.0-16.0' Grey, wet, soft, SILT, some clay, trace fine sand in layers. 55x110 mm vane raw torque readings: V2: 8.1/1.5 ft-lbs	G#210297 A-4, CL-ML WC=33.5% LL=25 PL=20 PI=5
	Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210622 A-4, CL-ML WC=30.2% Non-Plastic
	55x110 mm vane raw torque readings: V3: 15.1/5.2 ft-lbs V4: 14.1/2.0 ft-lbs	
	Grey, wet, medium stiff, SILT, some clay, trace fine sand. 55x110 mm vane raw torque readings: V5: 12.2/2.2 ft-lbs V6: 14.4/3.6 ft-lbs	G#210295 A-4, CL-ML WC=31.5% LL=27 PL=20 PI=7
	Grey, wet, medium stiff, SILT, some clay, trace fine sand in layers.	G,C#210623 A-6, CL WC=29.8% LL=34 PL=23 PI=11
	55x110 mm vane raw torque readings: V7: 8.6/2.3 ft-lbs V8: 11.8/2.1 ft-lbs	
	55x110 mm vane raw torque readings: V9: 13.8/3.0 ft-lbs Grey, wet, medium stiff, Clayey SILT, trace fine sand. V10: 16.8/3.5 ft-lbs	
	Failed Tube attempt, tube empty, took spoon sample. Grey, wet, medium stiff, Clayey SILT, with black staining, trace fine sand. Washed ahead 2.0' to 41.0' bgs, took tube sample 3U.	G#210298 A-4, ML WC=35.4% LL=35 PL=27 PI=8
	Grey, wet, medium stiff, silty CLAY with black staining, trace fine sand.	G,C#210624 A-4, CL WC=38.4% LL=27 PL=19 PI=8
	Grey, wet, medium stiff, silty CLAY with black staining, trace fine sand. 55x110 mm vane raw torque readings: V11: 15.0/1.5 ft-lbs V12: 16.0/1.7 ft-lbs	
	Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,#210625 A-4, CL-ML

Remarks: Hammer #283 0.7' Concrete Deck thickness. 20.9' from top of Bridge Deck to Ground Surface.

Page 1 of 2 — Boring No.: BB-ACNR-102

Boring No.: BB-ACNR-102 — PIN: 15600.00 (continued)

Elev. / Depth	Visual Description and Remarks	Laboratory Testing Results
		WC=30.7% LL=26 PL=19 PI=7
	55x110 mm vane raw torque readings: V13: 15.0/1.9 ft-lbs V14: 14.1/1.2 ft-lbs	
	Grey, wet, medium stiff, Clayey SILT, trace fine sand. 55x110 mm vane raw torque readings: V15: 13.0/1.8 ft-lbs V16: 19.0/2.0 ft-lbs	G#210299 A-4, CL WC=30.6% LL=29 PL=20 PI=9
	Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand.	G,C#210626 A-6, CL WC=36.8% LL=33 PL=22 PI=11
	55x110 mm vane raw torque readings: V17: 16.2/3.4 ft-lbs V18: 23.4/4.0 ft-lbs	
153.80 / 64.00	Grey, wet, dense, fine SAND, little silt, trace medium to coarse sand, uniform. ᵇ109 blows for 0.7'.	G#210300 A-2-4, SM WC=20.3%
	Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, with broken rock. ᶜ257 blows for 0.8'.	G#210601 A-1-b, SW-SM WC=10.4%
	Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles.	
	ᵈ765 blows for 0.8'. Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles, (Till). ᵉWashed ahead to 84.9' bgs.	
132.90 / 84.90	Grey, wet, very dense, fine to coarse SAND, little gravel, little silt, (Till).	G#210602 A-2-4, SP-SM WC=12.2%
129.90	R1: Pegmetite boulders with iron staining over Gneiss boulders with fine to coarse silty sand layers. R1:Core Times: (min:sec) 84.9-85.9' (2:08) 85.9-86.9' (2:00) 86.9-87.9' (4:03) 87.9-88.9' (3:46) 88.9-89.9' (3:40) 89.9-90.9' (3:15) 90.9-91.9' (4:31) 91.9-92.9' (5:04) 92.9-93.9' (5:45) 93.9-94.2' (3:11) 46% Recovery Core Blocked	
87.90	Grey, wet very dense, fine to coarse SAND, little gravel, little silt.	
123.80 / 94.00	Top of Bedrock at Elev. 123.8'. Bedrock: Black, white and grey, GNEISS interbedded with pegmetite intrusions, chlorite rich. Rock Mass Quality = Fair R2:Core Times (min:sec) 94.2-95.2' (3:32) 95.2-96.2' (3:35) 96.2-97.2' (3:15) 97.2-98.2' (2:16) 100% recovery Core Blocked	
119.60 / 98.20	Bottom of Exploration at 98.20 feet below ground surface.	

Remarks: Hammer #283 0.7' Concrete Deck thickness. 20.9' from top of Bridge Deck to Ground Surface.

Page 2 of 2 — Boring No.: BB-ACNR-102

STATE OF MAINE DEPARTMENT OF TRANSPORTATION

BR-A560(000)X

BRIDGE NO. 5952 — PIN 15600.00 — BRIDGE PLANS

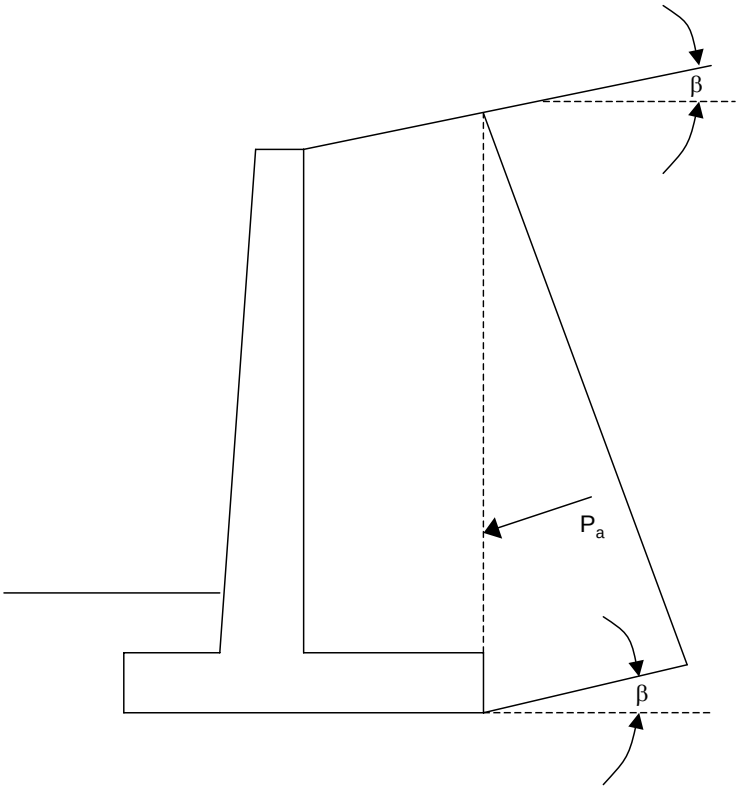
		BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	K. MAGUIRE	T. WHITE	SEPT 2008
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE — P.E. NUMBER — DATE

CNR CROSSING BRIDGE
CANADIAN NATIONAL RAILWAY
AUBURN ANDROSCOGGIN COUNTY

BORING LOGS

SHEET NUMBER 3 OF 5



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

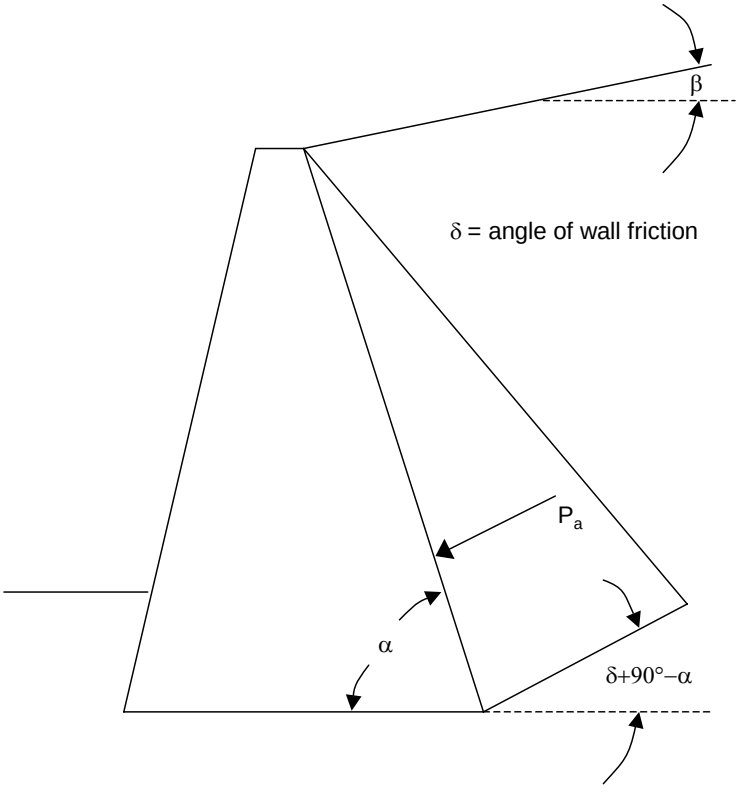
For a horizontal backfill surface, $\beta = 0°$:

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

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

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

P_a is oriented at β



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 $\delta + 90° - \alpha$

Rankine and Coulomb Active Earth Pressure Coefficients

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

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

January 2008

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-101
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	238.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	237.30		Pavement	
	1D	15.6/14	1.00 - 2.30	34/27/50(3.6")	---					0.90 Brown, damp, dense, fine to coarse SAND, trace gravel, trace silt, (Fill).	
5	2D	24/19	5.00 - 7.00	3/3/5/4	8	8		233.20		5.00 Brown, damp, loose, fine SAND, trace silt, trace medium sand, (Fill).	G#209920 A-3, SP-SM WC=4.4%
10	3D	24/20	10.00 - 12.00	5/5/6/8	11	12	60			Similar to 2D, medium dense.	
							102				
							149				
							177				
15							150				
	4D	24/15	15.00 - 17.00	14/17/19/21	36	38	65			Brown, damp, dense, fine SAND, trace silt, (Fill).	G#209921 A-3, SP-SM WC=11.3%
							118	221.20		17.00	
							140				
							149				
							120				
20	5D	24/15	20.00 - 22.00	6/6/7/7	13	14	62			Light brown to dark brown, wet, medium dense, fine SAND with iron staining, little silt, trace medium sand.	G#209922 A-2-4, SM WC=19.7%
							68				
							74	215.40		22.80	
							82				
25							105				

Remarks:

Hammer #283

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-101
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	238.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/24	25.00 - 27.00	4/5/6/6	11	12	[a]48			Olive brown, wet, stiff, SILT, trace sand in layers, trace gravel. [a]Washed ahead of Casing.	G#209923 A-4, ML WC=30.1%
							[a]48				
	V1		27.33 - 27.50	Su=>1045/943 psf			62			24.5x50.8 mm vane raw torque readings: V1:624/24 in-lbs	
	MV			Could not push			67			Failed 24.5x50.8 mm vane attempt.	
							61				
30	7D	24/24	30.00 - 32.00	-/-/5/6	---		51			(7D/A) 30.0-31.5' bgs. Brown/olive, wet, stiff, SILT, some clay, trace sand in layers.	G#209924 A-4, CL-ML WC=33.1%
	V2		30.63 - 31.00	Su=1317/357 psf						55x110 mm vane raw torque readings: V2: 29.5/8.0 ft-lbs	
	MV			Could not push			58			(7D/B) 31.5-32.0' bgs. Brown, wet, SILT, some sand with iron staining, trace clay.	G#209925 A-4, ML WC=25.3%
							70			Failed 55x110 mm vane attempt.	
							68				
							73				
35	8D/MU	24/22	35.00 - 37.00	3/2/2/2	4	4	56			Failed Tube sample with piston sampler, no recovery. Brown, wet, soft, SILT, little clay, little sand in layers.	G#210269 A-4, ML WC=26.9%
							65				
							68				
							78				
							70	198.70		39.50	
40	[a]1U	24/24	40.00 - 42.00	Piston Sampler	---		58			Grey, wet, medium stiff, SILT, some clay, trace fine sand in layers. [a]Piston sampler had sand in it, making it difficult to release, tube dropped when taken off sampler.	
							60				
	V3		42.63 - 43.00	Su=536/107 psf			55			55x110 mm vane raw torque readings: V3: 12.0/2.4 ft-lbs	
	V4		43.63 - 44.00	Su=634/156 psf			65			V4: 14.2/3.5 ft-lbs	
							53				
45	2U	24/24	45.00 - 47.00	Piston Sampler	---		69			Grey, wet, very soft to medium stiff, SILT, some clay, trace fine sand in layers.	G,C#210617 A-4, ML WC=30.6% LL=22 PL=19 PI=3
							62				
	V5		47.63 - 48.00	Su=156/89 psf			69			55x110 mm vane raw torque readings: V5: 3.5/2.0 ft-lbs	
	V6		48.63 - 49.00	Su=723/134 psf			65			V6: 16.2/3.0 ft-lbs	
50							56				

Remarks:

Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-101

PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	238.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	9D	24/24	50.00 - 52.00	sample thru vane	---		58			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#210270
	~~V7~~		~~50.63 - 51.00~~	~~Su=625/134 psf~~						55x110 mm vane raw torque readings:	A-4, CL-ML
	V8		51.63 - 52.00	Su=625/156 psf			57			V7: 14.0/3.0 ft-lbs	WC=32.4%
							56			V8: 14.0/3.5 ft-lbs	LL=28
							57				PL=22
							57				PI=6
55	3U	24/24	55.00 - 57.00	Piston Sampler	---		68			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,C#210618
							62				A-6, CL
	V9		57.63 - 58.00	Su=625/147 psf			70				WC=32.8%
	V10		58.63 - 59.00	Su=737/170 psf			69			55x110 mm vane raw torque readings:	LL=30
							86			V9: 14.0/3.3 ft-lbs	PL=19
60	10D	24/24	60.00 - 62.00	sample thru vane	---		83			V10: 16.5/3.8 ft-lbs	PI=11
	~~V11~~		~~60.63 - 61.00~~	~~Su=737/170 psf~~						Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#210271
	V12		61.63 - 62.00	Su=848/192 psf			91			55x110 mm vane raw torque readings:	A-4, CL
							105			V11: 16.5/3.8 ft-lbs	WC=30.9%
							108			V12: 19.0/4.3 ft-lbs	LL=30
							108				PL=22
65	4U	24/24	65.00 - 67.00	Piston Sampler	---		101				PI=8
							95			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,C#210619
	V13		67.63 - 68.00	Su=603/45 psf			97				A-6, CL
	V14		68.63 - 69.00	Su=589/45 psf			94			55x110 mm vane raw torque readings:	WC=38.0%
							97			V13: 13.5/1.0 ft-lbs	LL=35
70	11D	24/24	70.00 - 72.00	sample thru vane	---		120			V14: 13.2/1.0 ft-lbs	PL=24
	~~V15~~		~~70.63 - 71.00~~	~~Su=674/22 psf~~							PI=11
	V16		71.63 - 72.00	Su=562/40 psf			100			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#210272
							95			55x110 mm vane raw torque readings:	A-6, CL
							92			V15: 15.1/0.5 ft-lbs	WC=34.4%
75							80			V16: 12.6/0.9 ft-lbs	LL=31
											PL=12
											PI=19

Remarks:

Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-101
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	238.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75	5U	24/24	75.00 - 77.00	Piston Sampler	---		89			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210620 A-4, ML WC=26.4% LL=22 PL=20 PI=2
							102				
	V17		77.63 - 78.00	Su=625/67 psf			85			55x110 mm vane raw torque readings: V17: 14.0/1.5 ft-lbs V18: 13.8/1.2 ft-lbs	
	V18		78.63 - 79.00	Su=616/54 psf			73				
							74				
80	12D	24/24	80.00 - 82.00	sample thru vane	---		88			Grey, wet, stiff, Clayey SILT, trace fine sand.	
	~~V19~~		~~80.63 - 81.00~~	~~Su=1286/138 psf~~						55x110 mm vane raw torque readings: V19: 28.8/3.1 ft-lbs V20: >30.0/4.3 ft-lbs	
	V20		81.63 - 82.00	Su=>1339/192 psf			73				
							68				
							74				
							72				
85	6U	24/18	85.00 - 87.00	Piston Sampler	---		83			Grey, wet, stiff, Clayey SILT, trace fine sand. Lost bottom 6" of tube, fine sand seam.	G,C#210621 A-4, CL WC=35.2% LL=31 PL=22 PI=9
							66				
	MV			Could not push			99			Failed 55x110 mm vane attempt.	
							138				
							168				
90	13D	4.2/3	90.00 - 90.35	50(4.2")	---		123	148.20		90.00	G#210273 A-2-4, SC-SM WC=21.8%
							282			Grey, wet, dense, fine SAND, little silt, trace clay, trace medium to coarse sand, trace gravel. Refusal at 90.35' bgs. Roller Coned ahead to 92.0' bgs.	
	R1	18/10	92.00 - 93.50	RQD = N/A%			320 NQ CORE	146.20		92.00 R1:White, grey with garnets, Granite pegmetite BOULDER. R1:Core Times (min:sec) 92.0-93.0' (1:45)	
							323	144.70		93.0-93.5' (0:36) 53% Recovery 93.50	
							269			Roller Coned ahead from 93.5-95.0' bgs.	
95	14D	14.4/14.4	95.00 - 96.20	19/32/50(2.4")	---		192			Grey, wet, fine to coarse SAND, trace silt, trace gravel, broken rock in nose of spoon. Roller Coned ahead from 95.0-100.5' bgs.	G#210274 A-2-4, SP-SM WC=18.6%
							845				
							905 RC			Sand with frequent cobbles and boulders. Casing shoe bent, roller cone thru shoe. 60 blows/in movement on casing.	
100											

Remarks:

Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-101

PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	238.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/22/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+45.9, 12.9 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR/C = weight of rods or casing
WO1P = Weight of one person
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T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
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$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
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PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
100	R2	117.6/ 117.6	100.50 - 110.30	RQD = 77%			NQ	137.70		100.50	
							CORE			Top of Bedrock at Elev. 137.7'	
										Bedrock: White, green and grey, coarse grained pegmetite GRANITE with garnet and mica. Rock Mass Quality = Good	
										R2:Core Times (min:sec)	
										100.5-101.5' (5:01)	
										101.5-102.5' (5:01)	
										102.5-103.5' (4:39)	
										103.5-104.5' (5:40)	
105										104.5-105.5' (5:06)	
										105.5-106.5' (6:09)	
										106.5-107.5' (6:20)	
										107.5-108.5' (4:58)	
										108.5-109.5' (5:07)	
										109.5-110.3' (4:52) 100% Recovery	
110								127.90		110.30	
										Bottom of Exploration at 110.30 feet below ground surface.	
115											
120											
125											

Remarks:

Hammer #283

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-102A

PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	216.6	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/22/08; 13:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	4+93.9, 14.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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LL = Liquid Limit
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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/6	0.00 - 2.00	1/0/1/1	1	1	[a]HP			Grass and brush at surface.	G#210275
							13			Light brown/yellow, damp, loose, fine SAND, little silt, trace gravel, trace medium to coarse sand, (Fill).	A-2-4, SP-SM
							13			[a]Hydraulic Push	WC=9.6%
							17				
							28				
5							45			Casing refusal on top of Pier Pile Cap, abandon hole and moved 2.3' South to BB-ACNR-102.	
								210.60		6.00	
										Bottom of Exploration at 6.00 feet below ground surface.	
10											
15											
20											
25											

Remarks:

Hammer #283
0.7' Concrete Deck thickness.
23.2' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-102
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	217.8	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/22/08, 5/29/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+91.6, 14.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							4	217.50		Grass at Ground Surface.	
										0.30	
							8				
							11				
							14				
	1D	24/4	4.00 - 6.00	10/5/3/2	8	8	10			Brown, damp, loose, fine to coarse SAND, trace gravel, broken rock, 0.2' decomposed wood layer at top of spoon, (Fill).	
5							14				
							15				
							16				
							13				
	2D		9.00 - 11.00	1/0/1/0	1	1	1			Brown, damp, very loose, fine SAND, some silt, little clay in layers, trace medium to coarse sand, trace gravel, (Fill). [a]Hydraulic Push	G#210294 A-4, SC-SM WC=28.6%
10							[a]HP				
								205.80		12.00	
										Brown, wet, stiff, SILT, some clay, trace fine sand.	
	3D/AB		14.00 - 16.00	1/1/0/0	1	1				(3D/A) 14.0-15.0'	G#210296 A-4, CL-ML WC=32.9% LL=27 PL=22 PI=5
15	V1		14.63 - 15.00	Su=1116/134 psf				202.80		55x110 mm vane raw torque readings: V1: 25.0/3.0 ft-lbs	
	V2		15.63 - 16.00	Su=362/67 psf						15.00	
										(3D/B)15.0-16.0' Grey, wet, soft, SILT, some clay, trace fine sand in layers. 55x110 mm vane raw torque readings: V2: 8.1/1.5 ft-lbs	G#210297 A-4, CL-ML WC=33.5% LL=25 PL=20 PI=5
	1U	24/22	19.00 - 21.00	Piston Sampler						Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210622 A-4, CL-ML WC=30.2% Non-Plastic
20	V3		21.63 - 22.00	Su=674/232 psf						55x110 mm vane raw torque readings: V3: 15.1/5.2 ft-lbs	
	V4		22.63 - 23.00	Su=629/89 psf						V4: 14.1/2.0 ft-lbs	
	4D	24/24	24.00 - 26.00	sample thru vane						Grey, wet, medium stiff, SILT, some clay, trace fine sand. 55x110 mm vane raw torque readings:	G#210295 A-4, CL-ML
25											

Remarks:

Hammer #283
0.7' Concrete Deck thickness.
20.9' from top of Bridge Deck to Ground Surface.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-102
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	217.8	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/22/08, 5/29/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+91.6, 14.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	V5		24.63 - 25.00	Su=545/98 psf						V5: 12.2/2.2 ft-lbs	WC=31.5%
	V6		25.63 - 26.00	Su=643/161 psf						V6: 14.4/3.6 ft-lbs	LL=27 PL=20 PI=7
	2U	24/15	29.00 - 31.00	Piston Sampler						Grey, wet, medium stiff, SILT, some clay, trace fine sand in layers.	G,C#210623 A-6, CL WC=29.8% LL=34 PL=23 PI=11
30	V7		31.63 - 32.00	Su=384/103 psf						55x110 mm vane raw torque readings: V7: 8.6/2.3 ft-lbs	
	V8		32.63 - 33.00	Su=527/94 psf						V8: 11.8/2.1 ft-lbs	
	V9		33.63 - 34.00	Su=616/134 psf						55x110 mm vane raw torque readings: V9: 13.8/3.0 ft-lbs	
	5D	24/24	34.00 - 36.00	sample thru vane						Grey, wet, medium stiff, Clayey SILT, trace fine sand.	
35	~~V10~~		~~34.63 - 35.00~~	~~Su=750/156 psf~~						V10: 16.8/3.5 ft-lbs	
	6D/MU	24/24	39.00 - 41.00	0-24"(WO1P)	---					Failed Tube attempt, tube empty. took spoon sample. Grey, wet, medium stiff, Clayey SILT, with black staining, trace fine sand.	G#210298 A-4, ML WC=35.4%
40										Washed ahead 2.0' to 41.0' bgs, took tube sample 3U.	LL=35 PL=27 PI=8
	3U	24/24	41.00 - 43.00	Piston Sampler						Grey, wet, medium stiff, silty CLAY with black staining, trace fine sand.	G,C#210624 A-4, CL WC=38.4% LL=27 PL=19 PI=8
	7D	24/24	43.00 - 45.00	sample thru vane						Grey, wet, medium stiff, silty CLAY with black staining, trace fine sand.	
	~~V11~~		~~43.63 - 44.00~~	~~Su=670/67 psf~~						55x110 mm vane raw torque readings:	
	V12		44.63 - 45.00	Su=714/76 psf						V11: 15.0/1.5 ft-lbs V12: 16.0/1.7 ft-lbs	
45											
	4U	24/24	49.00 - 51.00	Piston Sampler						Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G,#210625 A-4, CL-ML
50											

Remarks:

Hammer #283
0.7' Concrete Deck thickness.
20.9' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-102
PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	217.8	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/22/08, 5/29/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+91.6, 14.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50											WC=30.7% LL=26 PL=19 PI=7
	V13		51.63 - 52.00	Su=670/85 psf						55x110 mm vane raw torque readings: V13: 15.0/1.9 ft-lbs	
	V14		52.63 - 53.00	Su=629/54 psf						V14: 14.1/1.2 ft-lbs	
	8D	24/24	54.00 - 56.00	sample thru vane						Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#210299 A-4, CL WC=30.6% LL=29 PL=20 PI=9
55	V15		54.63 - 55.00	Su=580/80 psf						55x110 mm vane raw torque readings: V15: 13.0/1.8 ft-lbs	
	V16		55.63 - 56.00	Su=848/89 psf						V16: 19.0/2.0 ft-lbs	
	5U	24/20	59.00 - 61.00	Piston Sampler						Grey, wet, medium stiff to stiff, Clayey SILT, trace fine sand.	G,C#210626 A-6, CL WC=36.8% LL=33 PL=22 PI=11
60											
	V17		61.63 - 62.00	Su=723/152 psf						55x110 mm vane raw torque readings: V17: 16.2/3.4 ft-lbs	
	V18		62.63 - 63.00	Su=1045/179 psf						V18: 23.4/4.0 ft-lbs	
								153.80		64.00	
	9D	24/14	64.00 - 66.00	12/24/23/29	47	50	90			Grey, wet, dense, fine SAND, little silt, trace medium to coarse sand, uniform.	G#210300 A-2-4, SM WC=20.3%
65							[b]109			[b]109 blows for 0.7'.	
							79				
							154				
70							137				
	10D	24/12	71.00 - 73.00	33/25/25/33	50	53	182			Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, with broken rock.	G#210601 A-1-b, SW-SM WC=10.4%
							[c]257			[c]257 blows for 0.8'.	
75	11D	9.6/8	74.00 - 74.80	36/50(3.6")	---		81			Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles.	

Remarks:

Hammer #283
0.7' Concrete Deck thickness.
20.9' from top of Bridge Deck to Ground Surface.

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

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-102

PIN: 15600.00

Driller:	Northern Test Boring	**Elevation (ft.)**	217.8	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	K. Maguire	**Rig Type:**	Diedrick D50	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/22/08, 5/29/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"x10'
Boring Location:	4+91.6, 14.3 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.633	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75							115				
							129				
							176				
							559				
	12D	9.6/2	79.00 - 79.80	48/50(3.6")	---		[d]765			[d]765 blows for 0.8'. Grey, wet, very dense, fine to coarse SAND, some gravel, trace silt, occasional cobbles, (Till).	
80							[e]WA			[e]Washed ahead to 84.9' bgs.	
	13D	7.2/2	84.00 - 84.60	79/50(1.2")	---		NQ			Grey, wet, very dense, fine to coarse SAND, little gravel, little silt, (Till).	G#210602
85	R1	111.6/55	84.90 - 94.20	RQD = N/A%			CORE	132.90		84.90	A-2-4, SP-SM
										R1: Pegmetite boulders with iron staining over Gneiss boulders with fine to coarse silty sand layers. R1:Core Times: (min:sec) 84.9-85.9' (2:08) 85.9-86.9' (2:00) 86.9-87.9' (4:03)	WC=12.2%
								129.90		87.9-88.9' (3:46) 88.9-89.9' (3:40) 89.9-90.9' (3:15) 90.9-91.9' (4:31) 91.9-92.9' (5:04) 92.9-93.9' (5:45) 93.9-94.2' (3:11) 46% Recovery Core Blocked	
90										87.90 Grey, wet very dense, fine to coarse SAND, little gravel, little silt.	
								123.80		94.00 Top of Bedrock at Elev. 123.8'.	
	R2	48/48	94.20 - 98.20	RQD = 65%						Bedrock: Black, white and grey, GNEISS interbedded with pegmetite intrusions, chlorite rich. Rock Mass Quality = Fair R2:Core Times (min:sec) 94.2-95.2' (3:32) 95.2-96.2' (3:35) 96.2-97.2' (3:15) 97.2-98.2' (2:16) 100% recovery	
95										Core Blocked	
								119.60		98.20 **Bottom of Exploration at 98.20 feet below ground surface.**	
100											

Remarks:

Hammer #283
0.7' Concrete Deck thickness.
20.9' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-103

PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	224.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/4	0.00 - 2.00	1/1/1/1	2	3	6			Light brown, dry, very loose, fine SAND with roots and wood, (Fill).	
							7				
							5				
							3				
							5				
5	2D	24/12	5.00 - 7.00	1/2/2/1	4	5	8			Light brown, wet, loose, fine Sandy SILT, some silt, trace medium to coarse sand, trace gravel, trace clay, (Fill).	G#210603 A-4, ML WC=25.8%
							2				
							2				
							3				
							2				
10	3D	24/14	10.00 - 12.00	WOR/WOH/WOH/ WOR	---		3			Similar to above.	
							7				
							10				
							6				
							3	210.40		14.00	
15	1U	24/24	15.00 - 17.00	WOR/WOR			WOC			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210627 A-4, CL-ML WC=34.3% LL=28 PL=21 PI=7
							WOC				
	V1		17.52 - 17.95	Su=632/110 psf			WOH			65x130 mm vane raw torque readings: V1: 23.0/4.0 ft-lbs V2: 23.0/3.0 ft-lbs	
	V2		18.52 - 18.95	Su=632/82 psf			WOH				
							WOH				
20	4D	24/24	20.50 - 22.50	push thru vane	---		WOC			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G#210604 A-6, CL WC=33.2% LL=36 PL=22 PI=14
	V3		21.07 - 21.50	Su=659/110 psf			WOC			65x130 mm vane raw torque readings: V3: 24.0/4.0 ft-lbs V4: 19.0/3.0 ft-lbs	
	V4		22.07 - 22.50	Su=522/82 psf			WOH				
							WOH				
25							WOH				

Remarks:

0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-103

PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	224.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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PL = Plastic Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	2U	24/24	25.00 - 27.00	WOR/WOR			WOH			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210628 A-4, ML WC=29.0% LL=27 PL=23 PI=4
							WOH				
							14				
	V5		28.57 - 29.00	Su=522/110 psf			22			65x130 mm vane raw torque readings: V5: 19.0/4.0 ft-lbs V6: 21.0/5.0 ft-lbs	
	V6		29.57 - 30.00	Su=577/137 psf			19				
30	5D	24/18	30.50 - 32.50	push thru vane	---		14			Similar to above.	
	V7		31.07 - 31.50	Su=838/165 psf			19			65x130 mm vane raw torque readings: V7: 30.5/6.0 ft-lbs V8: 32.0/7.0 ft-lbs	
	V8		32.07 - 32.50	Su=879/192 psf			21				
							21				
							22				
35	3U	24/24	35.00 - 37.00	WOR/HydralicPush			14			Grey, wet, stiff, SILT, some clay, trace fine sand.	G,C#210629 A-6, CL WC=34.3% LL=35 PL=21 PI=14
							21				
	V9		37.57 - 38.00	Su=1044/220 psf			27			65x130 mm vane raw torque readings: V9: 38.0/8.0 ft-lbs V10: 40.0/9.0 ft-lbs	
	V10		38.57 - 39.00	Su=1099/247 psf			29				
							30				
40	6D	24/24	40.50 - 42.50	push thru vane	---		12			Grey, wet, medium stiff, Clayey SILT, trace fine sand.	G#210605 A-6, CL WC=36.8% LL=36 PL=22 PI=14
	V11		41.13 - 41.50	Su=893/156 psf			22			55x110 mm vane raw torque readings: V11: 20.0/3.5 ft-lbs V12: 20.0/3.5 ft-lbs	
	V12		42.13 - 42.50	Su=893/156 psf			26				
							26				
							24				
45	4U	24/24	45.00 - 47.00	WOR/WOR			13			Grey, wet, medium stiff, SILT, some clay, trace fine sand.	G,C#210630 A-6, ML WC=40.8% LL=37 PL=27 PI=10
							18				
	V13		47.63 - 48.00	Su=871/134 psf			20			55x110 mm vane raw torque readings: V13: 19.5/3.0 ft-lbs V14: 22.0/5.0 ft-lbs	
	V14		48.63 - 49.00	Su=982/223 psf			19				
50							21				

Remarks:

0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

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

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-103
PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	224.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	7D	24/24	50.50 - 52.50	push thru vane	---		18			Grey, wet, stiff, SILT, trace fine sand.	
	V15		51.13 - 51.50	Su=1161/112 psf			21			55x110 mm vane raw torque readings: V15: 26.0/2.5 ft-lbs	
	V16		52.13 - 52.50	Su=1250/201 psf			24			V16: 28.0/4.5 ft-lbs	
							24				
							23				
55	5U	24/24	55.00 - 57.00	WOR/WOR			21			Gret, wet, stiff, Clayey SILT, trace fine sand.	G,C#210631 A-6, CL WC=40.2% LL=35 PL=23 PI=12
							26				
	V17		57.63 - 58.00	Su=1473/179 psf			26			55x110 mm vane raw torque readings: V17: 33.0/4.0 ft-lbs	
	V18		58.63 - 59.00	Su=1384/223 psf			26			V18: 31.0/5.0 ft-lbs	
							48	164.90		59.50	
60	8D	24/16	60.00 - 62.00	7/6/7/14	13	17	48			Brown, wet, medium dense, fine to medium SAND, little silt, trace coarse sand, trace gravel.	G#210606 A-2-4, SM WC=22.5%
							62				
							77				
							77				
							82				
65	9D	24/17	65.00 - 67.00	4/5/5/10	10	13	64			Brown, wet, medium dense, fine to medium SAND, little silt, trace coarse sand, trace gravel.	
							93				
							120				
							174				
							246				
70	10D	24/16	70.00 - 72.00	26/42/40/55	82	105	74			Brown, wet, very dense, fine to coarse SAND, little gravel, little silt.	G#210607 A-2-4, SM WC=13.0%
							127				
							175				
	R1	60/59	73.30 - 78.30	RQD = 65%			a125 NQ CORE	151.10		a125 blows for 0.3'. 73.30 Top of Bedrock at Elev. 151.1' Bedrock: White, green and grey, coarse grained, pegmetite GRANITE,	
75											

Remarks:

0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-103

PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	224.4	**Auger ID/OD:**	N/A
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/19/08, 5/21/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	5+60.6, 14.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
75										with garnet and mica, no bedding. Rock Mass Quality = Fair R1:Core Times (min:sec) 73.3-74.3' (7:41) 74.3-75.3' (7:06) 75.3-76.3' (6:35) 76.3-77.3' (6:41) 77.3-78.3' (6:52) 98% Recovery R2: Rock Quality = Poor Core Times (min:sec) 78.3-79.3' (5:14) 79.3-80.3' (5:08) 80.3-81.3' (5:24) 81.3-82.3' (5:29) 82.3-83.3' (5:00) 96% Recovery	
	R2	60/58	78.30 - 83.30	RQD = 45%							
80											
								141.10		83.30 **Bottom of Exploration at 83.30 feet below ground surface.**	
85											
90											
95											
100											

Remarks:

0.6' Concrete Deck thickness.
19.0' from top of Bridge Deck to Ground Surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-104
PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	241.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/21/08, 5/27/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+01.9, 11.7 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	240.60		Pavement	
										0.50	
	1D	13.2/13.2	1.00 - 2.10	10/25/20(1.2")	---					Brown, dry, very dense, fine to coarse SAND, little gravel, little silt, occasional cobbles, (Fill).	
										Boulder from 2.1-3.2' bgs.	
5	2D/A	24/18	5.00 - 7.00	4/6/5/4	11	14				(2D/A) 5.0-6.5'. Similar to above, medium dense.	G#210608 A-1-b, SM WC=4.5%
								234.60		6.50	
										(2D) 6.5-7.0' bgs. Light brown, damp, Sandy SILT, little clay, (Fill).	G#210609 A-4, ML WC=20.9%
								232.10		9.00	
10	3D	24/19	10.00 - 12.00	15/9/9/16	18	23				Golden brown, damp, medium dense, fine SAND, trace silt, trace medium to coarse sand, (Fill).	
15	4D	24/20	15.00 - 17.00	4/7/9/9	16	21	143			Golden brown, damp, medium dense, fine SAND, trace silt, trace medium to coarse sand, (Fill).	G#210610 A-3, SP-SM WC=8.6%
							157				
							123				
								223.10		18.00	
							79				
							87				
20	5D	24/20	20.50 - 22.50	2/4/8/7	12	15	64			Olive-brown, moist, medium dense, SILT, some fine sand, little clay, trace roots.	G#210611 A-4, ML WC=21.0%
							81				
							88				
							87				
25	6D	24/22	24.00 - 26.00	7/6/7/6	13	17	57				G#210612 A-4, ML

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202
Location: Auburn, Maine

Boring No.: BB-ACNR-104
PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	241.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/21/08, 5/27/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+01.9, 11.7 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							67			Olive-brown, wet, medium dense, SILT, little sand, little clay.	WC=25.5%
							68				
							67				
							66				
	7D/MV	24/24	29.50 - 31.50	3/WOH/WOH/1	---		53			Failed 55x110 mm vane attempt, could not push.	G#210613
30							59			Olive, wet, very soft, SILT, little clay, trace fine sand.	A-4, ML WC=30.7% LL=25 PL=22 PI=3
							61				
							55				
							54	207.70		33.40	
	1U	24/24	34.00 - 36.00	WOR/Hyd Push			46			Grey, wet, soft to medium stiff, Clayey SILT, trace fine sand.	G,C#210632
35							55				A-4, CL WC=36.7% LL=30 PL=22 PI=8
	V1		36.63 - 37.00	Su=513/89 psf			59			55x110 mm vane raw torque readings: V1: 11.5/2.0 ft-lbs	
	V2		37.63 - 38.00	Su=491/89 psf			54			V2: 11.0/2.0 ft-lbs	
							59				
							54				
40	8D	24/24	40.50 - 42.50	push thru vane	---		70			Similar to above, medium stiff.	G#210614
	V3		41.13 - 41.50	Su=625/89 psf			91			55x110 mm vane raw torque readings: V3: 14.0/2.0 ft-lbs	A-6, CL WC=31.9% LL=31 PL=19 PI=12
	V4		42.13 - 42.50	Su=737/134 psf			79			V4: 16.5/3.0 ft-lbs	
							73				
	2U	24/24	44.00 - 46.00	WOR/HydraulicPus			62			Grey, wet, stiff, Silty CLAY, trace fine sand.	G,C#210633
45							73				A-6, CL WC=36.9% LL=36 PL=24 PI=12
	V5		46.63 - 47.00	Su=1049/223 psf			79			55x110 mm vane raw torque readings: V5: 23.5/5.0 ft-lbs	
	V6		47.63 - 48.00	Su=1071/223 psf			77			V6: 24.0/5.0 ft-lbs	
							73				
50	9D	24/24	49.50 - 51.50	push thru vane	---		66			Similar to above, stiff.	G#210615

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: CNR Railroad Crossing, Routes 4/100/202

Location: Auburn, Maine

Boring No.: BB-ACNR-104

PIN: 15600.00

Driller:	MaineDOT	**Elevation (ft.)**	241.1	**Auger ID/OD:**	5" Solid Stem
Operator:	E. Giguere/C. Giles	**Datum:**	NAVD 88	**Sampler:**	24" Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	5/21/08, 5/27/08	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	6+01.9, 11.7 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	None Observed
Hammer Efficiency Factor:	0.77	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	V7		50.13 - 50.50	Su=1027/201 psf			67			55x110 mm vane raw torque readings: V7: 23.0/4.5 ft-lbs V8: 26.0/6.0 ft-lbs	A-6, CL WC=40.9% LL=39 PL=25 PI=14
	V8		51.13 - 51.50	Su=1161/268 psf			67				
							73				
							72				
	3U	12/12	54.00 - 55.00	Hydraulic Push			53	187.10		54.00 Grey, wet, soft, SILT, some clay, trace sand, trace gravel.	G,C#210634 A-4, CL-ML WC=29.5% LL=25 PL=20 PI=5
55							200				
	R1	60/60	56.10 - 61.10	RQD = 28%			NQ CORE	185.00		Roller Coned ahead to 56.1' bgs. 56.10 Top of Bedrock at Elev. 185.0' Bedrock: White, green and grey, coarse grained, pegmetite GRANITE, with garnet and mica. Rock Mass Quality = Poor R1: Core Times (min:sec) 56.1-57.1' (7:28) 57.1-58.1' (5:42) 58.1-59.1' (4:43) 59.1-60.1' (3:19) 60.1-61.1' (10:32) 100% Recovery Rock Mass Quality = Fair R2: Core Times (min:sec) 61.1-62.1' (7:08) 62.1-63.1' (4:28) 63.1-64.1' (7:48) 64.1-65.1' (8:00) 65.1-66.1' (8:51) 100% Recovery	
60											
	R2	60/60	61.10 - 66.10	RQD = 53%							
65								175.00		66.10 **Bottom of Exploration at 66.10 feet below ground surface.**	
70											
75											

Remarks:

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Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Auburn Project Number: 15600.00

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-ACNR-101, 2D	4+45.9	12.9 Rt.	5.0-7.0	209920	1	4.4			SP-SM	A-3	0
BB-ACNR-101,4D	4+45.9	12.9 Rt.	15.0-17.0	209921	1	11.3			SP-SM	A-3	0
BB-ACNR-101, 5D	4+45.9	12.9 Rt.	20.0-22.0	209922	1	19.7			SM	A-2-4	II
BB-ACNR-101, 6D	4+45.9	12.9 Rt.	25.0-27.0	209923	1	30.1			ML	A-4	IV
BB-ACNR-101, 7D/A	4+45.9	12.9 Rt.	30.0-31.5	209924	1	33.1			CL-ML	A-4	IV
BB-ACNR-101, 7D/B	4+45.9	12.9 Rt.	31.5-32.0	209925	1	25.3			ML	A-4	IV
BB-ACNR-101, 8D	4+45.9	12.9 Rt.	35.0-37.0	210269	2	26.9			ML	A-4	IV
BB-ACNR-101, 2U	4+45.9	12.9 Rt.	45.0-47.0	210617	2	30.6	22	3	ML	A-4	IV
BB-ACNR-101, 9D	4+45.9	12.9 Rt.	50.0-52.0	210270	2	32.4	28	6	CL-ML	A-4	IV
BB-ACNR-101, 3U	4+45.9	12.9 Rt.	55.0-57.0	210618	2	32.8	30	11	CL	A-6	IV
BB-ACNR-101, 10D	4+45.9	12.9 Rt.	60.0-62.0	210271	2	30.9	30	8	CL	A-4	IV
BB-ACNR-101, 4U	4+45.9	12.9 Rt.	65.0-67.0	210619	2	38.0	35	11	CL	A-6	IV
BB-ACNR-101, 11D	4+45.9	12.9 Rt.	70.0-72.0	210272	3	34.4	31	19	CL	A-6	III
BB-ACNR-101, 5U	4+45.9	12.9 Rt.	75.0-77.0	210620	3	26.4	22	2	ML	A-4	IV
BB-ACNR-101, 6U	4+45.9	12.9 Rt.	85.0-87.0	210621	3	35.2	31	9	CL	A-4	IV
BB-ACNR-101, 13D	4+45.9	12.9 Rt.	90.0-90.35	210273	3	21.8			SC-SM	A-2-4	III
BB-ACNR-101, 14D	4+45.9	12.9 Rt.	95.0-96.2	210274	3	18.6			SP-SM	A-2-4	0
BB-ACNR-102, 2D	4+91.6	14.3 Rt.	9.0-11.0	210294	4	28.6			SC-SM	A-4	III
BB-ACNR-102, 3D/A	4+91.6	14.3 Rt.	14.0-15.0	210296	4	32.9	27	5	CL-ML	A-4	IV
BB-ACNR-102, 3D/B	4+91.6	14.3 Rt.	15.0-16.0	210297	4	33.5	25	5	CL-ML	A-4	IV
BB-ACNR-102, 1U	4+91.6	14.3 Rt.	19.0-21.0	210622	4	30.2	-N	P-	CL-ML	A-4	IV
BB-ACNR-102, 4D	4+91.6	14.3 Rt.	24.0-26.0	210295	4	31.5	27	7	CL-ML	A-4	IV
BB-ACNR-102, 2U	4+91.6	14.3 Rt.	29.0-31.0	210623	5	29.8	34	11	CL	A-6	IV
BB-ACNR-102, 6D	4+91.6	14.3 Rt.	39.0-41.0	210298	5	35.4	35	8	ML	A-4	IV
BB-ACNR-102, 3U	4+91.6	14.3 Rt.	41.0-43.0	210624	5	38.4	27	8	CL	A-4	IV
BB-ACNR-102, 4U	4+91.6	14.3 Rt.	49.0-51.0	210625	5	30.7	26	7	CL-ML	A-4	IV
BB-ACNR-102, 8D	4+91.6	14.3 Rt.	54.0-56.0	210299	5	30.6	29	9	CL	A-4	IV
BB-ACNR-102, 5U	4+91.6	14.3 Rt.	59.0-61.0	210626	6	36.8	33	11	CL	A-6	IV
BB-ACNR-102, 9D	4+91.6	14.3 Rt.	64.0-66.0	210300	6	20.3			SM	A-2-4	II
BB-ACNR-102, 10D	4+91.6	14.3 Rt.	71.0-73.0	210601	6	10.4			SW-SM	A-1-b	0
BB-ACNR-102, 13D	4+91.6	14.3 Rt.	84.0-84.6	210602	6	12.2			SP-SM	A-2-4	0
BB-ACNR-102A/1D	4+93.9	14.3 Rt.	0.0-2.0	210275	6	9.6			SP-SM	A-2-4	0
BB-ACNR-103, 2D	5+60.6	14.2 Rt.	5.0-7.0	210603	7	25.8			ML	A-4	IV
BB-ACNR-103, 1U	5+60.6	14.2 Rt.	15.0-17.0	210627	7	34.3	28	7	CL-ML	A-4	IV
BB-ACNR-103, 4D	5+60.6	14.2 Rt.	20.5-22.5	210604	7	33.2	36	14	CL	A-6	III
BB-ACNR-103, 2U	5+60.6	14.2 Rt.	25.0-27.0	210628	7	29.0	27	4	ML	A-4	IV
BB-ACNR-103, 3U	5+60.6	14.2 Rt.	35.0-37.0	210629	7	34.3	35	14	CL	A-6	III
BB-ACNR-103, 6D	5+60.6	14.2 Rt.	40.5-42.5	210605	8	36.8	36	14	CL	A-6	III
BB-ACNR-103, 4U	5+60.6	14.2 Rt.	45.0-47.0	210630	8	40.8	37	10	ML	A-6	IV
BB-ACNR-103, 5U	5+60.6	14.2 Rt.	55.0-57.0	210631	8	40.2	35	12	CL	A-6	III
BB-ACNR-103, 8D	5+60.6	14.2 Rt.	60.0-62.0	210606	8	22.5			SM	A-2-4	II
BB-ACNR-103, 10D	5+60.6	14.2 Rt.	70.0-72.0	210607	8	13.0			SM	A-2-4	II

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

Laboratory Testing Summary Sheet

Town(s): Auburn Project Number: 15600.00

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-ACNR-104, 2D/A	6+01.9	11.7 Rt.	5.0-6.5	210608	9	4.5			SM	A-1-b	II
BB-ACNR-104, 2D/B	6+01.9	11.7 Rt.	6.5-7.0	210609	9	20.9			ML	A-4	IV
BB-ACNR-104, 4D	6+01.9	11.7 Rt.	15.0-17.0	210610	9	8.6			SP-SM	A-3	0
BB-ACNR-104, 5D	6+01.9	11.7 Rt.	20.5-22.5	210611	9	21.0			ML	A-4	IV
BB-ACNR-104, 6D	6+01.9	11.7 Rt.	24.0-26.0	210612	9	25.5			ML	A-4	IV
BB-ACNR-104, 7D	6+01.9	11.7 Rt.	29.5-31.5	210613	9	30.7	25	3	ML	A-4	IV
BB-ACNR-104, 1U	6+01.9	11.7 Rt.	34.0-36.0	210632	10	36.7	30	8	CL	A-4	IV
BB-ACNR-104, 8D	6+01.9	11.7 Rt.	40.5-42.5	210614	10	31.9	31	12	CL	A-6	III
BB-ACNR-104, 2U	6+01.9	11.7 Rt.	44.0-46.0	210633	10	36.9	36	12	CL	A-6	III
BB-ACNR-104, 9D	6+01.9	11.7 Rt.	49.5-51.5	210615	10	40.9	39	14	CL	A-6	III
BB-ACNR-104, 3U	6+01.9	11.7 Rt.	54.0-55.0	210634	10	29.5	25	5	CL-ML	A-4	IV

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

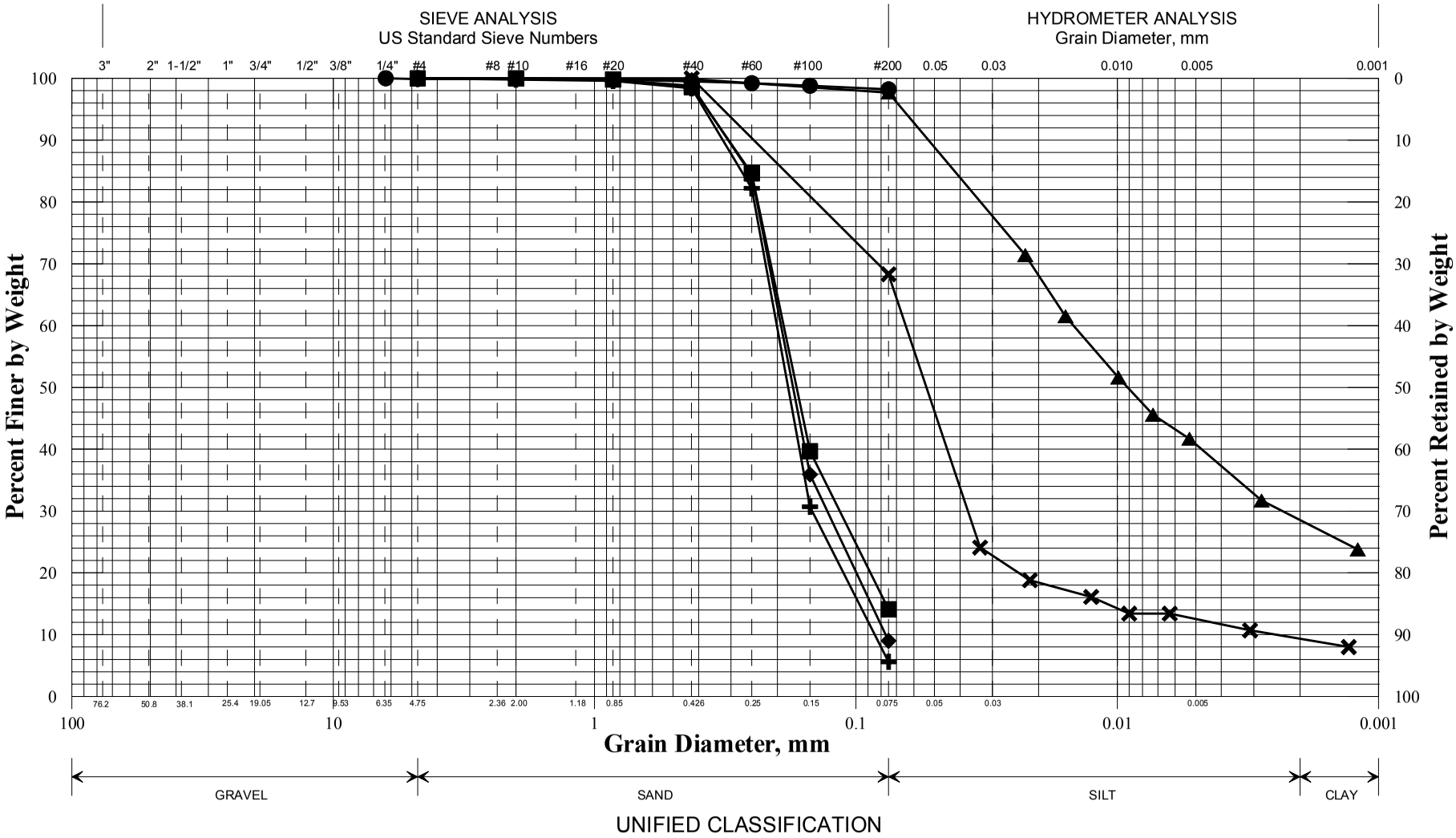
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-101/2D	4+45.9	12.9 RT.	5.0-7.0	SAND, trace silt.	4.4			
◆	BB-ACNR-101/4D	4+45.9	12.9 RT.	15.0-17.0	SAND, trace silt.	11.3			
■	BB-ACNR-101/5D	4+45.9	12.9 RT.	20.0-22.0	SAND, little silt.	19.7			
●	BB-ACNR-101/6D	4+45.9	12.9 RT.	25.0-27.0	SILT, trace sand, trace gravel.	30.1			
▲	BB-ACNR-101/7D/A	4+45.9	12.9 RT.	30.0-31.5	SILT, some clay, trace sand.	33.1			
✖	BB-ACNR-101/7D/B	4+45.9	12.9 RT.	31.5-32.0	SILT, some sand, trace clay.	25.3			

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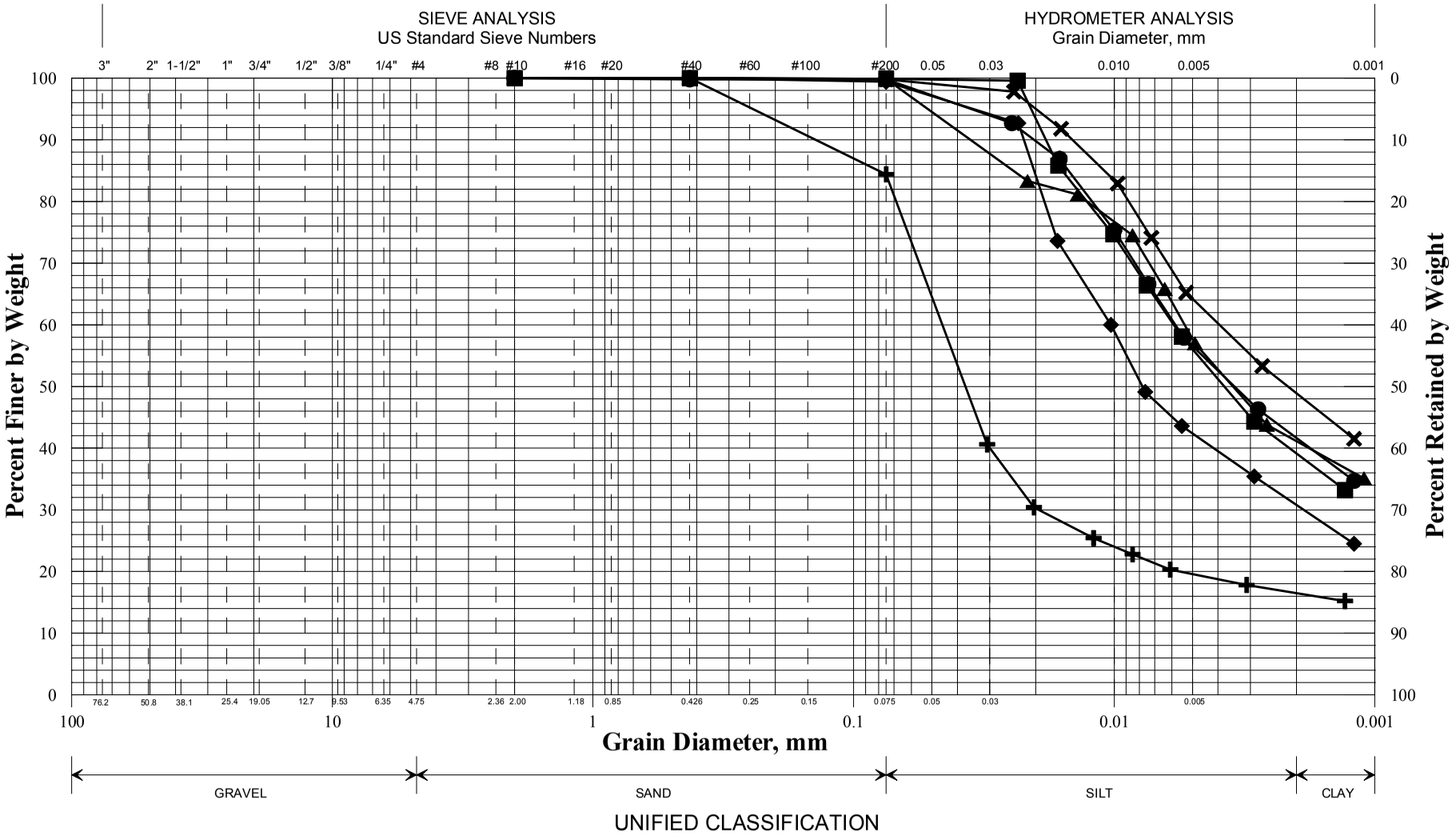
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-101/8D	4+45.9	12.9 RT.	35.0-37.0	SILT, little clay, little sand.	26.9			
◆	BB-ACNR-101/2U	4+45.9	12.9 RT.	45.0-47.0	SILT, some clay, trace sand.	30.6	22	19	3
■	BB-ACNR-101/9D	4+45.9	12.9 RT.	50.0-52.0	Clayey SILT, trace sand.	32.4	28	22	6
●	BB-ACNR-101/3U	4+45.9	12.9 RT.	55.0-57.0	Clayey SILT, trace sand.	32.8	30	19	11
▲	BB-ACNR-101/10D	4+45.9	12.9 RT.	60.0-62.0	Clayey SILT, trace sand.	30.9	30	22	8
✕	BB-ACNR-101/4U	4+45.9	12.9 RT.	65.0-67.0	Clayey SILT, trace sand.	38.0	35	24	11

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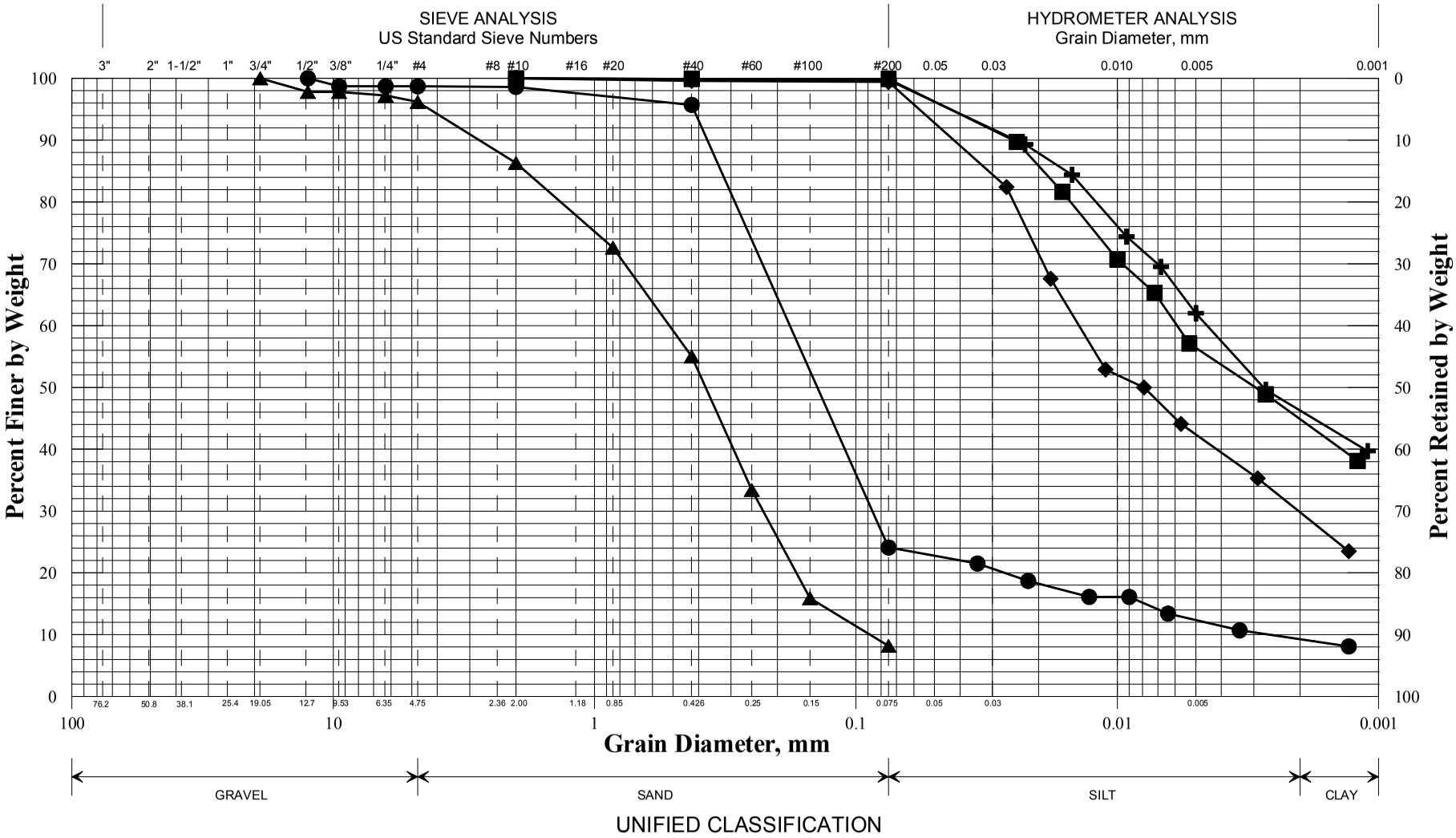
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-101/11D	4+45.9	12.9 RT.	70.0-72.0	Clayey SILT, trace sand.	34.4	31	12	19
◆	BB-ACNR-101/5U	4+45.9	12.9 RT.	75.0-77.0	SILT, some clay, trace sand.	26.4	22	20	2
■	BB-ACNR-101/6U	4+45.9	12.9 RT.	85.0-87.0	Clayey SILT, trace sand.	35.2	31	22	9
●	BB-ACNR-101/13D	4+45.9	12.9 RT.	90.0-90.35	SAND, little silt, trace clay, trace gravel.	21.8			
▲	BB-ACNR-101/14D	4+45.9	12.9 RT.	95.0-96.2	SAND, trace silt, trace gravel.	18.6			
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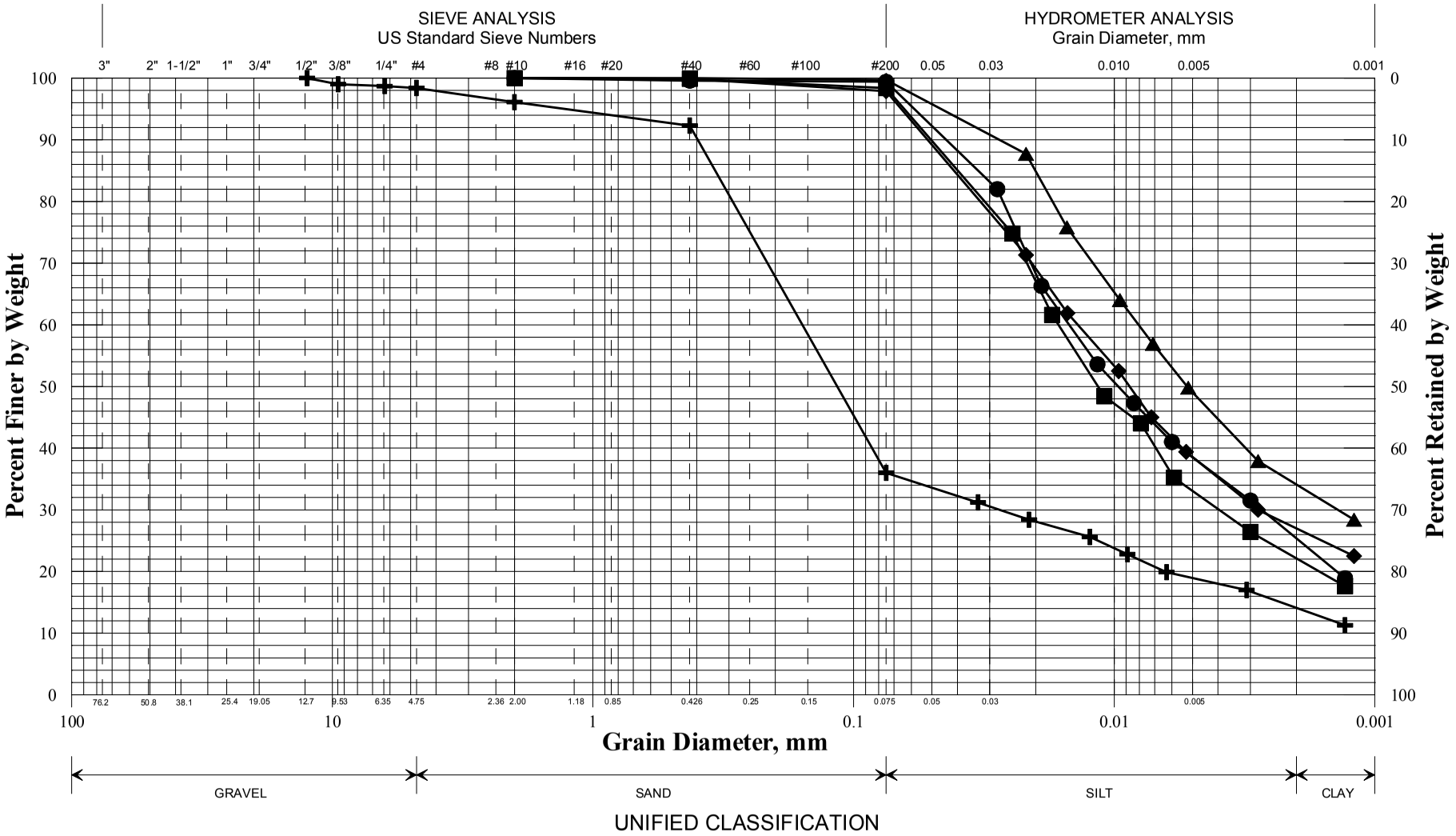
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-102/2D	4+91.6	14.3 RT.	9.0-11.0	SAND, some silt, little clay, trace gravel.	28.6			
◆	BB-ACNR-102/3D-A	4+91.6	14.3 RT.	14.0-15.0	SILT, some clay, trace sand.	32.9	27	22	5
■	BB-ACNR-102/3D-B	4+91.6	14.3 RT.	15.0-16.0	SILT, some clay, trace sand.	33.5	25	20	5
●	BB-ACNR-102/1U	4+91.6	14.3 RT.	19.0-21.0	SILT, some clay, trace sand.	30.2			NP
▲	BB-ACNR-102/4D	4+91.6	14.3 RT.	24.0-26.0	SILT, some clay, trace sand.	31.5	27	20	7
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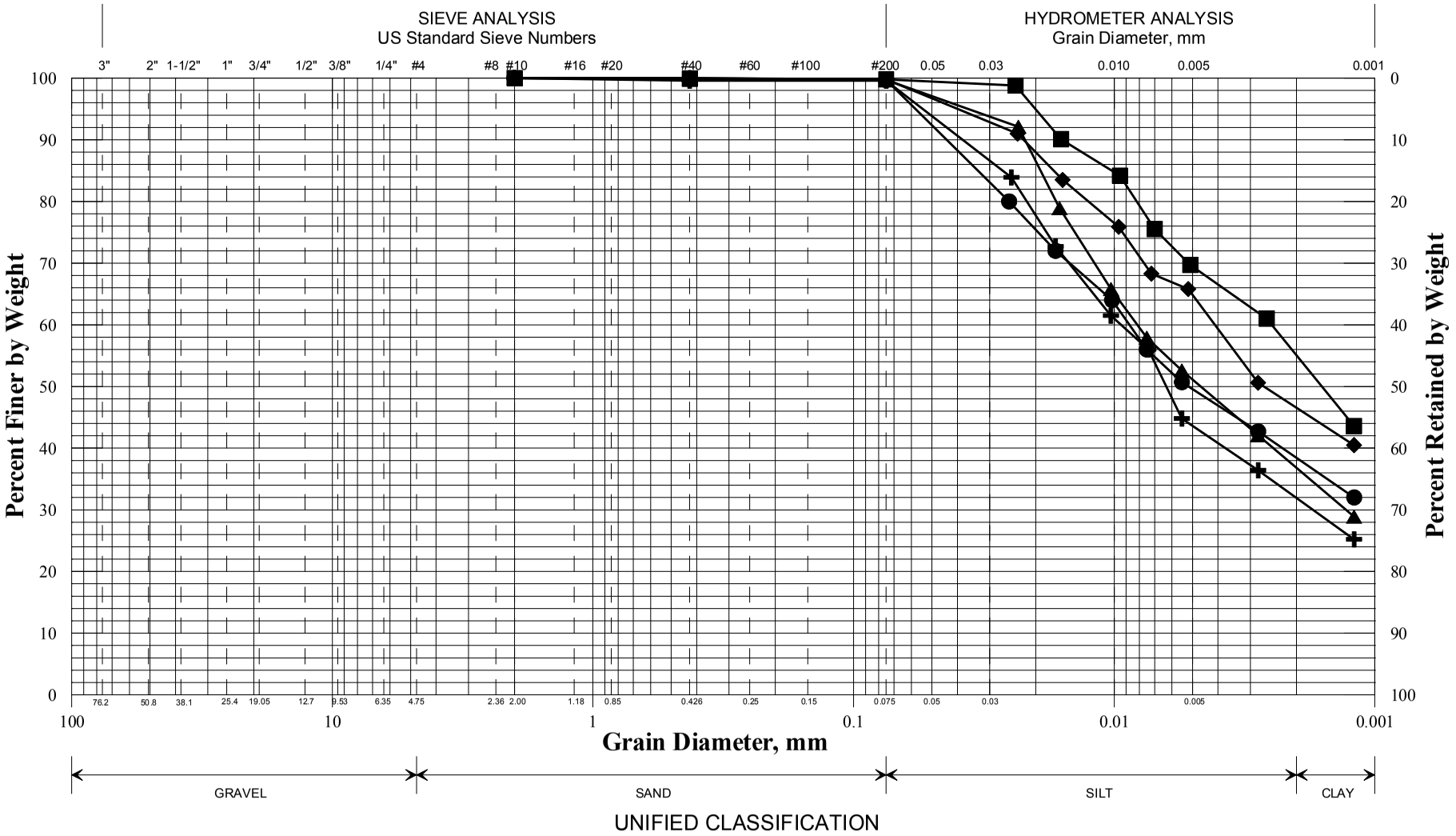
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-102/2U	4+91.6	14.3 RT.	29.0-31.0	SILT, some clay, trace sand.	29.8	34	23	11
◆	BB-ACNR-102/6D	4+91.6	14.3 RT.	39.0-41.0	Clayey SILT, trace sand.	35.4	35	27	8
■	BB-ACNR-102/3U	4+91.6	14.3 RT.	41.0-43.0	Silty CLAY, trace sand.	38.4	27	19	8
●	BB-ACNR-102/4U	4+91.6	14.3 RT.	49.0-51.0	Clayey SILT, trace sand.	30.7	26	19	7
▲	BB-ACNR-102/8D	4+91.6	14.3 RT.	54.0-56.0	Clayey SILT, trace sand.	30.6	29	20	9
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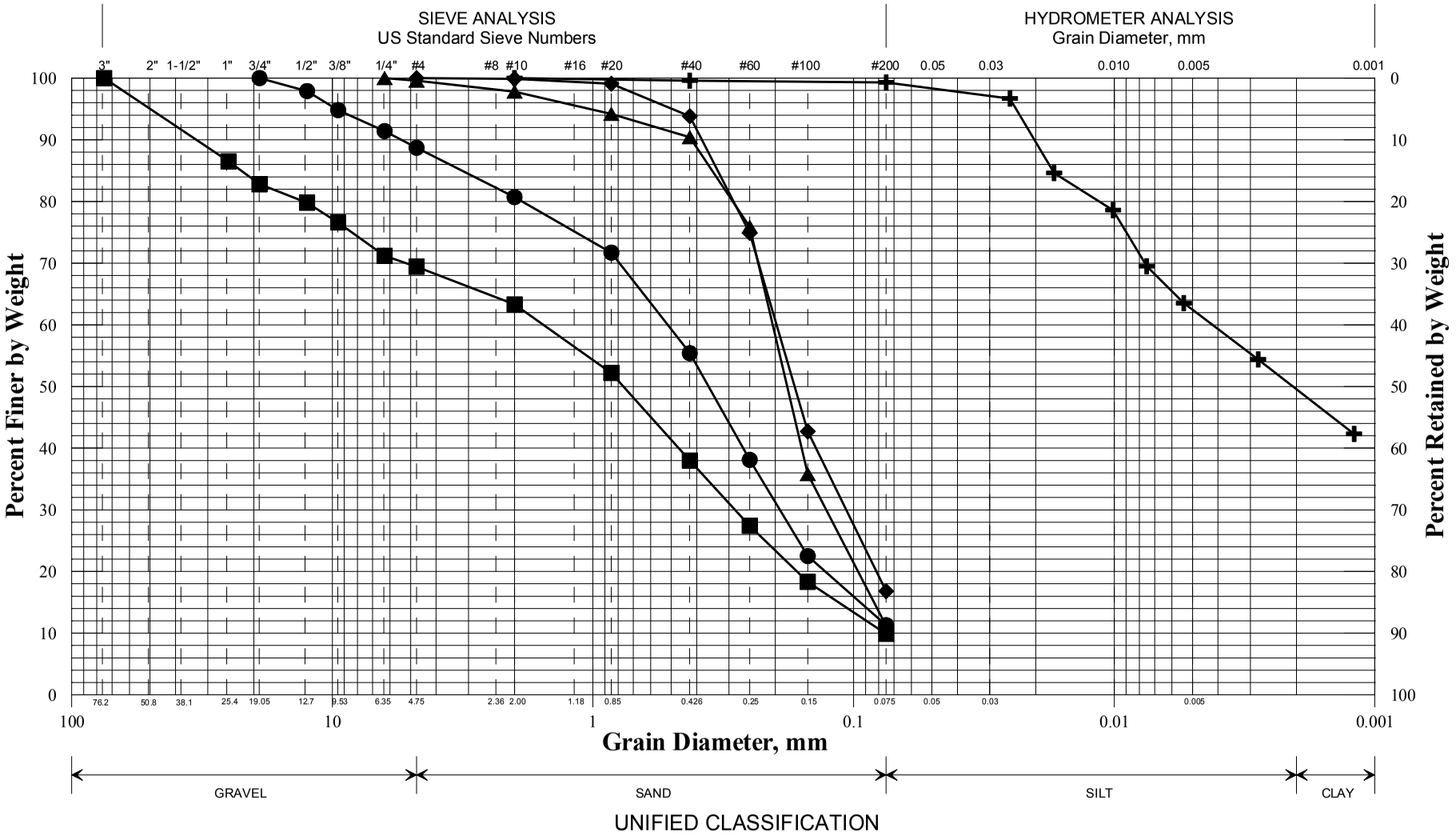

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-102/5U	4+91.6	14.3 RT.	59.0-61.0	Clayey SILT, trace sand.	36.8	33	22	11
◆	BB-ACNR-102/9D	4+91.6	14.3 RT.	64.0-66.0	SAND, little silt.	20.3			
■	BB-ACNR-102/10D	4+91.6	14.3 RT.	71.0-73.0	SAND, some gravel, trace silt.	10.4			
●	BB-ACNR-102/13D	4+91.6	14.3 RT.	84.0-84.6	SAND, little gravel, little silt.	12.2			
▲	BB-ACNR-102A/1D	4+93.9	14.3 RT.	0.0-2.0	SAND, little silt, trace gravel.	9.6			
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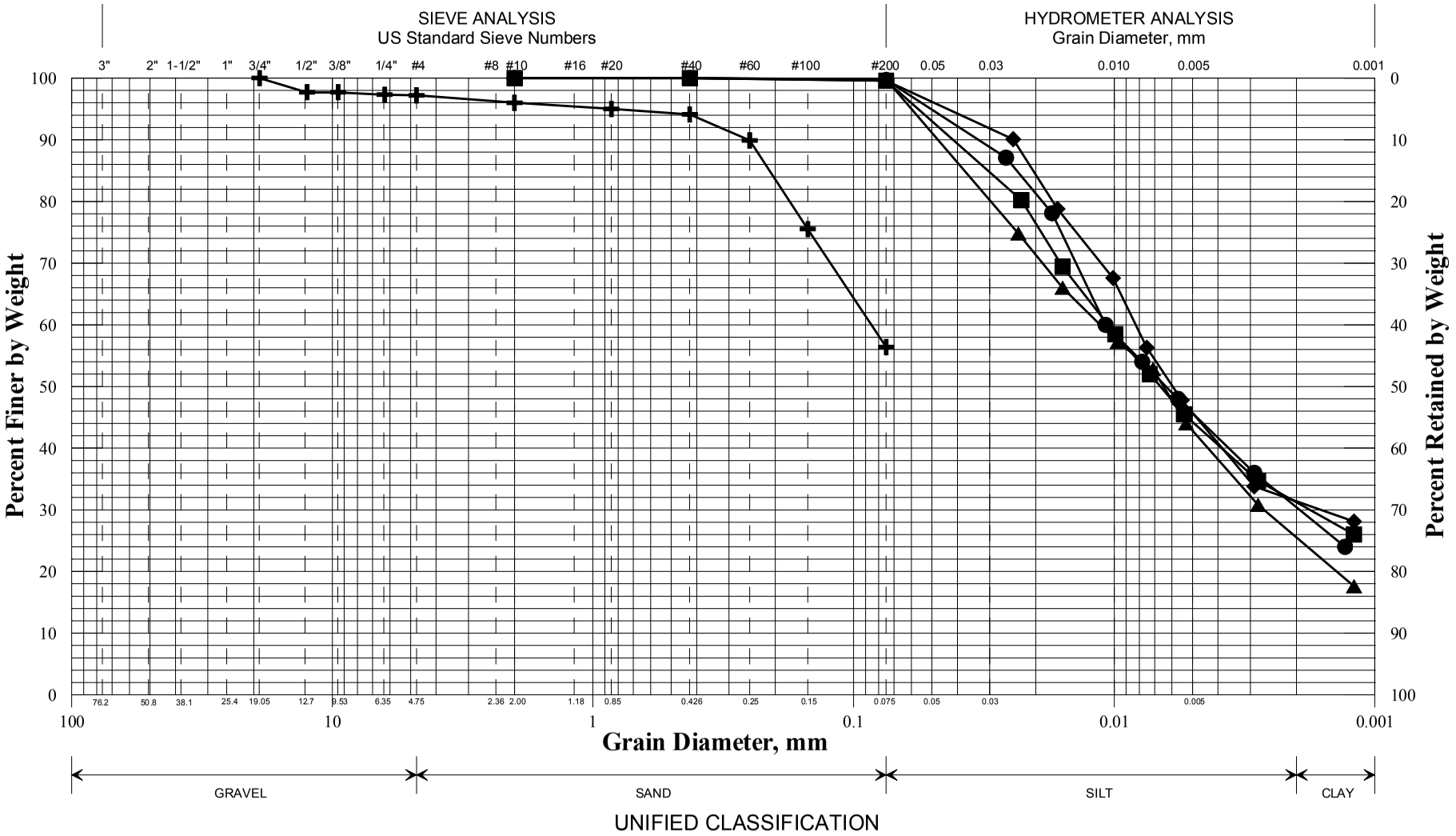
GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-103/2D	5+60.6	14.2 RT.	5.0-7.0	Sandy SILT with clay, trace gravel.	25.8			
◆	BB-ACNR-103/1U	5+60.6	14.2 RT.	15.0-17.0	SILT, some clay, trace sand.	34.3	28	21	7
■	BB-ACNR-103/4D	5+60.6	14.2 RT.	20.5-22.5	SILT, some clay, trace sand.	33.2	36	22	14
●	BB-ACNR-103/2U	5+60.6	14.2 RT.	25.0-27.0	SILT, some clay, trace sand.	29.0	27	23	4
▲	BB-ACNR-103/3U	5+60.6	14.2 RT.	35.0-37.0	SILT, some clay, trace sand.	34.3	35	21	14
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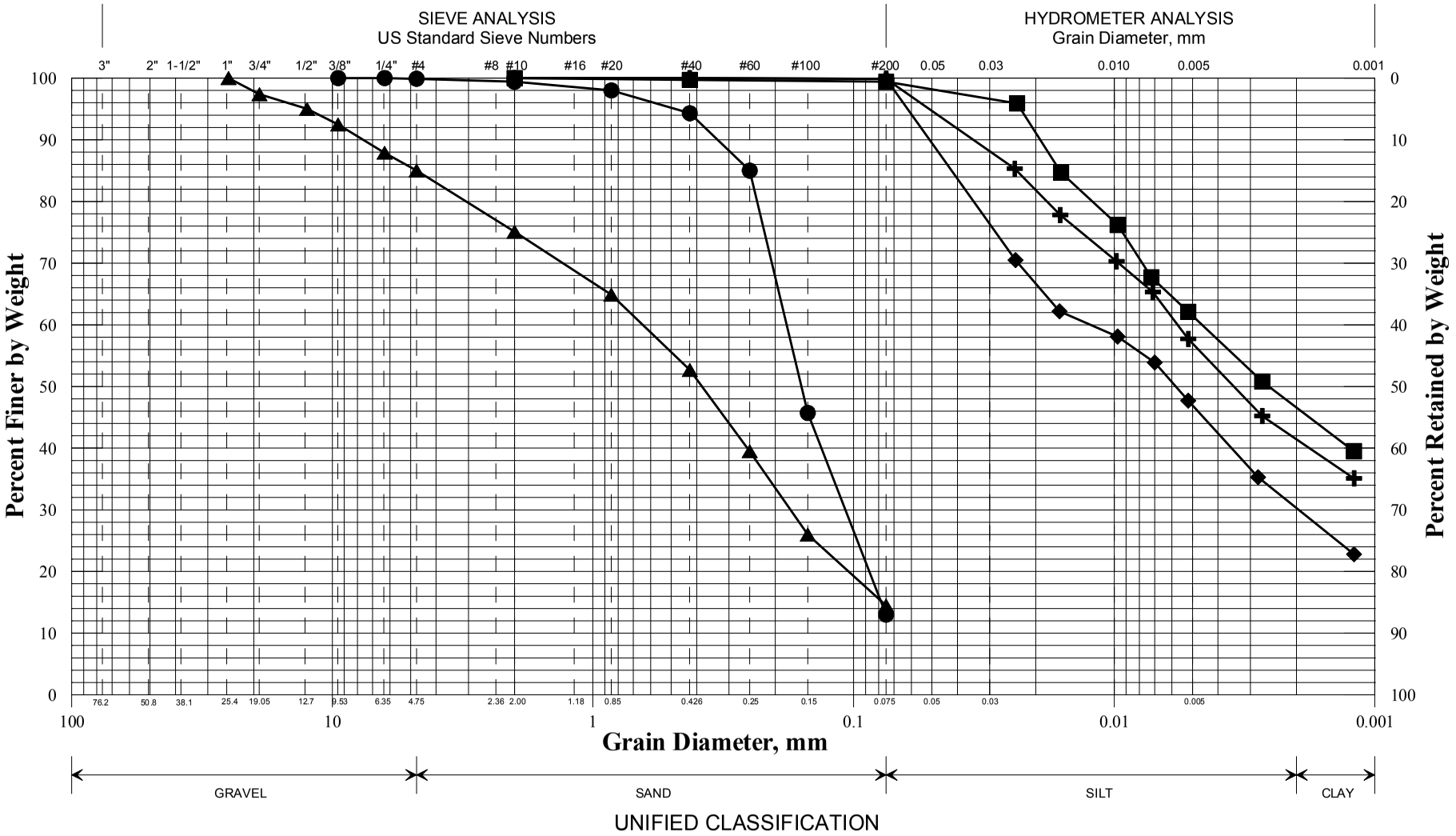

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-103/6D	5+60.6	14.2 RT.	40.5'-42.5'	Clayey SILT, trace sand.	36.8	36	22	14
◆	BB-ACNR-103/4U	5+60.6	14.2 RT.	45.0-47.0	SILT, some clay, trace sand.	40.8	37	27	10
■	BB-ACNR-103/5U	5+60.6	14.2 RT.	55.0-57.0	Clayey SILT, trace sand.	40.2	35	23	12
●	BB-ACNR-103/8D	5+60.6	14.2 RT.	60.0-62.0	SAND, little silt, trace gravel.	22.5			
▲	BB-ACNR-103/10D	5+60.6	14.2 RT.	70.0-72.0	SAND, little gravel, little silt.	13.0			
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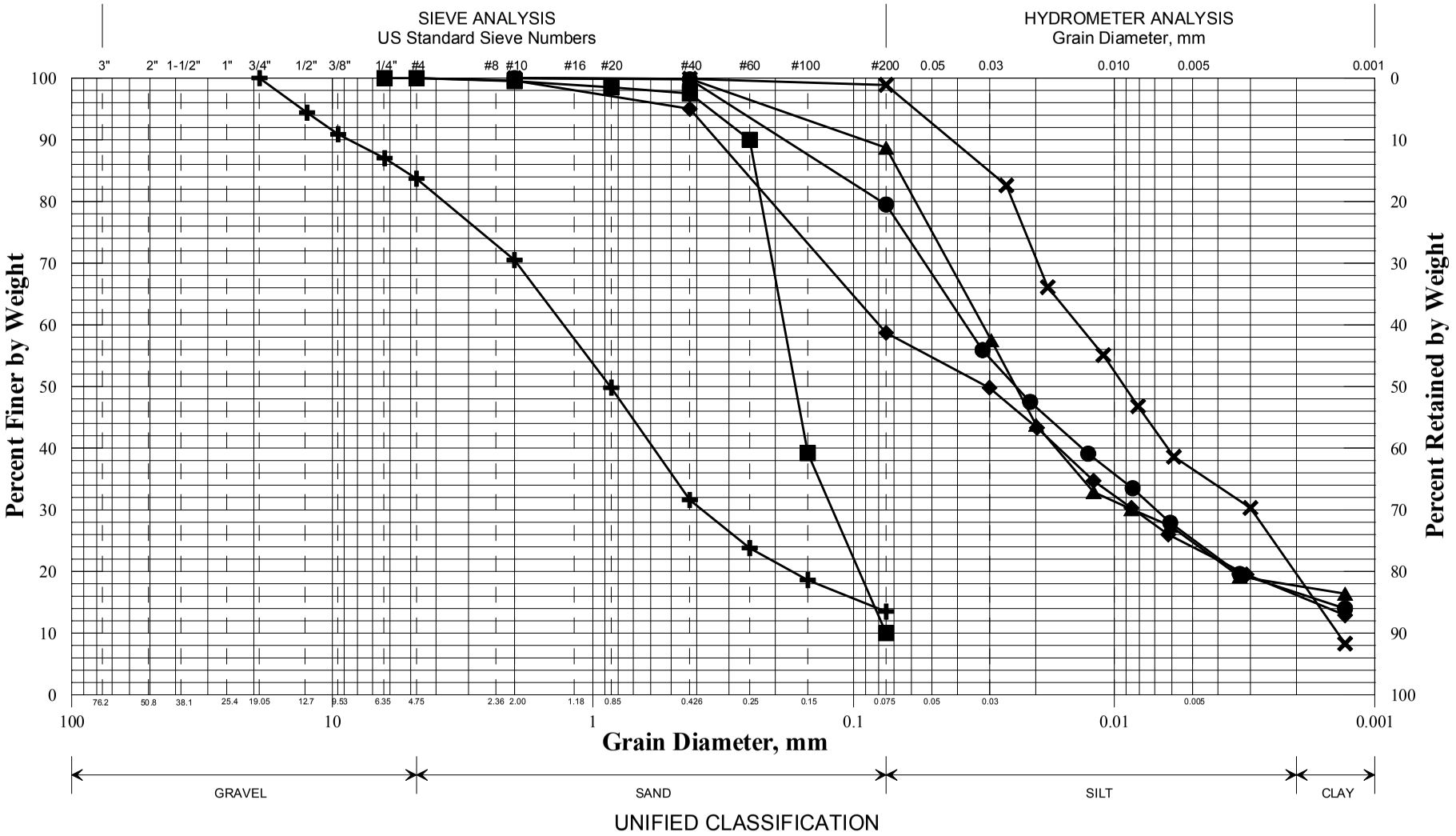

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-104/2DA	6+01.9	11.7 RT.	5.0-6.5	SAND, little gravel, little silt.	4.5			
◆	BB-ACNR-104/2DB	6+01.9	11.7 RT.	6.5-7.0	Sandy SILT, little clay.	20.9			
■	BB-ACNR-104/4D	6+01.9	11.7 RT.	15.0-17.0	SAND, trace silt.	8.6			
●	BB-ACNR-104/5D	6+01.9	11.7 RT.	20.5-22.5	SILT, some sand, little clay.	21.0			
▲	BB-ACNR-104/6D	6+01.9	11.7 RT.	24.0-26.0	SILT, little clay, little sand.	25.5			
✖	BB-ACNR-104/7D	6+01.9	11.7 RT.	29.5-31.5	SILT, little clay, trace sand.	30.7	25	22	3

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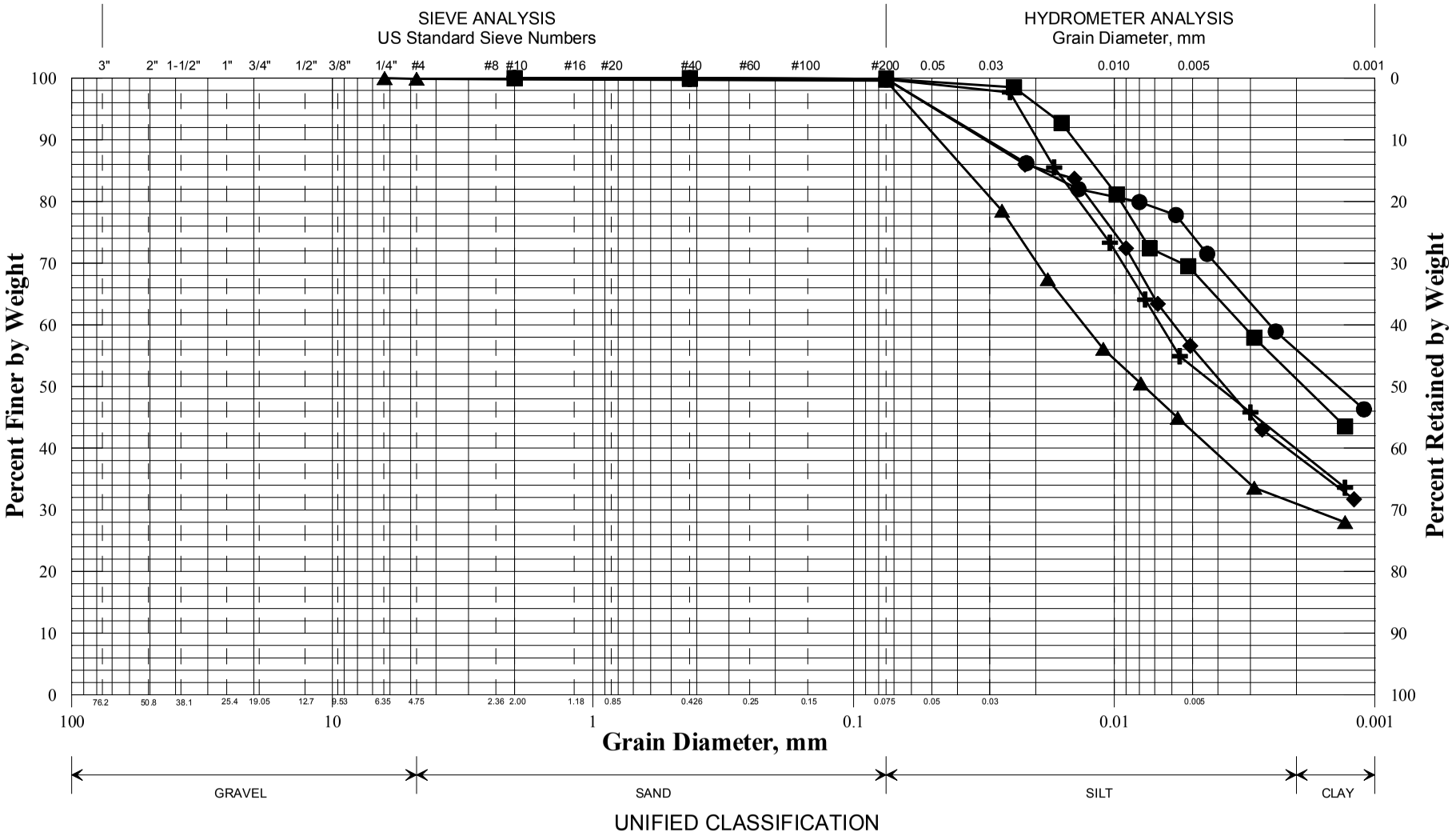

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-ACNR-104/1U	6+01.9	11.7 RT.	34.0-36.0	Clayey SILT, trace sand.	36.7	30	22	8
◆	BB-ACNR-104/8D	6+01.9	11.7 RT.	40.5-42.5	Clayey SILT, trace sand.	31.9	31	19	12
■	BB-ACNR-104/2U	6+01.9	11.7 RT.	44.0-46.0	Silty CLAY, trace sand.	36.9	36	24	12
●	BB-ACNR-104/9D	6+01.9	11.7 RT.	49.5-51.5	Silty CLAY, trace sand.	40.9	39	25	14
▲	BB-ACNR-104/3U	6+01.9	11.7 RT.	54.0-55.0	SILT, some clay, trace sand, trace gravel.	29.5	25	20	5
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Town
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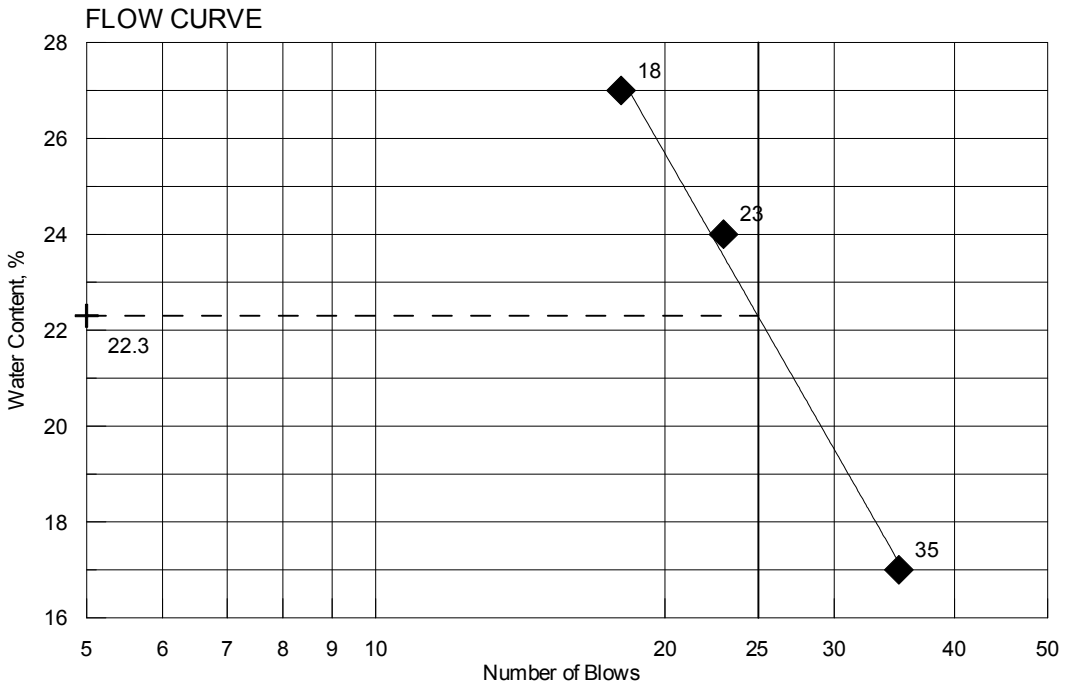
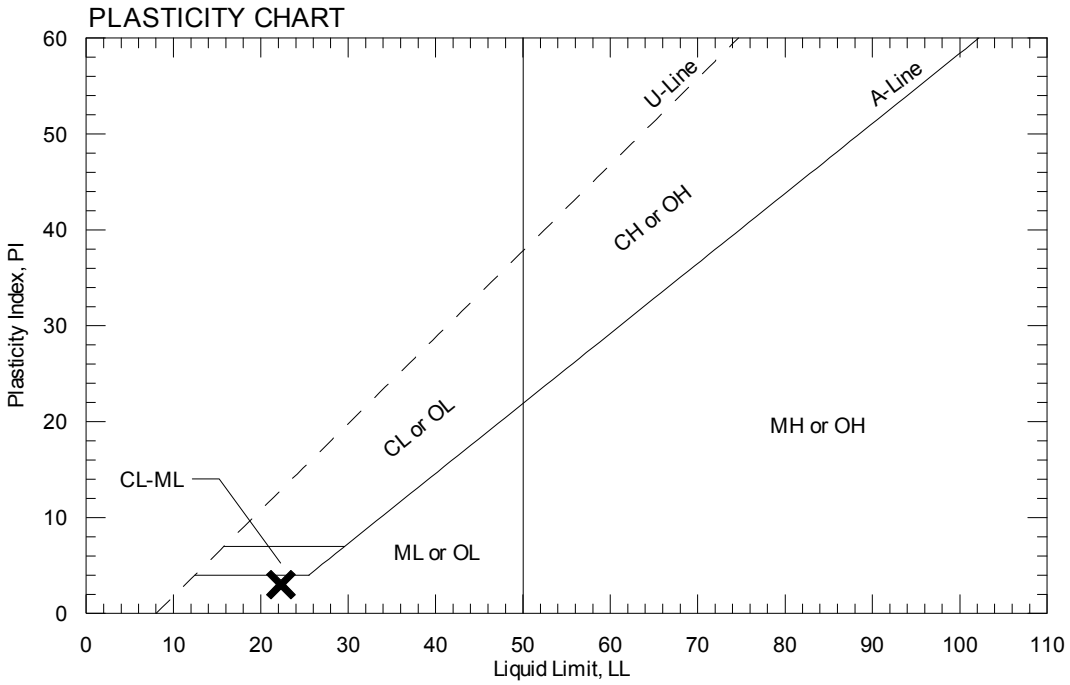
TOWN	Auburn	Reference No.	210617
PIN	015600.00	Water Content, %	30.6
Sampled		Plastic Limit	19
Boring No./Sample No.	BB-ACNR-101/2U	Liquid Limit	22
Station	4+45.9	Plasticity Index	3
Depth	45.0-47.0	Tested By	BBURR



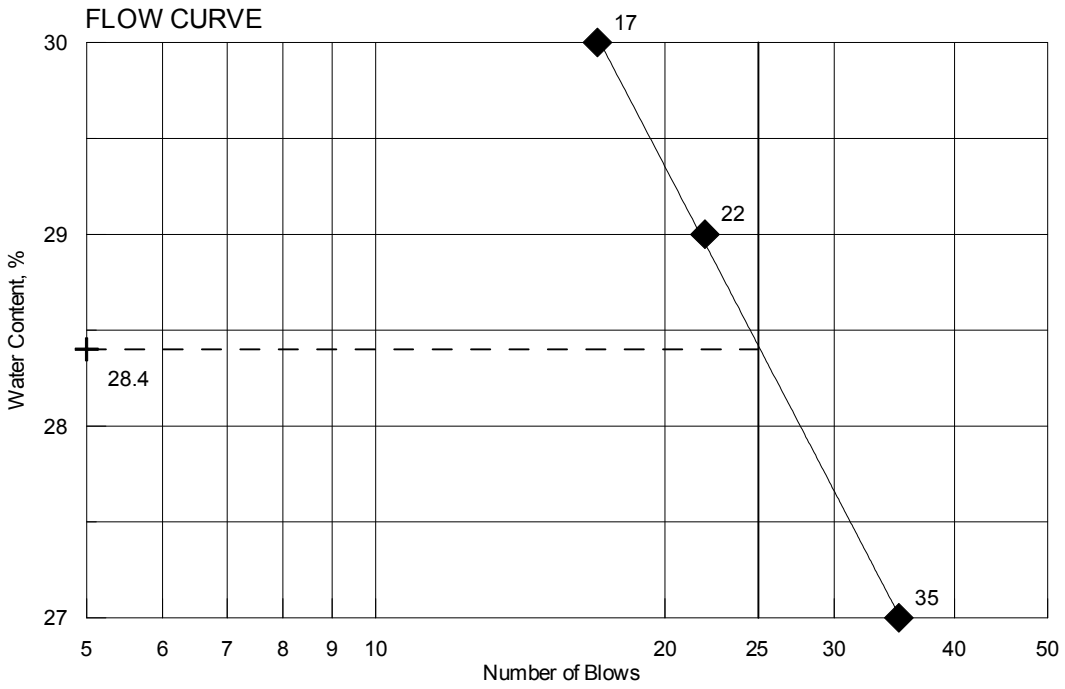


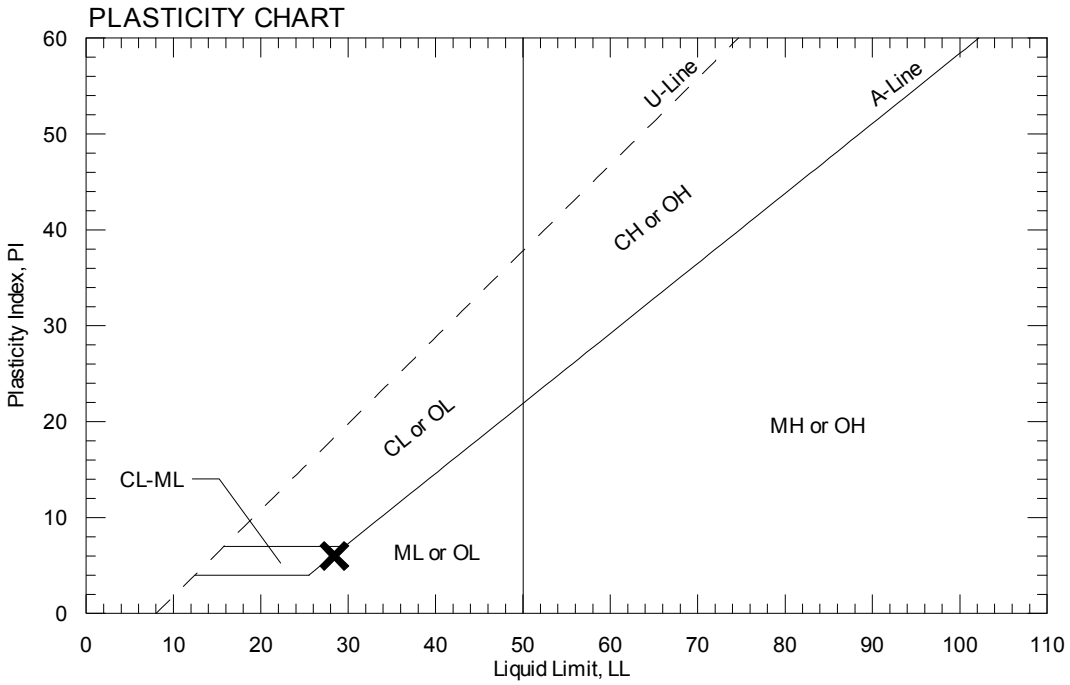
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/15/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210270
PIN	015600.00	Water Content, %	32.4
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-101/9D	Liquid Limit	28
Station	4+45.9	Plasticity Index	6
Depth	50.0-52.0	Tested By	BBURR



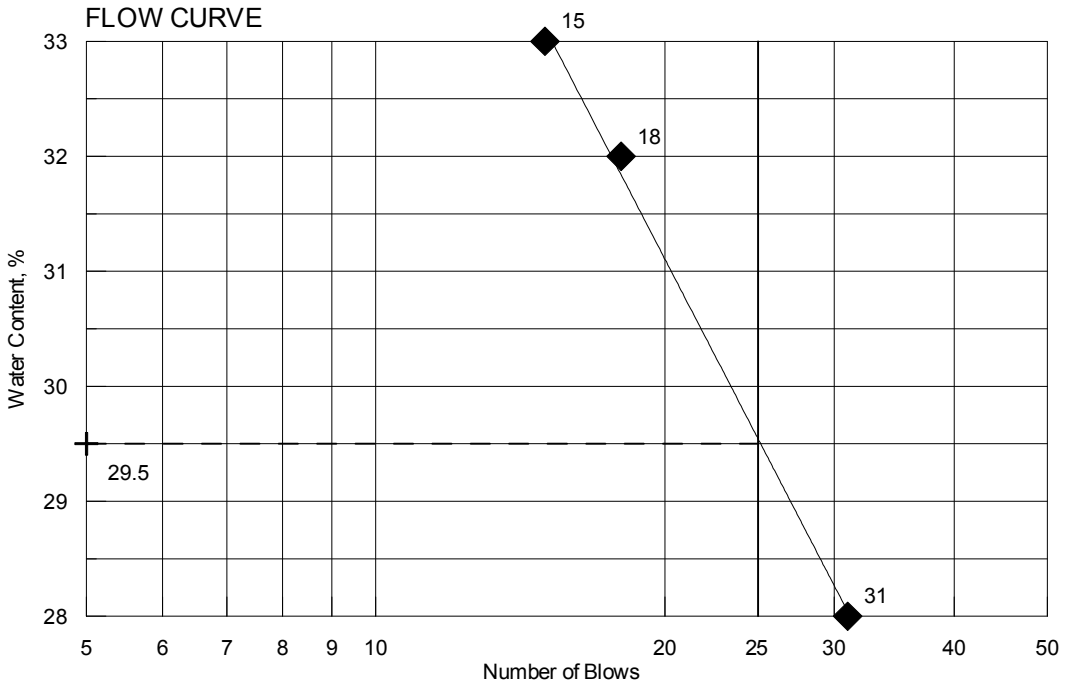


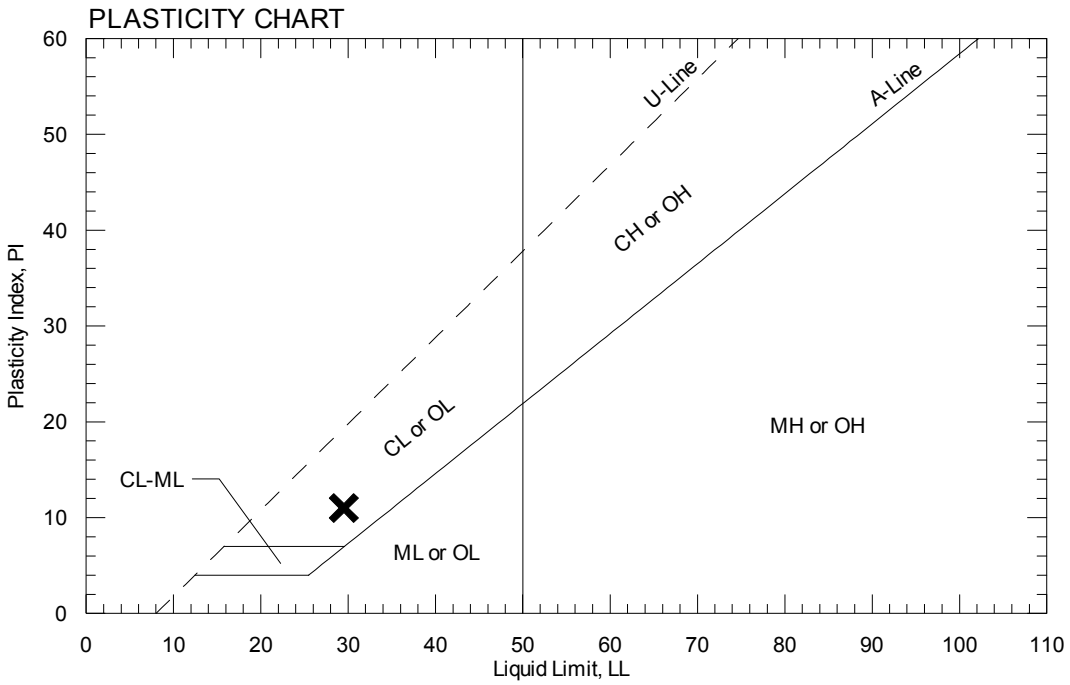
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/8/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210618
PIN	015600.00	Water Content, %	32.8
Sampled	5/20/2008	Plastic Limit	19
Boring No./Sample No.	BB-ACNR-101/3U	Liquid Limit	30
Station	4+45.9	Plasticity Index	11
Depth	55.0-57.0	Tested By	BBURR



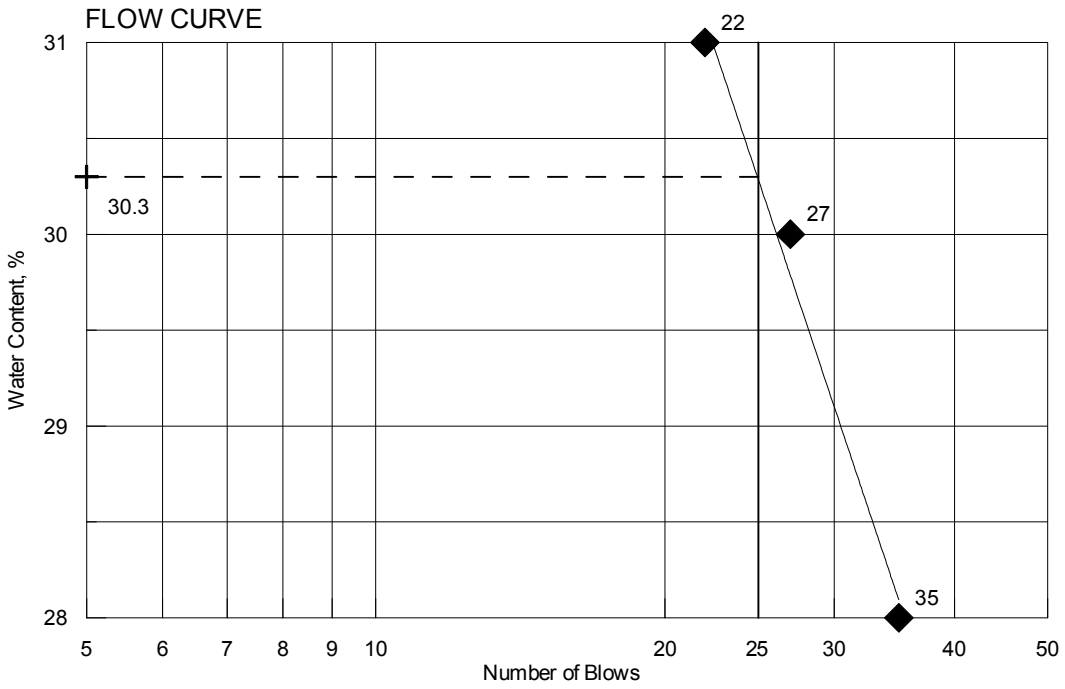


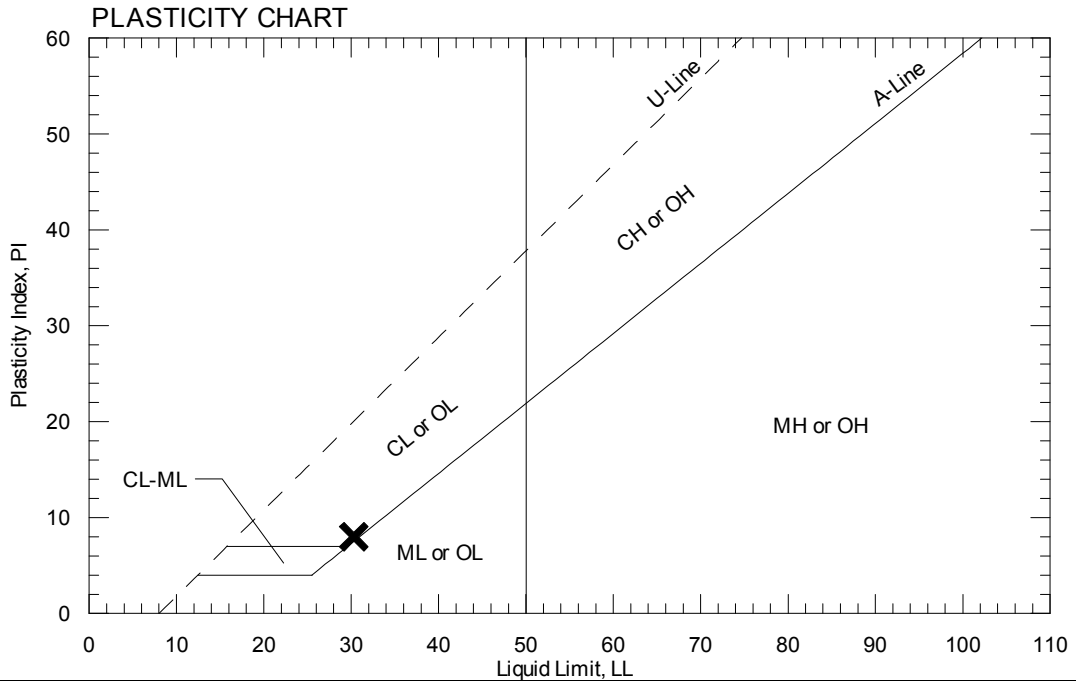
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/30/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210271
PIN	015600.00	Water Content, %	30.9
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-101/10D	Liquid Limit	30
Station	4+45.9	Plasticity Index	8
Depth	60.0-62.0	Tested By	BBURR



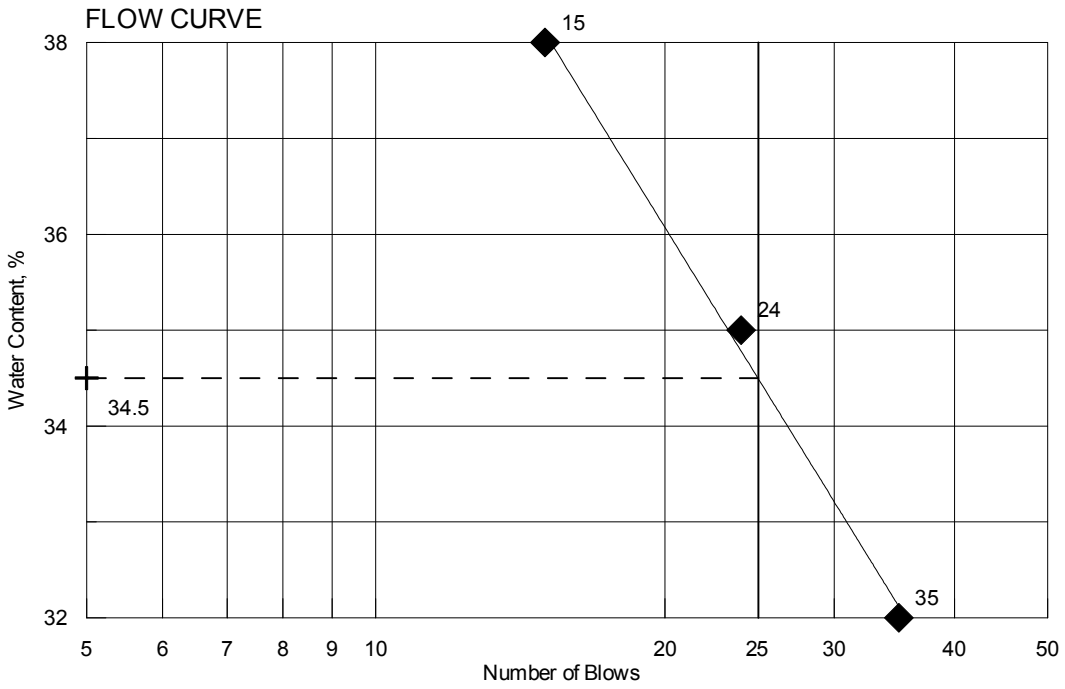


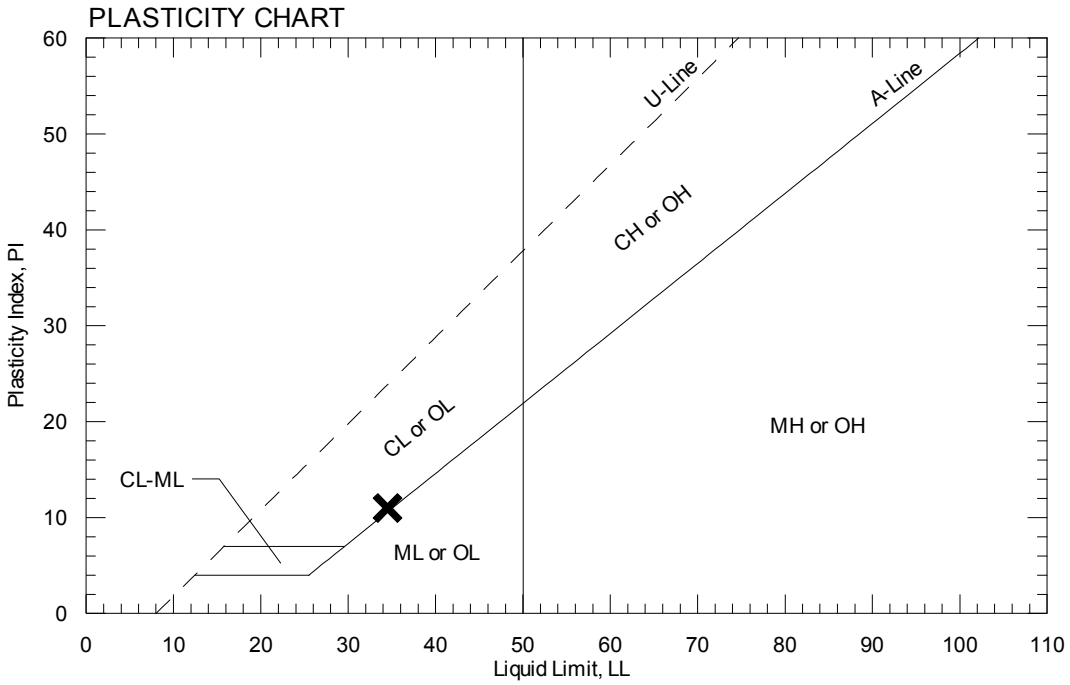
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210619
PIN	015600.00	Water Content, %	38
Sampled	5/20/2008	Plastic Limit	24
Boring No./Sample No.	BB-ACNR-101/4U	Liquid Limit	35
Station	4+45.9	Plasticity Index	11
Depth	65.0-67.0	Tested By	BBURR



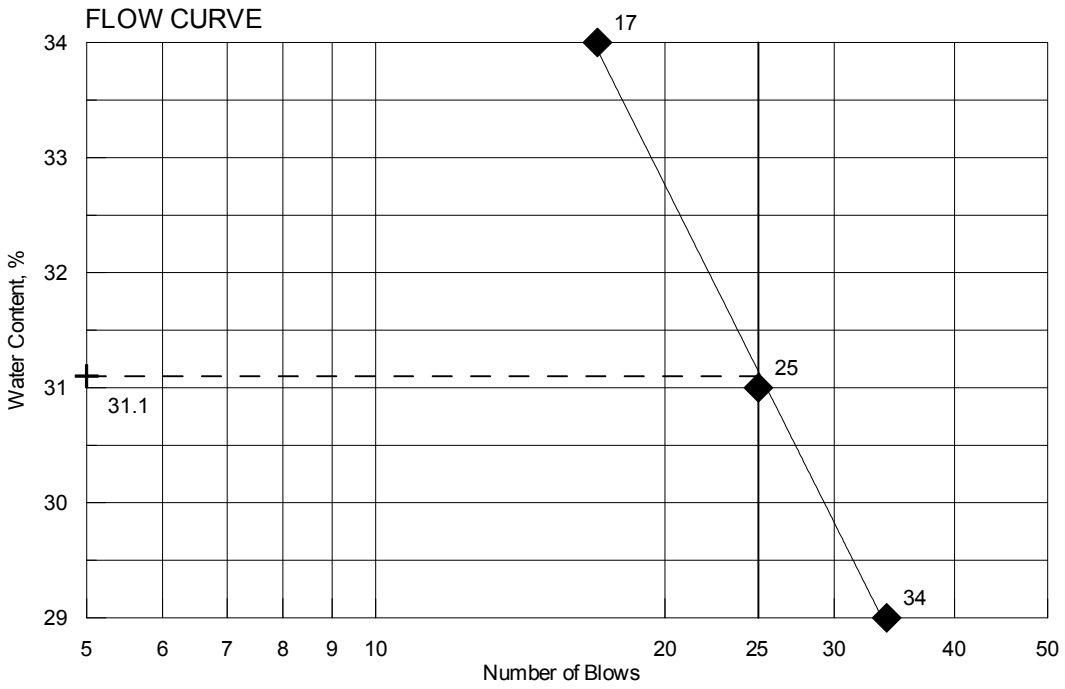


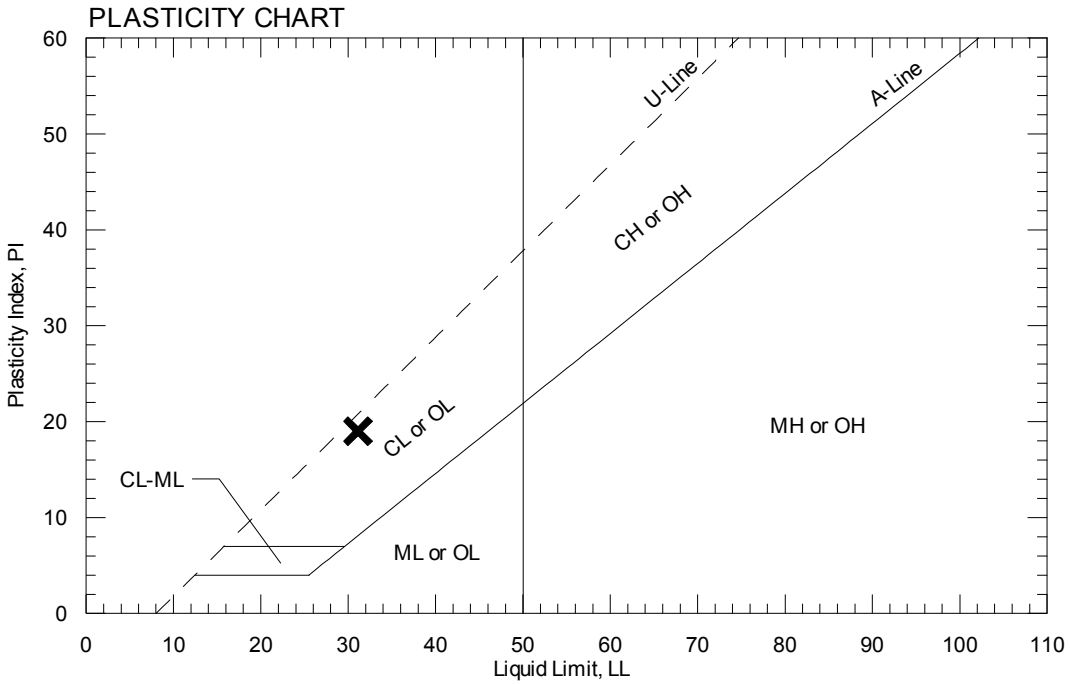
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/16/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210272
PIN	015600.00	Water Content, %	34.4
Sampled		Plastic Limit	12
Boring No./Sample No.	BB-ACNR-101/11D	Liquid Limit	31
Station	4+45.9	Plasticity Index	19
Depth	70.0-72.0	Tested By	BBURR



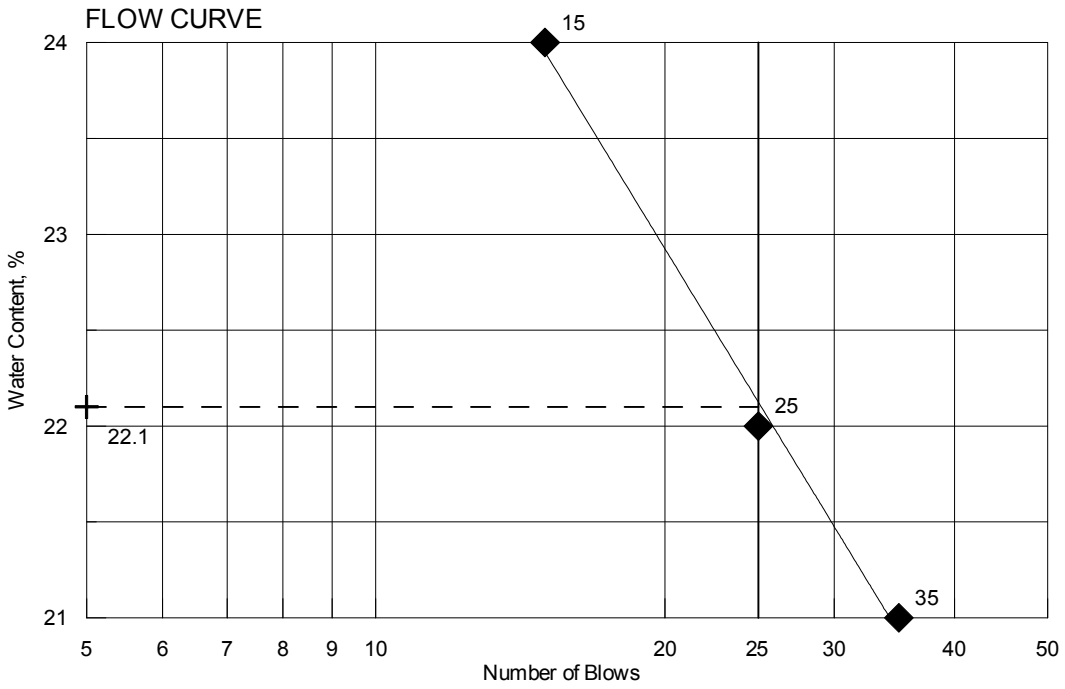


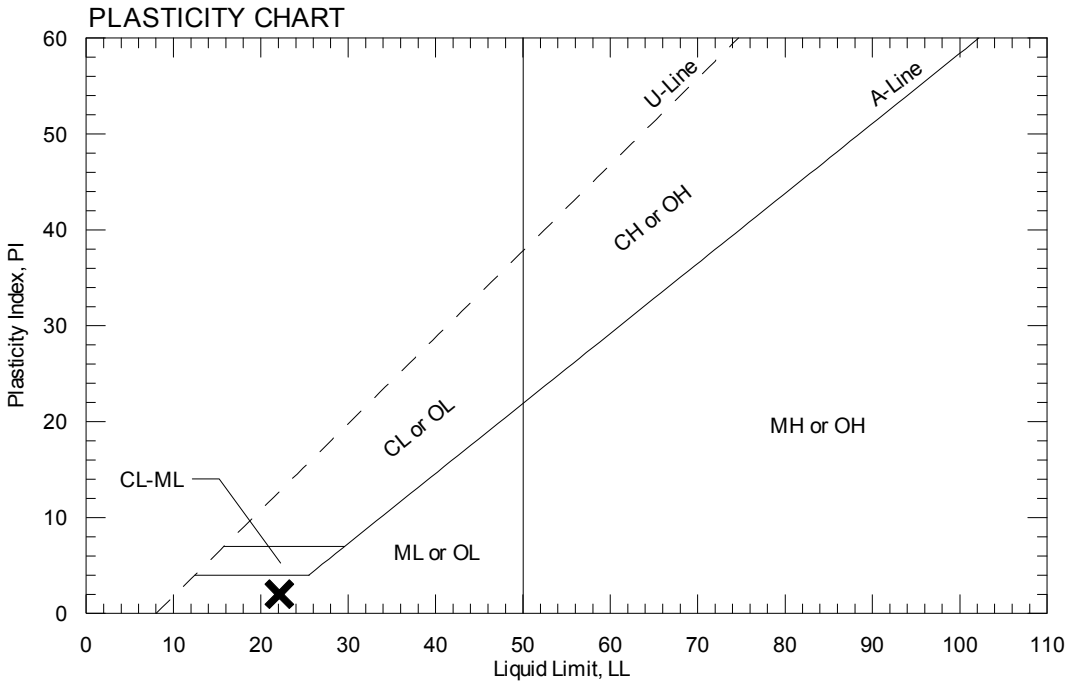
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210620
PIN	015600.00	Water Content, %	26.4
Sampled	5/20/2008	Plastic Limit	20
Boring No./Sample No.	BB-ACNR-101/5U	Liquid Limit	22
Station	4+45.9	Plasticity Index	2
Depth	75.0-77.0	Tested By	BBURR



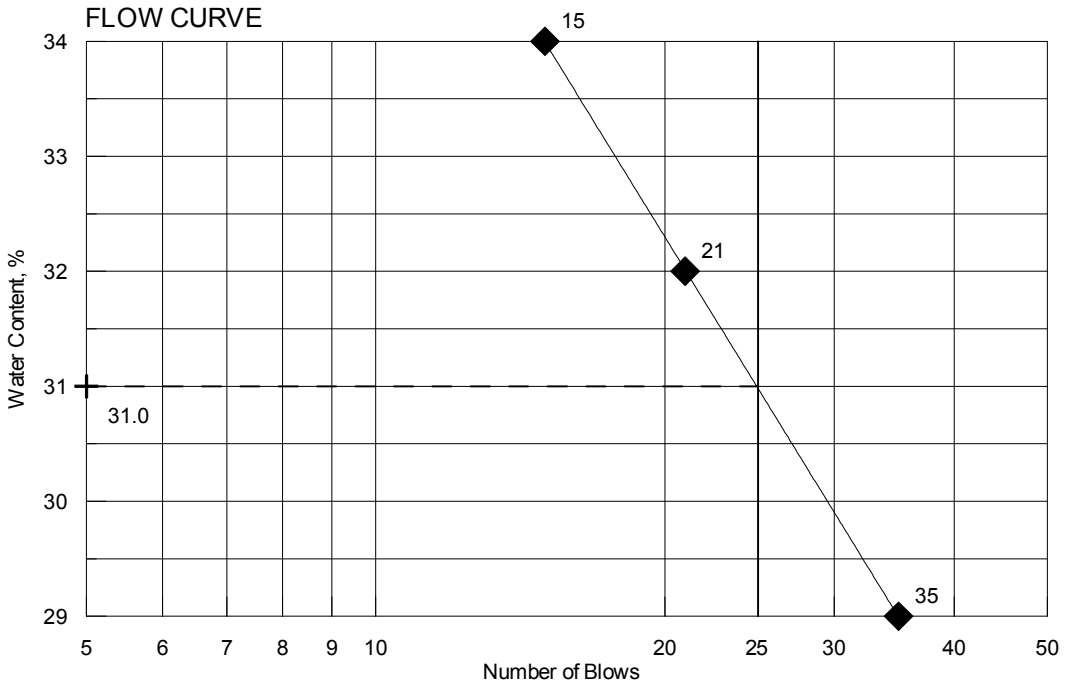


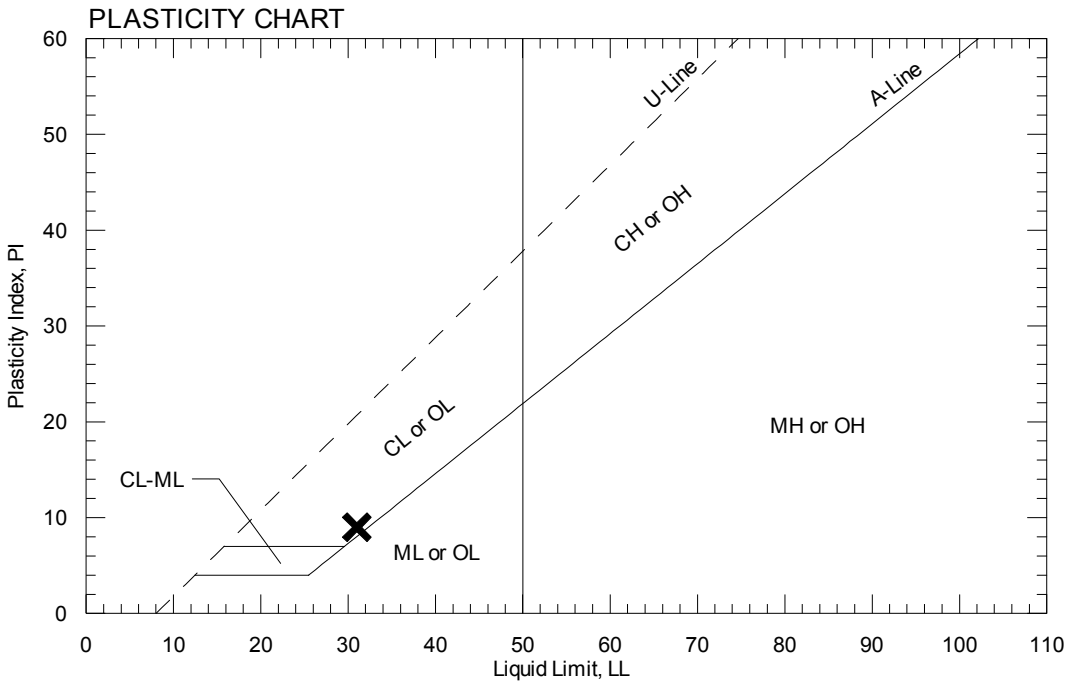
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/30/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210621
PIN	015600.00	Water Content, %	35.2
Sampled	5/20/2008	Plastic Limit	22
Boring No./Sample No.	BB-ACNR-101/6U	Liquid Limit	31
Station	4+45.9	Plasticity Index	9
Depth	85.0-87.0	Tested By	BBURR



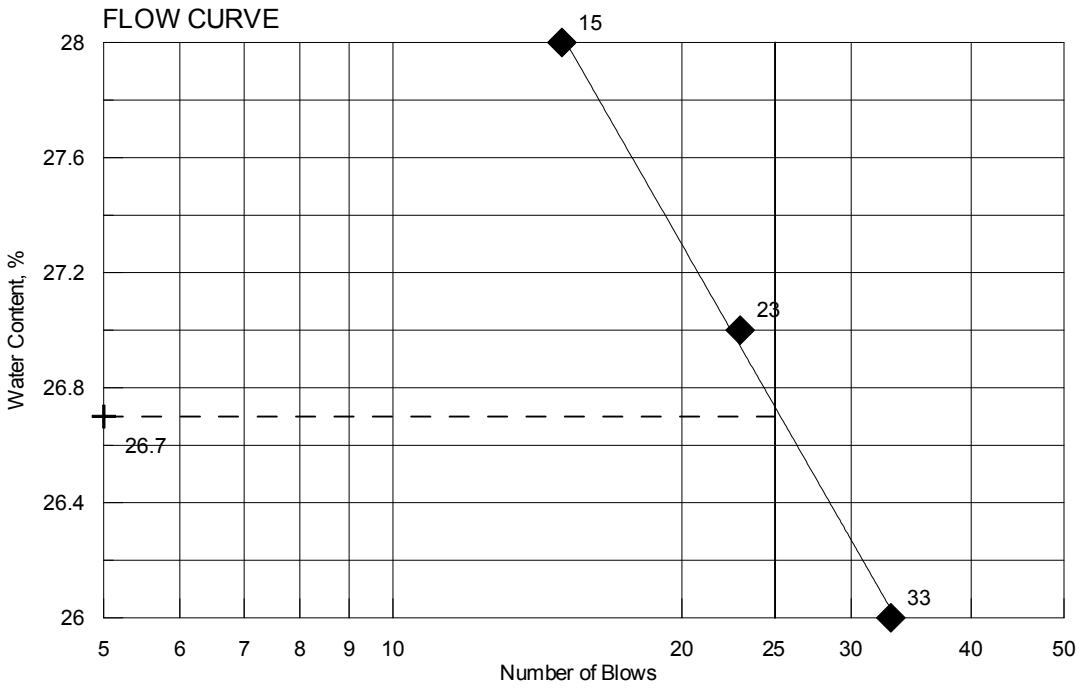


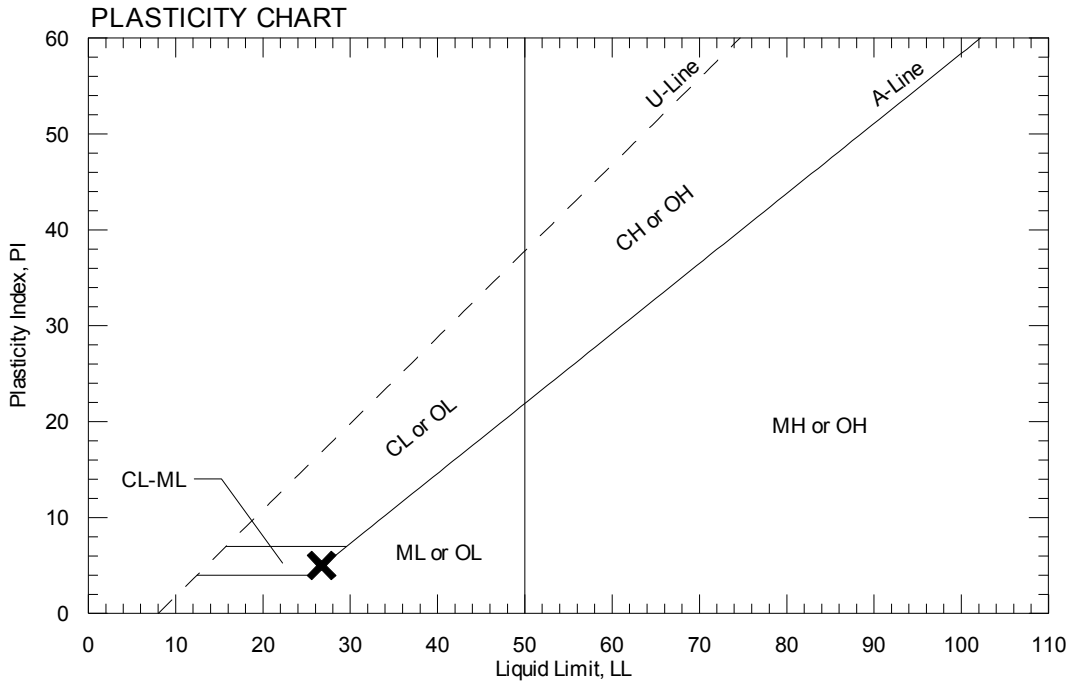
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/4/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210296
PIN	015600.00	Water Content, %	32.9
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-102/3D-A	Liquid Limit	27
Station	4+91.6	Plasticity Index	5
Depth	14.0-15.0	Tested By	BBURR



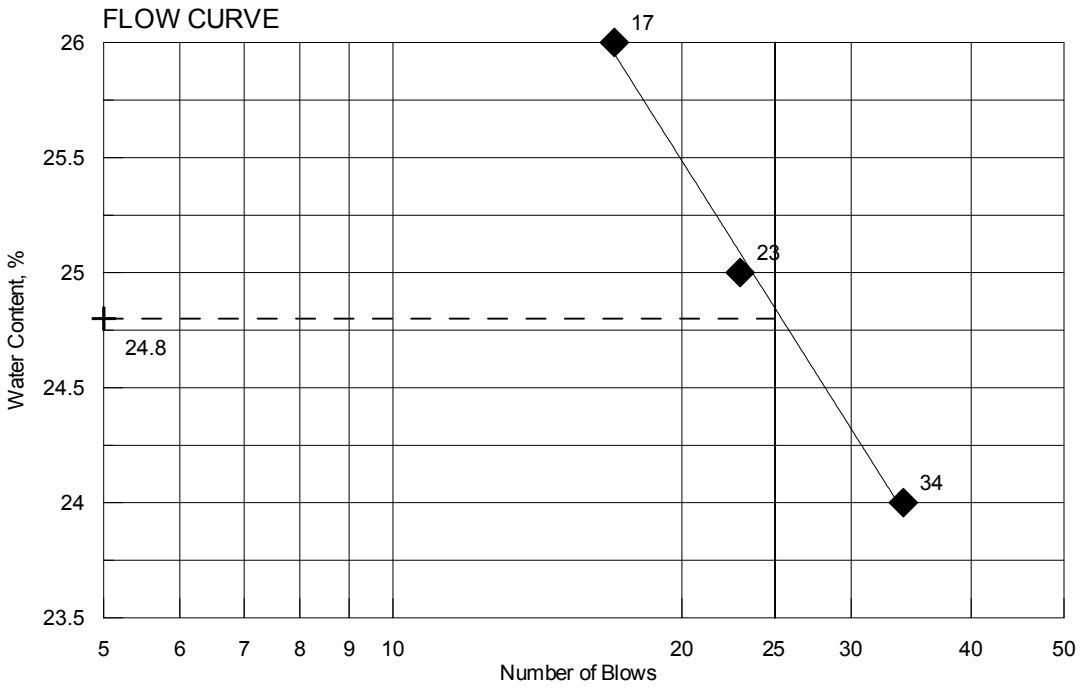


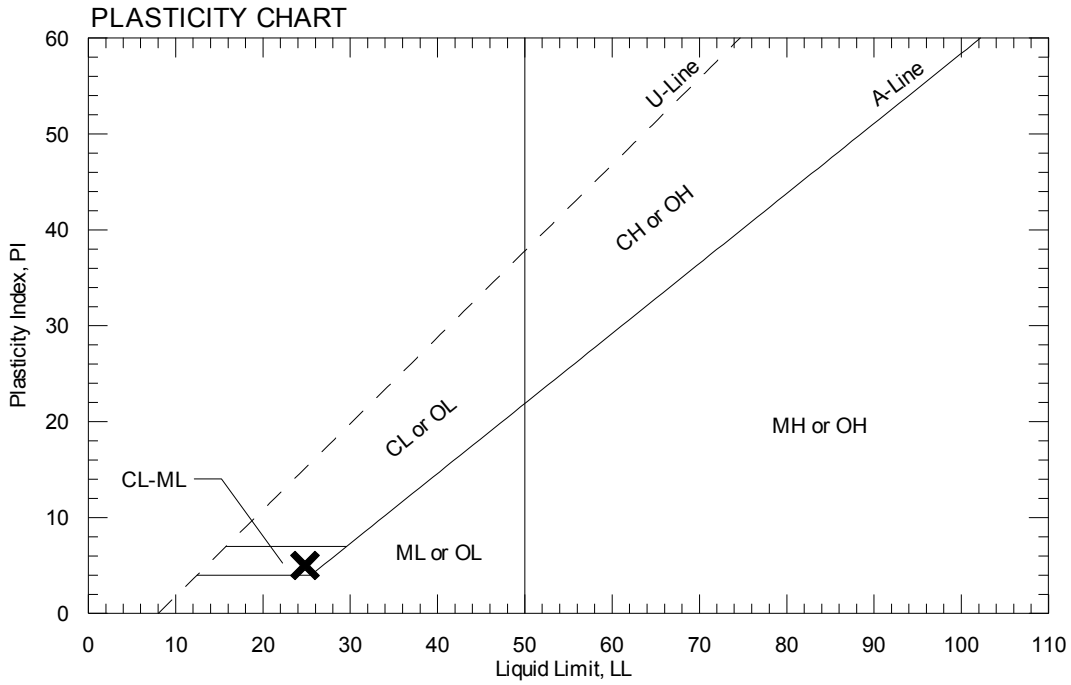
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210297
PIN	015600.00	Water Content, %	33.5
Sampled		Plastic Limit	20
Boring No./Sample No.	BB-ACNR-102/3D-B	Liquid Limit	25
Station	4+91.6	Plasticity Index	5
Depth	15.0-16.0	Tested By	BBURR



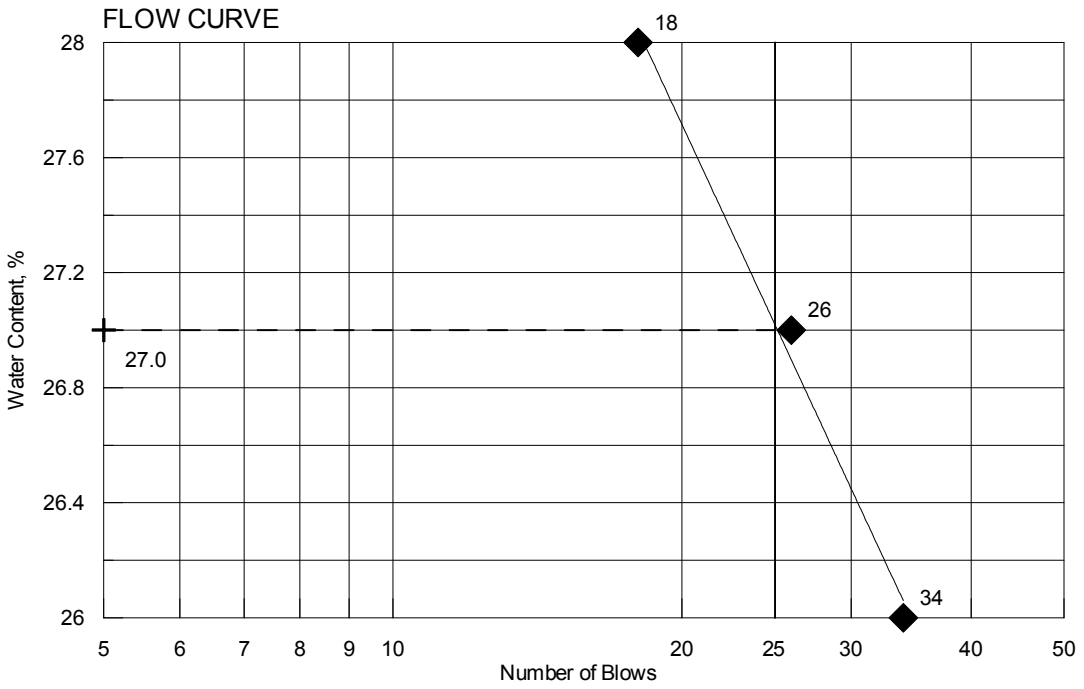


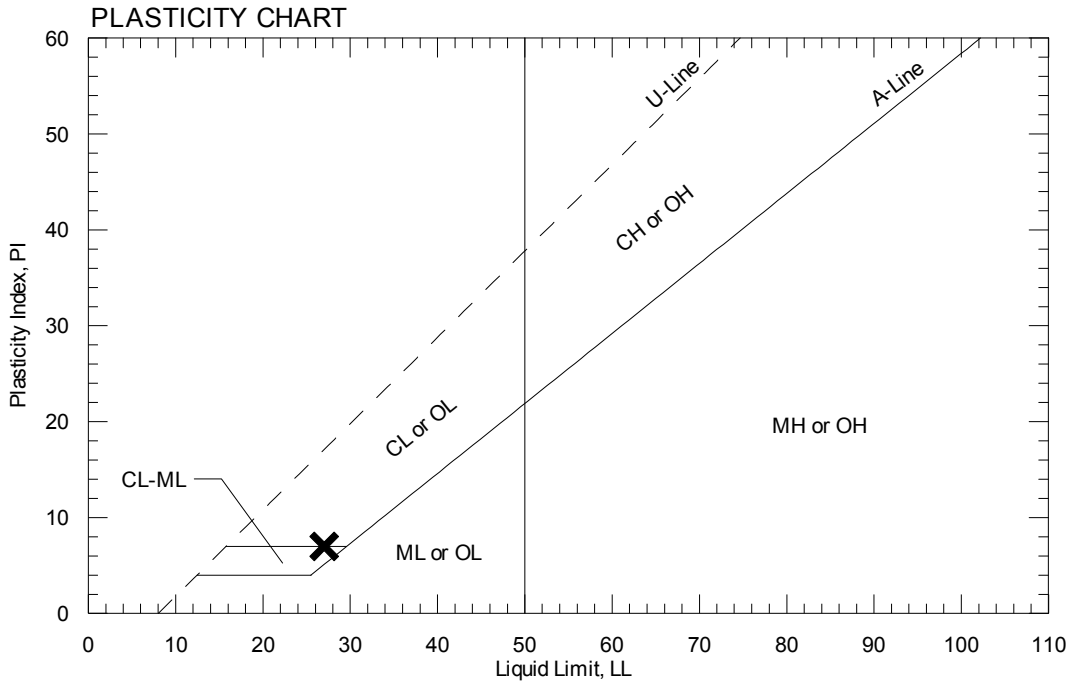
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/8/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210295
PIN	015600.00	Water Content, %	31.5
Sampled		Plastic Limit	20
Boring No./Sample No.	BB-ACNR-102/4D	Liquid Limit	27
Station	4+91.6	Plasticity Index	7
Depth	24.0-26.0	Tested By	BBURR



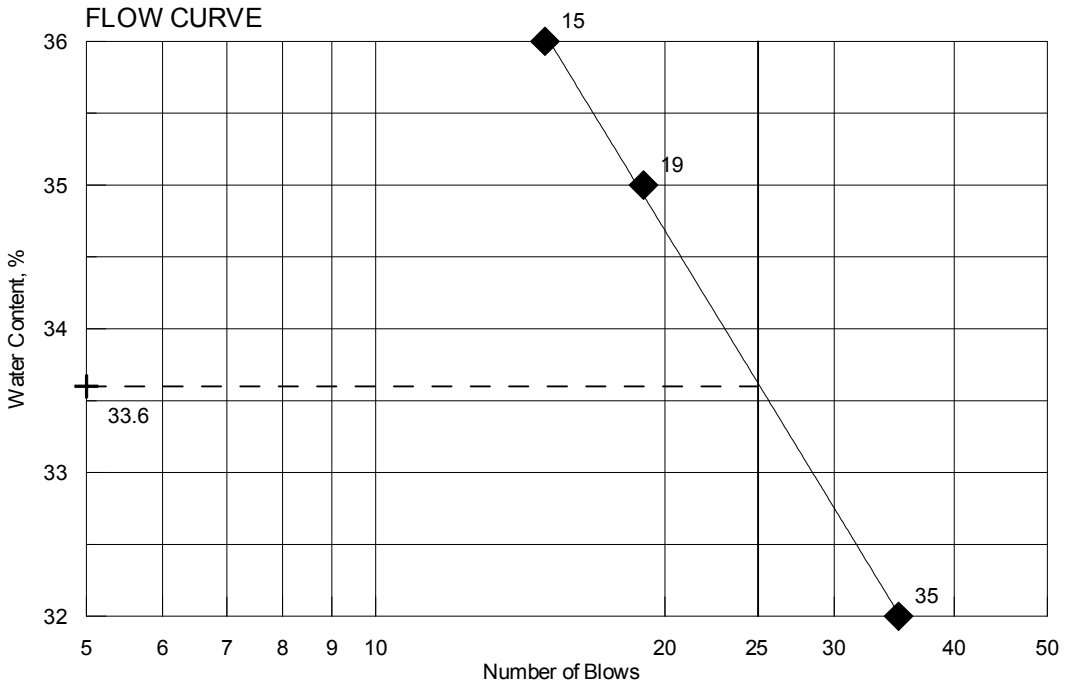


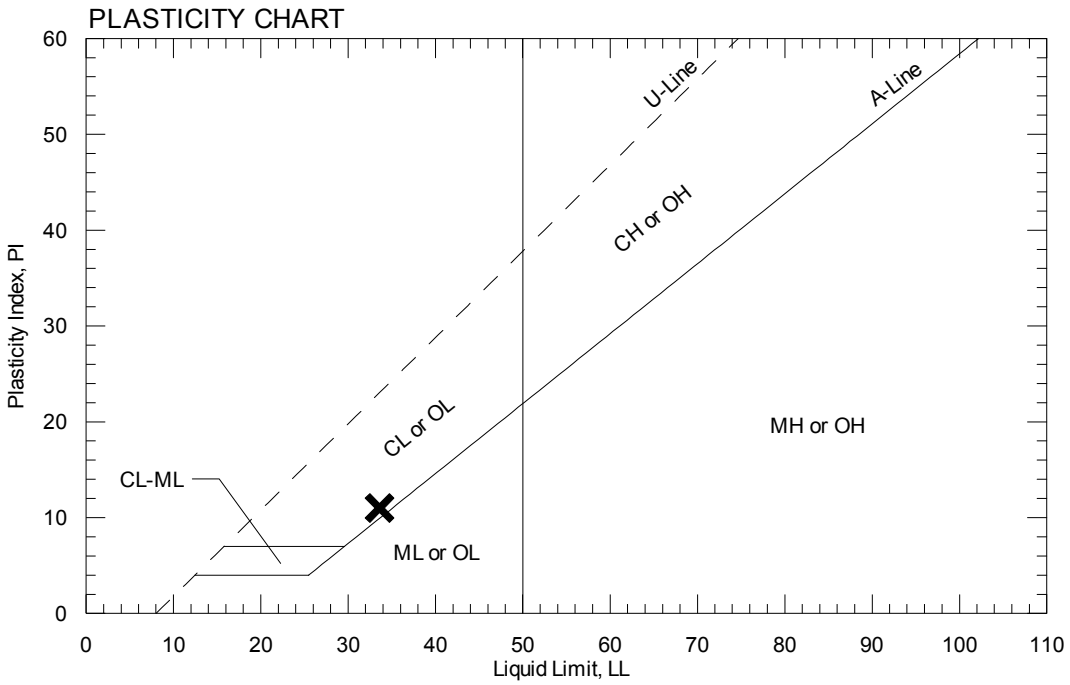
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/10/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210623
PIN	015600.00	Water Content, %	29.8
Sampled	5/20/2008	Plastic Limit	23
Boring No./Sample No.	BB-ACNR-102/2U	Liquid Limit	34
Station	4+91.6	Plasticity Index	11
Depth	29.0-31.0	Tested By	BBURR



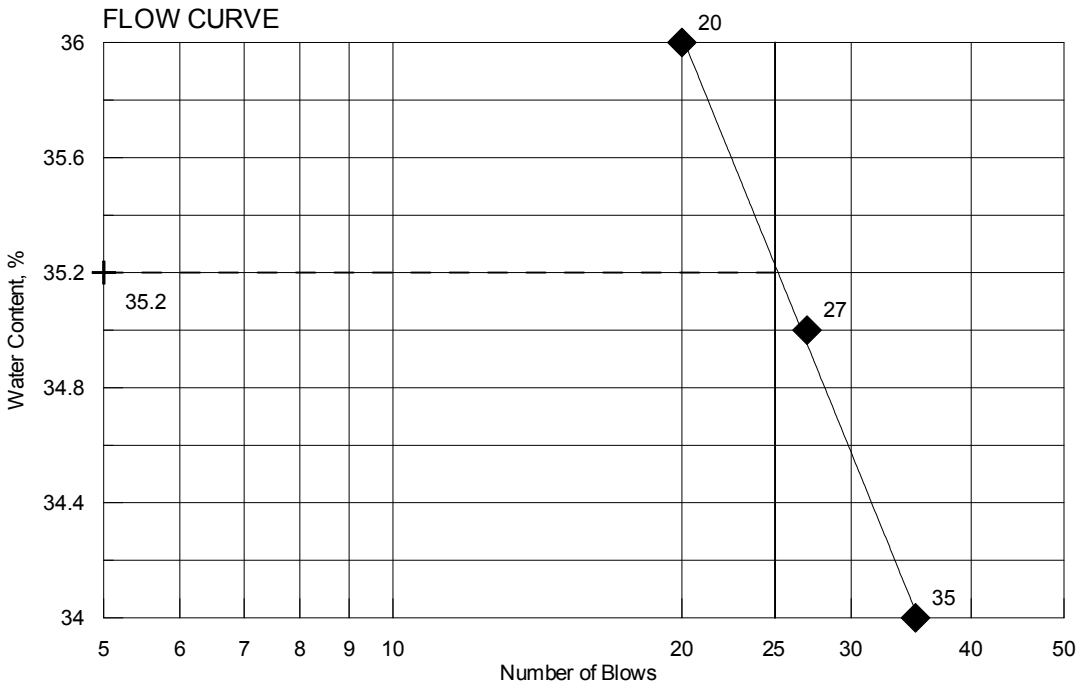


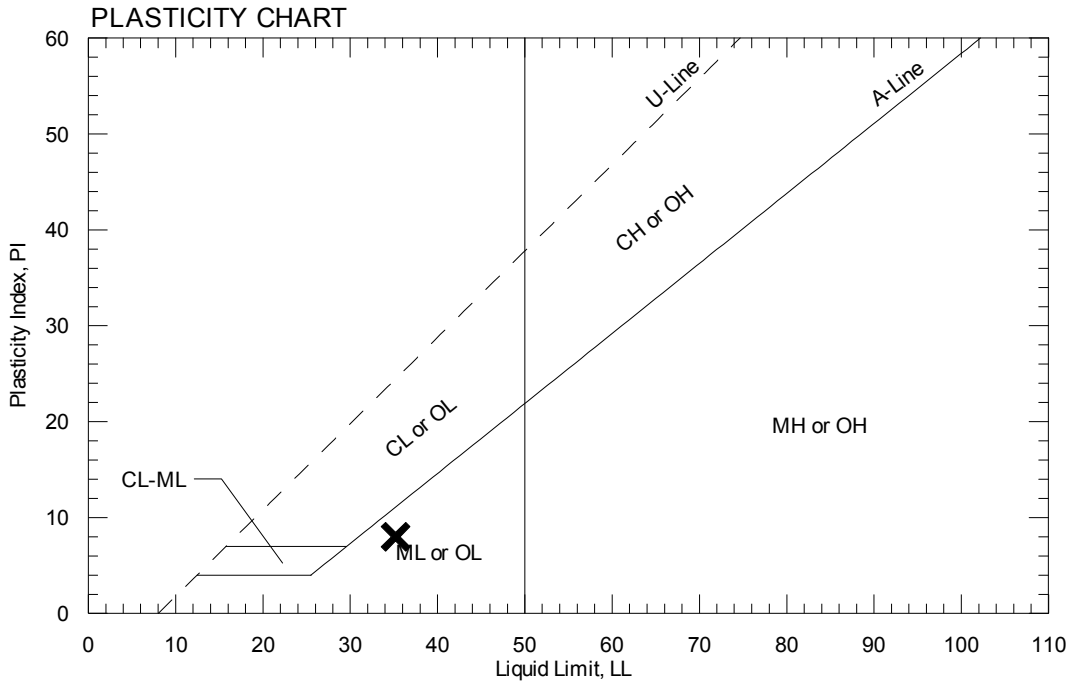
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/29/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210298
PIN	015600.00	Water Content, %	35.4
Sampled		Plastic Limit	27
Boring No./Sample No.	BB-ACNR-102/6D	Liquid Limit	35
Station	4+91.6	Plasticity Index	8
Depth	39.0-41.0	Tested By	BBURR



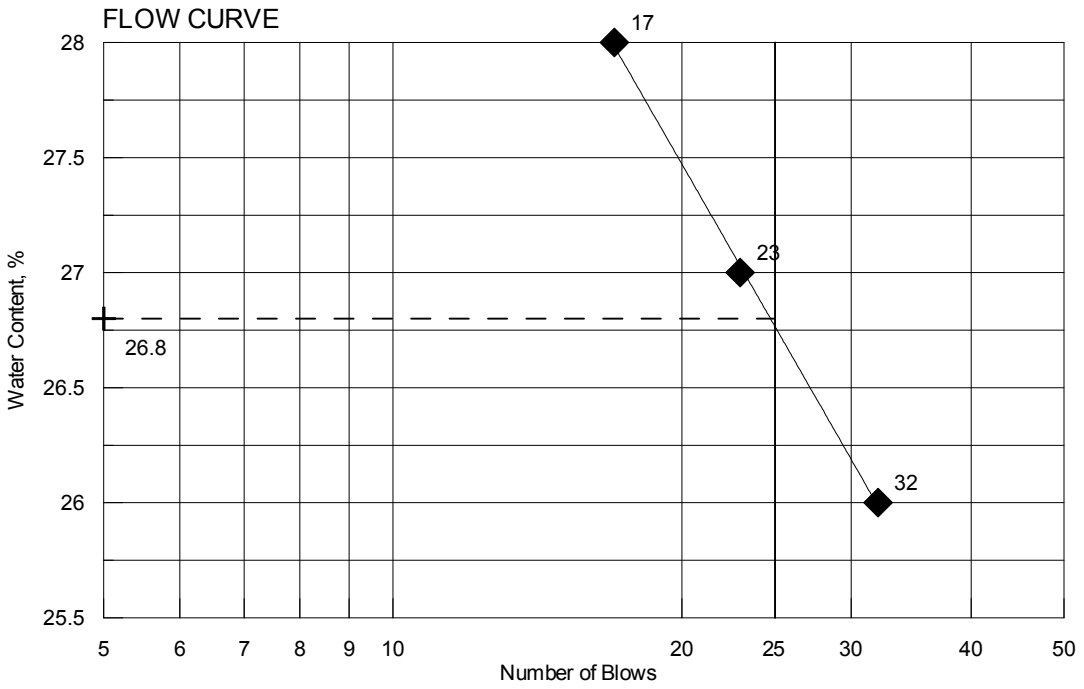


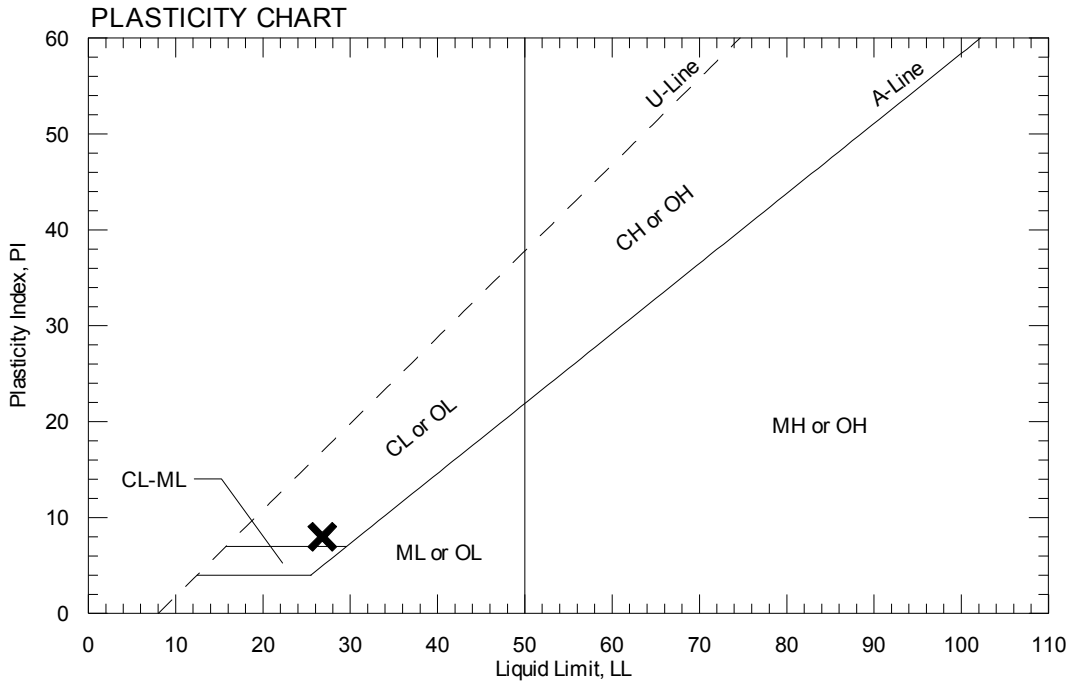
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/8/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210624
PIN	015600.00	Water Content, %	38.4
Sampled	5/20/2008	Plastic Limit	19
Boring No./Sample No.	BB-ACNR-102/3U	Liquid Limit	27
Station	4+91.6	Plasticity Index	8
Depth	41.0-43.0	Tested By	BBURR



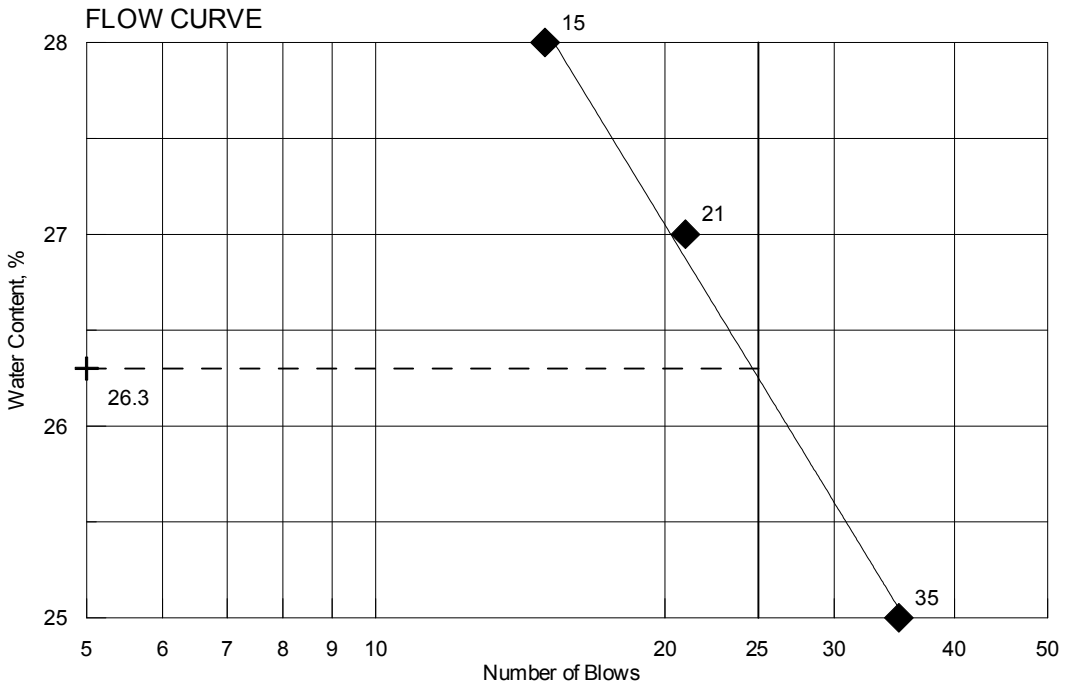


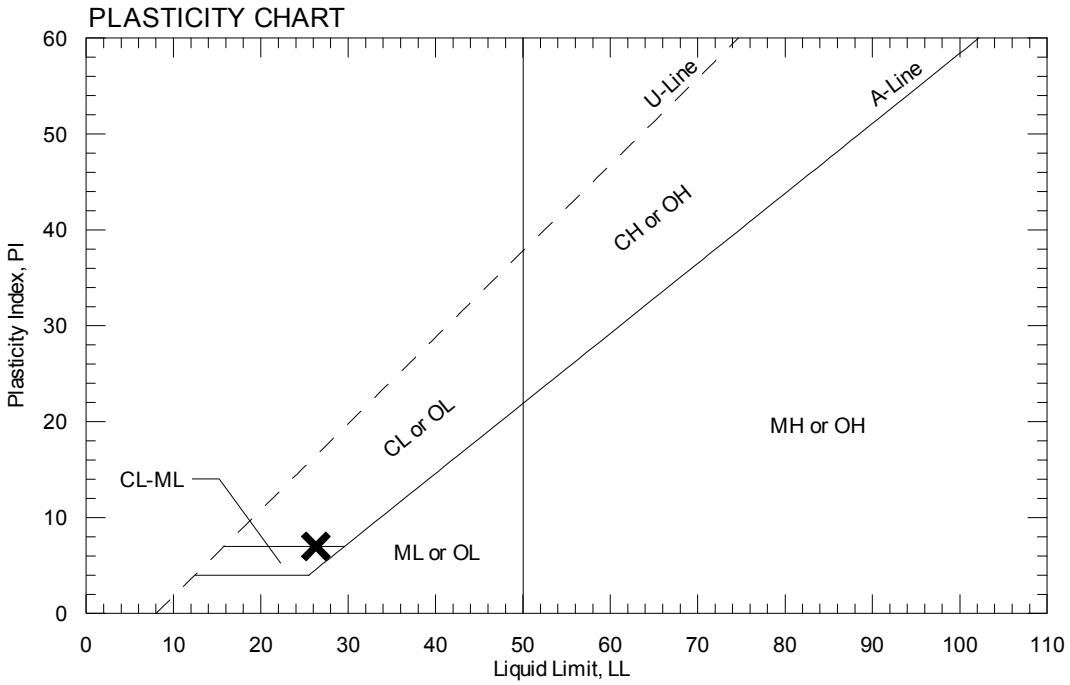
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/30/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210625
PIN	015600.00	Water Content, %	30.7
Sampled	5/20/2008	Plastic Limit	19
Boring No./Sample No.	BB-ACNR-102/4U	Liquid Limit	26
Station	4+91.6	Plasticity Index	7
Depth	49.0-51.0	Tested By	BBURR





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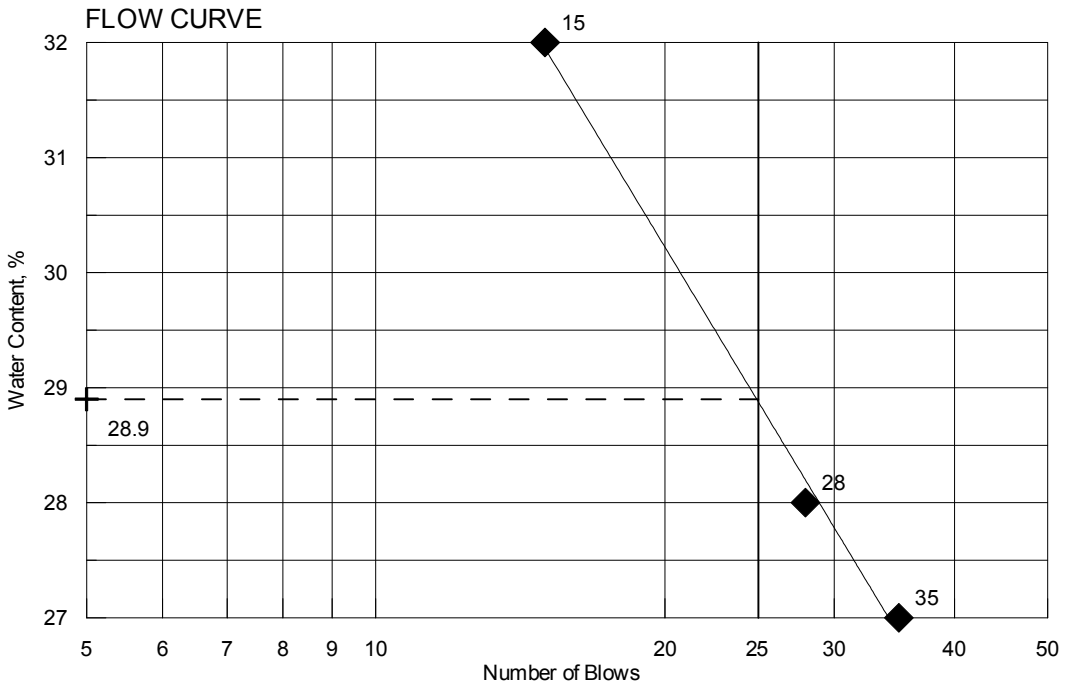
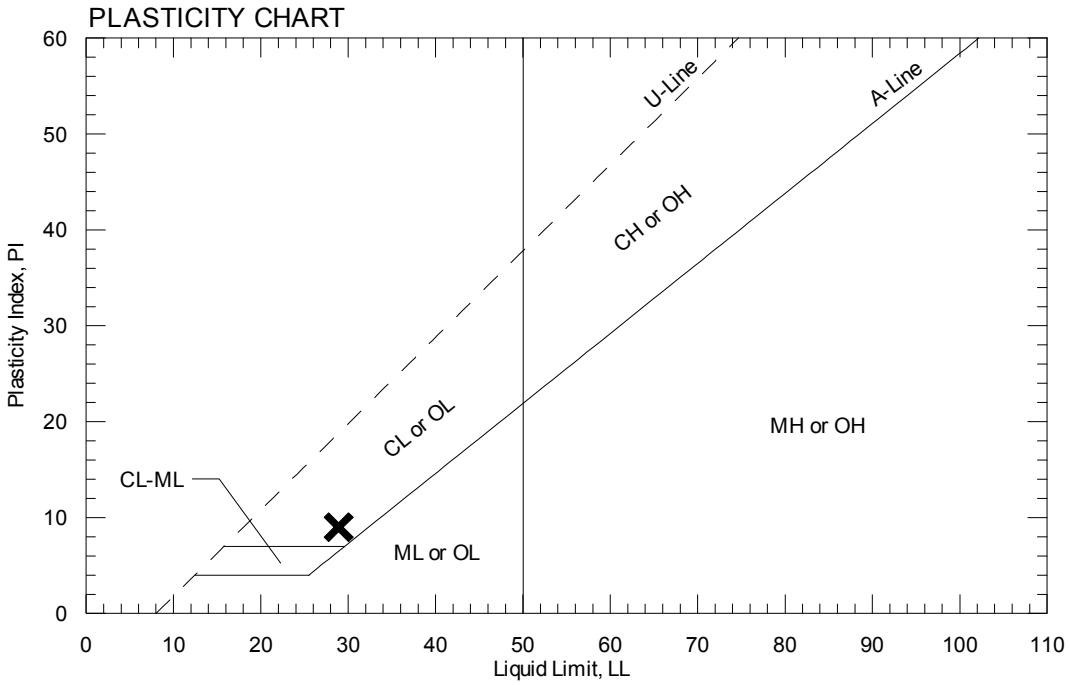
Reported by: **FOGG, BRIAN** Date Reported: **8/5/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210299
PIN	015600.00	Water Content, %	30.6
Sampled		Plastic Limit	20
Boring No./Sample No.	BB-ACNR-102/8D	Liquid Limit	29
Station	4+91.6	Plasticity Index	9
Depth	54.0-56.0	Tested By	BBURR



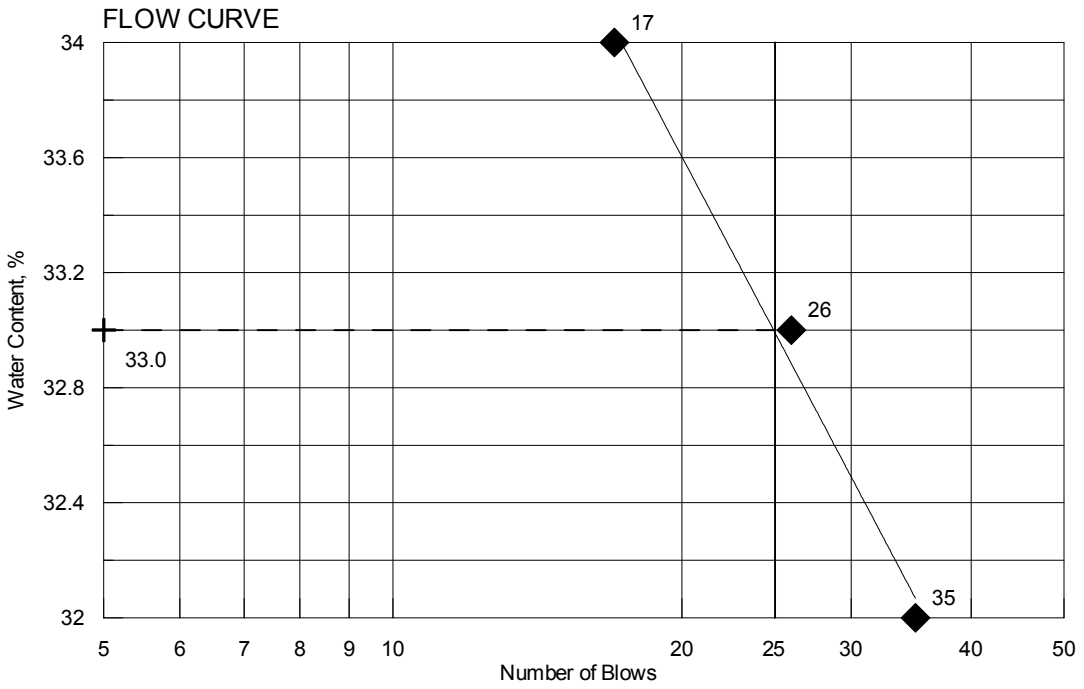


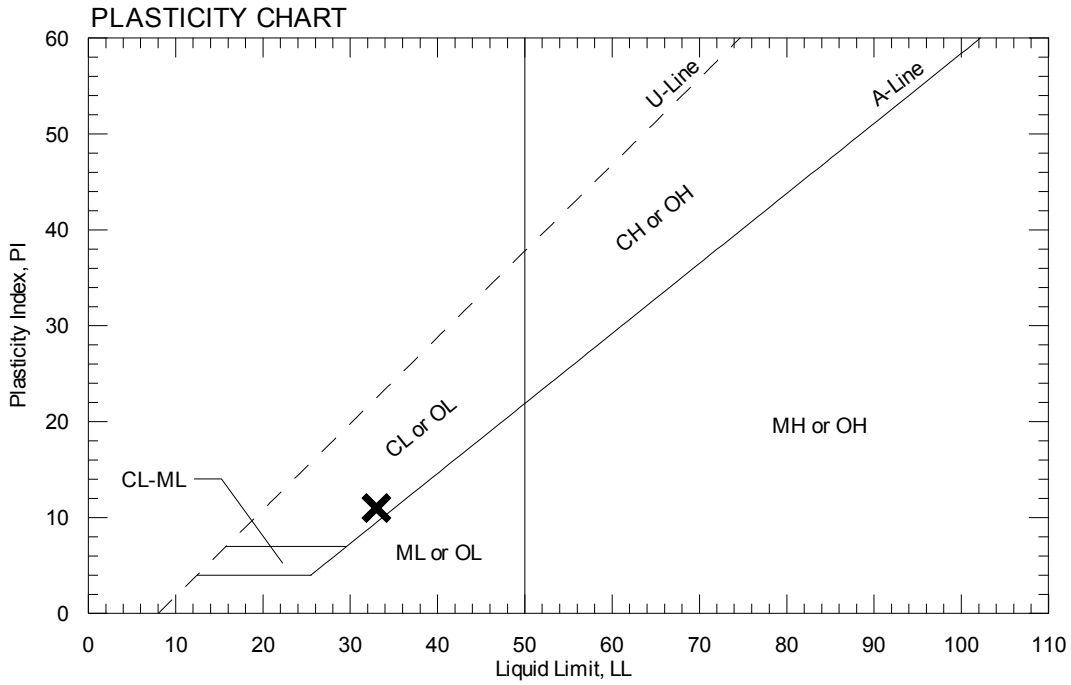
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210626
PIN	015600.00	Water Content, %	36.8
Sampled	5/20/2008	Plastic Limit	22
Boring No./Sample No.	BB-ACNR-102/5U	Liquid Limit	33
Station	4+91.6	Plasticity Index	11
Depth	59.0-61.0	Tested By	BBURR



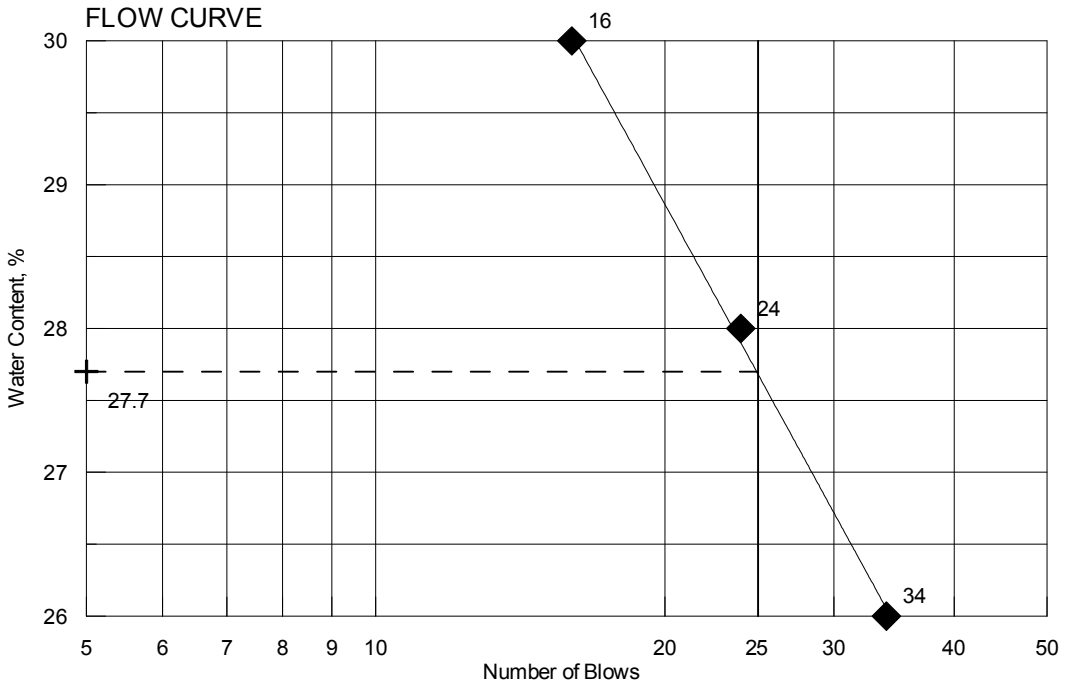


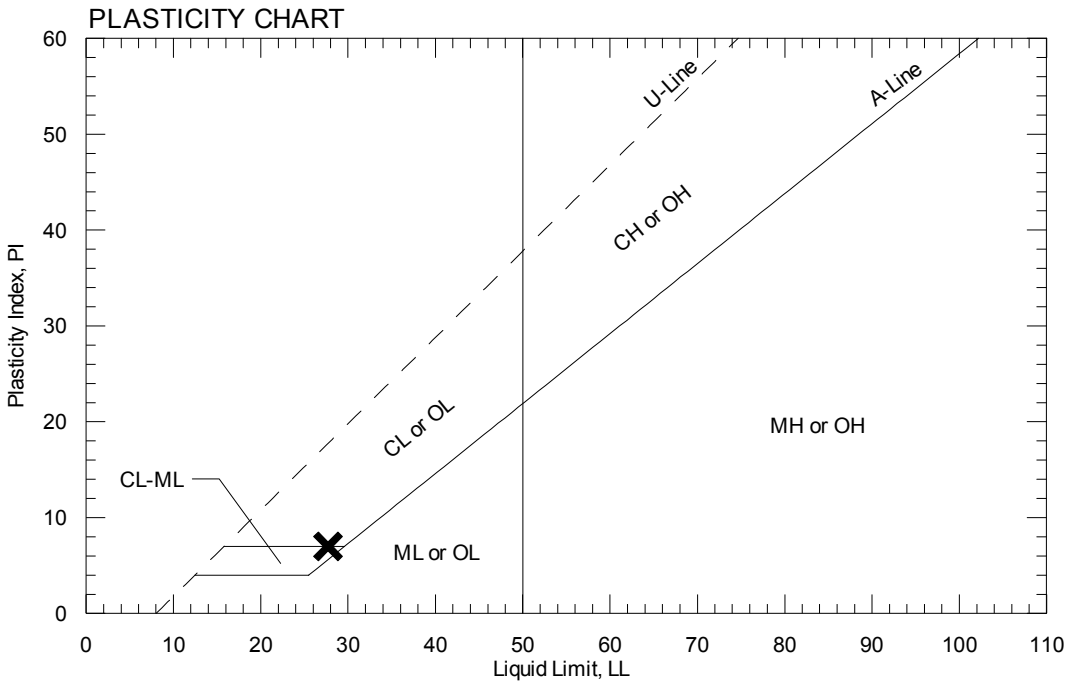
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/5/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210627
PIN	015600.00	Water Content, %	34.3
Sampled	5/20/2008	Plastic Limit	21
Boring No./Sample No.	BB-ACNR-103/1U	Liquid Limit	28
Station	5+60.6	Plasticity Index	7
Depth	15.0-17.0	Tested By	BBURR



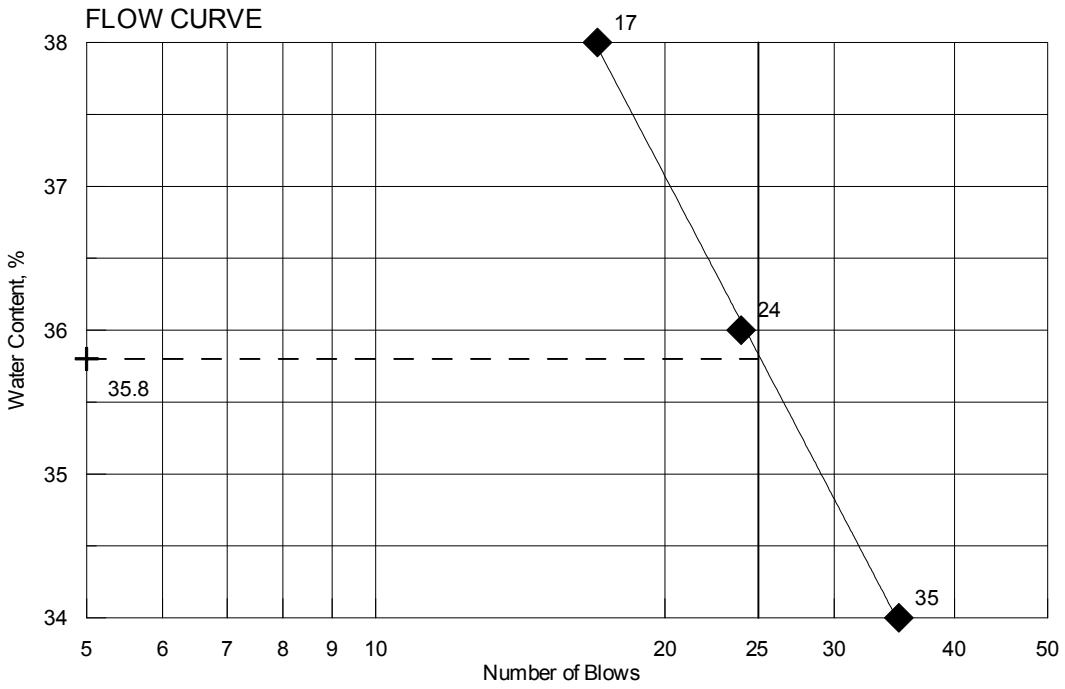


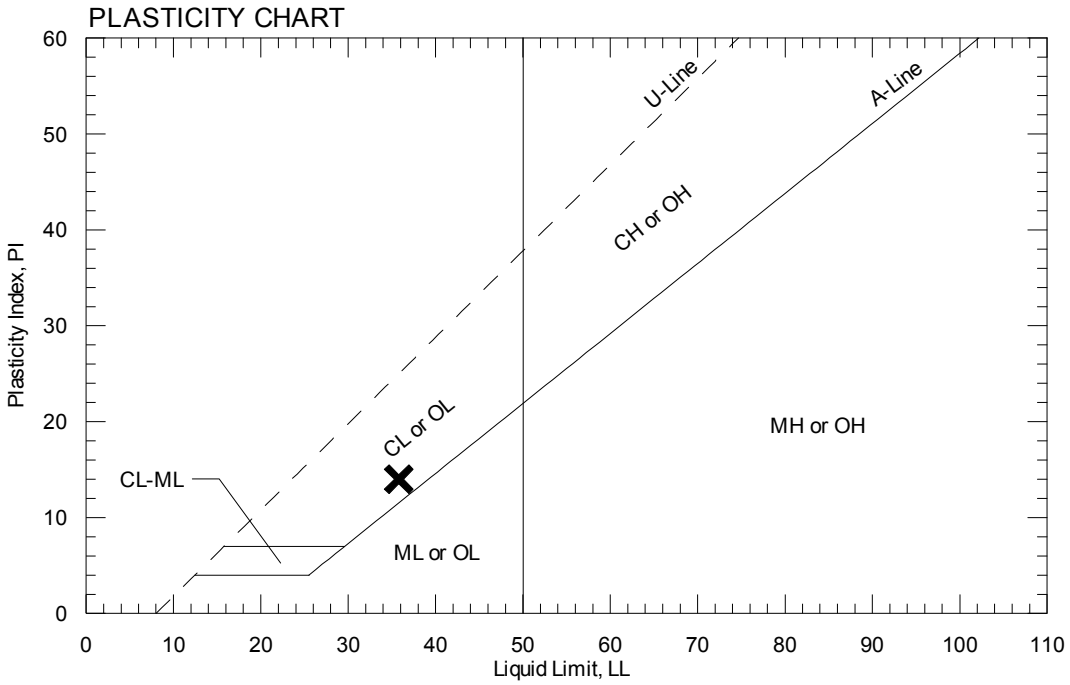
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/6/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210604
PIN	015600.00	Water Content, %	33.2
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-103/4D	Liquid Limit	36
Station	5+60.6	Plasticity Index	14
Depth	20.5-22.5	Tested By	BBURR



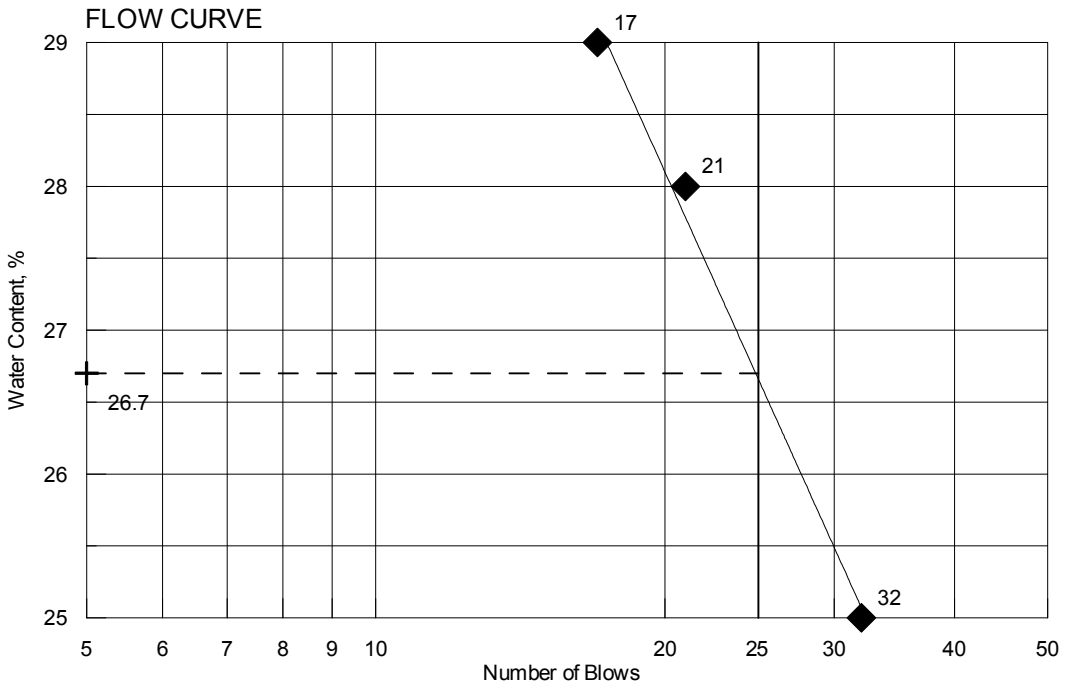


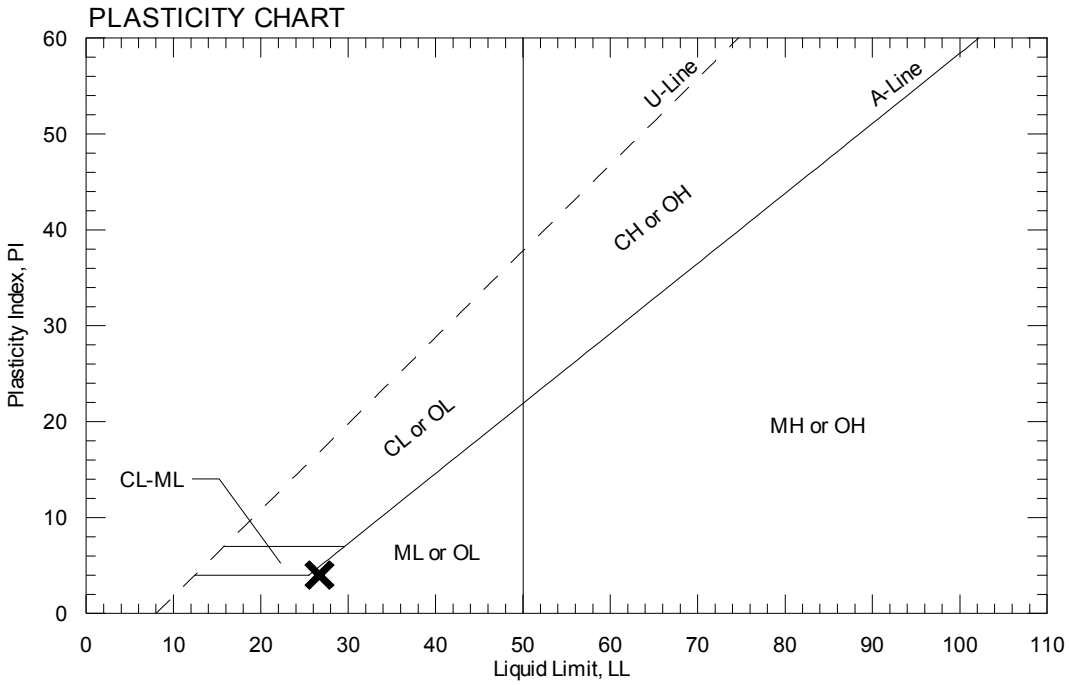
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/30/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210628
PIN	015600.00	Water Content, %	29
Sampled	5/20/2008	Plastic Limit	23
Boring No./Sample No.	BB-ACNR-103/2U	Liquid Limit	27
Station	5+60.6	Plasticity Index	4
Depth	25.0-27.0	Tested By	BBURR



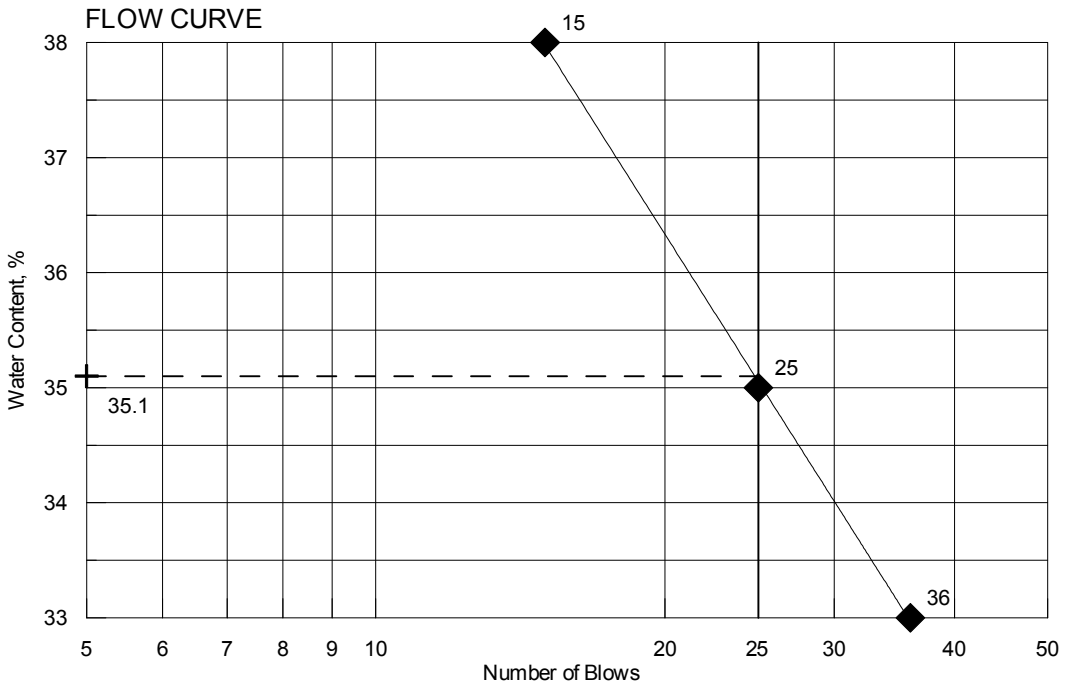


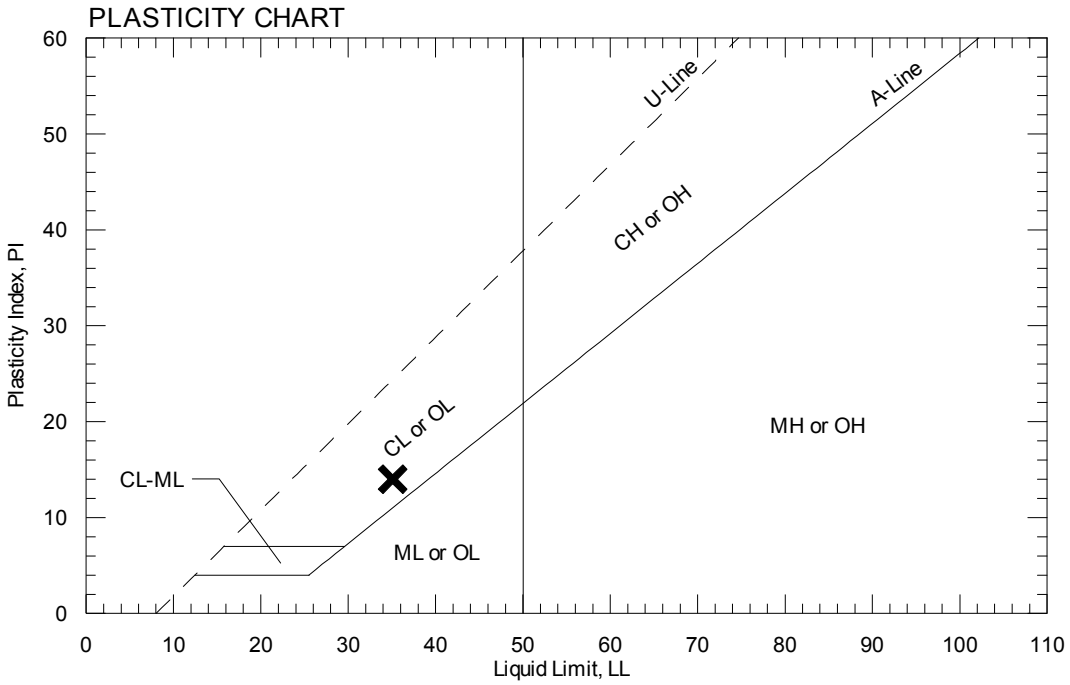
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/13/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210629
PIN	015600.00	Water Content, %	34.3
Sampled	5/20/2008	Plastic Limit	21
Boring No./Sample No.	BB-ACNR-103/3U	Liquid Limit	35
Station	5+60.6	Plasticity Index	14
Depth	35.0-37.0	Tested By	BBURR



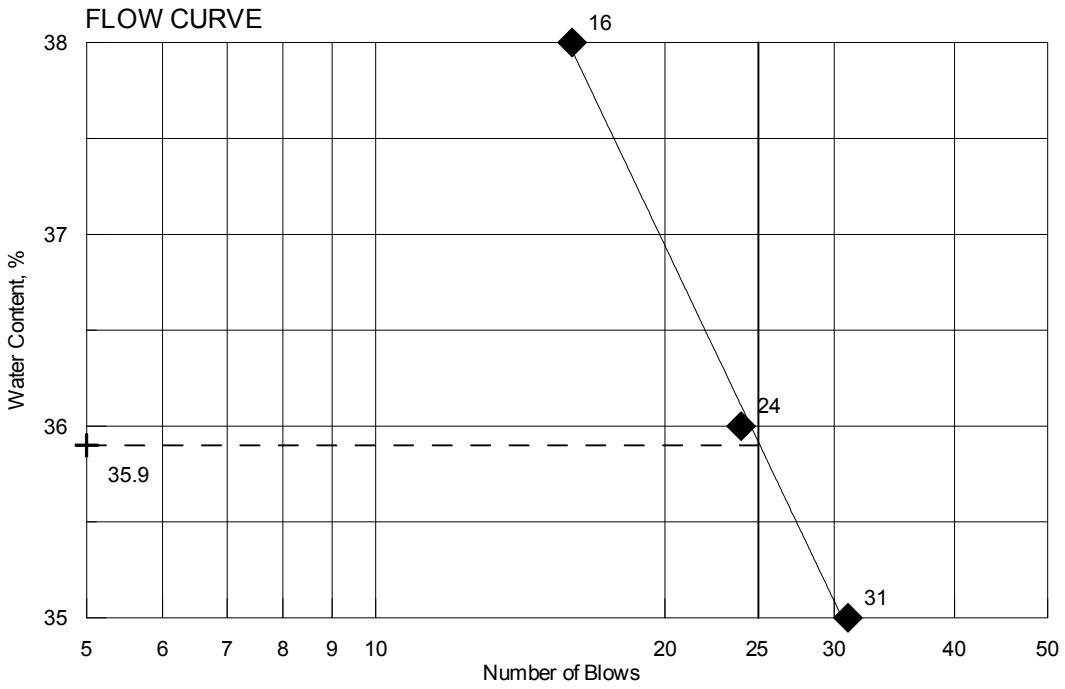


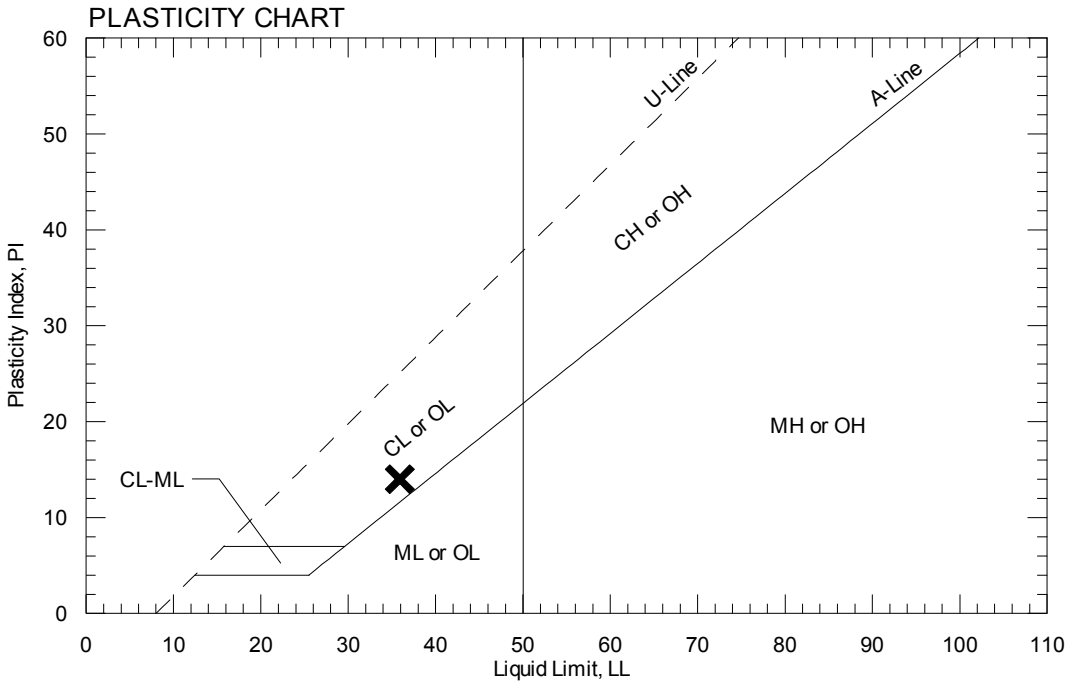
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/14/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210605
PIN	015600.00	Water Content, %	36.8
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-103/6D	Liquid Limit	36
Station	5+60.6	Plasticity Index	14
Depth	40.5'-42.5'	Tested By	BBURR



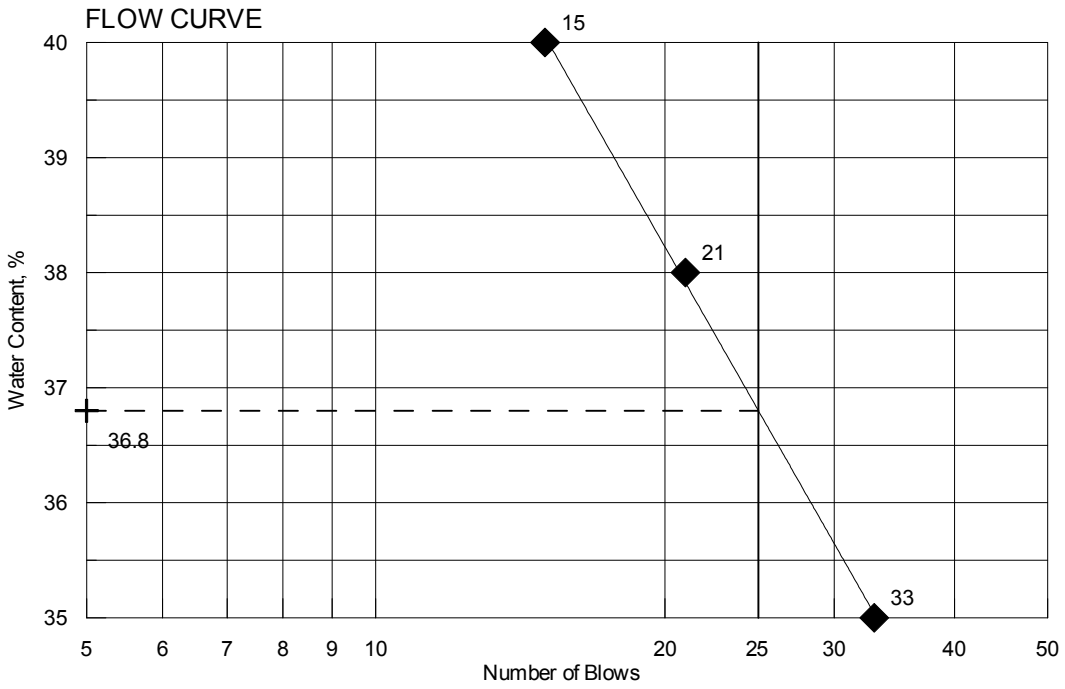


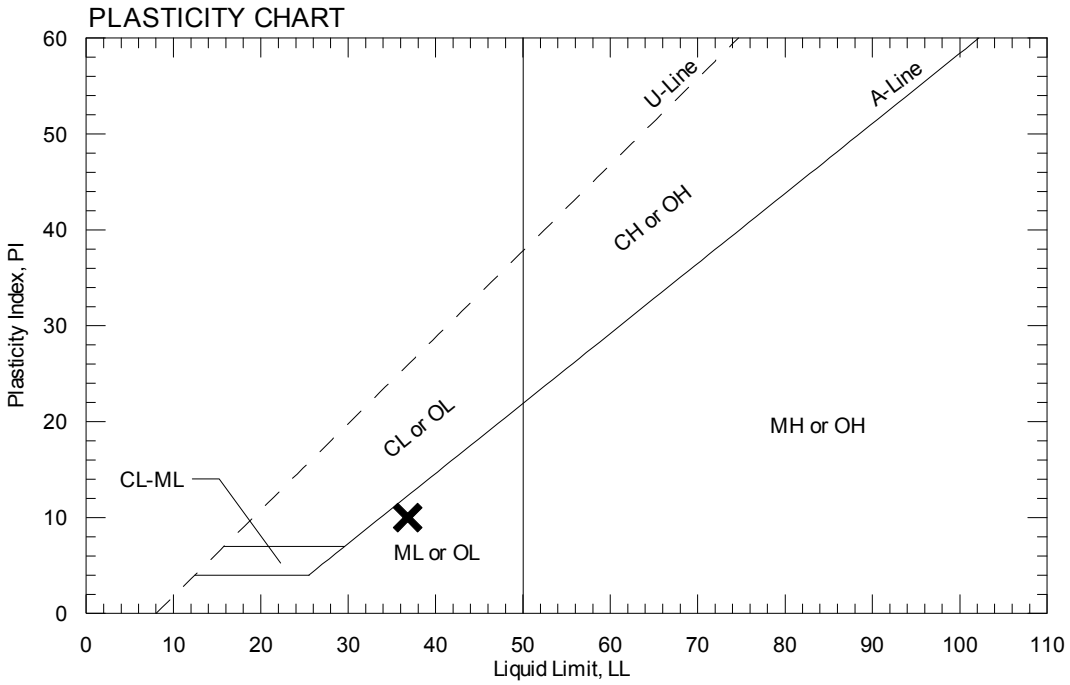
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210630
PIN	015600.00	Water Content, %	40.8
Sampled	5/20/2008	Plastic Limit	27
Boring No./Sample No.	BB-ACNR-103/4U	Liquid Limit	37
Station	5+60.6	Plasticity Index	10
Depth	45.0-47.0	Tested By	BBURR



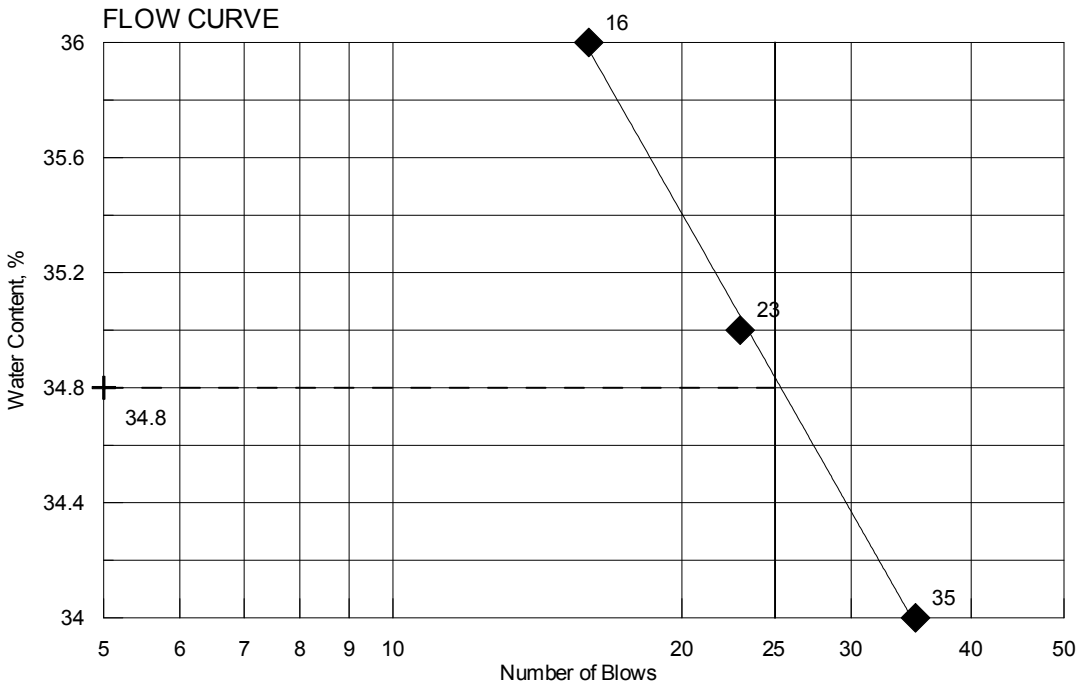


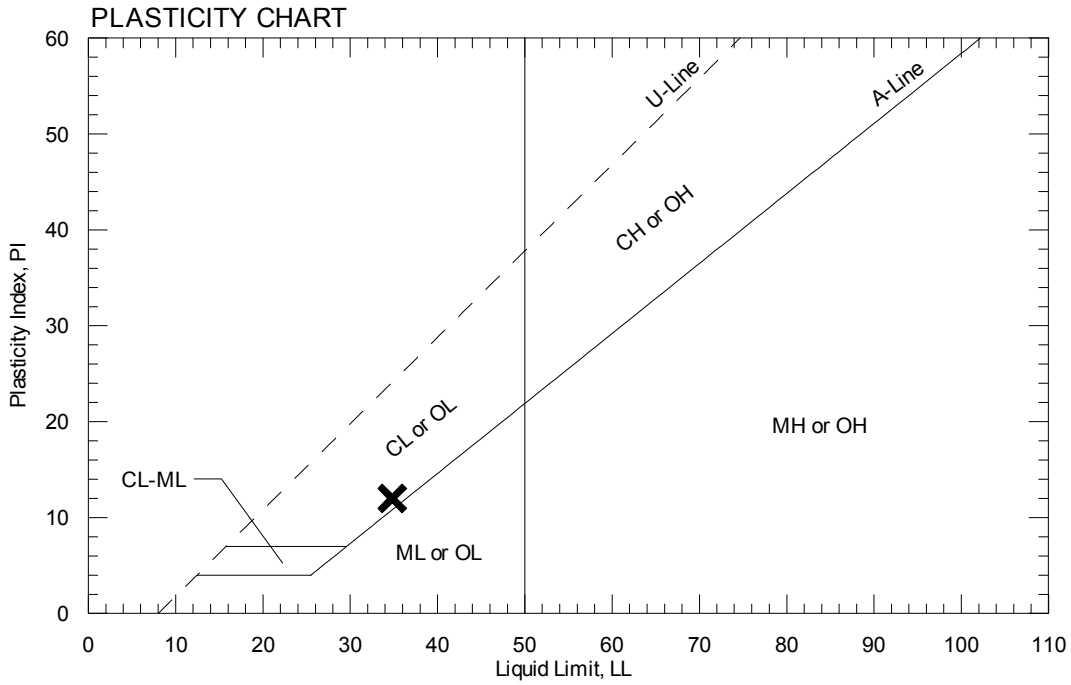
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/13/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210631
PIN	015600.00	Water Content, %	40.2
Sampled	5/20/2008	Plastic Limit	23
Boring No./Sample No.	BB-ACNR-103/5U	Liquid Limit	35
Station	5+60.6	Plasticity Index	12
Depth	55.0-57.0	Tested By	BBURR



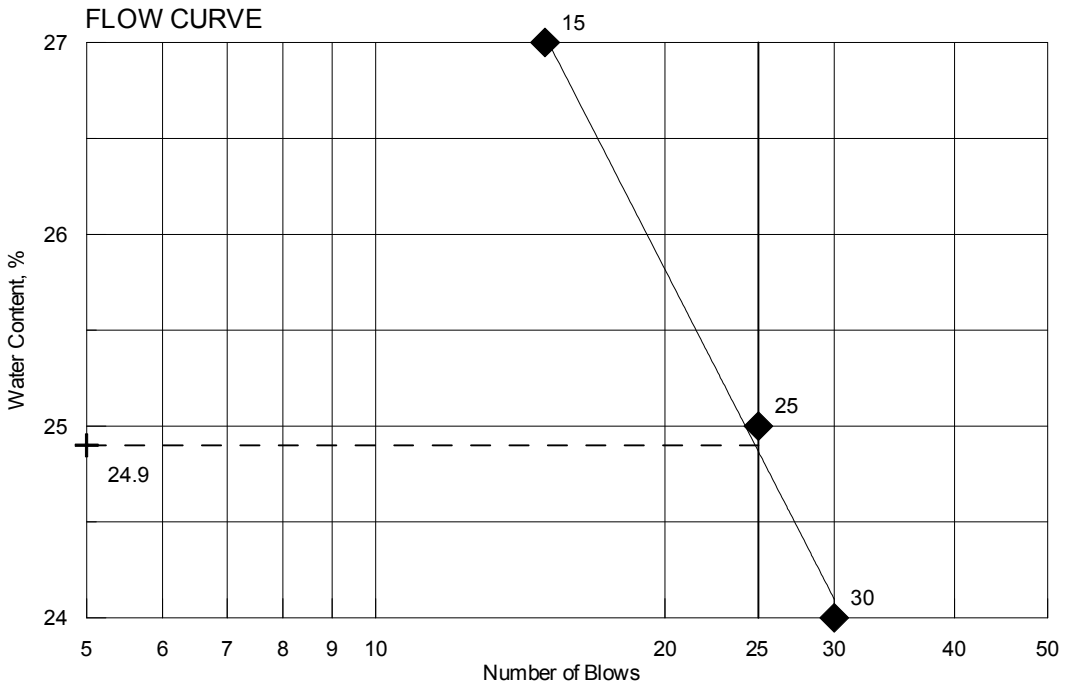


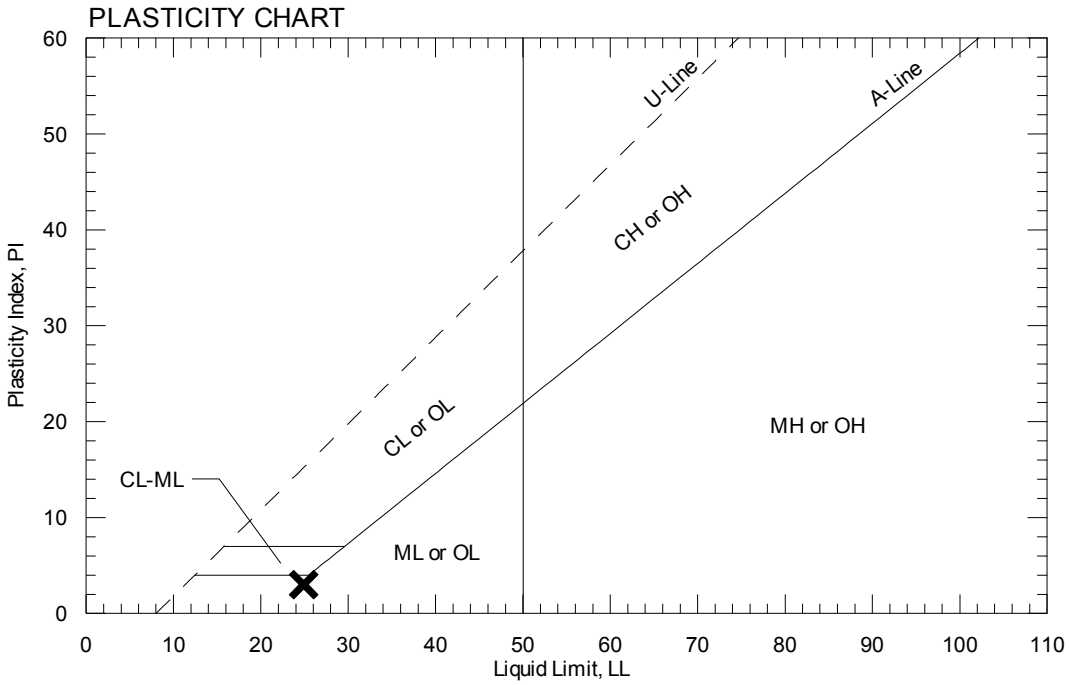
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/19/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210613
PIN	015600.00	Water Content, %	30.7
Sampled		Plastic Limit	22
Boring No./Sample No.	BB-ACNR-104/7D	Liquid Limit	25
Station	6+01.9	Plasticity Index	3
Depth	29.5-31.5	Tested By	BBURR



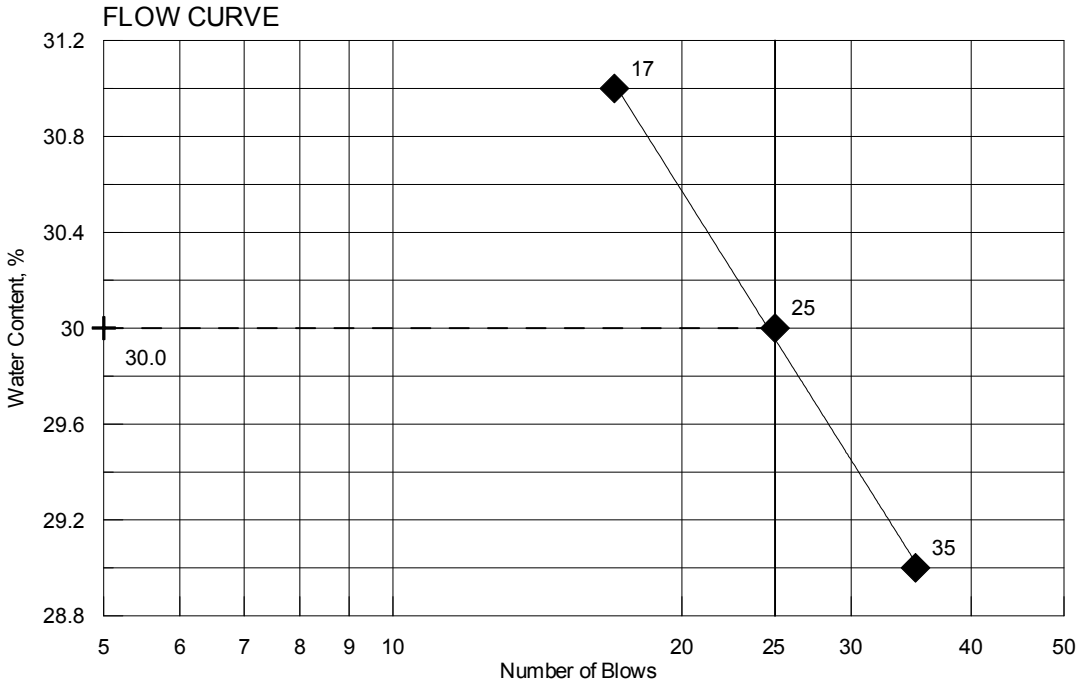


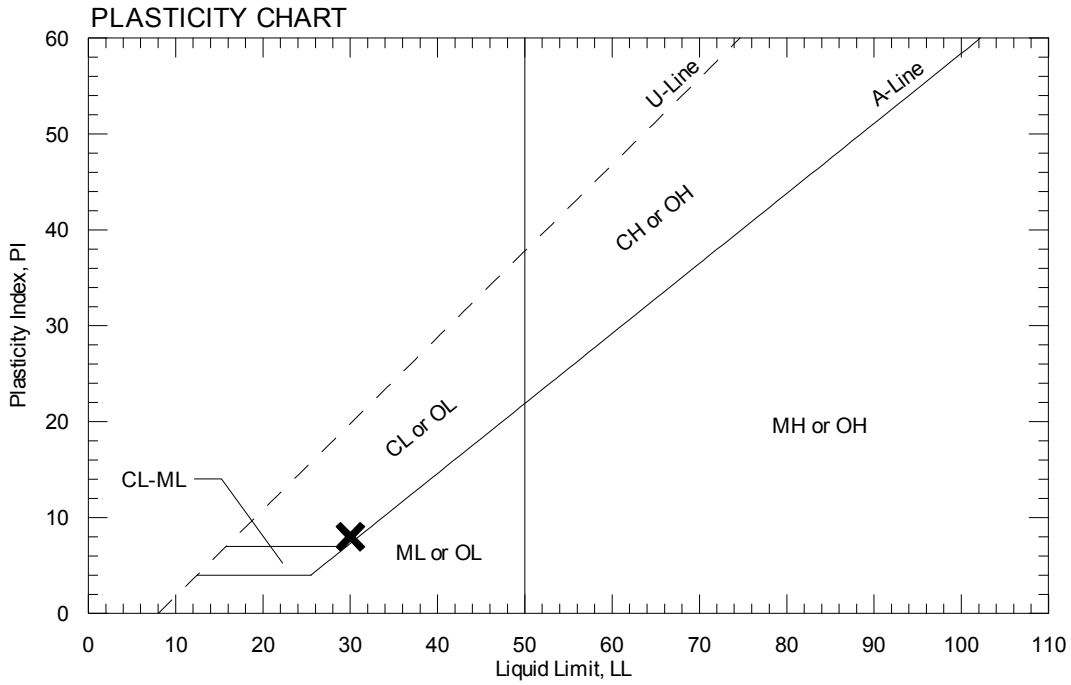
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210632
PIN	015600.00	Water Content, %	36.7
Sampled	5/20/2008	Plastic Limit	22
Boring No./Sample No.	BB-ACNR-104/1U	Liquid Limit	30
Station	6+01.9	Plasticity Index	8
Depth	34.0-36.0	Tested By	BBURR



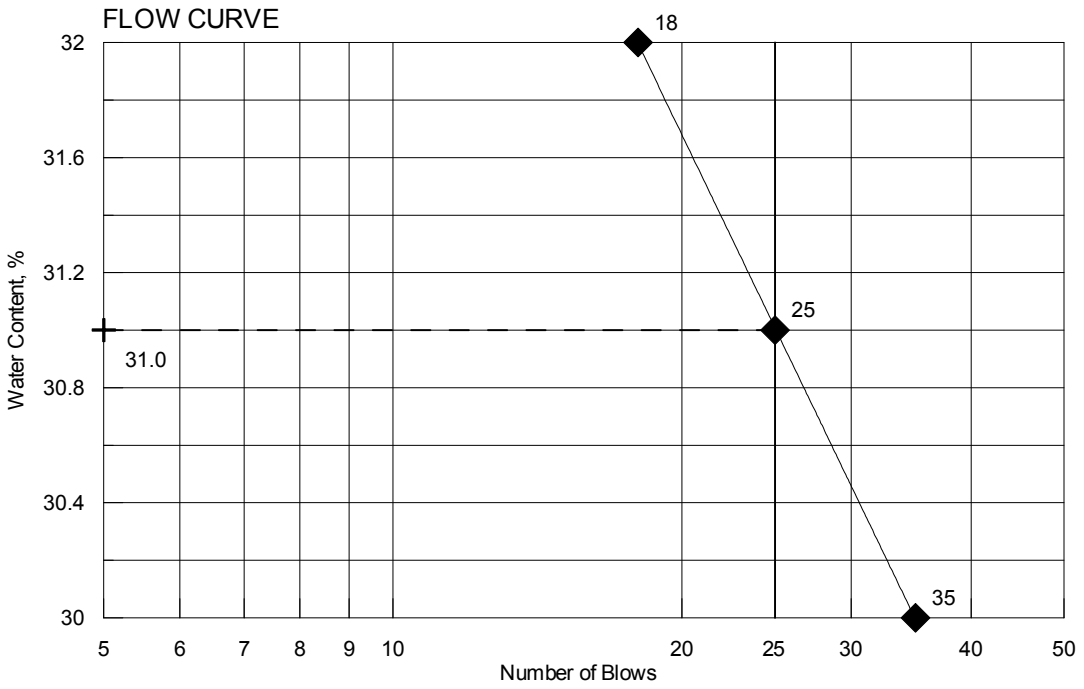


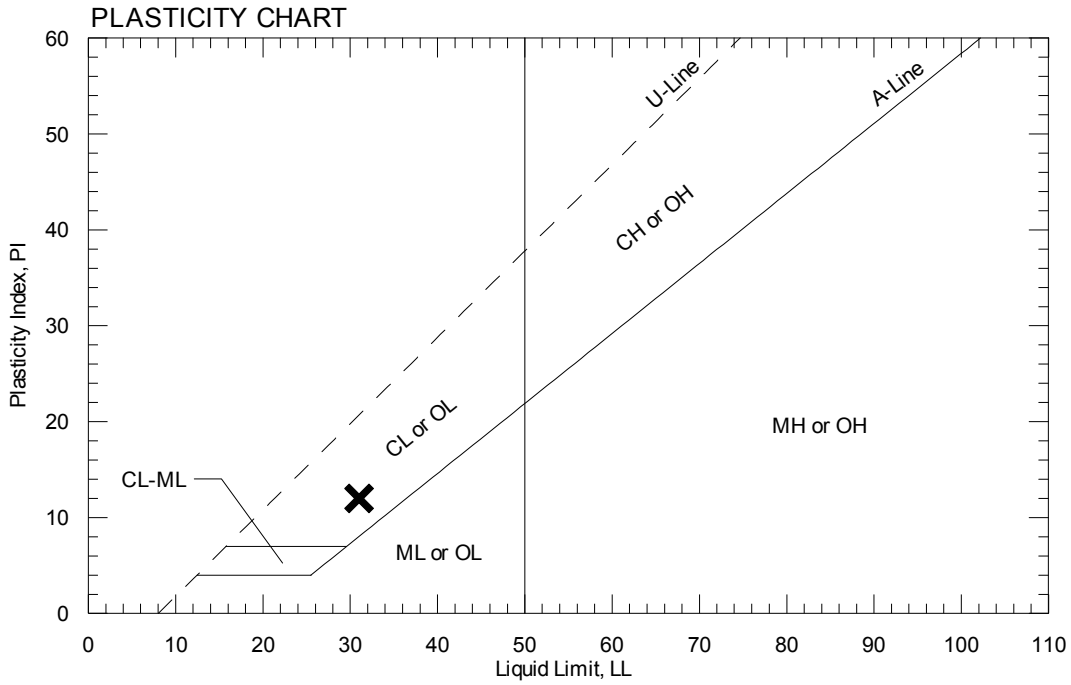
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/20/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210614
PIN	015600.00	Water Content, %	31.9
Sampled		Plastic Limit	19
Boring No./Sample No.	BB-ACNR-104/8D	Liquid Limit	31
Station	6+01.9	Plasticity Index	12
Depth	40.5-42.5	Tested By	BBURR



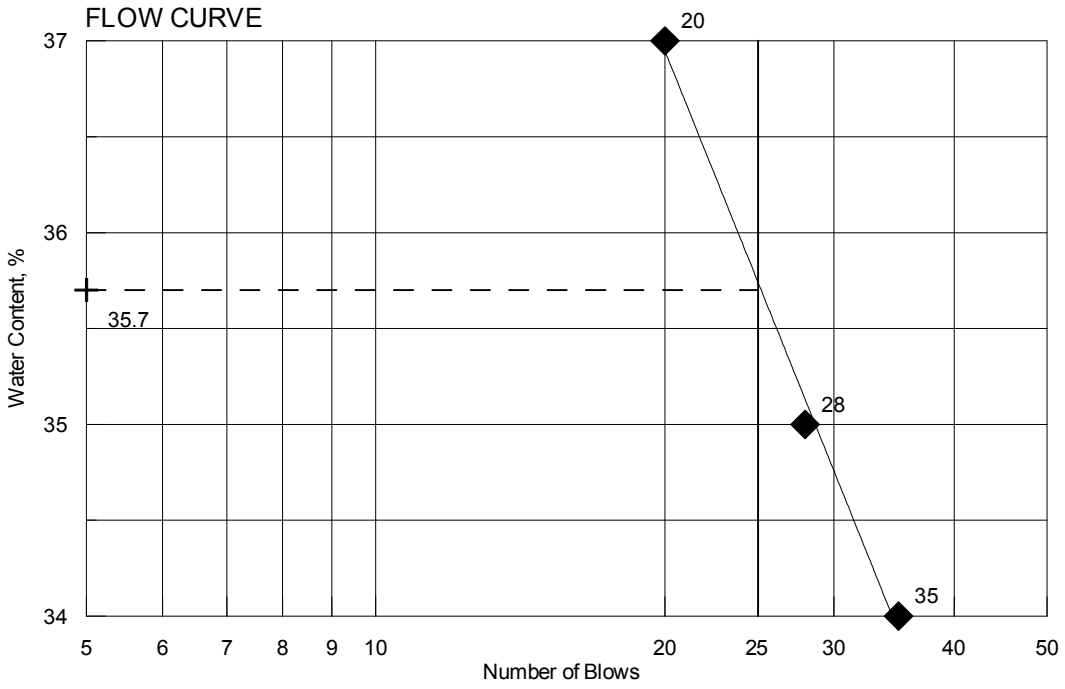


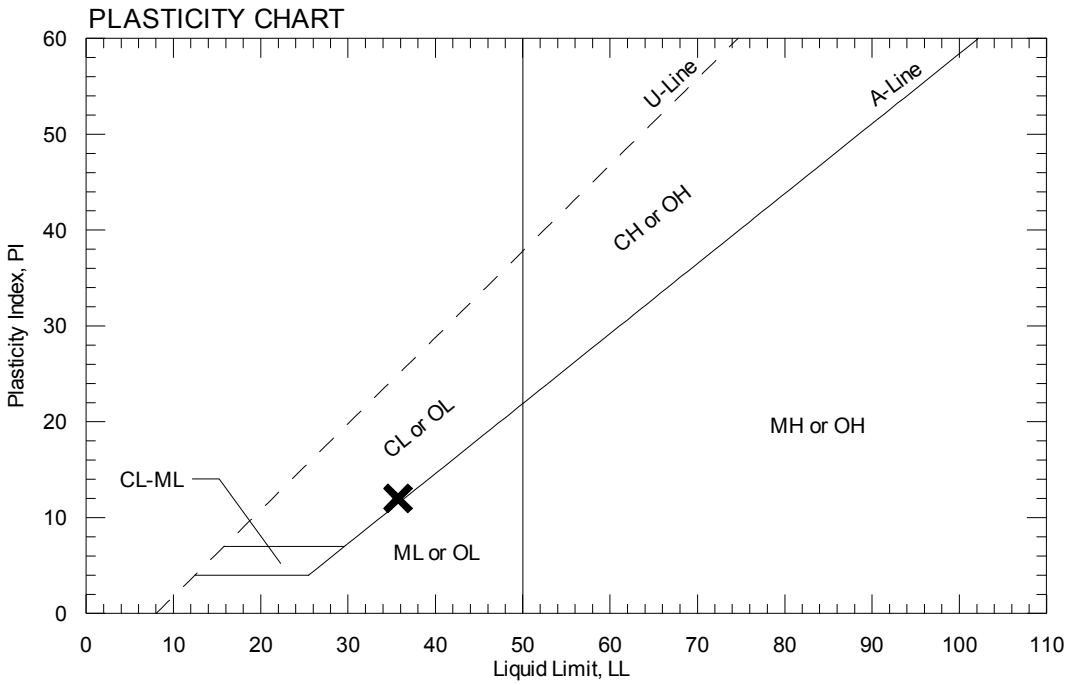
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210633
PIN	015600.00	Water Content, %	36.9
Sampled	5/20/2008	Plastic Limit	24
Boring No./Sample No.	BB-ACNR-104/2U	Liquid Limit	36
Station	6+01.9	Plasticity Index	12
Depth	44.0-46.0	Tested By	BBURR



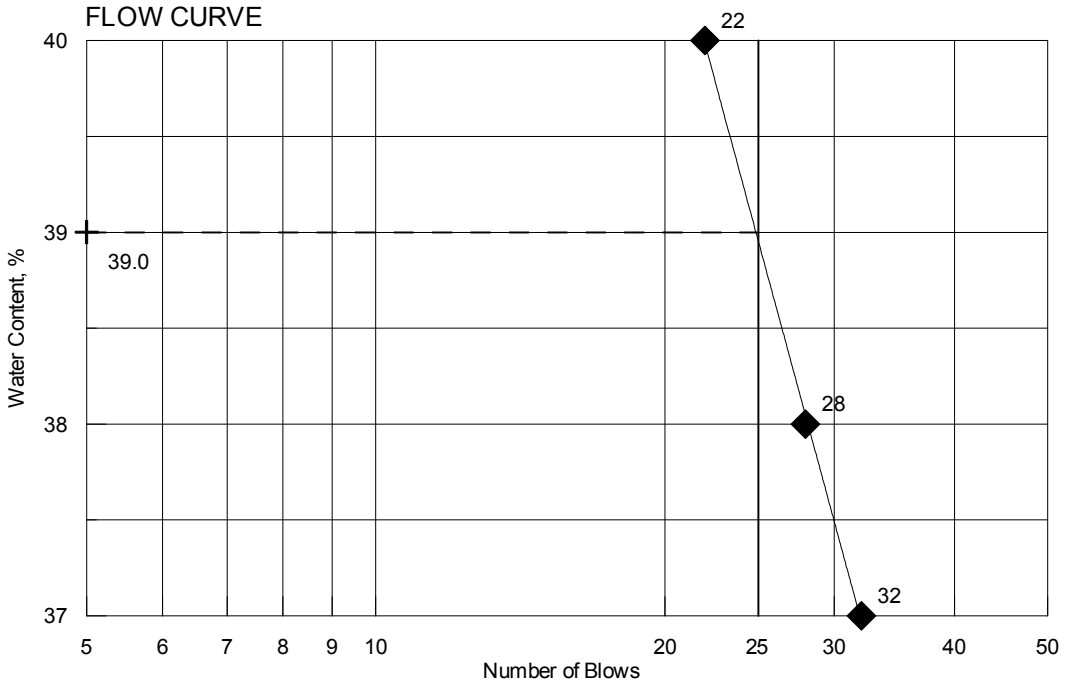


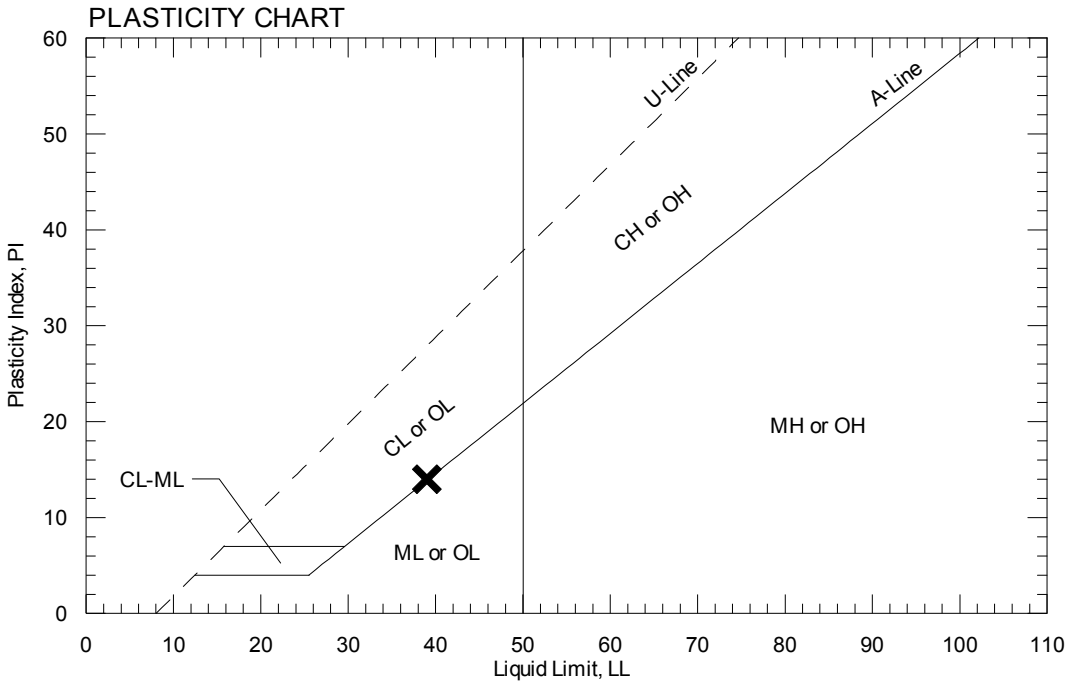
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **8/25/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210615
PIN	015600.00	Water Content, %	40.9
Sampled		Plastic Limit	25
Boring No./Sample No.	BB-ACNR-104/9D	Liquid Limit	39
Station	6+01.9	Plasticity Index	14
Depth	49.5-51.5	Tested By	BBURR



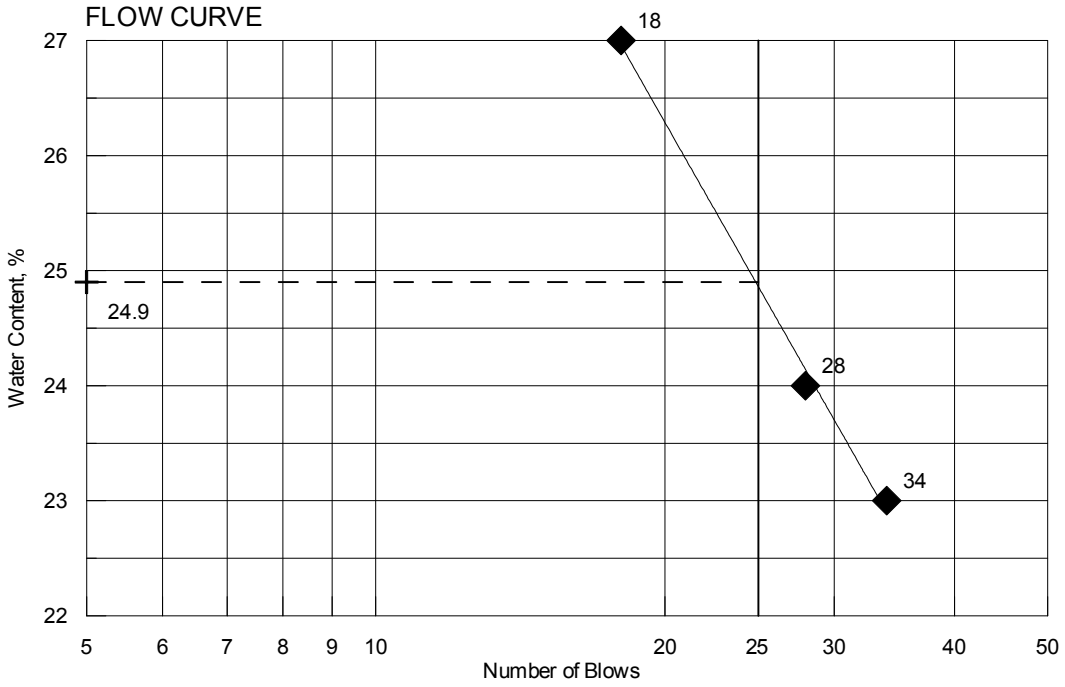


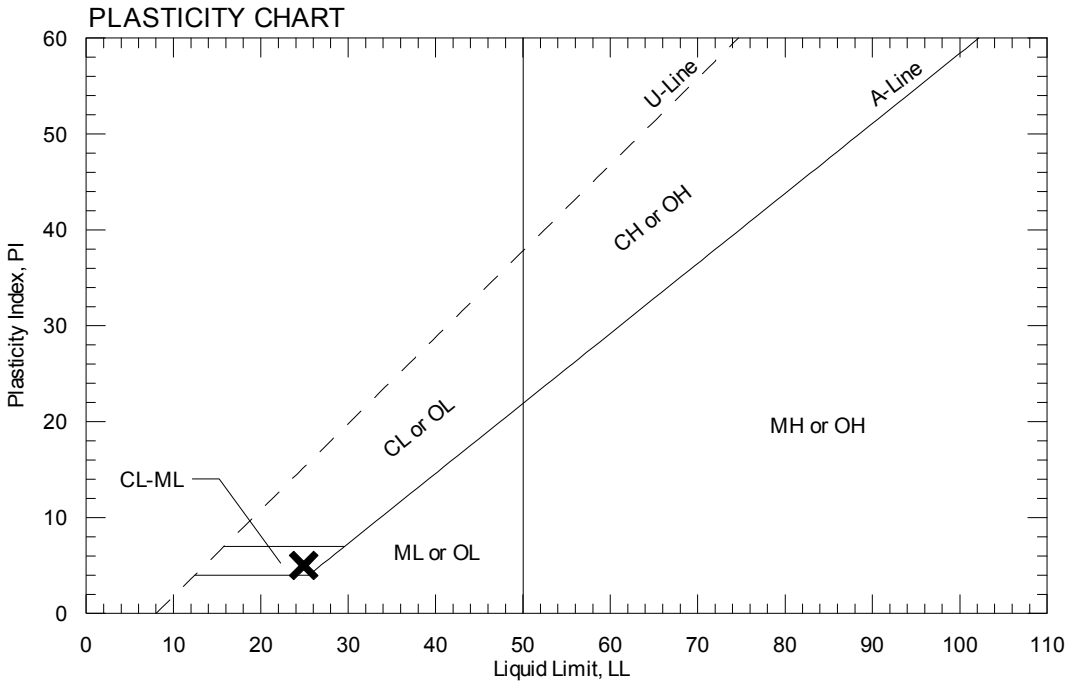
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **7/9/2008**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Auburn	Reference No.	210634
PIN	015600.00	Water Content, %	29.5
Sampled	5/20/2008	Plastic Limit	20
Boring No./Sample No.	BB-ACNR-104/3U	Liquid Limit	25
Station	6+01.9	Plasticity Index	5
Depth	54.0-55.0	Tested By	BBURR





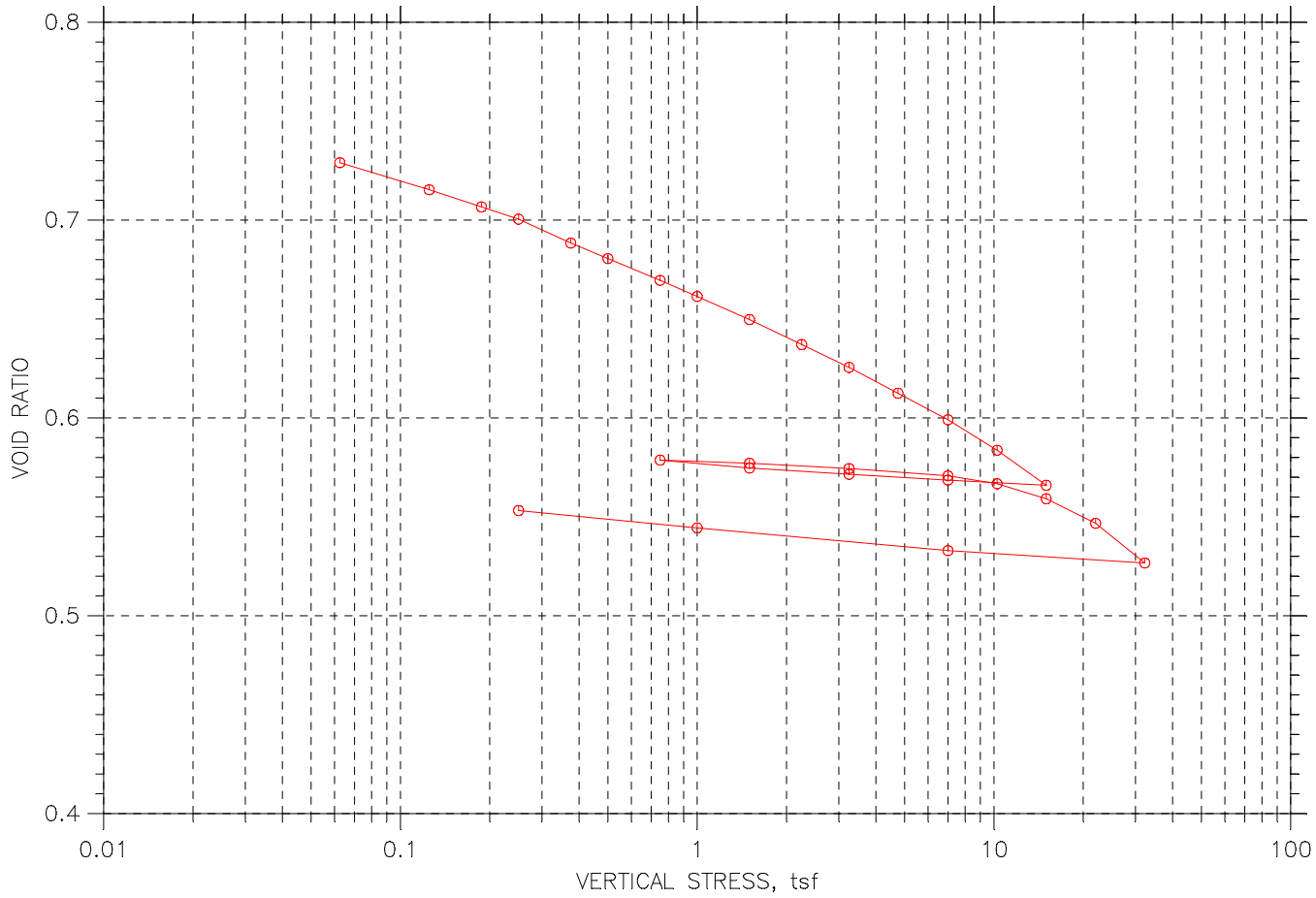
AUTHORIZATION AND DISTRIBUTION

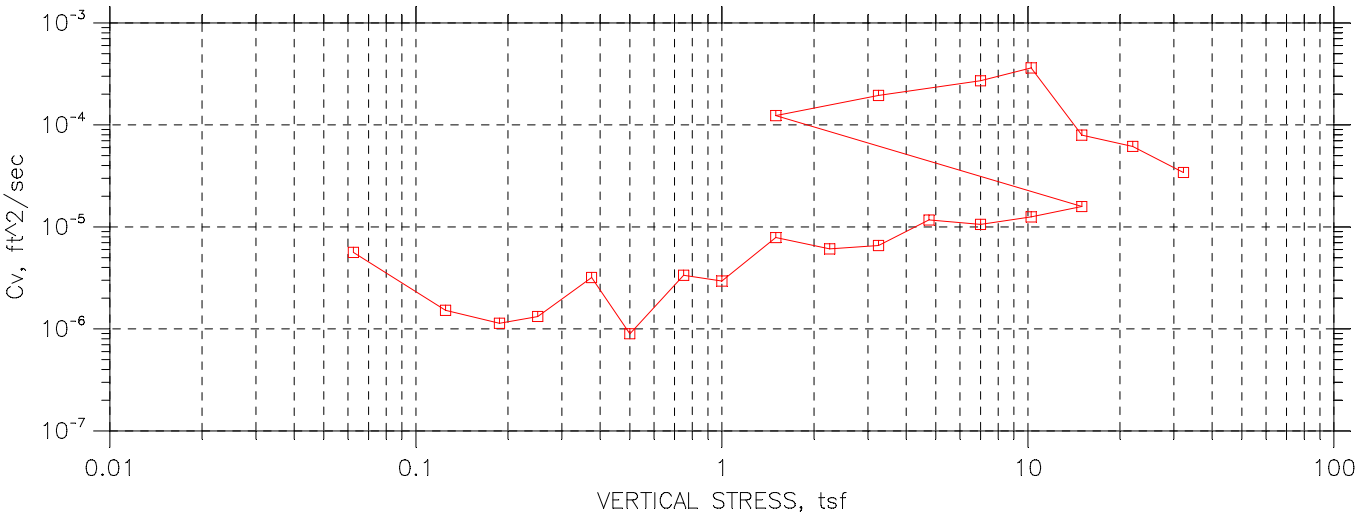
Reported by: **FOGG, BRIAN** Date Reported: **8/20/2008**

Paper Copy: Lab File; Project File; Geotech File

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-101	Tested By: Brian Fogg	Checked By:
	Sample No.: 2U	Test Date: 7/11/2008	Depth: 45-47 FT
	Test No.: 210617	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Tue, 02-SEP-2008 16:06:17

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 2U
Test No.: 210617

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/11/2008
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 45-47 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.81
Initial Void Ratio: 0.81
Final Void Ratio: 0.55

Liquid Limit: 22
Plastic Limit: 19
Plasticity Index: 3

Initial Height: 1.02 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	200	RING	RING	35
Wt. Container + Wet Soil, gm	193.76	422.61	413.18	215.56
Wt. Container + Dry Soil, gm	166.33	388.33	388.33	190.79
Wt. Container, gm	65.2	262.25	262.25	65.13
Wt. Dry Soil, gm	101.13	126.08	126.08	125.66
Water Content, %	27.12	27.19	19.71	19.71
Void Ratio	---	0.81	0.55	---
Degree of Saturation, %	---	93.78	100.14	---
Dry Unit Weight, pcf	---	96.663	112.95	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 2U
Test No.: 210617

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/11/2008
Sample Type: Shelby Tube

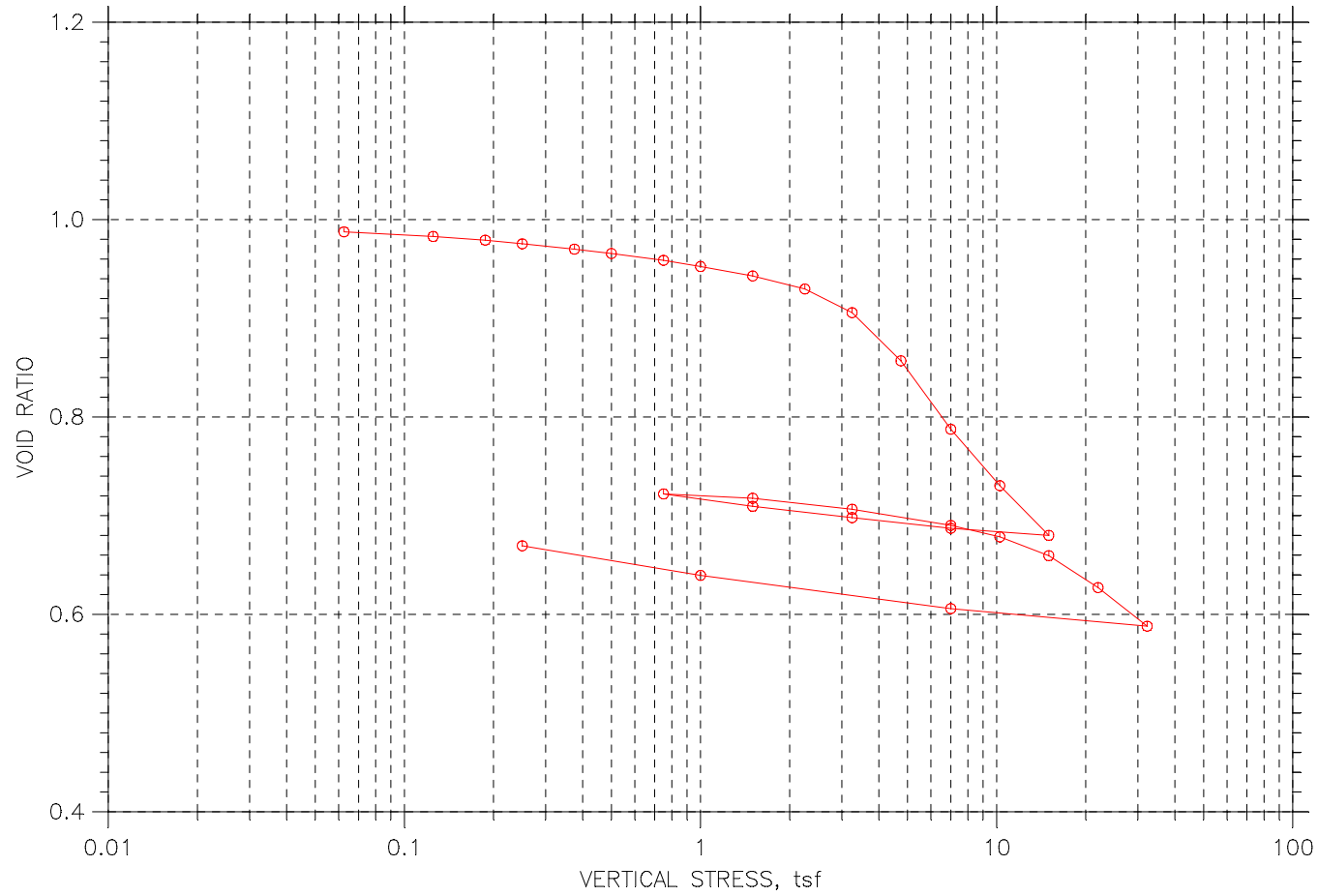
Project No.: 15600.00
Checked By:
Depth: 45-47 FT
Elevation: ---

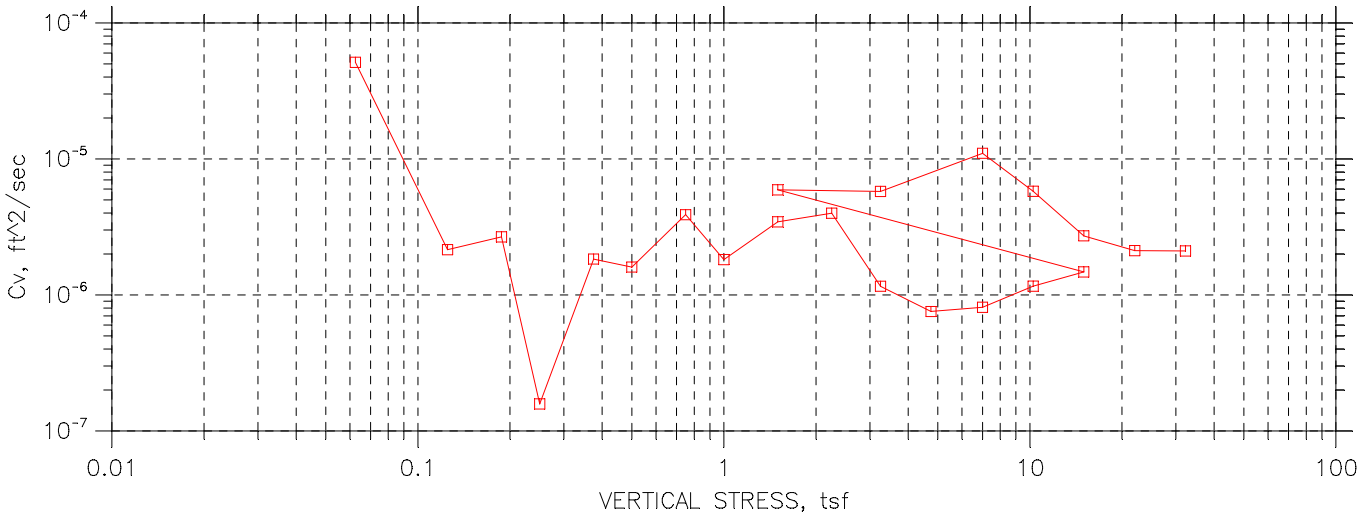
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.04842	0.729	4.73	1.6	0.4	3.47e-006	1.49e-005	5.63e-006
2	0.125	0.05615	0.715	5.48	3.5	3.6	1.53e-006	1.51e-006	1.52e-006
3	0.188	0.06105	0.707	5.96	4.5	4.9	1.19e-006	1.09e-006	1.14e-006
4	0.25	0.06454	0.700	6.30	4.3	3.7	1.23e-006	1.43e-006	1.32e-006
5	0.375	0.07137	0.688	6.97	1.6	1.7	3.33e-006	3.06e-006	3.19e-006
6	0.5	0.07579	0.681	7.40	8.8	2.7	5.86e-007	1.90e-006	8.96e-007
7	0.75	0.08201	0.670	8.00	1.5	1.6	3.51e-006	3.21e-006	3.35e-006
8	1	0.08659	0.661	8.45	1.7	1.7	2.95e-006	2.95e-006	2.95e-006
9	1.5	0.09319	0.650	9.10	0.9	0.4	5.63e-006	1.30e-005	7.86e-006
10	2.25	0.1003	0.637	9.79	0.8	0.0	6.09e-006	0.00e+000	6.09e-006
11	3.25	0.1068	0.626	10.43	0.9	0.6	5.53e-006	8.09e-006	6.57e-006
12	4.75	0.1142	0.612	11.15	0.6	0.2	8.32e-006	1.97e-005	1.17e-005
13	7	0.1218	0.599	11.89	0.7	0.2	6.60e-006	2.68e-005	1.06e-005
14	10.3	0.1305	0.584	12.74	0.6	0.2	8.11e-006	2.77e-005	1.25e-005
15	15	0.1405	0.566	13.71	0.5	0.1	9.63e-006	4.46e-005	1.58e-005
16	7	0.139	0.569	13.57	0.0	0.0	6.89e-004	0.00e+000	6.89e-004
17	3.25	0.1373	0.572	13.41	0.0	0.0	2.06e-004	0.00e+000	2.06e-004
18	1.5	0.1356	0.575	13.23	0.0	0.0	1.18e-004	1.55e-004	1.34e-004
19	0.75	0.1333	0.579	13.01	0.1	0.0	3.20e-005	0.00e+000	3.20e-005
20	1.5	0.1342	0.577	13.10	0.0	0.0	1.23e-004	0.00e+000	1.23e-004
21	3.25	0.1357	0.574	13.25	0.0	0.0	1.94e-004	0.00e+000	1.94e-004
22	7	0.1378	0.571	13.45	0.0	0.0	3.53e-004	2.20e-004	2.71e-004
23	10.3	0.14	0.567	13.67	0.0	0.0	3.62e-004	0.00e+000	3.62e-004
24	15	0.1443	0.559	14.09	0.1	0.0	5.43e-005	1.48e-004	7.94e-005
25	22	0.1513	0.547	14.77	0.1	0.0	4.41e-005	1.02e-004	6.16e-005
26	32.3	0.1627	0.527	15.88	0.2	0.0	2.04e-005	1.06e-004	3.42e-005
27	7	0.1591	0.533	15.53	0.0	0.0	2.93e-004	0.00e+000	2.93e-004
28	1	0.1526	0.544	14.90	0.1	0.0	8.47e-005	2.28e-004	1.24e-004
29	0.25	0.1477	0.553	14.42	0.5	0.2	9.25e-006	1.76e-005	1.21e-005

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-101	Tested By: Brian Fogg	Checked By:
	Sample No.: 3U	Test Date: 7/16/08	Depth: 55-57 FT
	Test No.: 210618	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Tue, 02-SEP-2008 16:21:02

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 3U
Test No.: 210618

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/16/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 55-57 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.76
Initial Void Ratio: 0.99
Final Void Ratio: 0.67

Liquid Limit: 30
Plastic Limit: 19
Plasticity Index: 11

Initial Height: 0.99 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	44	RING	RING	22
Wt. Container + Wet Soil, gm	144.76	408.7	398.01	187.15
Wt. Container + Dry Soil, gm	121.65	371.5	371.5	160.89
Wt. Container, gm	53.81	262.21	262.21	52.65
Wt. Dry Soil, gm	67.84	109.29	109.29	108.24
Water Content, %	34.07	34.04	24.26	24.26
Void Ratio	---	0.99	0.67	---
Degree of Saturation, %	---	94.51	100.01	---
Dry Unit Weight, pcf	---	86.404	103.21	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 3U
Test No.: 210618

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/16/08
Sample Type: Shelby Tube

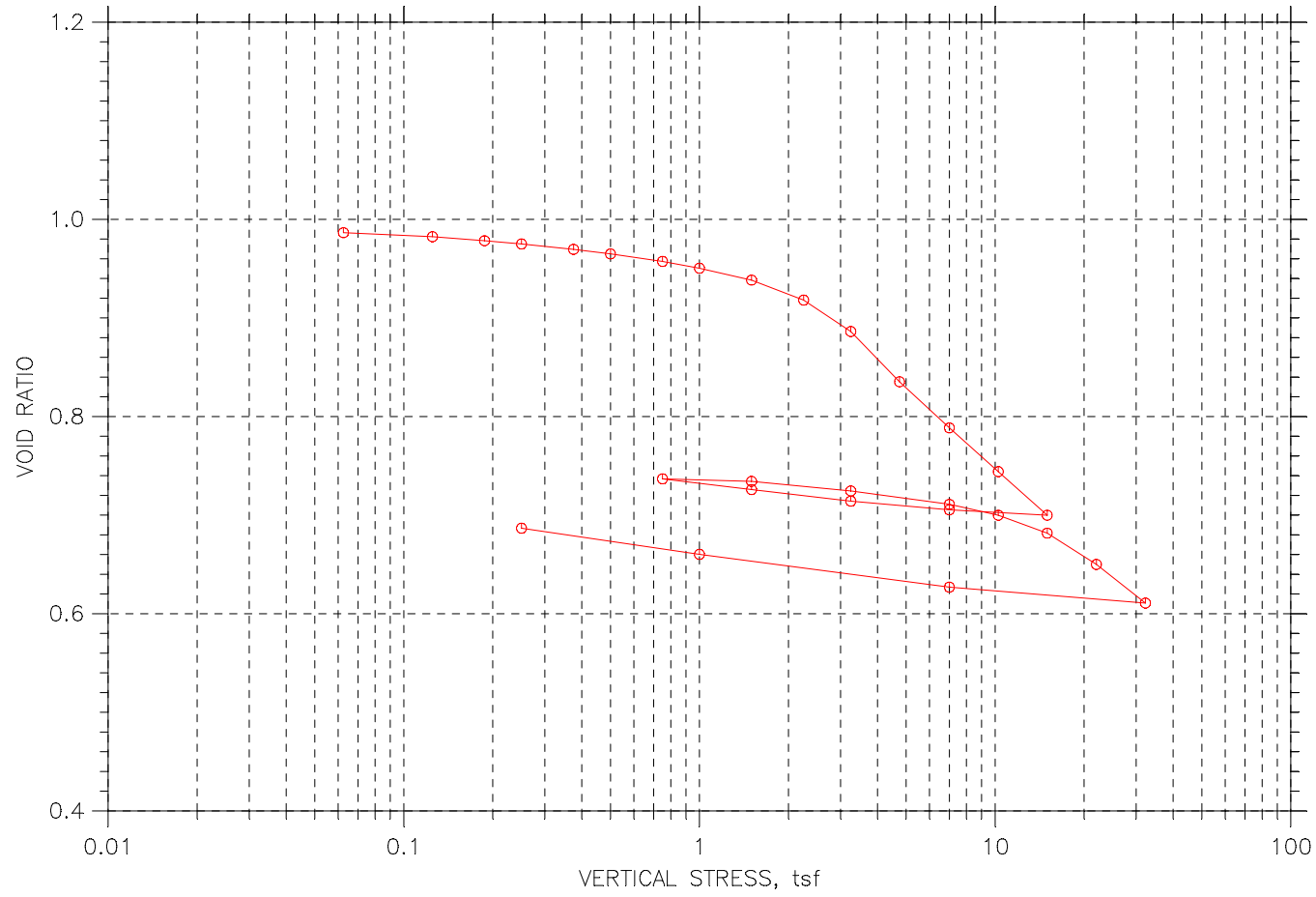
Project No.: 15600.00
Checked By:
Depth: 55-57 FT
Elevation: ---

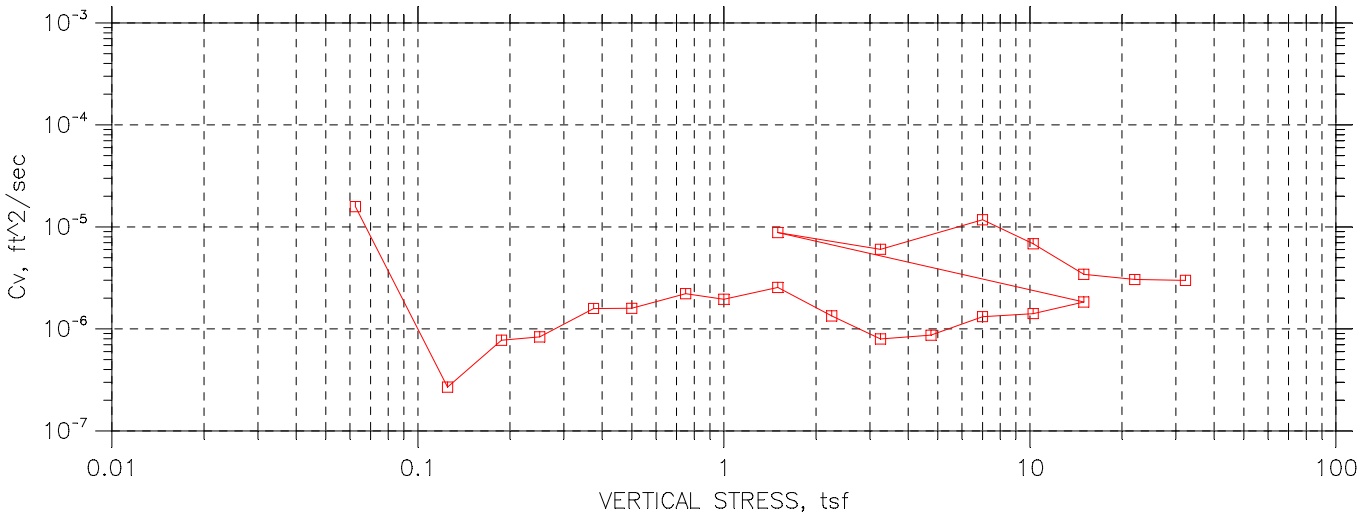
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.003247	0.988	0.33	0.1	0.0	5.13e-005	0.00e+000	5.13e-005
2	0.125	0.005603	0.983	0.56	2.8	2.4	1.99e-006	2.35e-006	2.15e-006
3	0.188	0.007458	0.979	0.75	2.0	2.2	2.83e-006	2.52e-006	2.67e-006
4	0.25	0.009327	0.975	0.94	35.1	0.0	1.58e-007	0.00e+000	1.58e-007
5	0.375	0.012	0.970	1.21	3.6	2.4	1.53e-006	2.28e-006	1.84e-006
6	0.5	0.01424	0.966	1.43	4.4	2.5	1.26e-006	2.21e-006	1.60e-006
7	0.75	0.01762	0.959	1.77	1.6	1.2	3.46e-006	4.43e-006	3.89e-006
8	1	0.02079	0.952	2.09	3.0	0.0	1.82e-006	0.00e+000	1.82e-006
9	1.5	0.02556	0.943	2.57	1.6	1.6	3.46e-006	3.43e-006	3.45e-006
10	2.25	0.03213	0.930	3.23	1.6	1.1	3.41e-006	4.81e-006	3.99e-006
11	3.25	0.04408	0.906	4.44	4.5	0.0	1.16e-006	0.00e+000	1.16e-006
12	4.75	0.0684	0.857	6.88	7.0	6.3	7.15e-007	8.00e-007	7.55e-007
13	7	0.1029	0.788	10.36	7.0	4.6	6.73e-007	1.02e-006	8.11e-007
14	10.3	0.1314	0.730	13.23	3.5	4.1	1.26e-006	1.08e-006	1.16e-006
15	15	0.1565	0.680	15.75	2.3	3.2	1.76e-006	1.27e-006	1.48e-006
16	7	0.1529	0.687	15.39	0.1	0.0	6.68e-005	0.00e+000	6.68e-005
17	3.25	0.1475	0.698	14.85	0.5	0.0	8.78e-006	0.00e+000	8.78e-006
18	1.5	0.1418	0.710	14.27	1.0	0.0	4.23e-006	0.00e+000	4.23e-006
19	0.75	0.1356	0.722	13.65	3.5	3.6	1.18e-006	1.17e-006	1.17e-006
20	1.5	0.1377	0.718	13.86	0.7	0.0	5.92e-006	0.00e+000	5.92e-006
21	3.25	0.1433	0.706	14.43	0.7	0.0	5.76e-006	0.00e+000	5.76e-006
22	7	0.1515	0.690	15.25	0.5	0.2	7.85e-006	1.84e-005	1.10e-005
23	10.3	0.1572	0.679	15.82	0.7	0.7	5.73e-006	5.84e-006	5.78e-006
24	15	0.1667	0.659	16.78	1.4	1.5	2.83e-006	2.62e-006	2.72e-006
25	22	0.1829	0.627	18.41	1.8	1.8	2.16e-006	2.08e-006	2.12e-006
26	32.3	0.2024	0.588	20.37	1.6	1.8	2.23e-006	1.99e-006	2.11e-006
27	7	0.1935	0.606	19.47	0.0	0.0	9.83e-005	0.00e+000	9.83e-005
28	1	0.1767	0.640	17.78	1.8	2.2	2.08e-006	1.72e-006	1.88e-006
29	0.25	0.1617	0.670	16.28	7.1	8.8	5.49e-007	4.41e-007	4.89e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-101	Tested By: Brian Fogg	Checked By:
	Sample No.: 4U	Test Date: 6/18/08	Depth: 65-67 FT
	Test No.: 210619	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Tue, 02-SEP-2008 16:24:32

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 4U
Test No.: 210619

Location: AUBURN
Tested By: Brian Fogg
Test Date: 6/18/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 65-67 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.76
Initial Void Ratio: 1.16
Final Void Ratio: 0.69

Liquid Limit: 35
Plastic Limit: 24
Plasticity Index: 11

Initial Height: 1.11 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	71	RING	RING	203
Wt. Container + Wet Soil, gm	199.52	413.68	402.51	204.54
Wt. Container + Dry Soil, gm	158.27	374.52	374.52	176.61
Wt. Container, gm	45	262.16	262.16	64.51
Wt. Dry Soil, gm	113.27	112.36	112.36	112.1
Water Content, %	36.42	34.86	24.92	24.92
Void Ratio	---	1.16	0.69	---
Degree of Saturation, %	---	83.13	100.15	---
Dry Unit Weight, pcf	---	79.867	102.16	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 4U
Test No.: 210619

Location: AUBURN
Tested By: Brian Fogg
Test Date: 6/18/08
Sample Type: Shelby Tube

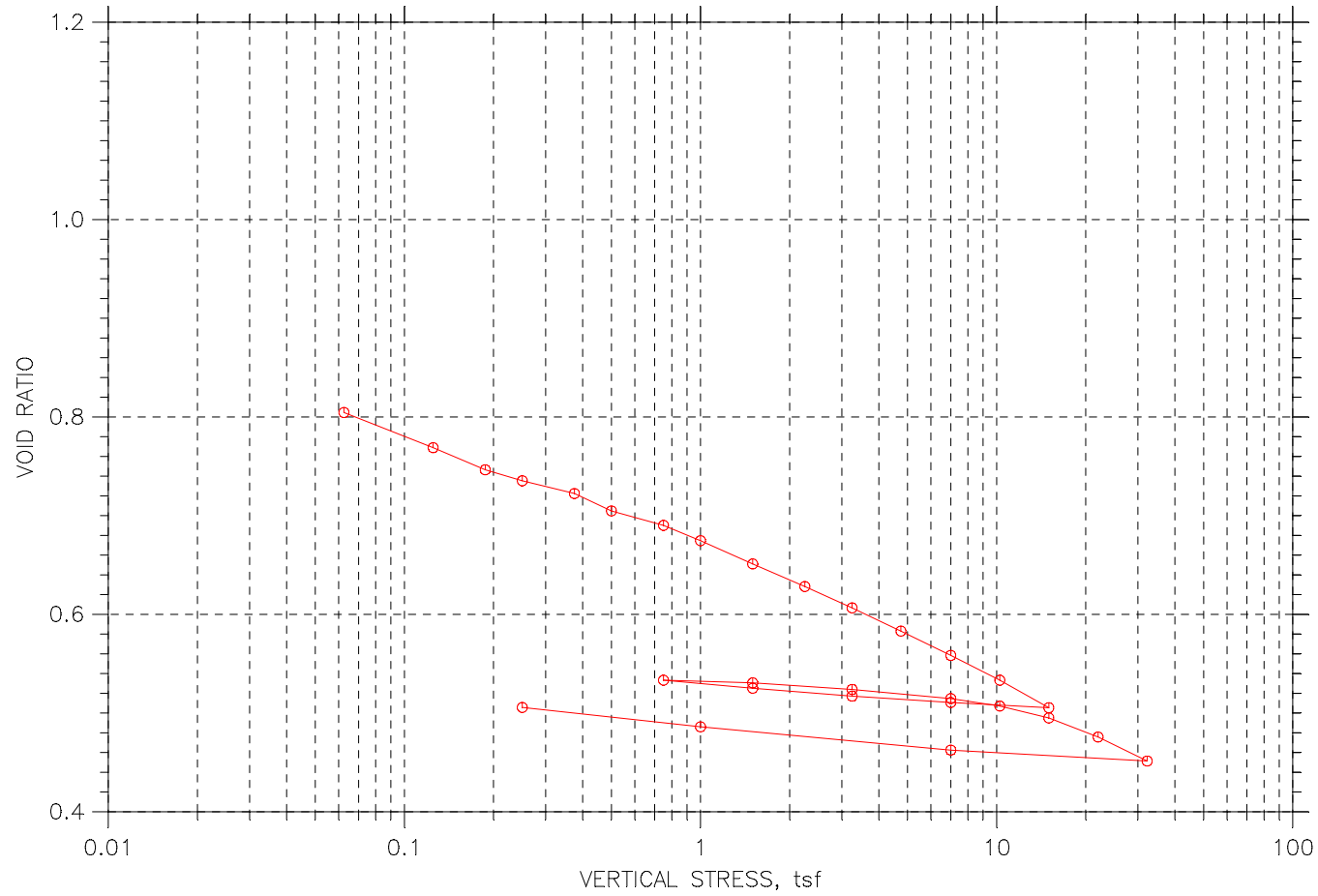
Project No.: 15600.00
Checked By:
Depth: 65-67 FT
Elevation: ---

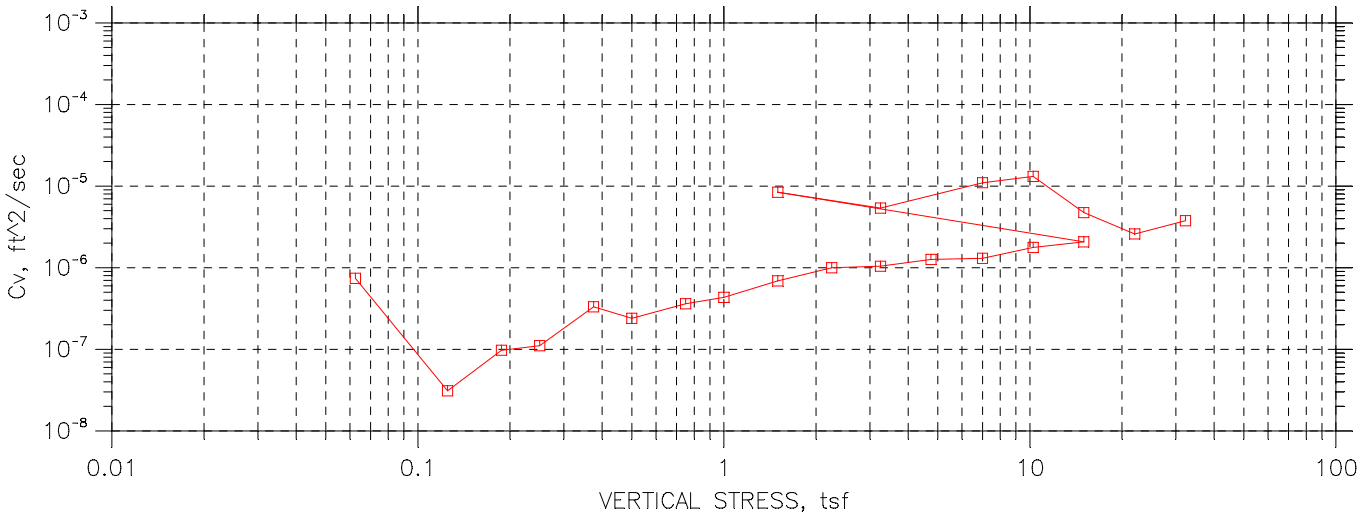
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.08751	0.986	7.92	0.4	0.0	1.58e-005	0.00e+000	1.58e-005
2	0.125	0.08961	0.982	8.11	21.9	0.0	2.69e-007	0.00e+000	2.69e-007
3	0.188	0.09172	0.978	8.30	7.0	8.1	8.33e-007	7.22e-007	7.74e-007
4	0.25	0.09339	0.975	8.45	7.0	0.0	8.35e-007	0.00e+000	8.35e-007
5	0.375	0.09616	0.970	8.70	3.7	3.7	1.59e-006	1.58e-006	1.59e-006
6	0.5	0.09855	0.965	8.92	3.9	3.4	1.48e-006	1.72e-006	1.59e-006
7	0.75	0.1024	0.957	9.27	2.3	2.9	2.52e-006	1.97e-006	2.22e-006
8	1	0.106	0.950	9.59	3.2	2.7	1.81e-006	2.11e-006	1.95e-006
9	1.5	0.1122	0.938	10.15	2.2	2.2	2.58e-006	2.52e-006	2.55e-006
10	2.25	0.1225	0.918	11.09	4.6	3.7	1.22e-006	1.49e-006	1.34e-006
11	3.25	0.1389	0.886	12.57	4.8	8.8	1.12e-006	6.17e-007	7.96e-007
12	4.75	0.165	0.835	14.93	6.9	5.1	7.53e-007	1.02e-006	8.68e-007
13	7	0.1889	0.789	17.10	3.6	3.9	1.37e-006	1.27e-006	1.32e-006
14	10.3	0.2117	0.744	19.15	3.5	3.1	1.35e-006	1.48e-006	1.41e-006
15	15	0.2343	0.700	21.21	2.1	2.8	2.14e-006	1.61e-006	1.84e-006
16	7	0.2315	0.705	20.95	0.0	0.0	2.70e-004	0.00e+000	2.70e-004
17	3.25	0.227	0.714	20.54	0.5	0.2	9.64e-006	1.79e-005	1.25e-005
18	1.5	0.221	0.726	20.00	1.0	0.0	4.63e-006	0.00e+000	4.63e-006
19	0.75	0.2154	0.737	19.49	2.4	3.3	1.85e-006	1.36e-006	1.57e-006
20	1.5	0.2167	0.734	19.61	0.5	0.0	8.80e-006	0.00e+000	8.80e-006
21	3.25	0.2217	0.724	20.06	0.7	0.0	6.04e-006	0.00e+000	6.04e-006
22	7	0.2286	0.711	20.69	0.5	0.3	9.25e-006	1.61e-005	1.18e-005
23	10.3	0.2343	0.700	21.20	0.9	0.4	4.75e-006	1.22e-005	6.84e-006
24	15	0.2436	0.682	22.04	1.1	1.4	3.78e-006	3.13e-006	3.43e-006
25	22	0.2599	0.650	23.52	1.4	1.3	3.02e-006	3.09e-006	3.05e-006
26	32.3	0.2799	0.611	25.33	1.1	1.5	3.47e-006	2.63e-006	2.99e-006
27	7	0.2717	0.627	24.59	0.0	0.0	1.68e-004	0.00e+000	1.68e-004
28	1	0.2546	0.660	23.04	1.2	0.0	3.41e-006	0.00e+000	3.41e-006
29	0.25	0.2411	0.687	21.82	9.3	9.4	4.49e-007	4.46e-007	4.48e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-101	Tested By: Brian Fogg	Checked By:
	Sample No.: 5U	Test Date: 7/15/08	Depth: 75-77 FT
	Test No.: 210620	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 09:47:57

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 5U
Test No.: 210620

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/15/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 75-77 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.74
Initial Void Ratio: 0.85
Final Void Ratio: 0.51

Liquid Limit: 22
Plastic Limit: 20
Plasticity Index: 2

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	9	RING	RING	157
Wt. Container + Wet Soil, gm	181.62	419.68	405.02	206.66
Wt. Container + Dry Soil, gm	150.47	382.75	382.75	184.41
Wt. Container, gm	47.58	262.17	262.17	63.93
Wt. Dry Soil, gm	102.89	120.58	120.58	120.48
Water Content, %	30.28	30.63	18.47	18.47
Void Ratio	---	0.85	0.51	---
Degree of Saturation, %	---	98.49	100.07	---
Dry Unit Weight, pcf	---	92.359	113.61	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 5U
Test No.: 210620

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/15/08
Sample Type: Shelby Tube

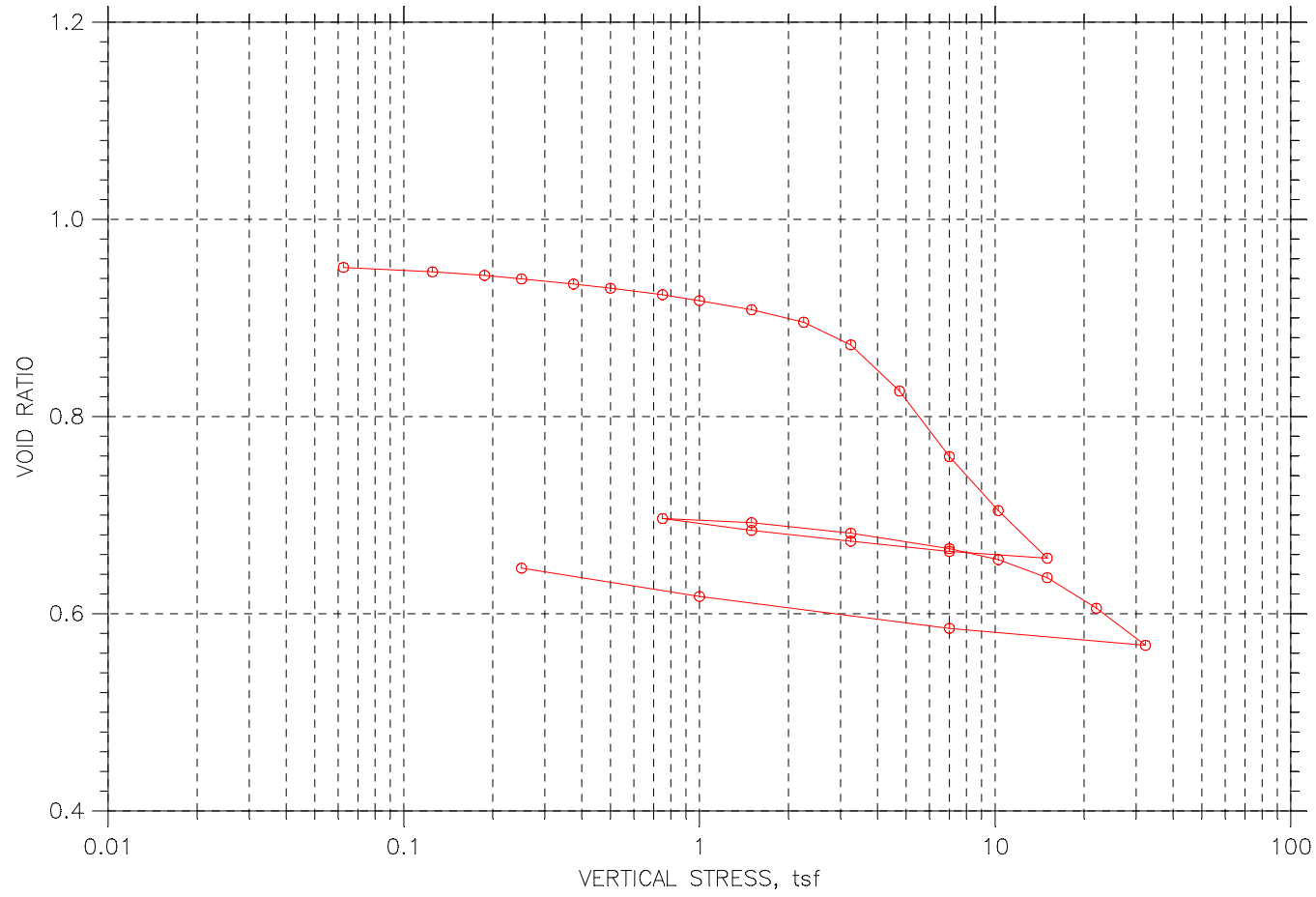
Project No.: 15600.00
Checked By:
Depth: 75-77 FT
Elevation: ---

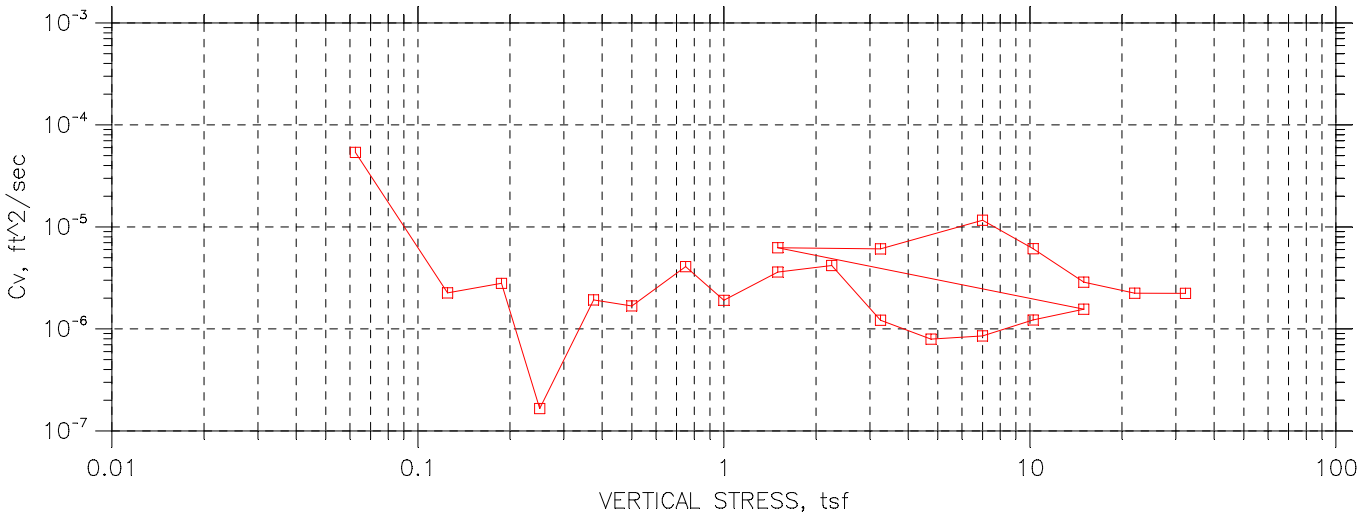
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.02632	0.805	2.57	7.9	0.0	7.41e-007	0.00e+000	7.41e-007
2	0.125	0.04601	0.769	4.49	174.6	185.5	3.20e-008	3.01e-008	3.10e-008
3	0.188	0.05846	0.746	5.70	55.5	0.0	9.74e-008	0.00e+000	9.74e-008
4	0.25	0.06459	0.735	6.30	47.9	0.0	1.11e-007	0.00e+000	1.11e-007
5	0.375	0.07178	0.722	7.00	15.8	0.0	3.32e-007	0.00e+000	3.32e-007
6	0.5	0.08152	0.705	7.95	19.5	23.3	2.63e-007	2.21e-007	2.40e-007
7	0.75	0.08961	0.690	8.74	13.9	0.0	3.62e-007	0.00e+000	3.62e-007
8	1	0.0982	0.675	9.58	11.4	0.0	4.34e-007	0.00e+000	4.34e-007
9	1.5	0.1113	0.651	10.85	7.0	7.1	6.94e-007	6.84e-007	6.89e-007
10	2.25	0.124	0.628	12.09	4.7	4.7	1.00e-006	9.94e-007	9.99e-007
11	3.25	0.1359	0.607	13.26	4.6	4.1	9.85e-007	1.11e-006	1.04e-006
12	4.75	0.149	0.583	14.53	3.5	3.6	1.27e-006	1.25e-006	1.26e-006
13	7	0.1626	0.558	15.86	3.4	3.2	1.26e-006	1.35e-006	1.30e-006
14	10.3	0.1765	0.533	17.21	2.3	2.5	1.84e-006	1.70e-006	1.77e-006
15	15	0.1919	0.505	18.72	1.8	2.1	2.28e-006	1.90e-006	2.07e-006
16	7	0.189	0.511	18.43	0.1	0.0	5.09e-005	0.00e+000	5.09e-005
17	3.25	0.1854	0.517	18.08	0.5	0.2	8.06e-006	2.50e-005	1.22e-005
18	1.5	0.181	0.525	17.65	1.2	0.0	3.45e-006	0.00e+000	3.45e-006
19	0.75	0.1765	0.533	17.21	3.5	0.0	1.15e-006	0.00e+000	1.15e-006
20	1.5	0.178	0.531	17.36	0.5	0.0	8.43e-006	0.00e+000	8.43e-006
21	3.25	0.1817	0.524	17.72	0.8	0.0	5.38e-006	0.00e+000	5.38e-006
22	7	0.1868	0.515	18.22	0.5	0.3	8.80e-006	1.47e-005	1.10e-005
23	10.3	0.191	0.507	18.62	0.4	0.2	9.35e-006	2.21e-005	1.31e-005
24	15	0.1977	0.495	19.27	1.1	0.6	3.53e-006	7.17e-006	4.73e-006
25	22	0.2084	0.476	20.32	1.5	0.0	2.59e-006	0.00e+000	2.59e-006
26	32.3	0.2219	0.451	21.64	0.9	1.1	4.10e-006	3.49e-006	3.77e-006
27	7	0.2157	0.462	21.04	0.0	0.0	1.71e-004	0.00e+000	1.71e-004
28	1	0.2027	0.486	19.76	1.0	0.0	3.79e-006	0.00e+000	3.79e-006
29	0.25	0.1918	0.506	18.70	7.0	6.6	5.59e-007	5.92e-007	5.75e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-101	Tested By: Brian Fogg	Checked By:
	Sample No.: 6U	Test Date: 7/17/08	Depth: 85-87 FT
	Test No.: 210621	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 09:54:36

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 6U
Test No.: 210621

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 85-87 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.73
Initial Void Ratio: 0.96
Final Void Ratio: 0.65

Liquid Limit: 31
Plastic Limit: 22
Plasticity Index: 9

Initial Height: 1.02 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	162	RING	RING	223
Wt. Container + Wet Soil, gm	212.77	413.47	401.55	195.42
Wt. Container + Dry Soil, gm	175.36	374.83	374.83	168.75
Wt. Container, gm	66.53	262.1	262.1	56.25
Wt. Dry Soil, gm	108.83	112.73	112.73	112.5
Water Content, %	34.37	34.28	23.71	23.71
Void Ratio	---	0.96	0.65	---
Degree of Saturation, %	---	97.74	100.15	---
Dry Unit Weight, pcf	---	87.064	103.53	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-101
Sample No.: 6U
Test No.: 210621

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

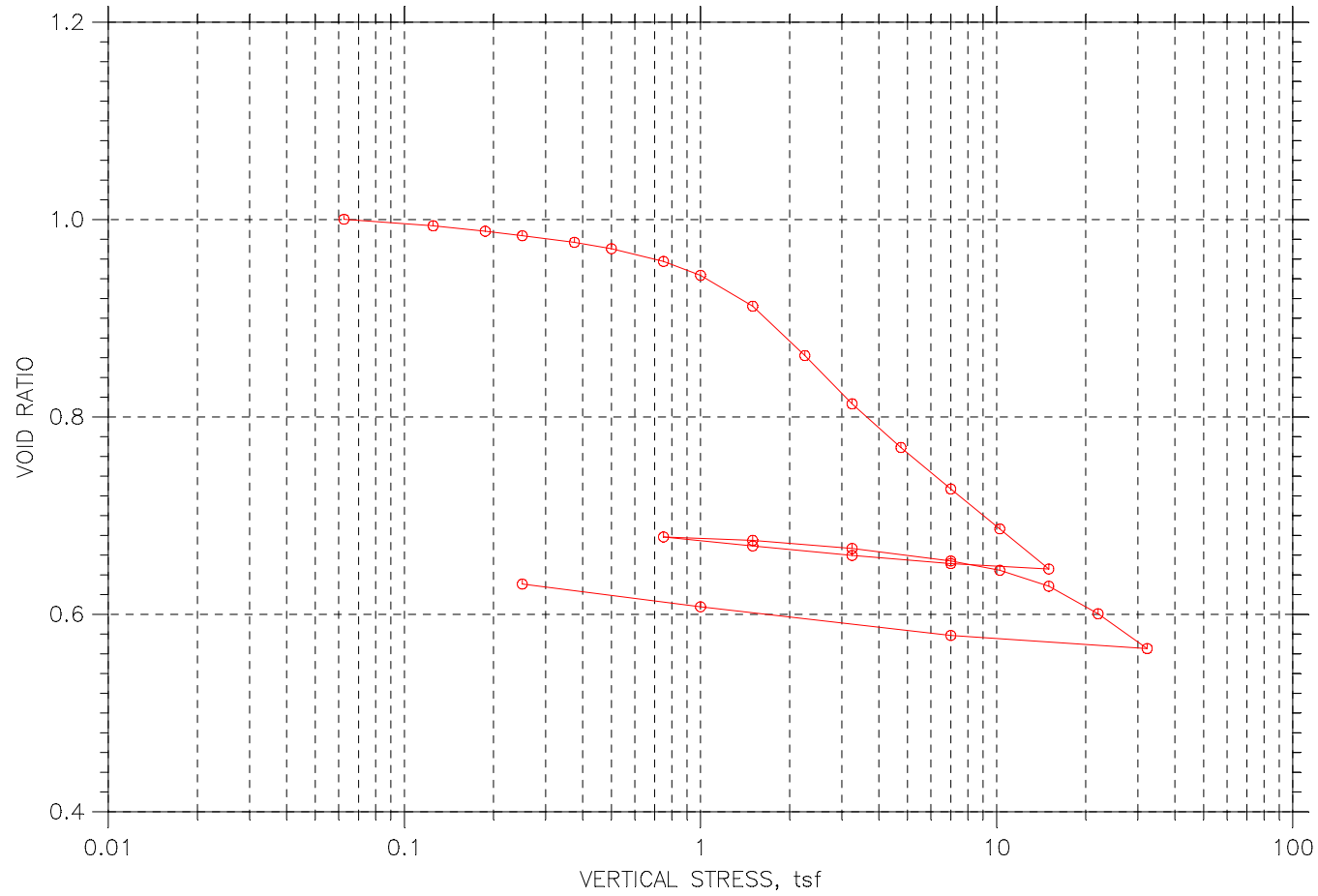
Project No.: 15600.00
Checked By:
Depth: 85-87 FT
Elevation: ---

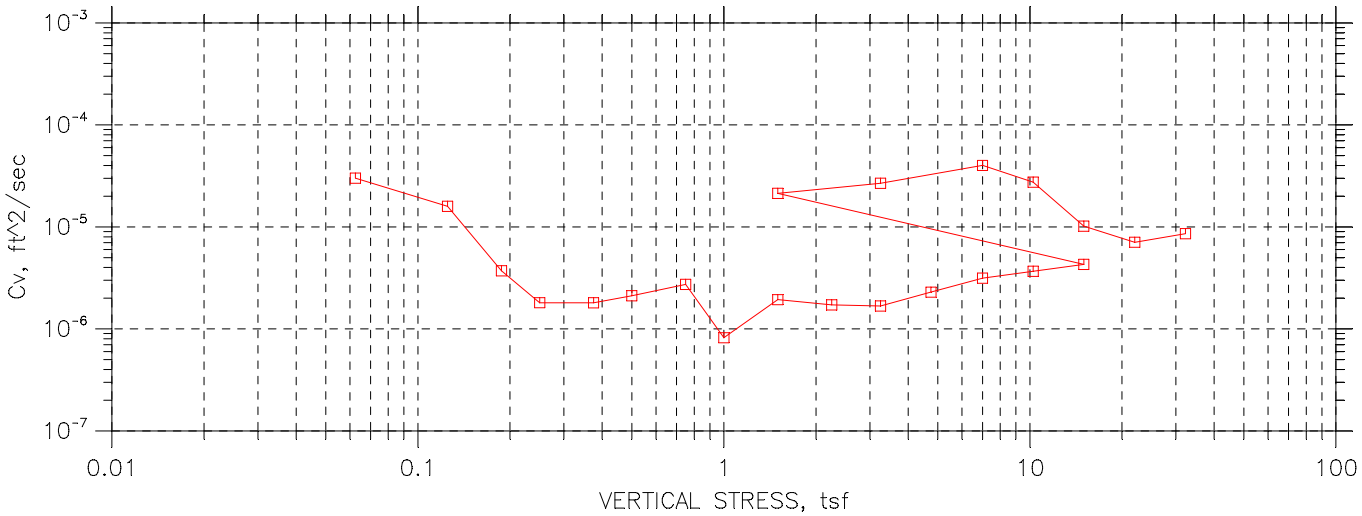
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.003247	0.951	0.32	0.1	0.0	5.37e-005	0.00e+000	5.37e-005
2	0.125	0.005603	0.947	0.55	2.8	2.4	2.08e-006	2.46e-006	2.26e-006
3	0.188	0.007458	0.943	0.73	2.0	2.2	2.97e-006	2.65e-006	2.80e-006
4	0.25	0.009327	0.940	0.92	35.1	0.0	1.66e-007	0.00e+000	1.66e-007
5	0.375	0.012	0.934	1.18	3.6	2.4	1.61e-006	2.39e-006	1.92e-006
6	0.5	0.01424	0.930	1.40	4.4	2.5	1.32e-006	2.31e-006	1.68e-006
7	0.75	0.01762	0.924	1.73	1.6	1.2	3.63e-006	4.65e-006	4.08e-006
8	1	0.02079	0.917	2.04	3.0	0.0	1.91e-006	0.00e+000	1.91e-006
9	1.5	0.02556	0.908	2.51	1.6	1.6	3.63e-006	3.60e-006	3.61e-006
10	2.25	0.03213	0.896	3.16	1.6	1.1	3.58e-006	5.05e-006	4.19e-006
11	3.25	0.04408	0.873	4.33	4.5	0.0	1.22e-006	0.00e+000	1.22e-006
12	4.75	0.0684	0.826	6.73	7.0	6.3	7.52e-007	8.40e-007	7.94e-007
13	7	0.1029	0.759	10.12	7.0	4.6	7.09e-007	1.07e-006	8.54e-007
14	10.3	0.1314	0.705	12.92	3.5	4.1	1.33e-006	1.13e-006	1.22e-006
15	15	0.1565	0.656	15.39	2.3	3.2	1.86e-006	1.34e-006	1.56e-006
16	7	0.1529	0.663	15.03	0.1	0.0	7.06e-005	0.00e+000	7.06e-005
17	3.25	0.1475	0.674	14.51	0.5	0.0	9.28e-006	0.00e+000	9.28e-006
18	1.5	0.1418	0.685	13.94	1.0	0.0	4.46e-006	0.00e+000	4.46e-006
19	0.75	0.1356	0.697	13.33	3.5	3.6	1.25e-006	1.23e-006	1.24e-006
20	1.5	0.1377	0.692	13.54	0.7	0.0	6.25e-006	0.00e+000	6.25e-006
21	3.25	0.1433	0.682	14.09	0.7	0.0	6.09e-006	0.00e+000	6.09e-006
22	7	0.1515	0.666	14.89	0.5	0.2	8.29e-006	1.94e-005	1.16e-005
23	10.3	0.1572	0.655	15.46	0.7	0.7	6.05e-006	6.17e-006	6.11e-006
24	15	0.1667	0.637	16.39	1.4	1.5	2.99e-006	2.77e-006	2.87e-006
25	22	0.1829	0.606	17.98	1.8	1.8	2.28e-006	2.20e-006	2.24e-006
26	32.3	0.2024	0.568	19.90	1.6	1.8	2.36e-006	2.11e-006	2.23e-006
27	7	0.1935	0.585	19.02	0.0	0.0	1.04e-004	0.00e+000	1.04e-004
28	1	0.1767	0.617	17.37	1.8	2.2	2.21e-006	1.82e-006	2.00e-006
29	0.25	0.1617	0.646	15.90	7.1	8.8	5.81e-007	4.66e-007	5.17e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-102	Tested By: Brian Fogg	Checked By:
	Sample No.: 1U	Test Date: 7/17/08	Depth: 19-21 FT
	Test No.: 210622	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:15:18

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 1U
Test No.: 210622

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 19-21 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.63
Initial Void Ratio: 1.01
Final Void Ratio: 0.63

Liquid Limit: NP
Plastic Limit: NP
Plasticity Index: NP

Initial Height: 1.05 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	118	RING	RING	20
Wt. Container + Wet Soil, gm	215.39	411.1	396.83	187.31
Wt. Container + Dry Soil, gm	177.47	370.76	370.76	161.3
Wt. Container, gm	69.54	262.17	262.17	52.95
Wt. Dry Soil, gm	107.93	108.59	108.59	108.35
Water Content, %	35.13	37.15	24.01	24.01
Void Ratio	---	1.01	0.63	---
Degree of Saturation, %	---	96.40	100.11	---
Dry Unit Weight, pcf	---	81.546	100.69	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 1U
Test No.: 210622

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

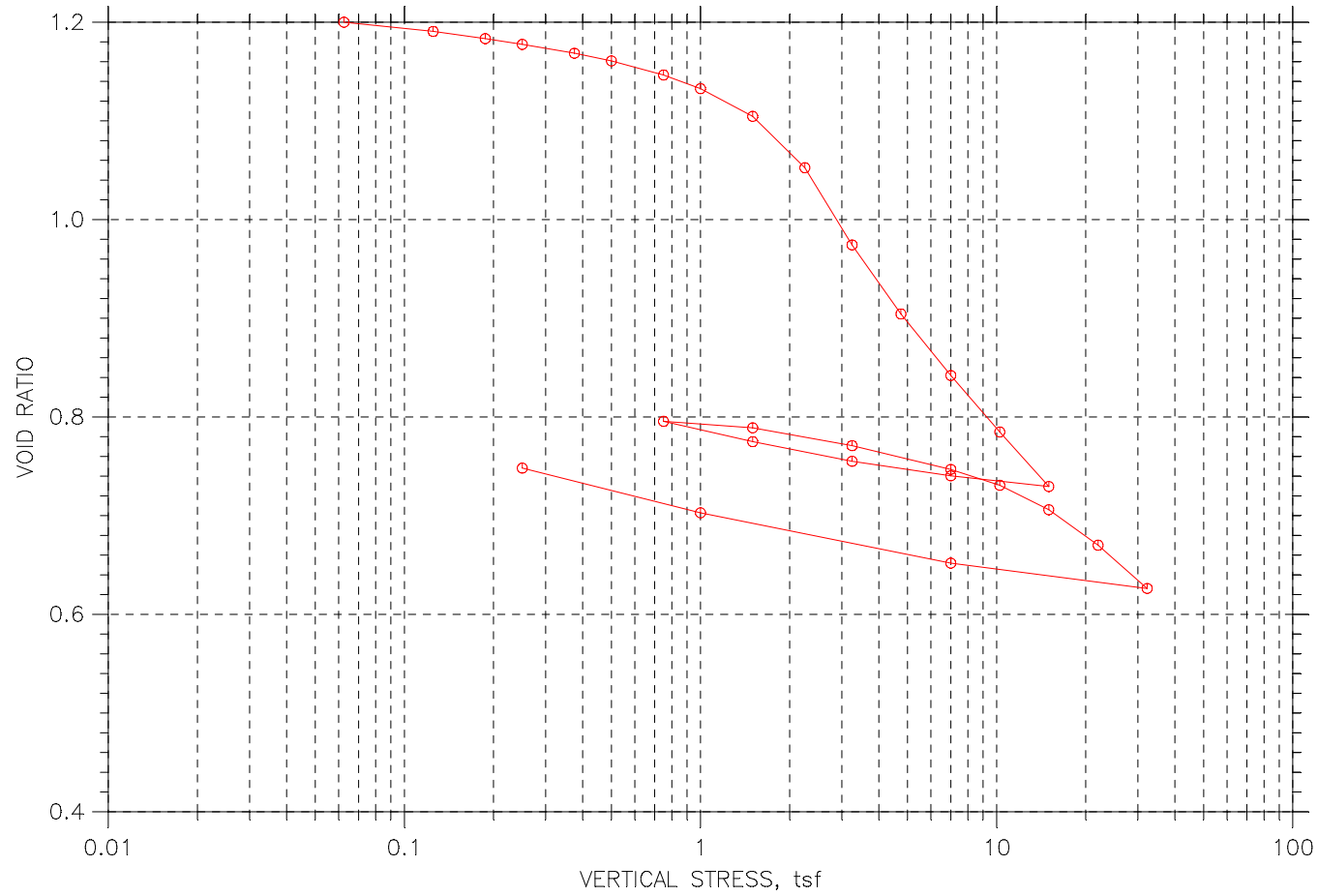
Project No.: 15600.00
Checked By:
Depth: 19-21 FT
Elevation: ---

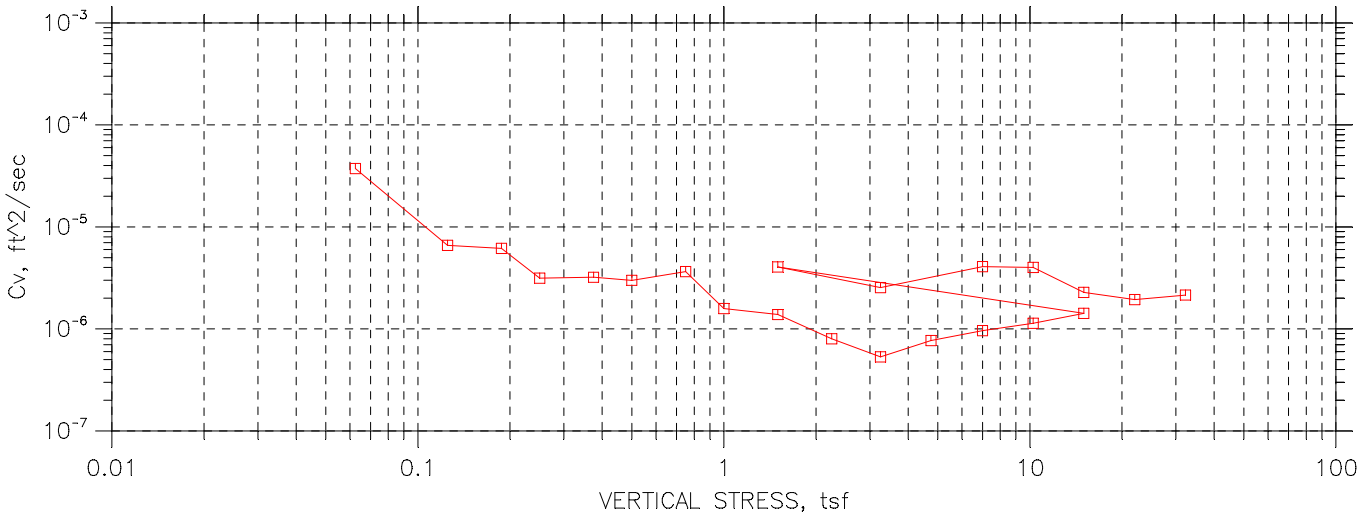
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.006796	1.000	0.65	0.2	0.2	2.50e-005	3.76e-005	3.01e-005
2	0.125	0.01027	0.994	0.98	0.5	0.3	1.32e-005	2.00e-005	1.59e-005
3	0.188	0.01309	0.988	1.25	1.6	0.0	3.71e-006	0.00e+000	3.71e-006
4	0.25	0.01546	0.984	1.48	3.4	0.0	1.81e-006	0.00e+000	1.81e-006
5	0.375	0.01895	0.977	1.81	3.3	0.0	1.80e-006	0.00e+000	1.80e-006
6	0.5	0.0223	0.970	2.13	2.8	0.0	2.12e-006	0.00e+000	2.12e-006
7	0.75	0.02902	0.958	2.77	2.2	0.0	2.73e-006	0.00e+000	2.73e-006
8	1	0.0363	0.944	3.47	7.2	0.0	8.19e-007	0.00e+000	8.19e-007
9	1.5	0.05258	0.912	5.03	3.4	2.5	1.67e-006	2.30e-006	1.94e-006
10	2.25	0.07857	0.862	7.51	3.4	3.0	1.61e-006	1.85e-006	1.72e-006
11	3.25	0.104	0.813	9.94	3.5	2.7	1.47e-006	1.96e-006	1.68e-006
12	4.75	0.127	0.769	12.14	2.2	2.1	2.22e-006	2.37e-006	2.29e-006
13	7	0.1488	0.727	14.23	1.5	1.5	3.08e-006	3.21e-006	3.14e-006
14	10.3	0.1698	0.687	16.23	1.3	1.1	3.36e-006	4.06e-006	3.68e-006
15	15	0.1909	0.646	18.25	1.3	0.7	3.29e-006	6.16e-006	4.29e-006
16	7	0.188	0.651	17.98	0.0	0.0	5.23e-004	0.00e+000	5.23e-004
17	3.25	0.1838	0.660	17.57	0.1	0.0	5.49e-005	0.00e+000	5.49e-005
18	1.5	0.1788	0.669	17.09	0.2	0.2	1.80e-005	2.03e-005	1.91e-005
19	0.75	0.174	0.678	16.64	0.9	1.0	4.77e-006	4.26e-006	4.50e-006
20	1.5	0.1759	0.675	16.81	0.2	0.2	1.83e-005	2.55e-005	2.13e-005
21	3.25	0.1801	0.667	17.22	0.1	0.2	2.87e-005	2.51e-005	2.68e-005
22	7	0.1867	0.654	17.85	0.1	0.1	4.29e-005	3.76e-005	4.01e-005
23	10.3	0.1916	0.645	18.32	0.2	0.1	2.27e-005	3.49e-005	2.75e-005
24	15	0.2	0.628	19.12	0.7	0.1	6.17e-006	2.91e-005	1.02e-005
25	22	0.2144	0.601	20.50	0.9	0.3	4.61e-006	1.52e-005	7.07e-006
26	32.3	0.2328	0.565	22.25	0.7	0.2	5.69e-006	1.74e-005	8.58e-006
27	7	0.2259	0.579	21.59	0.0	0.0	4.42e-004	0.00e+000	4.42e-004
28	1	0.2108	0.608	20.15	0.5	0.0	8.42e-006	0.00e+000	8.42e-006
29	0.25	0.1989	0.631	19.01	2.5	3.3	1.64e-006	1.21e-006	1.39e-006

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-102	Tested By: Brian Fogg	Checked By:
	Sample No.: 3U	Test Date: 07/14/2008	Depth: 41-43 FT
	Test No.: 210624	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:17:53

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 3U
Test No.: 210624

Location: AUBURN
Tested By: Brian Fogg
Test Date: 07/14/2008
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 41-43 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.77
Initial Void Ratio: 1.21
Final Void Ratio: 0.75

Liquid Limit: 27
Plastic Limit: 19
Plasticity Index: 8

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	1	RING	RING	70
Wt. Container + Wet Soil, gm	183.25	406.73	391.86	192.25
Wt. Container + Dry Soil, gm	147.67	364.25	364.25	164.68
Wt. Container, gm	64.26	262.15	262.15	62.73
Wt. Dry Soil, gm	83.41	102.1	102.1	101.95
Water Content, %	42.66	41.61	27.04	27.04
Void Ratio	---	1.21	0.75	---
Degree of Saturation, %	---	95.15	100.13	---
Dry Unit Weight, pcf	---	78.203	98.92	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 3U
Test No.: 210624

Location: AUBURN
Tested By: Brian Fogg
Test Date: 07/14/2008
Sample Type: Shelby Tube

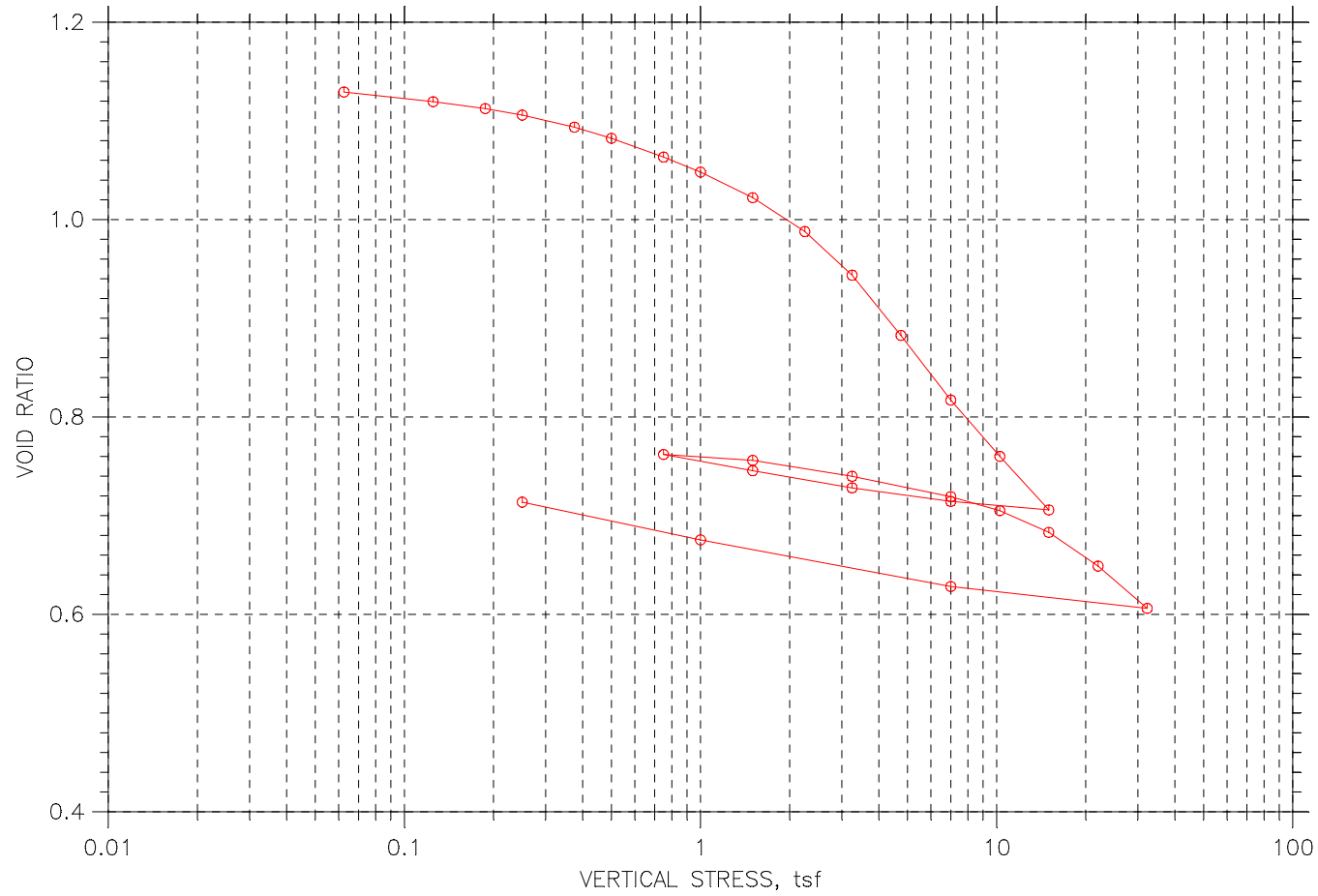
Project No.: 15600.00
Checked By:
Depth: 41-43 FT
Elevation: ---

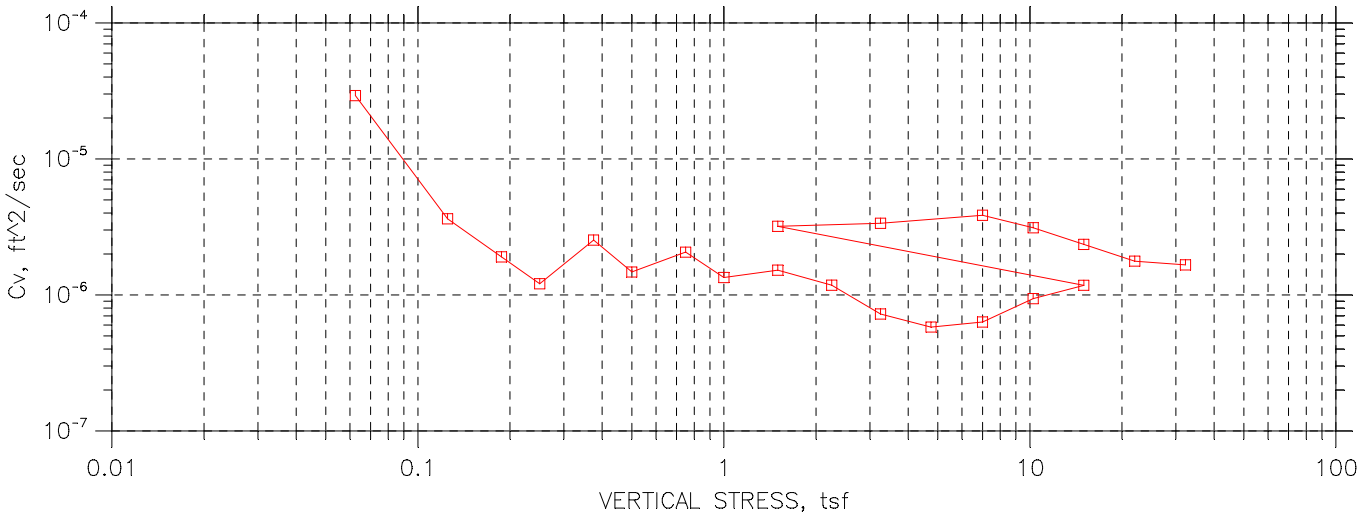
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.005224	1.200	0.51	0.2	0.2	3.66e-005	3.81e-005	3.73e-005
2	0.125	0.009557	1.191	0.93	0.9	0.0	6.60e-006	0.00e+000	6.60e-006
3	0.188	0.01293	1.183	1.26	1.0	0.0	6.16e-006	0.00e+000	6.16e-006
4	0.25	0.01561	1.178	1.52	2.0	1.7	2.94e-006	3.39e-006	3.14e-006
5	0.375	0.01981	1.169	1.93	1.8	1.8	3.17e-006	3.25e-006	3.21e-006
6	0.5	0.02342	1.161	2.28	2.1	1.8	2.79e-006	3.23e-006	3.00e-006
7	0.75	0.03004	1.146	2.93	1.6	1.5	3.48e-006	3.83e-006	3.65e-006
8	1	0.03645	1.133	3.55	3.5	3.6	1.60e-006	1.57e-006	1.58e-006
9	1.5	0.04949	1.105	4.83	4.6	3.3	1.19e-006	1.65e-006	1.38e-006
10	2.25	0.07361	1.053	7.18	6.9	6.4	7.72e-007	8.29e-007	7.99e-007
11	3.25	0.1099	0.974	10.72	9.1	9.6	5.45e-007	5.18e-007	5.31e-007
12	4.75	0.1423	0.904	13.88	6.9	5.1	6.69e-007	9.04e-007	7.69e-007
13	7	0.1712	0.842	16.70	4.6	4.3	9.31e-007	9.94e-007	9.62e-007
14	10.3	0.1978	0.785	19.28	3.5	3.7	1.17e-006	1.10e-006	1.13e-006
15	15	0.2234	0.729	21.79	2.3	3.0	1.61e-006	1.27e-006	1.42e-006
16	7	0.2184	0.740	21.29	0.2	0.0	1.77e-005	0.00e+000	1.77e-005
17	3.25	0.2116	0.755	20.63	0.7	0.0	5.50e-006	0.00e+000	5.50e-006
18	1.5	0.2023	0.775	19.73	2.2	0.0	1.75e-006	0.00e+000	1.75e-006
19	0.75	0.1928	0.796	18.80	4.7	4.0	8.41e-007	9.82e-007	9.06e-007
20	1.5	0.1958	0.789	19.10	1.0	0.0	4.05e-006	0.00e+000	4.05e-006
21	3.25	0.2042	0.771	19.91	1.5	0.0	2.54e-006	0.00e+000	2.54e-006
22	7	0.2154	0.747	21.00	0.9	0.0	4.08e-006	0.00e+000	4.08e-006
23	10.3	0.2228	0.731	21.73	1.1	0.8	3.37e-006	4.93e-006	4.01e-006
24	15	0.2343	0.706	22.85	1.6	1.6	2.29e-006	2.27e-006	2.28e-006
25	22	0.251	0.670	24.48	1.6	2.0	2.22e-006	1.72e-006	1.94e-006
26	32.3	0.2713	0.626	26.46	1.4	1.7	2.39e-006	1.95e-006	2.15e-006
27	7	0.2594	0.652	25.30	0.1	0.0	3.44e-005	1.81e-004	5.78e-005
28	1	0.2358	0.703	22.99	2.0	2.9	1.71e-006	1.19e-006	1.41e-006
29	0.25	0.2148	0.748	20.94	11.8	13.5	3.11e-007	2.71e-007	2.90e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-102	Tested By: Brian Fogg	Checked By:
	Sample No.: 5U	Test Date: 7/17/2008	Depth: 59-61 FT
	Test No.: 210626	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:20:56

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 5U
Test No.: 210626

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/2008
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 59-61 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.76
Initial Void Ratio: 1.14
Final Void Ratio: 0.71

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

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	69	RING	RING	52
Wt. Container + Wet Soil, gm	138.94	408.46	395.03	196.12
Wt. Container + Dry Soil, gm	118.38	367.71	367.71	169.1
Wt. Container, gm	62.45	262.1	262.1	64.64
Wt. Dry Soil, gm	55.93	105.61	105.61	104.46
Water Content, %	36.76	38.58	25.87	25.87
Void Ratio	---	1.14	0.71	---
Degree of Saturation, %	---	93.21	100.02	---
Dry Unit Weight, pcf	---	80.423	100.54	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-102
Sample No.: 5U
Test No.: 210626

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/2008
Sample Type: Shelby Tube

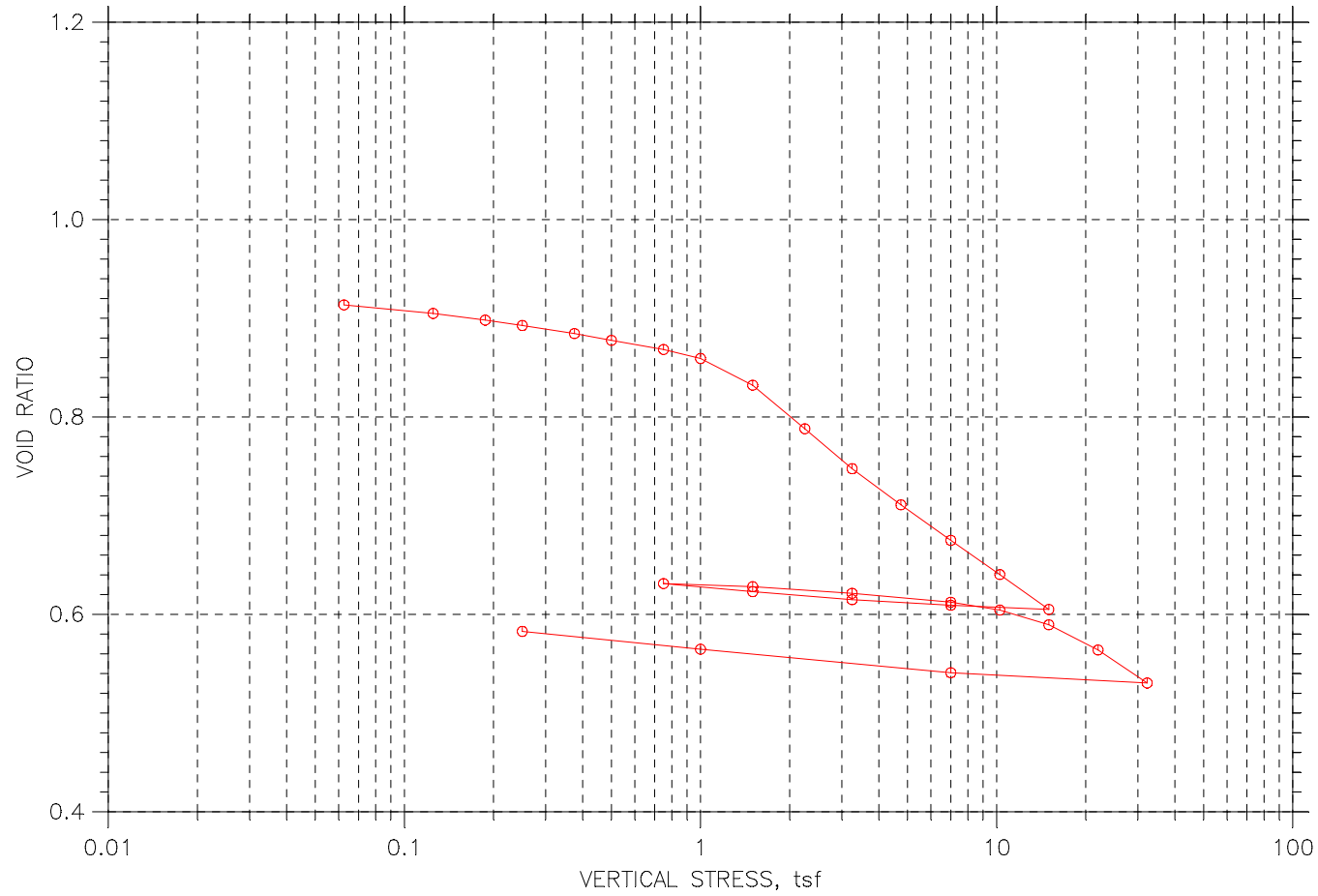
Project No.: 15600.00
Checked By:
Depth: 59-61 FT
Elevation: ---

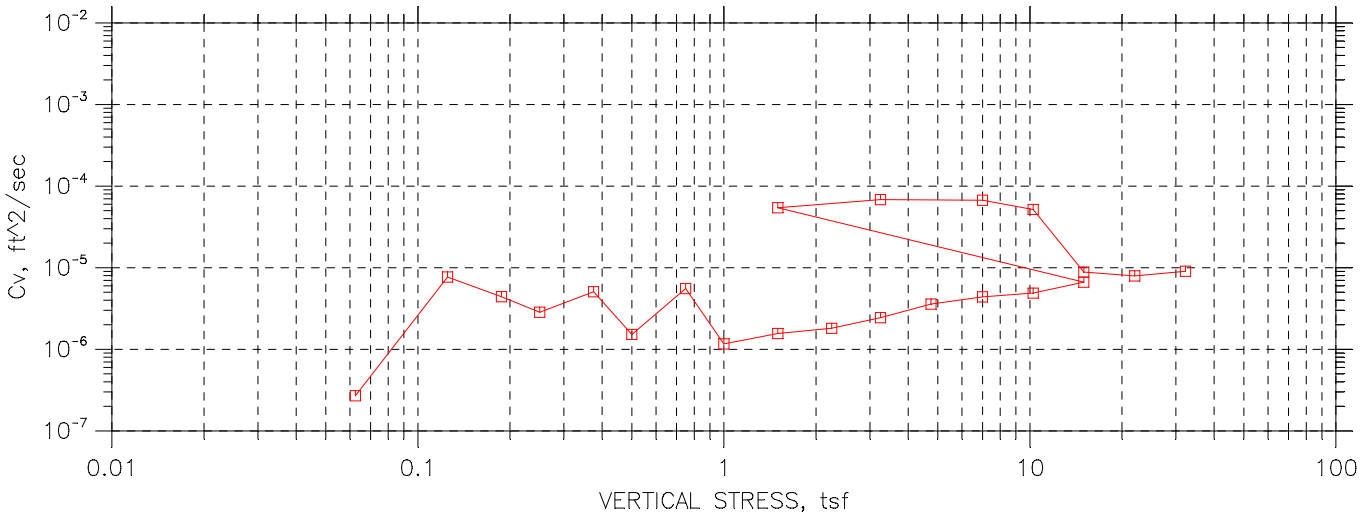
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.00634	1.129	0.61	0.2	0.2	2.43e-005	3.66e-005	2.92e-005
2	0.125	0.01108	1.119	1.07	1.6	0.0	3.63e-006	0.00e+000	3.63e-006
3	0.188	0.01445	1.112	1.40	3.4	2.9	1.77e-006	2.06e-006	1.90e-006
4	0.25	0.01764	1.106	1.71	4.9	0.0	1.21e-006	0.00e+000	1.21e-006
5	0.375	0.02351	1.094	2.28	2.3	2.3	2.56e-006	2.50e-006	2.53e-006
6	0.5	0.02894	1.082	2.81	4.5	3.3	1.27e-006	1.75e-006	1.47e-006
7	0.75	0.0381	1.063	3.69	3.6	1.9	1.59e-006	2.94e-006	2.06e-006
8	1	0.04543	1.048	4.40	4.6	3.7	1.21e-006	1.51e-006	1.34e-006
9	1.5	0.0579	1.022	5.61	3.6	3.6	1.54e-006	1.50e-006	1.52e-006
10	2.25	0.07442	0.988	7.21	5.0	4.1	1.07e-006	1.31e-006	1.18e-006
11	3.25	0.09569	0.944	9.28	7.1	7.0	7.17e-007	7.30e-007	7.23e-007
12	4.75	0.1251	0.883	12.13	9.2	7.5	5.29e-007	6.43e-007	5.80e-007
13	7	0.1567	0.817	15.19	7.0	7.3	6.43e-007	6.23e-007	6.33e-007
14	10.3	0.1841	0.760	17.85	4.8	4.3	8.91e-007	9.94e-007	9.40e-007
15	15	0.2102	0.706	20.38	3.4	3.4	1.18e-006	1.18e-006	1.18e-006
16	7	0.2061	0.714	19.98	0.2	0.0	2.51e-005	0.00e+000	2.51e-005
17	3.25	0.1994	0.728	19.34	0.7	0.0	5.53e-006	0.00e+000	5.53e-006
18	1.5	0.1911	0.746	18.52	1.8	2.4	2.28e-006	1.70e-006	1.95e-006
19	0.75	0.1831	0.762	17.76	4.8	4.3	8.42e-007	9.55e-007	8.95e-007
20	1.5	0.1861	0.756	18.04	1.3	0.0	3.20e-006	0.00e+000	3.20e-006
21	3.25	0.1938	0.740	18.79	1.2	0.0	3.36e-006	0.00e+000	3.36e-006
22	7	0.2038	0.719	19.75	1.0	0.0	3.85e-006	0.00e+000	3.85e-006
23	10.3	0.2106	0.705	20.41	1.2	1.3	3.30e-006	2.96e-006	3.12e-006
24	15	0.2212	0.683	21.44	1.6	1.6	2.36e-006	2.35e-006	2.35e-006
25	22	0.2376	0.649	23.04	2.1	2.1	1.78e-006	1.76e-006	1.77e-006
26	32.3	0.2582	0.606	25.03	1.7	2.5	2.10e-006	1.38e-006	1.66e-006
27	7	0.2476	0.628	24.00	0.1	0.0	2.78e-005	0.00e+000	2.78e-005
28	1	0.2248	0.675	21.80	2.4	2.8	1.51e-006	1.29e-006	1.39e-006
29	0.25	0.2064	0.714	20.01	18.9	0.0	2.01e-007	0.00e+000	2.01e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-103	Tested By: Brian Fogg	Checked By:
	Sample No.: 1U	Test Date: 7/17/08	Depth: 15-17 FT
	Test No.: 210627	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:23:13

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 1U
Test No.: 210627

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 15-17 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.78
Initial Void Ratio: 0.92
Final Void Ratio: 0.58

Liquid Limit: 28
Plastic Limit: 21
Plasticity Index: 7

Initial Height: 1.00 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	57	RING	RING	65
Wt. Container + Wet Soil, gm	179.43	415.18	401.77	199.52
Wt. Container + Dry Soil, gm	151.51	377.58	377.58	175.35
Wt. Container, gm	61.53	262.17	262.17	60.04
Wt. Dry Soil, gm	89.98	115.41	115.41	115.31
Water Content, %	31.03	32.58	20.96	20.96
Void Ratio	---	0.92	0.58	---
Degree of Saturation, %	---	98.63	100.00	---
Dry Unit Weight, pcf	---	90.471	109.65	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 1U
Test No.: 210627

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/17/08
Sample Type: Shelby Tube

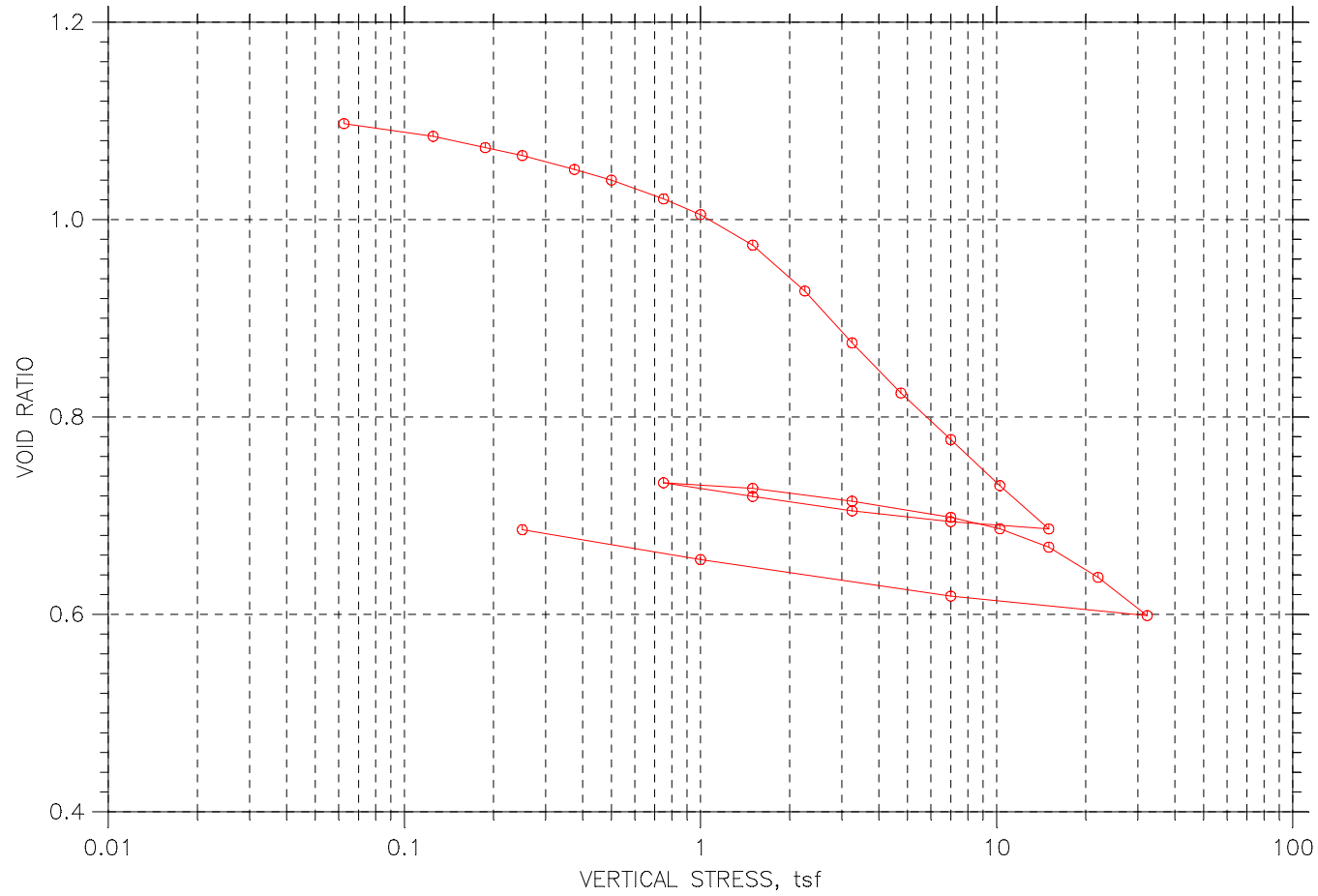
Project No.: 15600.00
Checked By:
Depth: 15-17 FT
Elevation: ---

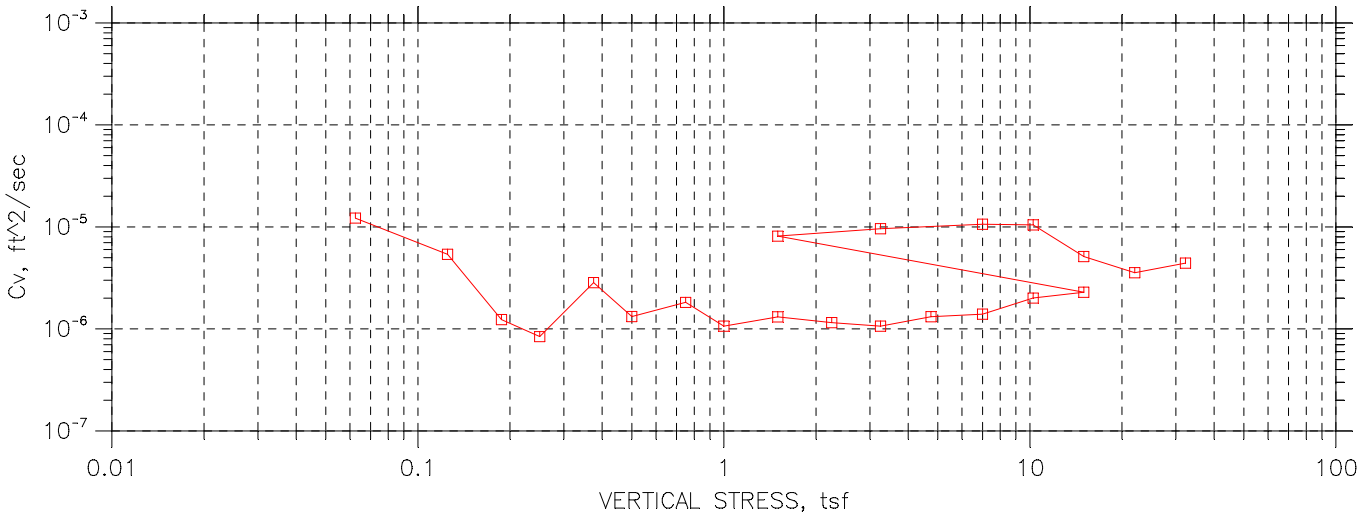
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.002486	0.914	0.25	21.2	0.0	2.70e-007	0.00e+000	2.70e-007
2	0.125	0.007022	0.905	0.70	0.9	0.6	6.38e-006	9.67e-006	7.69e-006
3	0.188	0.0105	0.898	1.05	1.3	0.0	4.42e-006	0.00e+000	4.42e-006
4	0.25	0.01338	0.893	1.34	2.0	0.0	2.85e-006	0.00e+000	2.85e-006
5	0.375	0.01763	0.885	1.76	1.5	0.7	3.67e-006	8.20e-006	5.07e-006
6	0.5	0.02124	0.878	2.12	3.6	0.0	1.52e-006	0.00e+000	1.52e-006
7	0.75	0.02608	0.868	2.60	1.4	0.5	3.85e-006	1.00e-005	5.56e-006
8	1	0.03082	0.859	3.08	4.6	0.0	1.17e-006	0.00e+000	1.17e-006
9	1.5	0.04497	0.832	4.49	3.4	0.0	1.56e-006	0.00e+000	1.56e-006
10	2.25	0.06803	0.788	6.79	3.4	2.3	1.51e-006	2.26e-006	1.81e-006
11	3.25	0.08915	0.748	8.90	2.2	1.8	2.24e-006	2.70e-006	2.44e-006
12	4.75	0.1083	0.711	10.81	1.5	1.1	3.12e-006	4.19e-006	3.58e-006
13	7	0.1271	0.675	12.69	1.3	0.7	3.46e-006	6.04e-006	4.40e-006
14	10.3	0.1452	0.640	14.49	1.1	0.7	3.92e-006	6.47e-006	4.88e-006
15	15	0.1636	0.605	16.33	0.9	0.4	4.75e-006	1.11e-005	6.66e-006
16	7	0.1615	0.609	16.11	0.0	0.0	5.93e-004	0.00e+000	5.93e-004
17	3.25	0.1585	0.615	15.82	0.1	0.0	6.57e-005	3.64e-003	1.29e-004
18	1.5	0.1543	0.623	15.40	0.1	0.1	2.99e-005	3.35e-005	3.16e-005
19	0.75	0.15	0.631	14.97	0.5	0.2	8.89e-006	1.88e-005	1.21e-005
20	1.5	0.1516	0.628	15.13	0.1	0.0	5.43e-005	0.00e+000	5.43e-005
21	3.25	0.1551	0.621	15.48	0.1	0.0	6.83e-005	0.00e+000	6.83e-005
22	7	0.1598	0.612	15.95	0.1	0.1	6.62e-005	6.80e-005	6.71e-005
23	10.3	0.164	0.604	16.37	0.1	0.1	3.96e-005	7.46e-005	5.18e-005
24	15	0.1718	0.589	17.15	0.8	0.1	4.95e-006	4.08e-005	8.83e-006
25	22	0.1851	0.564	18.48	0.8	0.2	4.81e-006	2.28e-005	7.94e-006
26	32.3	0.2026	0.531	20.22	0.7	0.1	5.49e-006	2.57e-005	9.04e-006
27	7	0.1972	0.541	19.68	0.0	0.0	4.77e-004	0.00e+000	4.77e-004
28	1	0.1847	0.565	18.44	0.2	0.0	2.46e-005	0.00e+000	2.46e-005
29	0.25	0.1753	0.583	17.49	1.2	0.0	3.18e-006	0.00e+000	3.18e-006

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-103	Tested By: Brian Fogg	Checked By:
	Sample No.: 2U	Test Date: 7/21/2008	Depth: 25-27 FT
	Test No.: 210628	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:26:02

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 2U
Test No.: 210628

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/21/2008
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 25-27 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.76
Initial Void Ratio: 1.16
Final Void Ratio: 0.69

Liquid Limit: 27
Plastic Limit: 23
Plasticity Index: 4

Initial Height: 1.04 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	218	RING	RING	35
Wt. Container + Wet Soil, gm	264.9	407.49	393.86	196.14
Wt. Container + Dry Soil, gm	217.95	367.61	367.61	170.03
Wt. Container, gm	65.67	262.14	262.14	65.1
Wt. Dry Soil, gm	152.28	105.47	105.47	104.93
Water Content, %	30.83	37.81	24.88	24.88
Void Ratio	---	1.16	0.69	---
Degree of Saturation, %	---	90.21	100.15	---
Dry Unit Weight, pcf	---	79.892	102.21	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 2U
Test No.: 210628

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/21/2008
Sample Type: Shelby Tube

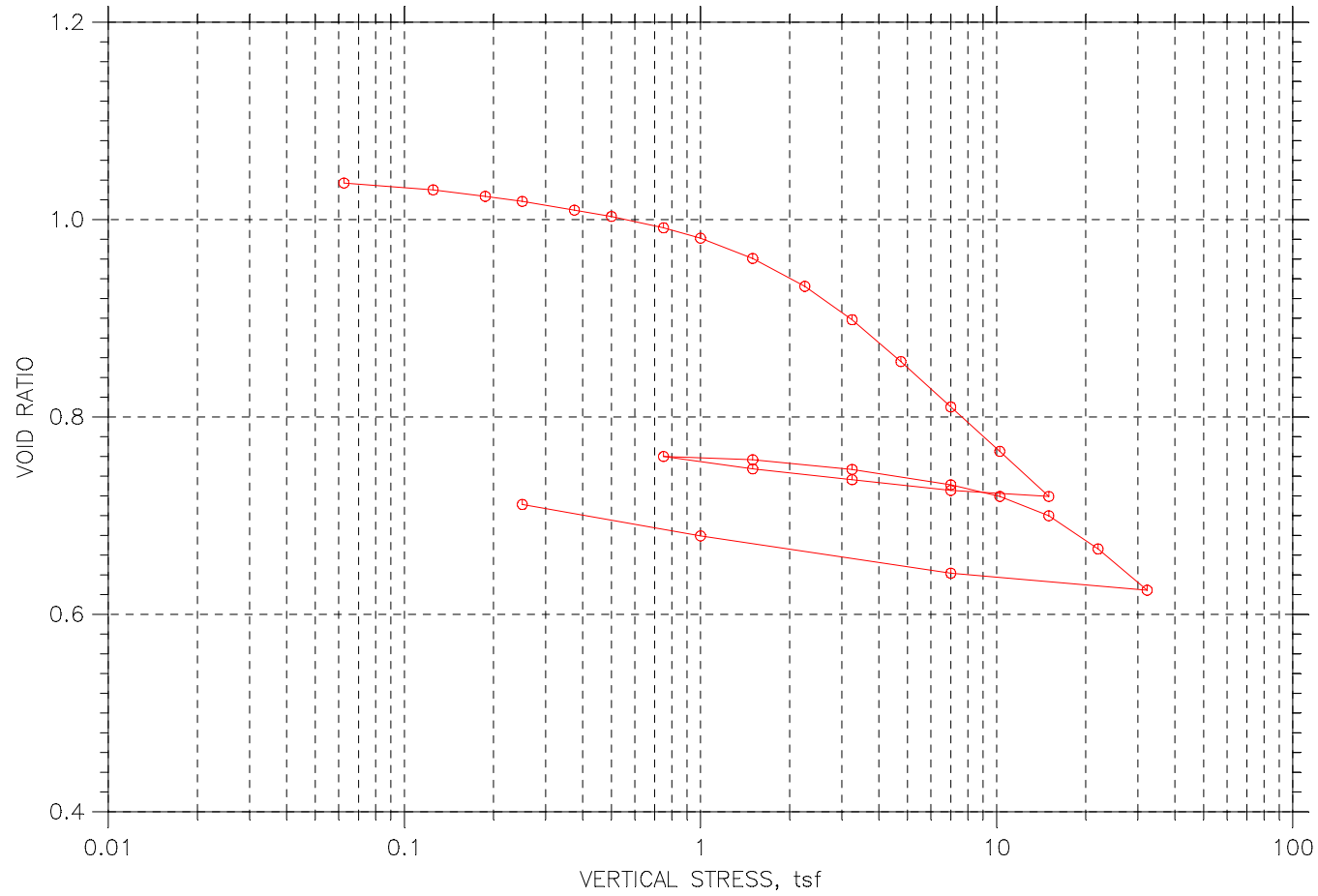
Project No.: 15600.00
Checked By:
Depth: 25-27 FT
Elevation: ---

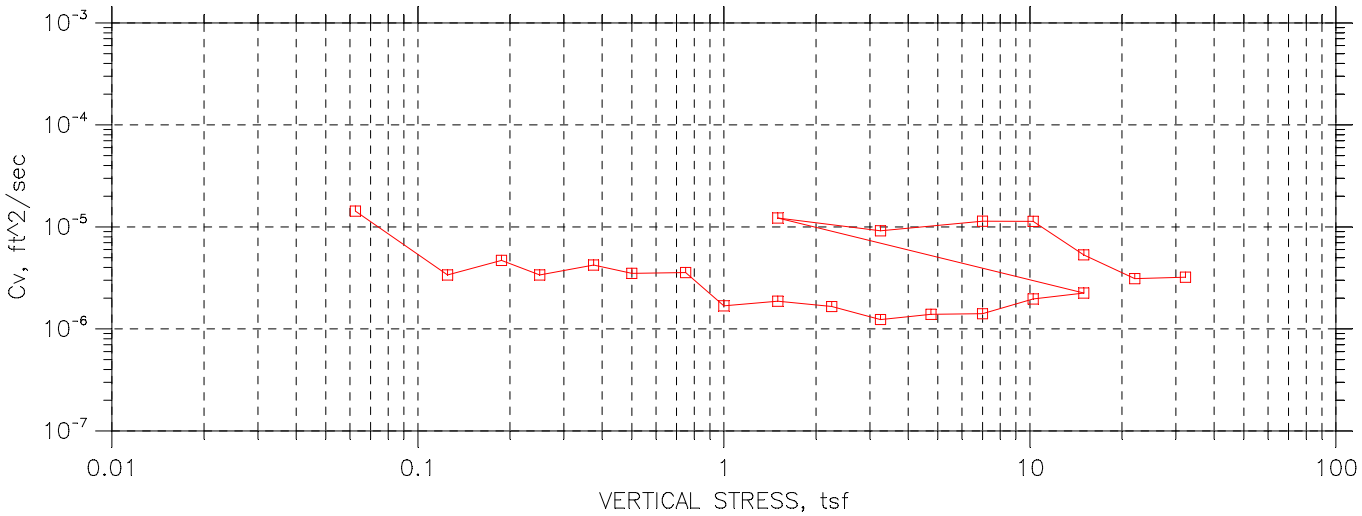
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.0286	1.097	2.76	0.6	0.4	1.07e-005	1.43e-005	1.22e-005
2	0.125	0.03476	1.084	3.35	1.4	0.8	4.25e-006	7.34e-006	5.38e-006
3	0.188	0.04031	1.073	3.89	4.6	0.0	1.23e-006	0.00e+000	1.23e-006
4	0.25	0.04416	1.065	4.26	6.7	0.0	8.40e-007	0.00e+000	8.40e-007
5	0.375	0.05089	1.051	4.91	2.2	1.7	2.49e-006	3.32e-006	2.84e-006
6	0.5	0.05612	1.040	5.41	3.4	4.9	1.61e-006	1.12e-006	1.32e-006
7	0.75	0.06527	1.021	6.29	3.5	2.4	1.54e-006	2.23e-006	1.82e-006
8	1	0.0729	1.005	7.03	6.7	3.3	7.97e-007	1.60e-006	1.06e-006
9	1.5	0.08777	0.974	8.46	4.5	3.5	1.16e-006	1.50e-006	1.31e-006
10	2.25	0.1102	0.928	10.62	4.6	4.1	1.09e-006	1.22e-006	1.15e-006
11	3.25	0.1354	0.875	13.06	4.6	4.3	1.03e-006	1.10e-006	1.06e-006
12	4.75	0.1599	0.824	15.42	3.5	3.4	1.29e-006	1.34e-006	1.32e-006
13	7	0.1825	0.777	17.60	3.5	2.6	1.21e-006	1.64e-006	1.39e-006
14	10.3	0.2049	0.731	19.76	2.0	2.0	2.02e-006	1.99e-006	2.01e-006
15	15	0.2261	0.687	21.80	1.6	1.8	2.46e-006	2.13e-006	2.29e-006
16	7	0.2225	0.694	21.46	0.1	0.0	6.01e-005	0.00e+000	6.01e-005
17	3.25	0.2172	0.705	20.95	0.3	0.2	1.43e-005	1.99e-005	1.66e-005
18	1.5	0.2102	0.720	20.27	0.7	0.0	5.43e-006	0.00e+000	5.43e-006
19	0.75	0.2036	0.733	19.63	2.0	0.0	1.95e-006	0.00e+000	1.95e-006
20	1.5	0.2063	0.728	19.89	0.5	0.5	8.52e-006	7.76e-006	8.12e-006
21	3.25	0.2125	0.715	20.49	0.5	0.3	7.65e-006	1.29e-005	9.61e-006
22	7	0.2204	0.698	21.25	0.5	0.3	8.35e-006	1.46e-005	1.06e-005
23	10.3	0.226	0.687	21.79	0.5	0.3	8.15e-006	1.46e-005	1.05e-005
24	15	0.2349	0.668	22.65	0.9	0.6	4.25e-006	6.46e-006	5.13e-006
25	22	0.2496	0.638	24.07	1.1	0.9	3.27e-006	3.90e-006	3.56e-006
26	32.3	0.2683	0.599	25.87	0.9	0.7	3.79e-006	5.31e-006	4.42e-006
27	7	0.2588	0.619	24.95	0.0	0.0	1.76e-004	0.00e+000	1.76e-004
28	1	0.241	0.655	23.24	0.7	0.0	4.82e-006	0.00e+000	4.82e-006
29	0.25	0.2264	0.686	21.84	4.8	4.4	7.71e-007	8.43e-007	8.05e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-103	Tested By: Brian Fogg	Checked By:
	Sample No.: 3U	Test Date: 7/24/08	Depth: 35-37 FT
	Test No.: 210629	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:28:46

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 3U
Test No.: 210629

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/24/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 35-37 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.80
Initial Void Ratio: 1.09
Final Void Ratio: 0.71

Liquid Limit: 35
Plastic Limit: 21
Plasticity Index: 14

Initial Height: 1.04 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	67	RING	RING	40
Wt. Container + Wet Soil, gm	180.77	408.5	401.35	200.54
Wt. Container + Dry Soil, gm	151.94	373.15	373.15	172.39
Wt. Container, gm	66.91	262.14	262.14	61.56
Wt. Dry Soil, gm	85.03	111.01	111.01	110.83
Water Content, %	33.91	31.84	25.40	25.40
Void Ratio	---	1.09	0.71	---
Degree of Saturation, %	---	81.54	99.98	---
Dry Unit Weight, pcf	---	83.5	102.14	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 3U
Test No.: 210629

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/24/08
Sample Type: Shelby Tube

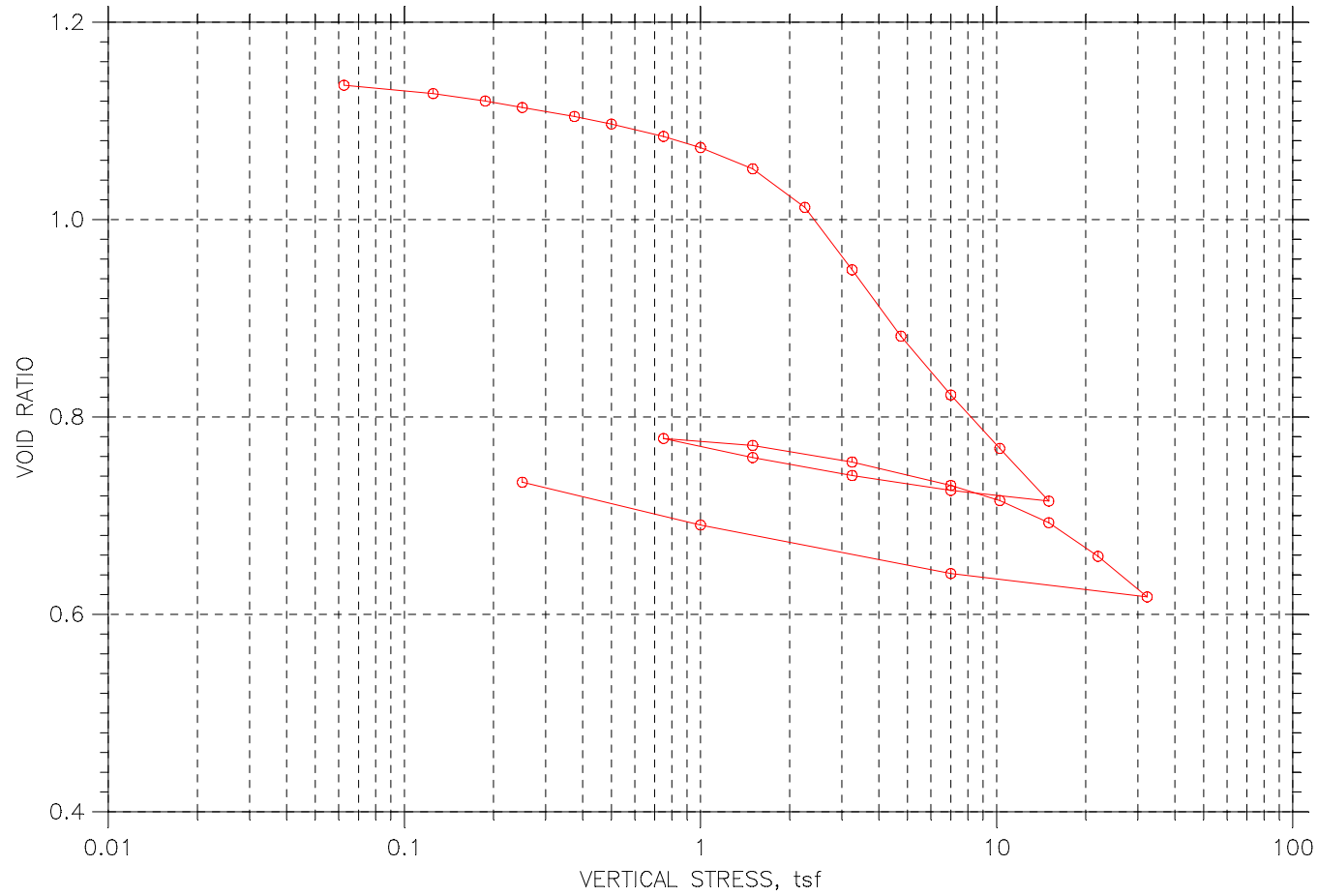
Project No.: 15600.00
Checked By:
Depth: 35-37 FT
Elevation: ---

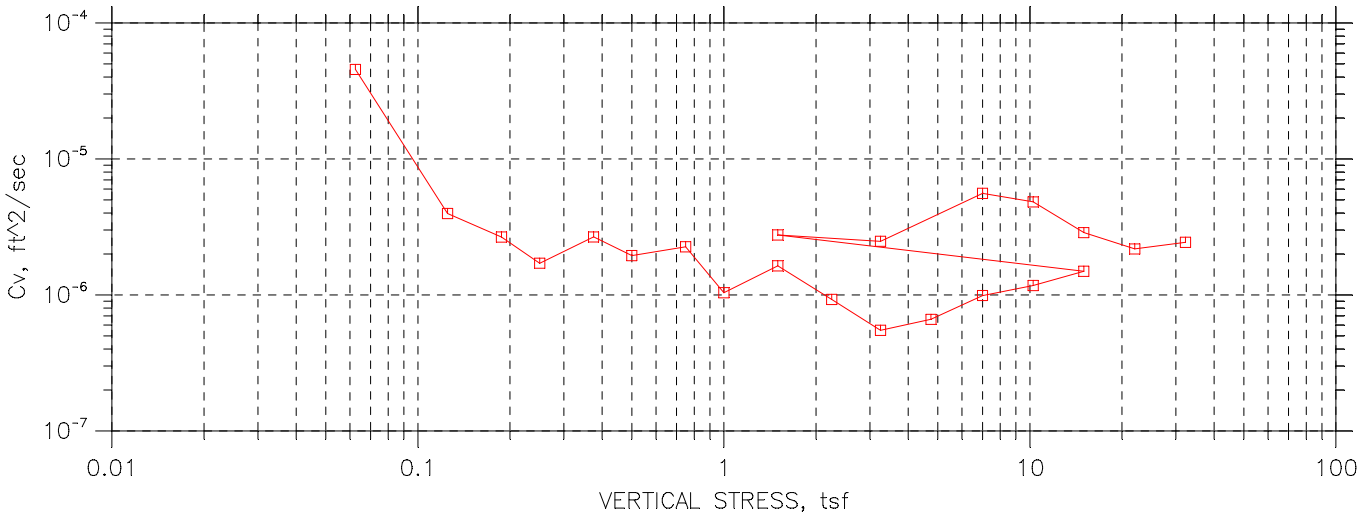
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.02814	1.037	2.69	0.4	0.0	1.43e-005	0.00e+000	1.43e-005
2	0.125	0.03156	1.030	3.02	1.7	0.0	3.38e-006	0.00e+000	3.38e-006
3	0.188	0.03484	1.024	3.34	1.3	1.2	4.38e-006	5.06e-006	4.70e-006
4	0.25	0.03741	1.018	3.58	1.7	1.7	3.38e-006	3.39e-006	3.38e-006
5	0.375	0.04181	1.010	4.00	1.7	1.1	3.45e-006	5.44e-006	4.22e-006
6	0.5	0.04501	1.003	4.31	1.9	1.4	3.07e-006	4.10e-006	3.51e-006
7	0.75	0.05077	0.992	4.86	1.5	1.7	3.73e-006	3.42e-006	3.57e-006
8	1	0.05602	0.981	5.36	3.4	3.3	1.65e-006	1.72e-006	1.69e-006
9	1.5	0.06622	0.961	6.34	3.4	2.5	1.63e-006	2.19e-006	1.87e-006
10	2.25	0.08035	0.932	7.69	3.4	3.1	1.60e-006	1.72e-006	1.66e-006
11	3.25	0.09721	0.899	9.31	4.7	3.7	1.11e-006	1.40e-006	1.24e-006
12	4.75	0.1184	0.856	11.34	3.5	3.7	1.45e-006	1.34e-006	1.39e-006
13	7	0.1413	0.810	13.53	3.4	3.4	1.40e-006	1.42e-006	1.41e-006
14	10.3	0.1637	0.765	15.68	2.4	2.3	1.93e-006	2.01e-006	1.97e-006
15	15	0.1866	0.719	17.86	1.8	2.1	2.42e-006	2.10e-006	2.25e-006
16	7	0.1835	0.726	17.57	0.0	0.0	2.63e-004	0.00e+000	2.63e-004
17	3.25	0.1781	0.736	17.05	0.5	0.2	9.33e-006	2.06e-005	1.28e-005
18	1.5	0.1726	0.747	16.53	1.0	0.0	4.11e-006	0.00e+000	4.11e-006
19	0.75	0.1664	0.760	15.93	3.3	3.1	1.32e-006	1.40e-006	1.35e-006
20	1.5	0.168	0.757	16.08	0.5	0.2	9.10e-006	1.86e-005	1.22e-005
21	3.25	0.1729	0.747	16.55	0.5	0.0	9.15e-006	0.00e+000	9.15e-006
22	7	0.1807	0.731	17.30	0.5	0.3	9.18e-006	1.50e-005	1.14e-005
23	10.3	0.1865	0.719	17.86	0.5	0.3	9.07e-006	1.51e-005	1.13e-005
24	15	0.1963	0.700	18.80	0.9	0.7	4.60e-006	6.31e-006	5.32e-006
25	22	0.2131	0.666	20.40	1.5	1.1	2.62e-006	3.83e-006	3.11e-006
26	32.3	0.2339	0.624	22.40	1.2	1.2	3.34e-006	3.09e-006	3.21e-006
27	7	0.2254	0.642	21.58	0.0	0.0	1.50e-004	0.00e+000	1.50e-004
28	1	0.2065	0.679	19.77	1.2	0.0	3.20e-006	0.00e+000	3.20e-006
29	0.25	0.1906	0.711	18.25	9.4	9.4	4.34e-007	4.34e-007	4.34e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-103	Tested By: Brian Fogg	Checked By:
	Sample No.: 4U	Test Date: 7/28/08	Depth: 45-47 FT
	Test No.: 210630	Sample Type: Shelby Tube	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:31:23

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 4U
Test No.: 210630

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/28/08
Sample Type: Shelby Tube

Project No.: 15600.00
Checked By:
Depth: 45-47 FT
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.76
Initial Void Ratio: 1.14
Final Void Ratio: 0.73

Liquid Limit: 37
Plastic Limit: 27
Plasticity Index: 10

Initial Height: 1.02 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	128	RING	RING	35
Wt. Container + Wet Soil, gm	185.95	408.05	394.34	196.43
Wt. Container + Dry Soil, gm	150.83	366.56	366.56	168.82
Wt. Container, gm	61.27	262.19	262.19	65.07
Wt. Dry Soil, gm	89.56	104.37	104.37	103.75
Water Content, %	39.21	39.75	26.61	26.61
Void Ratio	---	1.14	0.73	---
Degree of Saturation, %	---	95.83	100.09	---
Dry Unit Weight, pcf	---	80.337	99.375	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 4U
Test No.: 210630

Location: AUBURN
Tested By: Brian Fogg
Test Date: 7/28/08
Sample Type: Shelby Tube

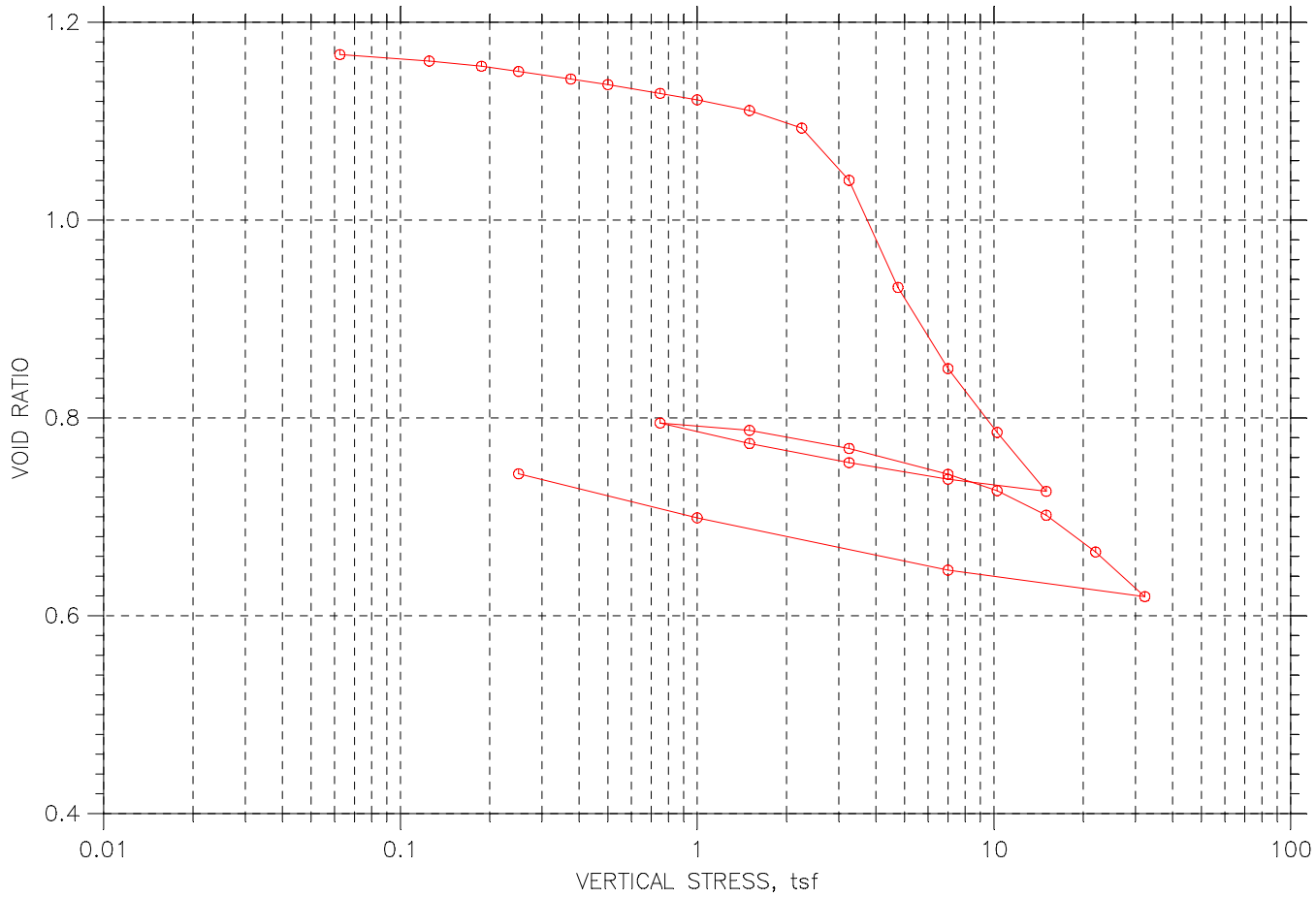
Project No.: 15600.00
Checked By:
Depth: 45-47 FT
Elevation: ---

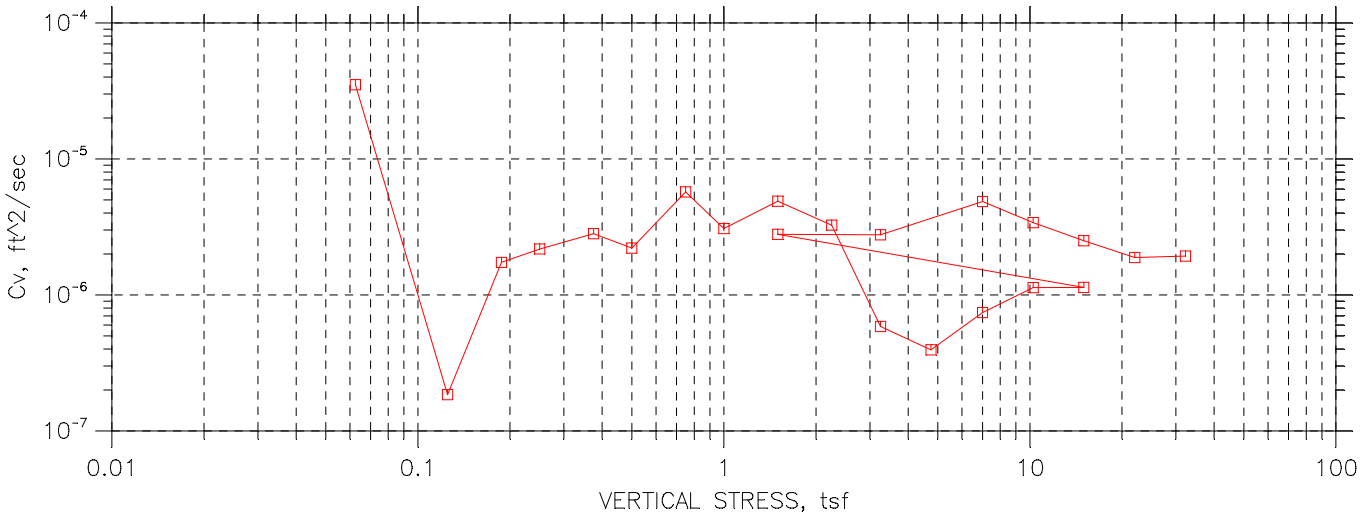
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.004109	1.136	0.40	0.1	0.1	4.55e-005	4.55e-005	4.55e-005
2	0.125	0.008138	1.128	0.80	1.5	0.0	3.97e-006	0.00e+000	3.97e-006
3	0.188	0.01172	1.120	1.15	2.1	2.3	2.81e-006	2.53e-006	2.66e-006
4	0.25	0.01485	1.114	1.46	3.6	3.2	1.61e-006	1.83e-006	1.71e-006
5	0.375	0.01915	1.104	1.88	1.9	2.4	3.03e-006	2.38e-006	2.67e-006
6	0.5	0.02286	1.097	2.24	3.4	2.5	1.68e-006	2.31e-006	1.95e-006
7	0.75	0.02877	1.084	2.82	2.2	2.8	2.58e-006	2.01e-006	2.26e-006
8	1	0.03417	1.073	3.35	7.0	3.7	7.92e-007	1.50e-006	1.04e-006
9	1.5	0.04442	1.051	4.35	3.3	3.4	1.67e-006	1.60e-006	1.64e-006
10	2.25	0.06301	1.012	6.17	7.0	4.6	7.67e-007	1.17e-006	9.27e-007
11	3.25	0.09305	0.949	9.12	9.2	9.3	5.53e-007	5.47e-007	5.50e-007
12	4.75	0.1251	0.882	12.26	6.9	7.4	6.83e-007	6.41e-007	6.61e-007
13	7	0.1534	0.822	15.03	4.6	4.3	9.61e-007	1.02e-006	9.90e-007
14	10.3	0.1792	0.768	17.56	3.5	3.6	1.20e-006	1.14e-006	1.17e-006
15	15	0.2046	0.715	20.05	2.3	2.9	1.67e-006	1.35e-006	1.49e-006
16	7	0.1994	0.726	19.54	0.1	0.0	3.13e-005	0.00e+000	3.13e-005
17	3.25	0.1922	0.741	18.84	0.5	0.0	7.81e-006	0.00e+000	7.81e-006
18	1.5	0.1836	0.759	17.99	2.2	0.0	1.84e-006	0.00e+000	1.84e-006
19	0.75	0.1744	0.778	17.09	4.7	4.5	8.68e-007	8.96e-007	8.81e-007
20	1.5	0.1777	0.771	17.42	1.4	1.6	2.91e-006	2.63e-006	2.76e-006
21	3.25	0.1858	0.754	18.20	1.6	0.0	2.48e-006	0.00e+000	2.48e-006
22	7	0.197	0.731	19.31	0.7	0.0	5.58e-006	0.00e+000	5.58e-006
23	10.3	0.2043	0.715	20.02	0.9	0.7	4.37e-006	5.39e-006	4.83e-006
24	15	0.215	0.693	21.07	1.4	1.3	2.78e-006	2.97e-006	2.87e-006
25	22	0.2312	0.659	22.66	1.6	1.7	2.27e-006	2.09e-006	2.18e-006
26	32.3	0.2508	0.618	24.57	1.2	1.6	2.86e-006	2.12e-006	2.44e-006
27	7	0.2395	0.641	23.47	0.2	0.0	2.13e-005	0.00e+000	2.13e-005
28	1	0.2161	0.691	21.18	2.1	2.7	1.73e-006	1.31e-006	1.49e-006
29	0.25	0.1955	0.734	19.16	11.7	0.0	3.24e-007	0.00e+000	3.24e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-103	Tested By: KLD	Checked By:
	Sample No.: 5U	Test Date: 8/5/2008	Depth: 55'-57'
	Test No.: 210631	Sample Type: SHELBY TUBE	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:35:36

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 5U
Test No.: 210631

Location: AUBURN
Tested By: KLD
Test Date: 8/5/2008
Sample Type: SHELBY TUBE

Project No.: 15600.00
Checked By:
Depth: 55'-57'
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.77
Initial Void Ratio: 1.19
Final Void Ratio: 0.74

Liquid Limit: 35
Plastic Limit: 23
Plasticity Index: 12

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	142	RING	RING	12
Wt. Container + Wet Soil, gm	192.74	407.83	393.45	199.84
Wt. Container + Dry Soil, gm	157.83	365.68	365.68	172.12
Wt. Container, gm	62.45	262.28	262.28	68.9
Wt. Dry Soil, gm	95.38	103.4	103.4	103.22
Water Content, %	36.60	40.76	26.86	26.86
Void Ratio	---	1.19	0.74	---
Degree of Saturation, %	---	95.16	100.06	---
Dry Unit Weight, pcf	---	79.084	99.185	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-103
Sample No.: 5U
Test No.: 210631

Location: AUBURN
Tested By: KLD
Test Date: 8/5/2008
Sample Type: SHELBY TUBE

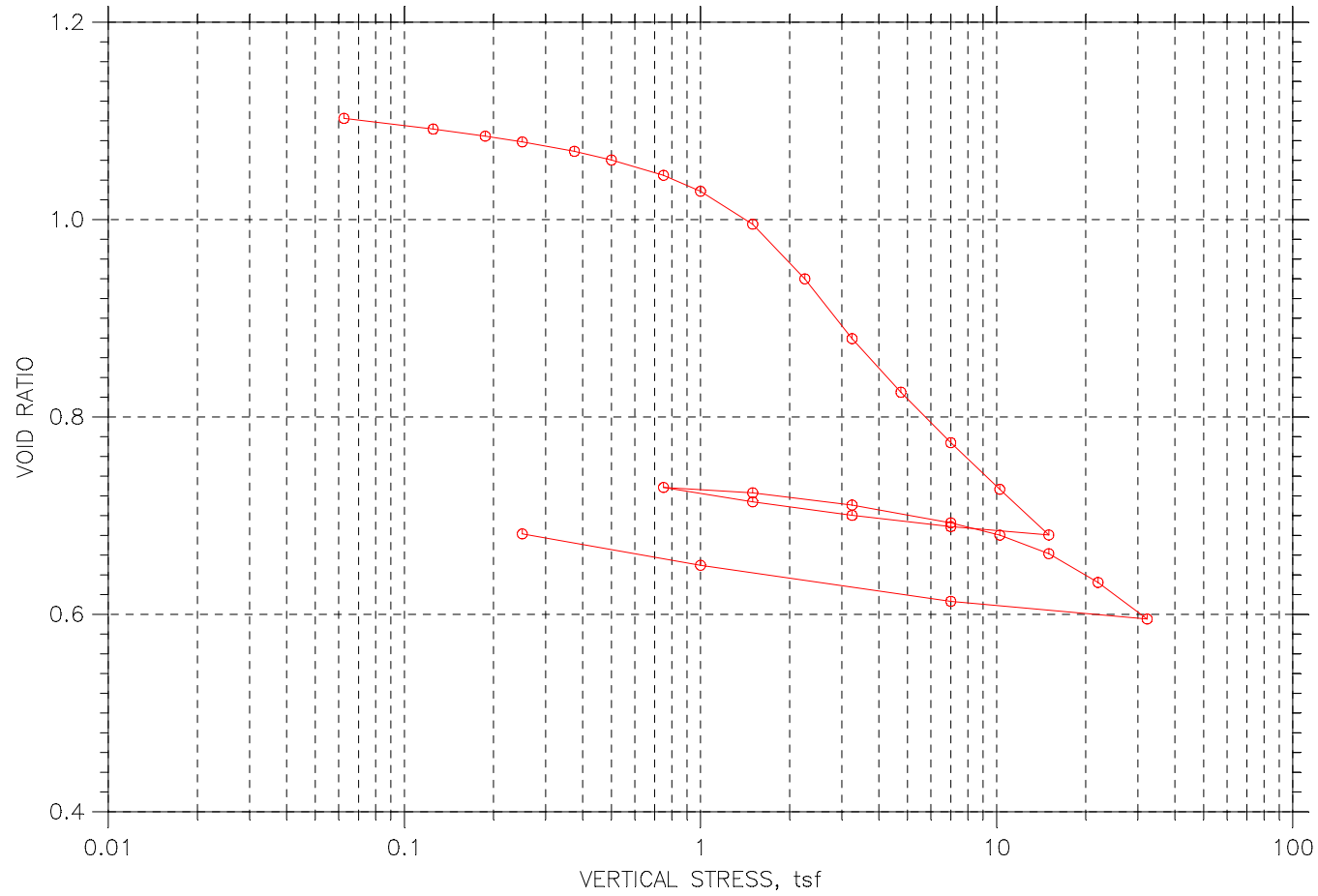
Project No.: 15600.00
Checked By:
Depth: 55'-57'
Elevation: ---

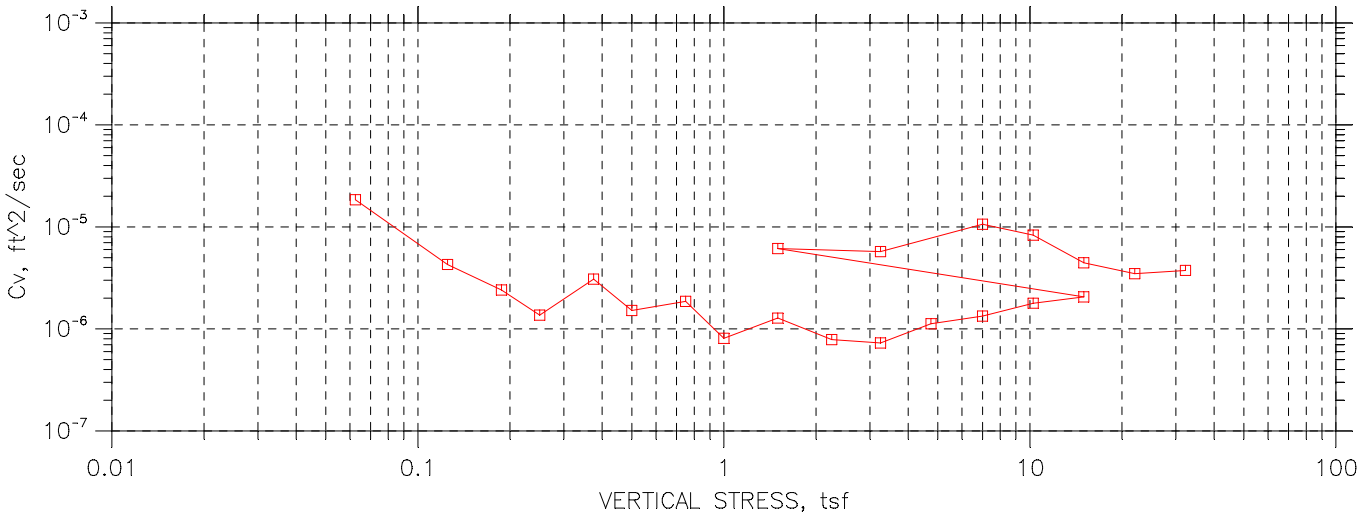
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.009077	1.167	0.88	0.2	0.0	3.51e-005	0.00e+000	3.51e-005
2	0.125	0.01214	1.161	1.18	31.9	0.0	1.85e-007	0.00e+000	1.85e-007
3	0.188	0.01461	1.155	1.42	3.5	3.2	1.65e-006	1.82e-006	1.73e-006
4	0.25	0.01708	1.150	1.66	3.3	2.1	1.79e-006	2.79e-006	2.18e-006
5	0.375	0.02067	1.143	2.01	1.8	2.4	3.30e-006	2.46e-006	2.82e-006
6	0.5	0.02331	1.137	2.27	2.3	2.9	2.46e-006	2.00e-006	2.21e-006
7	0.75	0.0275	1.128	2.68	0.9	1.1	6.50e-006	5.12e-006	5.72e-006
8	1	0.03057	1.122	2.98	1.6	2.1	3.50e-006	2.74e-006	3.07e-006
9	1.5	0.03565	1.111	3.47	1.1	1.3	5.32e-006	4.51e-006	4.88e-006
10	2.25	0.04395	1.093	4.28	2.0	1.4	2.75e-006	4.01e-006	3.26e-006
11	3.25	0.06877	1.040	6.70	9.2	0.0	5.87e-007	0.00e+000	5.87e-007
12	4.75	0.1196	0.932	11.65	11.3	13.9	4.40e-007	3.59e-007	3.95e-007
13	7	0.1582	0.850	15.40	6.9	5.2	6.51e-007	8.64e-007	7.42e-007
14	10.3	0.1884	0.785	18.35	3.5	3.8	1.19e-006	1.08e-006	1.13e-006
15	15	0.2165	0.726	21.08	3.4	3.4	1.13e-006	1.15e-006	1.14e-006
16	7	0.2107	0.738	20.52	0.2	0.0	2.28e-005	0.00e+000	2.28e-005
17	3.25	0.2028	0.755	19.75	0.7	0.0	5.27e-006	0.00e+000	5.27e-006
18	1.5	0.1937	0.774	18.86	2.0	2.3	2.01e-006	1.68e-006	1.83e-006
19	0.75	0.184	0.795	17.92	4.6	4.1	8.66e-007	9.71e-007	9.15e-007
20	1.5	0.1875	0.787	18.26	1.4	0.0	2.79e-006	0.00e+000	2.79e-006
21	3.25	0.1961	0.769	19.09	1.4	0.0	2.77e-006	0.00e+000	2.77e-006
22	7	0.2084	0.743	20.29	0.8	0.0	4.86e-006	0.00e+000	4.86e-006
23	10.3	0.2161	0.726	21.04	1.0	1.3	3.97e-006	2.98e-006	3.40e-006
24	15	0.2278	0.702	22.18	1.4	1.6	2.65e-006	2.37e-006	2.50e-006
25	22	0.2453	0.664	23.88	1.7	2.1	2.14e-006	1.69e-006	1.88e-006
26	32.3	0.2664	0.619	25.94	1.4	2.1	2.37e-006	1.63e-006	1.93e-006
27	7	0.2539	0.646	24.72	0.1	0.0	2.42e-005	0.00e+000	2.42e-005
28	1	0.229	0.699	22.30	2.0	2.8	1.75e-006	1.28e-006	1.48e-006
29	0.25	0.2081	0.743	20.27	11.8	0.0	3.17e-007	0.00e+000	3.17e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-104	Tested By: KLD	Checked By:
	Sample No.: 1U	Test Date: 8/12/08	Depth: 34-36'
	Test No.: 210632	Sample Type: SHELBY TUBE	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:38:14

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-104
Sample No.: 1U
Test No.: 210632

Location: AUBURN
Tested By: KLD
Test Date: 8/12/08
Sample Type: SHELBY TUBE

Project No.: 15600.00
Checked By:
Depth: 34-36'
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.70
Initial Void Ratio: 1.12
Final Void Ratio: 0.68

Liquid Limit: 30
Plastic Limit: 22
Plasticity Index: 8

Initial Height: 1.04 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	26	RING	RING	45
Wt. Container + Wet Soil, gm	167.15	408.54	394.02	195.54
Wt. Container + Dry Soil, gm	137.51	367.43	367.43	169.04
Wt. Container, gm	61.28	262.18	262.18	64.13
Wt. Dry Soil, gm	76.23	105.25	105.25	104.91
Water Content, %	38.88	39.06	25.26	25.26
Void Ratio	---	1.12	0.68	---
Degree of Saturation, %	---	94.47	100.08	---
Dry Unit Weight, pcf	---	79.648	100.24	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-104
Sample No.: 1U
Test No.: 210632

Location: AUBURN
Tested By: KLD
Test Date: 8/12/08
Sample Type: SHELBY TUBE

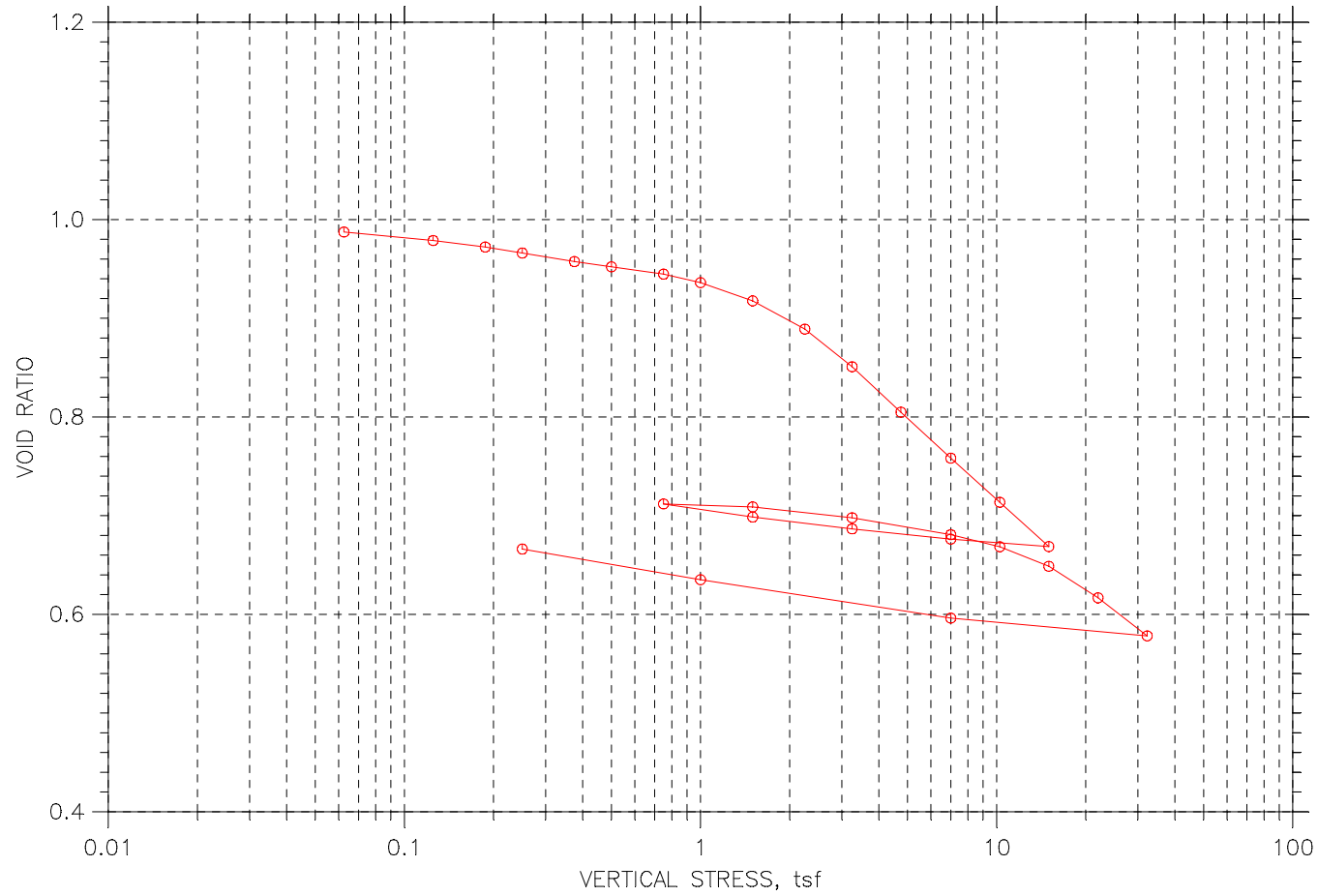
Project No.: 15600.00
Checked By:
Depth: 34-36'
Elevation: ---

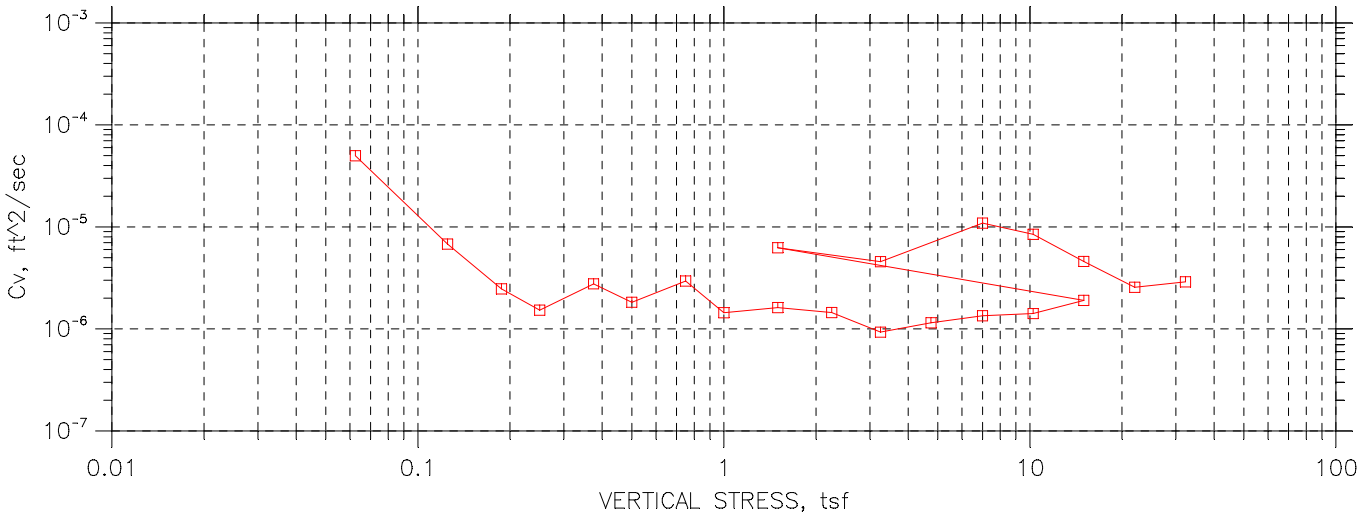
Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.006745	1.103	0.65	0.5	0.2	1.28e-005	3.36e-005	1.85e-005
2	0.125	0.01204	1.092	1.16	1.4	0.0	4.28e-006	0.00e+000	4.28e-006
3	0.188	0.01557	1.085	1.50	2.2	2.7	2.68e-006	2.19e-006	2.41e-006
4	0.25	0.0184	1.079	1.77	4.4	0.0	1.37e-006	0.00e+000	1.37e-006
5	0.375	0.02311	1.069	2.23	1.9	1.9	3.07e-006	3.08e-006	3.08e-006
6	0.5	0.02742	1.060	2.64	4.6	3.1	1.27e-006	1.87e-006	1.51e-006
7	0.75	0.03506	1.045	3.38	3.3	2.9	1.75e-006	1.99e-006	1.86e-006
8	1	0.04294	1.029	4.14	7.0	0.0	8.09e-007	0.00e+000	8.09e-007
9	1.5	0.05932	0.995	5.72	4.7	4.0	1.19e-006	1.37e-006	1.28e-006
10	2.25	0.08648	0.940	8.33	6.8	6.7	7.79e-007	7.94e-007	7.87e-007
11	3.25	0.1162	0.879	11.20	6.8	6.9	7.32e-007	7.26e-007	7.29e-007
12	4.75	0.1428	0.825	13.76	4.6	3.8	1.02e-006	1.25e-006	1.13e-006
13	7	0.1679	0.774	16.18	3.6	3.1	1.25e-006	1.43e-006	1.34e-006
14	10.3	0.1911	0.727	18.41	2.2	2.5	1.88e-006	1.70e-006	1.79e-006
15	15	0.2138	0.680	20.60	2.0	1.9	2.03e-006	2.10e-006	2.06e-006
16	7	0.2096	0.689	20.20	0.0	0.0	1.10e-004	0.00e+000	1.10e-004
17	3.25	0.204	0.700	19.65	0.2	0.2	1.67e-005	1.90e-005	1.78e-005
18	1.5	0.1973	0.714	19.01	1.0	0.0	4.20e-006	0.00e+000	4.20e-006
19	0.75	0.1901	0.729	18.32	2.4	2.9	1.69e-006	1.41e-006	1.54e-006
20	1.5	0.1928	0.723	18.57	0.7	0.6	5.67e-006	6.71e-006	6.14e-006
21	3.25	0.1989	0.711	19.17	0.7	0.0	5.73e-006	0.00e+000	5.73e-006
22	7	0.2078	0.693	20.02	0.5	0.3	8.31e-006	1.47e-005	1.06e-005
23	10.3	0.2139	0.680	20.60	0.7	0.3	5.74e-006	1.51e-005	8.31e-006
24	15	0.2231	0.661	21.49	1.1	0.6	3.52e-006	6.05e-006	4.45e-006
25	22	0.2374	0.632	22.87	1.1	1.0	3.32e-006	3.66e-006	3.48e-006
26	32.3	0.2555	0.595	24.62	0.9	1.0	3.91e-006	3.59e-006	3.75e-006
27	7	0.2467	0.613	23.77	0.0	0.0	1.84e-004	0.00e+000	1.84e-004
28	1	0.2288	0.650	22.05	1.0	0.0	3.74e-006	0.00e+000	3.74e-006
29	0.25	0.2132	0.681	20.54	6.9	9.1	5.55e-007	4.21e-007	4.78e-007

CONSOLIDATION TEST DATA

SUMMARY REPORT





	Project: CNR CROSSING BRIDGE	Location: AUBURN	Project No.: 15600.00
	Boring No.: BB-ACNR-104	Tested By: BRUCE BURRI	Checked By:
	Sample No.: 2U	Test Date: 08/14/08	Depth: 44'-46'
	Test No.: 210633	Sample Type: SHELBY TUBE	Elevation: ---
	Description: GREY SILTY CLAY		
	Remarks:		

Wed, 03-SEP-2008 10:40:51

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-104
Sample No.: 2U
Test No.: 210633

Location: AUBURN
Tested By: BRUCE BURRI
Test Date: 08/14/08
Sample Type: SHELBY TUBE

Project No.: 15600.00
Checked By:
Depth: 44'-46'
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

Measured Specific Gravity: 2.69
Initial Void Ratio: 0.99
Final Void Ratio: 0.67

Liquid Limit: 36
Plastic Limit: 24
Plasticity Index: 12

Initial Height: 1.03 in
Specimen Diameter: 2.48 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	140	RING	RING	216
Wt. Container + Wet Soil, gm	218.67	408.59	400.1	200.02
Wt. Container + Dry Soil, gm	178.56	372.73	372.73	172.69
Wt. Container, gm	62.76	262.23	262.23	62.36
Wt. Dry Soil, gm	115.8	110.5	110.5	110.33
Water Content, %	34.64	32.45	24.77	24.77
Void Ratio	---	0.99	0.67	---
Degree of Saturation, %	---	87.84	100.02	---
Dry Unit Weight, pcf	---	84.225	100.79	---

CONSOLIDATION TEST DATA

Project: CNR CROSSING BRIDGE
Boring No.: BB-ACNR-104
Sample No.: 2U
Test No.: 210633

Location: AUBURN
Tested By: BRUCE BURRI
Test Date: 08/14/08
Sample Type: SHELBY TUBE

Project No.: 15600.00
Checked By:
Depth: 44'-46'
Elevation: ---

Soil Description: GREY SILTY CLAY
Remarks:

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting Sq.Rt. min	T50 Fitting Log min	Coefficient of Consolidation Sq.Rt. ft^2/sec	Coefficient of Consolidation Log ft^2/sec	Coefficient of Consolidation Ave. ft^2/sec
1	0.0625	0.003348	0.987	0.32	0.1	0.1	4.78e-005	5.23e-005	4.99e-005
2	0.125	0.007783	0.979	0.76	0.9	0.0	6.79e-006	0.00e+000	6.79e-006
3	0.188	0.01116	0.972	1.08	1.9	2.9	3.09e-006	2.06e-006	2.47e-006
4	0.25	0.01435	0.966	1.39	4.5	3.3	1.32e-006	1.81e-006	1.53e-006
5	0.375	0.0188	0.957	1.82	2.4	1.9	2.46e-006	3.15e-006	2.77e-006
6	0.5	0.02154	0.952	2.09	3.6	2.8	1.62e-006	2.10e-006	1.83e-006
7	0.75	0.02537	0.945	2.46	1.9	2.1	3.09e-006	2.81e-006	2.94e-006
8	1	0.02981	0.936	2.89	4.7	3.3	1.23e-006	1.75e-006	1.44e-006
9	1.5	0.0394	0.918	3.82	3.6	3.4	1.58e-006	1.66e-006	1.62e-006
10	2.25	0.05419	0.889	5.26	3.5	4.1	1.56e-006	1.35e-006	1.45e-006
11	3.25	0.07389	0.851	7.17	6.8	4.7	7.87e-007	1.14e-006	9.30e-007
12	4.75	0.09765	0.805	9.48	4.6	4.2	1.10e-006	1.20e-006	1.15e-006
13	7	0.1218	0.758	11.82	3.5	3.7	1.39e-006	1.30e-006	1.35e-006
14	10.3	0.1448	0.714	14.05	3.4	3.1	1.34e-006	1.50e-006	1.41e-006
15	15	0.1681	0.669	16.31	2.0	2.5	2.13e-006	1.72e-006	1.90e-006
16	7	0.1642	0.676	15.93	0.1	0.0	8.17e-005	0.00e+000	8.17e-005
17	3.25	0.1588	0.687	15.41	0.5	0.2	9.13e-006	1.76e-005	1.20e-005
18	1.5	0.1526	0.699	14.81	1.2	1.4	3.63e-006	3.11e-006	3.35e-006
19	0.75	0.1457	0.712	14.14	3.5	4.1	1.27e-006	1.09e-006	1.17e-006
20	1.5	0.1473	0.709	14.29	0.7	0.0	6.25e-006	0.00e+000	6.25e-006
21	3.25	0.153	0.698	14.84	1.0	0.0	4.56e-006	0.00e+000	4.56e-006
22	7	0.1618	0.681	15.70	0.5	0.3	8.78e-006	1.43e-005	1.09e-005
23	10.3	0.1681	0.669	16.31	0.7	0.3	6.14e-006	1.36e-005	8.46e-006
24	15	0.1784	0.649	17.31	1.1	0.7	3.79e-006	5.80e-006	4.59e-006
25	22	0.1949	0.617	18.91	1.6	1.6	2.60e-006	2.52e-006	2.56e-006
26	32.3	0.2148	0.578	20.85	1.4	1.3	2.83e-006	2.97e-006	2.90e-006
27	7	0.2055	0.596	19.94	0.0	0.0	1.18e-004	0.00e+000	1.18e-004
28	1	0.1854	0.635	17.99	1.7	0.0	2.36e-006	0.00e+000	2.36e-006
29	0.25	0.1693	0.666	16.43	9.3	0.0	4.48e-007	0.00e+000	4.48e-007

Appendix C

Calculations

CNR Crossing Bridge
Over Canadian National Railway
Auburn, Maine
PIN 15600.00

By: Kate Maguire
July 2008
Checked by: LK 10-1-2008

Definition of Units:

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

LIQUIDITY INDEX (LI):

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

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

Sample	Soil	WC	LL	PL	PI	LI	
BB-ACNR-102 3D A	Silt	32.9	27	22	5	2.18	viscous liquid when remolded
BB-ACNR-104 7D	Silt	30.7	25	22	3	2.90	viscous liquid when remolded
BB-ACNR-101 2U	Silt	30.6	22	19	3	3.87	viscous liquid when remolded
BB-ACNR-101 9D	Clayey Silt	32.4	28	22	6	1.73	viscous liquid when remolded
BB-ACNR-101 3U	Clayey Silt	32.8	30	19	11	1.25	viscous liquid when remolded
BB-ACNR-101 10D	Clayey Silt	30.9	30	22	8	1.11	normally consolidated
BB-ACNR-101 4U	Clayey Silt	38.0	35	24	11	1.27	viscous liquid when remolded
BB-ACNR-101 11D	Clayey Silt	34.4	31	12	19	1.18	viscous liquid when remolded
BB-ACNR-101 5U	Silt	26.4	22	20	2	3.20	viscous liquid when remolded
BB-ACNR-101 6U	Clayey Silt	35.2	31	22	9	1.47	viscous liquid when remolded
BB-ACNR-102 3D B	Silt	33.5	25	20	5	2.70	viscous liquid when remolded
BB-ACNR-102 1U	Silt	30.2	NP	NP	NP	NP	Non-Plastic
BB-ACNR-102 4D	Silt	31.5	27	20	7	1.64	viscous liquid when remolded
BB-ACNR-102 2U	Silt	29.8	34	23	11	0.62	over consolidated
BB-ACNR-102 6D	Clayey Silt	35.4	35	27	8	1.05	normally consolidated
BB-ACNR-102 3U	Silty Clay	38.4	27	19	8	2.43	viscous liquid when remolded
BB-ACNR-102 4U	Clayey Silt	30.7	26	19	7	1.67	viscous liquid when remolded
BB-ACNR-102 8D	Clayey Silt	30.6	29	20	9	1.18	viscous liquid when remolded
BB-ACNR-102 5U	Clayey Silt	36.8	33	22	11	1.35	viscous liquid when remolded
BB-ACNR-103 1U	Silt	34.3	28	21	7	1.90	viscous liquid when remolded
BB-ACNR-103 4D	Silt	33.2	36	22	14	0.80	over consolidated
BB-ACNR-103 2U	Silt	29.0	27	23	4	1.50	viscous liquid when remolded
BB-ACNR-103 3U	Silt	34.3	35	21	14	0.95	normally consolidated
BB-ACNR-103 6D	Clayey Silt	36.8	36	22	14	1.06	normally consolidated
BB-ACNR-103 4U	Silt	40.8	37	27	10	1.38	viscous liquid when remolded
BB-ACNR-103 5U	Clayey Silt	40.2	35	23	12	1.43	viscous liquid when remolded
BB-ACNR-104 1U	Clayey Silt	36.7	30	22	8	1.84	viscous liquid when remolded
BB-ACNR-104 8D	Clayey Silt	31.9	31	19	12	1.08	normally consolidated
BB-ACNR-104 2U	Silty Clay	36.9	36	24	12	1.08	normally consolidated
BB-ACNR-104 9D	Silty Clay	40.9	39	25	14	1.14	viscous liquid when remolded
BB-ACNR-104 3U	Silt	29.5	25	20	5	1.90	viscous liquid when remolded

CONSOLIDATION TEST RESULTS

BB-ACNR-101 Sample 2U

Determine in-situ over burden stress:

Sample depth = 45.0 ft below ground surface

Groundwater table at 17.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.81$

Clay is overlain by:

17.0 ft of fill at 125 pcf
5.8 ft of sand at 120 pcf
16.7 ft of silt at 115 pcf and
5.5 ft of clay at 115 pcf

$\sigma'_{vo} := 17 \cdot ft \cdot 125 \cdot pcf + 5.8 \cdot ft \cdot (120 - 62.4) \cdot pcf + 22.2 \cdot ft \cdot (115 - 62.4) \cdot pcf$

$\sigma'_{vo} = 3627 \cdot psf$ or $\sigma'_{vo} = 1.813 \cdot tsf$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 3.2 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.7646$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 10.3 \cdot tsf$ $e_1 := 0.584$ $p_2 := 32.3 \cdot tsf$ $e_2 := 0.527$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.1148$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{12.74}{100}$ $\varepsilon_2 := \frac{15.88}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.0633$$ or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.0634$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 7 \cdot tsf$ $e_1 := 0.569$ $p_2 := 0.75 \cdot tsf$ $e_2 := 0.579$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0103$$

BB-ACNR-101 Sample 3U

Determine in-situ over burden stress:

Sample depth = 55.0 ft below ground surface

Groundwater table at 17.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.99$

Clay is overlain by:
17 ft of fill at 125 pcf
5.8 ft of sand at 120 pcf
16.7 ft silt at 115 pcf and
15.5 ft of clay at 115 pcf

$\sigma'_{vo} := 17 \cdot ft \cdot 125 \cdot pcf + 5.8 \cdot ft \cdot (120 - 62.4) \cdot pcf + 32.2 \cdot ft \cdot (115 - 62.4) \cdot pcf$

$\sigma'_{vo} = 4153 \cdot psf$ or $\sigma'_{vo} = 2.076 \cdot tsf$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 3.7 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.7819$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 4.75 \cdot tsf$ $e_1 := 0.857$ $p_2 := 7 \cdot tsf$ $e_2 := 0.788$

$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_c = 0.4097$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{6.88}{100}$ $\varepsilon_2 := \frac{10.36}{100}$ strain is given in percent

$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)}$ $C'_c = 0.2066$ or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.2059$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.722$ $p_2 := 7 \cdot tsf$ $e_2 := 0.690$

$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_r = 0.033$

BB-ACNR-101 Sample 4U

Determine in-situ over burden stress:

Sample depth = 65.0 ft below ground surface

Groundwater table at 17.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.16$

Clay is overlain by:
17 ft of fill at 125 pcf
5.8 ft of sand at 120 pcf
16.7 ft silt at 115 pcf and
25.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 17 \cdot ft \cdot 125 \cdot pcf + 5.8 \cdot ft \cdot (120 - 62.4) \cdot pcf + 42.2 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 4679 \cdot psf \text{ or } \sigma'_{vo} = 2.339 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.8 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.1969$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 0.886$ $p_2 := 15 \cdot tsf$ $e_2 := 0.700$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.28$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{12.57}{100}$ $\varepsilon_2 := \frac{21.21}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1301 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1296$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.737$ $p_2 := 7 \cdot tsf$ $e_2 := 0.711$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0268$$

BB-ACNR-101 Sample 5U

Determine in-situ over burden stress:

Sample depth = 75.0 ft below ground surface

Groundwater table at 17.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.85$

Clay is overlain by:
17 ft of Fill at 125 pcf
5.8 ft of sand at 120 pcf
16.7 ft of silt at 115 pcf and
35.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 17 \cdot ft \cdot 125 \cdot pcf + 5.8 \cdot ft \cdot (120 - 62.4) \cdot pcf + 52.2 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 5205 \cdot psf \quad \text{or} \quad \sigma'_{vo} = 2.602 \cdot tsf$$

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

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 0.4227$ *under consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 0.607$ $p_2 := 15 \cdot tsf$ $e_2 := 0.505$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.1536$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{13.26}{100}$ $\varepsilon_2 := \frac{18.72}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.0822$$

or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.083$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.533$ $p_2 := 7 \cdot tsf$ $e_2 := 0.515$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0186$$

BB-ACNR-101 Sample 6U

Determine in-situ over burden stress:

Sample depth = 85.0 ft below ground surface

Groundwater table at 17.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.96$

Clay is overlain by:

17.0 ft of fill at 125 pcf
5.8 ft of sand at 120 pcf
16.7 ft silt at 115 pcf and
45.5 ft of clay at 115 pcf

$$\sigma'_{vo} := 17 \cdot ft \cdot 125 \cdot pcf + 5.8 \cdot ft \cdot (120 - 62.4) \cdot pcf + 62.2 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 5731 \cdot psf \text{ or } \sigma'_{vo} = 2.865 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 3.8 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.3262$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 4.75 \cdot tsf$ $e_1 := 0.826$ $p_2 := 10.3 \cdot tsf$ $e_2 := 0.705$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.36$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{6.73}{100}$ $\varepsilon_2 := \frac{12.92}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1841 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1837$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.697$ $p_2 := 7 \cdot tsf$ $e_2 := 0.666$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.032$$

BB-ACNR-102 Sample 1U

Determine in-situ over burden stress:

Sample depth = 19.0 ft below ground surface

Groundwater table at 12.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.01$

Clay is overlain by:
12.0 ft of fill at 125 pcf
3.0 ft of silt at 115 pcf and
4.0 ft of clay at 115 pcf

$\sigma'_{vo} := 12 \cdot ft \cdot 125 \cdot pcf + 7 \cdot ft \cdot (115 - 62.4) \cdot pcf$

$\sigma'_{vo} = 1868 \cdot psf$ or $\sigma'_{vo} = 0.934 \cdot tsf$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.5 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.6058$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 0.862$ $p_2 := 7.0 \cdot tsf$ $e_2 := 0.727$

$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_c = 0.2739$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{7.51}{100}$ $\varepsilon_2 := \frac{14.23}{100}$ strain is given in percent

$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)}$ $C'_c = 0.1363$ or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.1363$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.678$ $p_2 := 7 \cdot tsf$ $e_2 := 0.654$

$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_r = 0.0247$

BB-ACNR-102 Sample 3U

Determine in-situ over burden stress:

Sample depth = 41.0 ft below ground surface

Groundwater table at 12.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.21$

Clay is overlain by:
12.0 ft of fill at 125 pcf
3.0 ft of silt at 115 pcf and
26.0 ft of clay at 115 pcf

$\sigma'_{vo} := 12 \cdot ft \cdot 125 \cdot pcf + 29 \cdot ft \cdot (115 - 62.4) \cdot pcf$

$\sigma'_{vo} = 3025 \cdot psf$ or $\sigma'_{vo} = 1.513 \cdot tsf$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.8 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.1899$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 1.053$ $p_2 := 4.75 \cdot tsf$ $e_2 := 0.904$

$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_c = 0.4592$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{7.18}{100}$ $\varepsilon_2 := \frac{13.88}{100}$ strain is given in percent

$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)}$ $C'_c = 0.2065$ or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.2078$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.796$ $p_2 := 7 \cdot tsf$ $e_2 := 0.747$

$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)}$ $C_r = 0.0505$

BB-ACNR-102 Sample 5U

Determine in-situ over burden stress:

Sample depth = 59.0 ft below ground surface

Groundwater table at 12.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.14$

Clay is overlain by:
12.0 ft of fill at 125 pcf
3.0 ft of silt at 115 pcf and
44.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 12 \cdot ft \cdot 125 \cdot pcf + 47 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3972 \cdot psf \text{ or } \sigma'_{vo} = 1.986 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.6 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.3091$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 0.944$ $p_2 := 7.0 \cdot tsf$ $e_2 := 0.817$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.3811$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{9.28}{100}$ $\varepsilon_2 := \frac{15.19}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1774$$

or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.1781$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.762$ $p_2 := 7 \cdot tsf$ $e_2 := 0.719$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0443$$

BB-ACNR-103 Sample 1U

Determine in-situ over burden stress:

Sample depth = 15.0 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.92$

Clay is overlain by:
14.0 ft of fill at 125 pcf and
1.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 14 \cdot ft \cdot 125 \cdot pcf + 1 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 1803 \cdot psf \text{ or } \sigma'_{vo} = 0.901 \cdot tsf$$

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

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.2205$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.832$ $p_2 := 3.25 \cdot tsf$ $e_2 := 0.748$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.2502$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{4.49}{100}$ $\varepsilon_2 := \frac{8.90}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1313 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1303$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.631$ $p_2 := 7 \cdot tsf$ $e_2 := 0.612$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0196$$

BB-ACNR-103 Sample 2U

Determine in-situ over burden stress:

Sample depth = 25.0 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.16$

Clay is overlain by:
14.0 ft of fill at 125 pcf and
11.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 14 \cdot ft \cdot 125 \cdot pcf + 11 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 2329 \cdot psf \text{ or } \sigma'_{vo} = 1.164 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.2 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.0307$ *Normally consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 0.928$ $p_2 := 7.0 \cdot tsf$ $e_2 := 0.777$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.3063$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{10.62}{100}$ $\varepsilon_2 := \frac{17.60}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1416 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1418$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.733$ $p_2 := 7 \cdot tsf$ $e_2 := 0.698$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0361$$

BB-ACNR-103 Sample 3U

Determine in-situ over burden stress:

Sample depth = 35.0 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.09$

Clay is overlain by:
14.0 ft of fill at 125 pcf and
21.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 14 \cdot ft \cdot 125 \cdot pcf + 21 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 2855 \cdot psf \text{ or } \sigma'_{vo} = 1.427 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.1 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.4713$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 4.75 \cdot tsf$ $e_1 := 0.856$ $p_2 := 15.0 \cdot tsf$ $e_2 := 0.719$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.2743$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{11.34}{100}$ $\varepsilon_2 := \frac{17.86}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1306$$

or: $C'c := \frac{C_c}{1 + e_0}$ $C'c = 0.1313$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 0.75 \cdot tsf$ $e_1 := 0.760$ $p_2 := 7 \cdot tsf$ $e_2 := 0.731$

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

BB-ACNR-103 Sample 4U

Determine in-situ over burden stress:

Sample depth = 45.0 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.14$

Clay is overlain by:
14.0 ft of fill at 125 pcf and
31.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 14 \cdot ft \cdot 125 \cdot pcf + 31 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3381 \cdot psf \text{ or } \sigma'_{vo} = 1.69 \cdot tsf$$

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

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.1241$ over consolidated

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 1.012$ $p_2 := 7.0 \cdot tsf$ $e_2 := 0.822$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.3855$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{6.17}{100}$ $\varepsilon_2 := \frac{15.03}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1797 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1801$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.771$ $p_2 := 7 \cdot tsf$ $e_2 := 0.731$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0598$$

BB-ACNR-103 Sample 5U

Determine in-situ over burden stress:

Sample depth = 55.0 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.19$

Clay is overlain by:
14.0 ft of fill at 125 pcf and
41.0 ft of clay at 115 pcf

$$\sigma'_{vo} := 14 \cdot ft \cdot 125 \cdot pcf + 41 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3907 \cdot psf \text{ or } \sigma'_{vo} = 1.953 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.8 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.4335$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 1.04$ $p_2 := 7.0 \cdot tsf$ $e_2 := 0.850$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.5702$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{6.7}{100}$ $\varepsilon_2 := \frac{15.4}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.2611 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.2604$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.787$ $p_2 := 7 \cdot tsf$ $e_2 := 0.743$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0658$$

BB-ACNR-104 Sample 1U

Determine in-situ over burden stress:

Sample depth = 34.0 ft below ground surface

Groundwater table at 18.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.12$

Clay is overlain by:
18.0 ft of fill at 125 pcf
15.4 ft sand at 120 pcf and
0.6 ft of clay at 115 pcf

$$\sigma'_{vo} := 18 \cdot ft \cdot 125 \cdot pcf + 15.4 \cdot ft \cdot (120 - 62.4) \cdot pcf + 0.6 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3169 \cdot psf \text{ or } \sigma'_{vo} = 1.584 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.8 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.1361$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 2.25 \cdot tsf$ $e_1 := 0.940$ $p_2 := 3.25 \cdot tsf$ $e_2 := 0.879$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.382$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{8.33}{100}$ $\varepsilon_2 := \frac{11.20}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1797 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1802$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.723$ $p_2 := 7 \cdot tsf$ $e_2 := 0.693$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0448$$

BB-ACNR-104 Sample 2U

Determine in-situ over burden stress:

Sample depth = 44.0 ft below ground surface

Groundwater table at 18.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 0.99$

Clay is overlain by:
18.0 ft of fill at 125 pcf
15.4 ft sand at 120 pcf and
10.6 ft of clay at 115 pcf

$$\sigma'_{vo} := 18 \cdot ft \cdot 125 \cdot pcf + 15.4 \cdot ft \cdot (120 - 62.4) \cdot pcf + 10.6 \cdot ft \cdot (115 - 62.4) \cdot pcf$$

$$\sigma'_{vo} = 3695 \cdot psf \text{ or } \sigma'_{vo} = 1.847 \cdot tsf$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 2.6 \cdot tsf$

Determine OCR: $OCR := \frac{\sigma'_p}{\sigma'_{vo}}$ $OCR = 1.4075$ *over consolidated*

Determine Cc:

from consolidation curve and lab results:

$p_1 := 3.25 \cdot tsf$ $e_1 := 0.851$ $p_2 := 15.0 \cdot tsf$ $e_2 := 0.669$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_c = 0.274$$

Determine C'c:

from consolidation curve and lab results:

$\varepsilon_1 := \frac{7.17}{100}$ $\varepsilon_2 := \frac{16.31}{100}$ strain is given in percent

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \qquad C'_c = 0.1376 \text{ or: } C'c := \frac{C_c}{1 + e_0} \qquad C'c = 0.1377$$

Determine Cr:

from consolidation curve and lab results:

$p_1 := 1.5 \cdot tsf$ $e_1 := 0.709$ $p_2 := 7 \cdot tsf$ $e_2 := 0.681$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \qquad C_r = 0.0419$$

Abutment Foundations: Integral driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at the following piles:

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

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

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

Where λ=normalized column slenderness factor

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

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

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

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

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

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

Where λ=normalized column slenderness factor

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

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

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

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

Resistance Factors for Service and Extreme Limit States ϕ = 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 $P_f := \phi \cdot P_n$ $P_f = \begin{pmatrix} 775 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$

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:

Granite RQD ranges from 28 to 77%.

Use RQD = 50% and ϕ = 34 to 40 deg (Tomlinson 4th Ed. pg 139)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Look at these piles:

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

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

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

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

Calculate pile box area:

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

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

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

q_u for granite compressive strength ranges from 2100 to 49000 psi

use $\sigma_{cG} := 30000 \cdot psi$

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

Spacing of discontinuities: $c := 36 \cdot in$ Assumed based on rock core

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

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

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

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.5633 \\ 0.5144 \\ 0.5126 \\ 0.5097 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

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

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

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

$$q_a := \sigma_{cG} \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 2434 \\ 2222 \\ 2215 \\ 2202 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

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

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 786 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$$

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

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

$R_f := \phi_{stat} \cdot R_p$ $R_f = \begin{pmatrix} 354 \\ 446 \\ 542 \\ 710 \end{pmatrix} \cdot kip$

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

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

$R_{fse} := \phi \cdot R_p$ $R_{fse} = \begin{pmatrix} 786 \\ 991 \\ 1204 \\ 1578 \end{pmatrix} \cdot kip$

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

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
σ_{dr} = 0.9 x ϕ_{da} x f_y (eq. 10.7.8-1)

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

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

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

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

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

$\phi_{dyn.reduced} := 0.65 \cdot 0.8$ $\phi_{dyn.reduced} = 0.52$

Pile Size = 12 x 53

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

State of Maine Dept. Of Transportation — 17-Sep-2008
Auburn CNR Crossing Bridge — 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	33.60	4.47	4.5	7.89	21.39
350.0	35.41	5.49	7.2	8.19	22.17
400.0	36.68	6.57	11.9	8.50	22.95
405.0	36.75	6.67	12.6	8.52	23.00
410.0	36.84	6.78	13.3	8.55	23.11
415.0	36.65	6.83	14.6	8.49	22.91
420.0	36.69	6.93	15.6	8.50	22.97
425.0	36.73	7.00	16.8	8.52	22.97
450.0	36.89	7.41	25.0	8.59	23.19
500.0	37.04	7.86	78.8	8.70	23.43

DELMAG D 19-42

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_12x53\_factored} := 418 \cdot kip \cdot \phi_{dyn.reduced}$

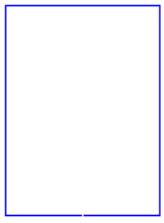
$R_{dr\_12x53\_factored} = 217 \cdot kip$

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

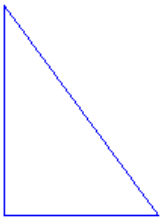
$R_{dr\_12x53\_servext} := 418 \cdot kip$

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 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	95.00 ft
Pile Penetration	95.00 ft
Pile Top Area	15.50 in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D 36-32 hammer to install 14 x 73 piles

State of Maine Dept. Of Transportation — 17-Sep-2008
Auburn CNR Crossing Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
475.0	34.14	4.20	3.2	7.72	46.73
500.0	34.49	4.39	3.7	7.85	47.31
550.0	35.20	4.73	5.0	8.05	48.31
600.0	35.89	4.96	6.9	8.32	49.57
650.0	36.01	5.10	10.3	8.41	49.99
680.0	36.01	5.13	13.8	8.49	50.22
685.0	36.01	5.14	14.5	8.49	50.28
690.0	35.97	5.14	15.5	8.50	50.23
695.0	35.99	5.13	16.4	8.52	50.34
700.0	36.00	5.15	17.3	8.52	50.44

DELMAG D 36-32

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x73\_factored} := 688 \cdot kip \cdot \phi_{dyn.reduced}$

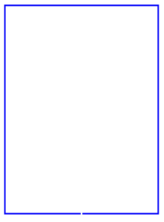
$R_{dr\_14x73\_factored} = 358 \cdot kip$

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

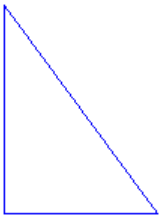
$R_{dr\_14x73\_servext} := 688 \cdot kip$

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 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	95.00 ft
Pile Penetration	95.00 ft
Pile Top Area	21.40 in2

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Pile Size = 14 x 89

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

State of Maine Dept. Of Transportation — 17-Sep-2008
Auburn CNR Crossing Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
475.0	37.42	4.00	2.7	7.71	45.10
500.0	38.35	4.15	3.0	7.84	45.79
600.0	41.40	4.71	4.7	8.34	48.28
650.0	42.66	5.18	5.8	8.61	49.77
700.0	43.11	5.63	7.6	8.70	50.22
750.0	43.60	6.01	10.2	8.86	50.85
800.0	44.00	6.27	13.7	8.98	51.49
815.0	44.08	6.35	15.1	9.01	51.62
820.0	44.12	6.38	15.6	9.02	51.70
825.0	44.17	6.41	16.2	9.04	51.77

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x89\_factored} := 815 \cdot kip \cdot \phi_{dyn.reduced}$

$R_{dr\_14x89\_factored} = 424 \cdot kip$

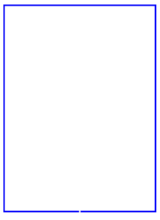
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x89\_servext} := 815 \cdot kip$

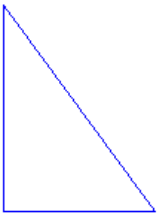
DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	95.00 ft
Pile Penetration	95.00 ft
Pile Top Area	26.10 in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag D 46-32 hammer to install 14 x 117 piles

State of Maine Dept. Of Transportation — 17-Sep-2008
Auburn CNR Crossing Bridge — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
925.0	38.72	2.30	9.4	9.61	63.97
950.0	38.74	2.29	10.4	9.65	64.33
975.0	38.67	2.23	11.7	9.69	64.51
1000.0	38.56	2.15	13.3	9.73	64.54
1010.0	38.56	2.15	13.9	9.75	64.77
1020.0	38.51	2.10	14.8	9.77	64.77
1025.0	38.50	2.09	15.1	9.78	64.88
1030.0	38.43	2.06	15.7	9.77	64.70
1040.0	38.43	2.05	16.4	9.79	64.92
1050.0	38.34	2.00	17.5	9.80	64.84

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_14x117\_factored} := 1025 \cdot kip \cdot \phi_{dyn.reduced}$

$R_{dr\_14x117\_factored} = 533 \cdot kip$

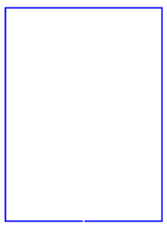
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_14x117\_servext} := 1025 \cdot kip$

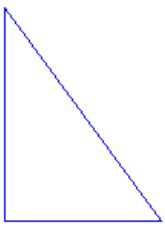
DELMAG D 46-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 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	95.00 ft
Pile Penetration	95.00 ft
Pile Top Area	34.40 in2

Pile Model



Skin Friction Distribution



Res. Shaft = 20 %
(Proportional)

Pipe Pile Pier Bent

Calculate Depth to Fixity for pipe piles:

Soil conditions:

15 ft of fill sand
50 ft of soft clay (Su=500 psf)
30 ft of sand with cobbles and boulders over bedrock

$S_u := 500 \cdot psf$

Consider Pile sizes:

24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

Piles will not be exposed to water therefore no corrosion is applied.
Bridge is a railroad crossing therefore no scour is considered .

Diameter of piles: Pipe pile wall thickness:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

$$dia_{conccore\_0.5} := dia_{steel} - 2 \cdot \frac{1}{2} \cdot in \qquad dia_{conccore\_0.5} = \begin{pmatrix} 23 \\ 25 \\ 27 \\ 29 \end{pmatrix} \cdot in$$

Diameter concrete core for 1/2" thick wall

$$dia_{conccore\_0.625} := dia_{steel} - 2 \cdot \frac{5}{8} \cdot in \qquad dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 36.9 \\ 40.1 \\ 43.2 \\ 46.3 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 45.9 \\ 49.8 \\ 53.8 \\ 57.7 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES

Transformed pile properties of 1/2 inch wall pile:

unit weight of concrete: $wc := 0.15$ in kips per cubic foot

compressive strength of concrete: $f_c := 4.45$ in ksi

Modulus of elasticity of concrete: $E_c := 33000 \cdot wc^{1.5} \cdot \sqrt{f_c} \cdot 1000 \cdot psi \qquad E_c = 4044 \cdot ksi$

Steel modulus: $E_{steel} := 29000 \cdot ksi$

$$n := \frac{E_{steel}}{E_c} \qquad n = 7.17$$

MaineDOT Structural engineers routinely use: $n := 7.6$

Moment of inertia of concrete core:

$$I_{c\_0.5} := \frac{\pi \cdot dia_{conccore\_0.5}^{\ 4}}{64} \qquad I_{c\_0.5} = \begin{pmatrix} 0.662 \\ 0.925 \\ 1.258 \\ 1.674 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.5} := \frac{\pi \cdot \overrightarrow{\left(dia_{steel}^{\ 4} - dia_{conccore\_0.5}^{\ 4}\right)}}{64} \qquad I_{s\_0.5} = \begin{pmatrix} 0.123 \\ 0.157 \\ 0.197 \\ 0.243 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.5} := \overrightarrow{\left(\frac{I_{c\_0.5}}{n} + I_{s\_0.5}\right)} \qquad I_{t\_0.5} = \begin{pmatrix} 0.21 \\ 0.279 \\ 0.363 \\ 0.463 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.5} := \pi \cdot \frac{dia_{conccore\_0.5}^{2}}{4} \qquad A_{conc\_0.5} = \begin{pmatrix} 415.48 \\ 490.87 \\ 572.56 \\ 660.52 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.5} := A_{0.5} + \frac{A_{conc\_0.5}}{n} \qquad A_{t\_0.5} = \begin{pmatrix} 0.636 \\ 0.727 \\ 0.823 \\ 0.925 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-1 for fixity in feet: $1.4*(E_p I_w/E_s)^{0.25}$ (in clays)

E_p in ksi

I_w in ft^4

Es $0.465*S_u$ (Su must be in ksf, results E_s in ksi)

Use same equation in NCHRP#343 pg 61:

$L_{eq}=L_u+1.4R$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

for clays: $R=(E_p*I_p/E_s)^{0.25}$

Average from Field Vanes: $S_u = 500 \cdot psf$

Soil modulus of clay: $E_{soil} := 67 \cdot S_u \qquad E_{soil} = 33.5 \cdot ksf$

R parameter:

$$R_{0.5} := \left(\frac{E_{steel} \cdot I_{t\_0.5}}{E_{soil}} \right)^{0.25} \qquad R_{0.5} = \begin{pmatrix} 12.72 \\ 13.65 \\ 14.58 \\ 15.5 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix\_0.5} := 1.4 \cdot R_{0.5} \qquad D_{fix\_0.5} = \begin{pmatrix} 18 \\ 19 \\ 20 \\ 22 \end{pmatrix} ft$$

Depth to fixity for 1/2" wall pipe piles

Check with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000 \cdot ksi \qquad E_{soil} = 0.2326 \cdot ksi$

$$I_{t\_0.5} = \begin{pmatrix} 0.2101 \\ 0.2787 \\ 0.3625 \\ 0.4635 \end{pmatrix} ft^4 \qquad Check := 1.4 \cdot \left(\frac{29000 \cdot I_{t\_0.5}}{0.2326} \right)^{0.25} \qquad Check = \begin{pmatrix} 17.81 \\ 19.12 \\ 20.41 \\ 21.71 \end{pmatrix} ft$$

OK

Transformed pile properties of 5/8 inch wall pile:

$S_u = 500 \cdot psf$ $n = 7.6$

Diameter of concrete core:

$$dia_{conccore\_0.625} = \begin{pmatrix} 22.75 \\ 24.75 \\ 26.75 \\ 28.75 \end{pmatrix} \cdot in$$

Diameter concrete core for 5/8" thick wall

Diameter of steel pipe

$$dia_{steel} = \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$$

Moment of inertia of concrete core:

$$I_{c\_0.625} := \frac{\pi \cdot dia_{conccore\_0.625}^{4}}{64} \qquad I_{c\_0.625} = \begin{pmatrix} 0.634 \\ 0.888 \\ 1.212 \\ 1.617 \end{pmatrix} ft^4$$

Moment of inertia of steel pipe:

$$I_{s\_0.625} := \frac{\pi \cdot \overrightarrow{\left(dia_{steel}^{4} - dia_{conccore\_0.625}^{4}\right)}}{64} \qquad I_{s\_0.625} = \begin{pmatrix} 0.151 \\ 0.194 \\ 0.243 \\ 0.3 \end{pmatrix} ft^4$$

Composite Moment of Inertia:

$$I_{t\_0.625} := \frac{I_{c\_0.625}}{n} + I_{s\_0.625} \qquad I_{t\_0.625} = \begin{pmatrix} 0.235 \\ 0.31 \\ 0.402 \\ 0.513 \end{pmatrix} ft^4$$

Transformed Area:

$$A_{conc\_0.625} := \pi \cdot \frac{dia_{conccore\_0.625}^{2}}{4} \qquad A_{conc\_0.625} = \begin{pmatrix} 406.49 \\ 481.11 \\ 562 \\ 649.18 \end{pmatrix} \cdot in^2$$

$$A_{t\_0.625} := A_{0.625} + \frac{A_{conc\_0.625}}{n} \qquad A_{t\_0.625} = \begin{pmatrix} 0.69 \\ 0.786 \\ 0.887 \\ 0.994 \end{pmatrix} \cdot ft^2$$

LRFD Eq.10.7.3.13.4-1 for fixity in feet: $1.4*(E_p I_w/E_s)^{0.25}$ (in clays)

E_p in ksi

I_w in ft^4

Es $0.465*S_u$ (Su must be in ksf, results E_s in ksi)

Use same equation in NCHRP#343 pg 61:

$L_{eq}=L_u+1.4R$ where:

L_{eq} = equivalent free standing length of pile

L_u = unsupported length of pile extending above ground

for clays: $R=(E_p*I_p/E_s)^{0.25}$

Average from Field Vanes: $S_u = 500 \cdot psf$

Soil modulus of clay: $E_{soil} := 67 \cdot S_u$ $E_{soil} = 33.5 \cdot ksf$ $E_{soil} = 0.2326 \cdot ksi$

R parameter:

$$R_{0.625} := \left(\frac{E_{steel} \cdot I_{t\_0.625}}{E_{soil}}\right)^{0.25} \qquad R_{0.625} = \begin{pmatrix} 13.08 \\ 14.03 \\ 14.97 \\ 15.9 \end{pmatrix} ft$$

Depth of Fixity:

$$D_{fix\_0.625} := 1.4 \cdot R_{0.625} \qquad D_{fix\_0.625} = \begin{pmatrix} 18 \\ 20 \\ 21 \\ 22 \end{pmatrix} ft$$

Depth to fixity for 5/8" wall pipe piles

Check with LRFD Eq. 10.7.3.13.4-1

$E_{steel} = 29000 \cdot ksi$ $E_{soil} = 0.2326 \cdot ksi$

$$I_{t\_0.625} = \begin{pmatrix} 0.2347 \\ 0.3104 \\ 0.4024 \\ 0.513 \end{pmatrix} ft^4 \qquad Check := 1.4 \cdot \left(\frac{29000 \cdot I_{t\_0.625}}{0.2326}\right)^{0.25} \qquad Check = \begin{pmatrix} 18.31 \\ 19.64 \\ 20.95 \\ 22.26 \end{pmatrix} ft$$

OK

Nominal Axial Structural Resistance of pipe piles

Ref: AASHTO LRFD Bridge Design Specifications 4th Edition 2007

Pier - Pipe Pile driven to bedrock, assume driven through cohesive soils to bedrock (refusal)

Axial pile resistance may be controlled by structural resistance if piles are driven to bedrock.
Check concurrent axial loading and moments with LRFD Equation 6.9.2.2-1 or 6.9.2.2-2
Use LRFD Equation 6.9.5.1-1 or 6.9.5.1-2 to compute the nominal compressive structural resistance for pipe pile sections.

λ in Equation 6.9.5.1-2 has to be computed for the pipe piles since they have an unbraced length.

Yield strength of steel shell: $F_y := 45 \cdot ksi$

Compressive strength of concrete core: $f_c := 4000 \cdot psi$

Yield strength of longitudinal reinforcement: $F_{yr} := 60 \cdot ksi$

Compute λ per 6.9.5.1-3 for composite members:

Effective length factor per LRFD Article 4.6.2.5:

Use case (c) in table C4.6.2.5-1

$K := 1.0$ Because piles are fixed at the end

Exposed length of pile:

Bottom of pile cap to track level is approximately 20 ft at Pier 1

$L_{ex} := 20 \cdot ft$

Unbraced length of column:

$L_{UB\_0.5} := L_{ex} + D_{fix\_0.5}$ $L_{UB\_0.5} = \begin{pmatrix} 37.81 \\ 39.11 \\ 40.41 \\ 41.71 \end{pmatrix} ft$

$L_{UB\_0.625} := L_{ex} + D_{fix\_0.625}$ $L_{UB\_0.625} = \begin{pmatrix} 38.31 \\ 39.64 \\ 40.95 \\ 42.26 \end{pmatrix} ft$

Longitudinal reinforcement:

Assume longitudinal reinforcement of 12 - #8 bars (1-inch) bars equally spaced for all pile sections.

$A_r := 12 \cdot \frac{\pi \cdot (1 \cdot in)^2}{4}$ $A_r = 9.42 \cdot in^2$

Composite Column Constant per Table 6.9.5.1.1

for tube filled sections: $C1 := 1.0$ $C2 := 0.85$ $C3 := 0.40$

Variable Fe:

$$F_{e\_0.5} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.5}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.5}}{A_{0.5}} \qquad F_{e\_0.5} = \begin{pmatrix} 98.59 \\ 100.78 \\ 103.16 \\ 105.67 \end{pmatrix} \cdot ksi$$ for 1/2" walls

$$F_{e\_0.625} := F_y + C1 \cdot F_{yr} \cdot \frac{A_r}{A_{0.625}} + C2 \cdot f_c \cdot \frac{A_{conc\_0.625}}{A_{0.625}} \qquad F_{e\_0.625} = \begin{pmatrix} 87.43 \\ 89.18 \\ 91.07 \\ 93.07 \end{pmatrix} \cdot ksi$$ for 5/8" walls

Radius of gyration of both sets of steel sections:

$$r_{s\_0.5} := \sqrt{\frac{I_{s\_0.5}}{A_{0.5}}} \qquad r_{s\_0.5} = \begin{pmatrix} 0.6925 \\ 0.7514 \\ 0.8104 \\ 0.8693 \end{pmatrix} ft$$ for 1/2" walls

$$r_{s\_0.625} := \sqrt{\frac{I_{s\_0.625}}{A_{0.625}}} \qquad r_{s\_0.625} = \begin{pmatrix} 0.6889 \\ 0.7478 \\ 0.8068 \\ 0.8657 \end{pmatrix} ft$$ for 5/8" walls

E_e term:

$$E_{e\_0.5} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{conc\_0.5}}{A_{0.5}}\right) \qquad E_{e\_0.5} = \begin{pmatrix} 46179 \\ 47705 \\ 49231 \\ 50756 \end{pmatrix} \cdot ksi$$ for 1/2" walls

$$E_{e\_0.625} := E_{steel} \cdot \left(1 + \frac{C3}{n} \cdot \frac{A_{conc\_0.625}}{A_{0.625}}\right) \qquad E_{e\_0.625} = \begin{pmatrix} 42518 \\ 43738 \\ 44959 \\ 46179 \end{pmatrix} \cdot ksi$$ for 5/8" walls

Lamda (λ) term for composite members LRFD Eq. 6.9.5.1-3

$$\lambda_{0.5} := \left[\left(\frac{K \cdot L_{UB\_0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.5}}{E_{e\_0.5}}\right] \qquad \lambda_{0.5} = \begin{pmatrix} 0.6448 \\ 0.58 \\ 0.528 \\ 0.4855 \end{pmatrix}$$ for 1/2" walls

$$\lambda_{0.625} := \left[\left(\frac{K \cdot L_{UB\_0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_{e\_0.625}}{E_{e\_0.625}}\right] \qquad \lambda_{0.625} = \begin{pmatrix} 0.6443 \\ 0.5803 \\ 0.5289 \\ 0.4867 \end{pmatrix}$$ for 5/8" walls

Lamda (λ) term for non composite members LRFD Eq. 6.9.4.1-3

$$\lambda_{0.5\_tip} := \left[\left(\frac{K \cdot L_{UB\_0.5}}{r_{s\_0.5} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right] \qquad \lambda_{0.5\_tip} = \begin{pmatrix} 0.4687 \\ 0.426 \\ 0.391 \\ 0.3619 \end{pmatrix}$$ for 1/2" walls

$$\lambda_{0.625\_tip} := \left[\left(\frac{K \cdot L_{UB\_0.625}}{r_{s\_0.625} \cdot \pi}\right)^2 \cdot \frac{F_y}{E_{steel}}\right] \qquad \lambda_{0.625\_tip} = \begin{pmatrix} 0.4862 \\ 0.4416 \\ 0.4051 \\ 0.3747 \end{pmatrix}$$ for 5/8" walls

Nominal Axial Structural Resistance of 1/2-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.5} := \left(0.66^{\lambda_{0.5}} \cdot F_{e\_0.5} \cdot A_{0.5}\right) \qquad P_{n\_0.5} = \begin{pmatrix} 2784 \\ 3172 \\ 3578 \\ 4002 \end{pmatrix} \cdot kip$$

At the bottom of open-ended piles, or closed ended piles where the conical tip or closed tip experiences breeching, the nominal compressive resistance is a function of only the steel pipe.

$$P_{n\_0.5tip} := \left(0.66^{\lambda_{0.5\_tip}} \cdot F_y \cdot A_{0.5}\right) \qquad P_{n\_0.5tip} = \begin{pmatrix} 1367 \\ 1510 \\ 1652 \\ 1794 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN

for 1/2" walls

Nominal Axial Structural Resistance of 5/8-inch wall

Since λ<2.25 use LRFD Eq. 6.9.5.1-1

$$P_{n\_0.625} := \left(0.66^{\lambda_{0.625}} \cdot F_{e\_0.625} \cdot A_{0.625}\right)$$

$$P_{n\_0.625} = \begin{pmatrix} 3070 \\ 3491 \\ 3929 \\ 4385 \end{pmatrix} \cdot kip$$

At the bottom of open-ended piles, or closed ended piles where the conical tip or closed tip experiences breeching, the nominal compressive resistance is a function of only the steel pipe.

$$P_{n\_0.625tip} := \left(0.66^{\lambda_{0.625\_tip}} \cdot F_y \cdot A_{0.625}\right)$$

$$P_{n\_0.625tip} = \begin{pmatrix} 1688 \\ 1866 \\ 2044 \\ 2221 \end{pmatrix} \cdot kip$$

USE THIS FOR DESIGN
for 5/8" walls

Factored Axial Structural Resistance of a single Pipe Pile:

Strength limit state resistance factor for pipe piles in compression, no damage anticipated - LRFD 6.5.4.2

$$\phi_c := 0.7$$

Factored Structural Resistance (Pr):

$$P_{r\_0.5} := \phi_c \cdot P_{n\_0.5}$$

$$P_{r\_0.5} = \begin{pmatrix} 1949 \\ 2221 \\ 2505 \\ 2801 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r\_0.625} := \phi_c \cdot P_{n\_0.625}$$

$$P_{r\_0.625} = \begin{pmatrix} 2149 \\ 2444 \\ 2751 \\ 3070 \end{pmatrix} \cdot kip$$

for 5/8" walls

Factored Structural Resistance (Pr) for the lower portion of open-ended piles or breached close-ended piles is a function of only the steel shell.

$$P_{r\_0.5tip} := \phi_c \cdot P_{n\_0.5tip}$$

$$P_{r\_0.5tip} = \begin{pmatrix} 957 \\ 1057 \\ 1157 \\ 1256 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{r\_0.625tip} := \phi_c \cdot P_{n\_0.625tip}$$

$$P_{r\_0.625tip} = \begin{pmatrix} 1181 \\ 1306 \\ 1431 \\ 1555 \end{pmatrix} \cdot kip$$

for 5/8" walls

USE THESE FOR STRENGTH LIMIT STATE FACTORED STRUCTURAL RESISTANCE

Service and Extreme Limit States Axial Structural Resistance

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$$\phi := 1.0$$

Factored Compressive Resistance for Service and Extreme Limit States:

$$P_{\_0.5tipf} := \phi \cdot P_{n\_0.5tip} \qquad P_{\_0.5tipf} = \begin{pmatrix} 1367 \\ 1510 \\ 1652 \\ 1794 \end{pmatrix} \cdot kip$$

for 1/2" walls

$$P_{\_0.625tipf} := \phi \cdot P_{n\_0.625tip} \qquad P_{\_0.625tipf} = \begin{pmatrix} 1688 \\ 1866 \\ 2044 \\ 2221 \end{pmatrix} \cdot kip$$

for 5/8" walls

USE THESE FOR SERVICE AND EXTREME LIMIT STATE FACTORED STRUCTURAL RESISTANCE

COMPUTE GEOTECHNICAL RESISTANCE OF PIPE PILES

Pipe pile capacity based on steel shell end bearing on bedrock - driven through soft glaciomarine silt clay deposit.

Pipe piles evaluated:
24 in diameter 1/2 in wall
26 in diameter 1/2 in wall
28 in diameter 1/2 in wall
30 in diameter 1/2 in wall

24 in diameter 5/8 in wall
26 in diameter 5/8 in wall
28 in diameter 5/8 in wall
30 in diameter 5/8 in wall

RQD of bedrock in pier locations ranged from: 45 to 65%.
Bedrock is identified as: GRANITE

Uniaxial Compressive Strength of GRANITE from AASHTO Standard Spec for Highway Bridges 17th Ed. Table 4.4.8.1.2B pg 64
Granite 2100 - 49000 psi Use 20000 psi

$$Q_{uc} := 30000 \cdot psi$$

Reference: Pile Design and Construction Practice, M.J. Tomlinson, Fourth Edition pg 139
Friction angle = 34 to 40 degrees

$$\phi_1 := 34 \cdot deg$$

Piles will not be exposed to water therefore no corrosion is applied.
Bridge is a railroad crossing therefore no scour is considered .

Diameter of piles: Pipe pile wall thickness:

$$dia_{steel} := \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in \qquad wall_t := \begin{pmatrix} \frac{1}{2} \\ \frac{5}{8} \end{pmatrix} \cdot in$$

$$A_{0.5} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.5}}{2}\right)^2 \qquad A_{0.5} = \begin{pmatrix} 36.9 \\ 40.1 \\ 43.2 \\ 46.3 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 1/2" PILES

$$A_{0.625} := \pi \cdot \left(\frac{dia_{steel}}{2}\right)^2 - \pi \cdot \left(\frac{dia_{conccore\_0.625}}{2}\right)^2 \qquad A_{0.625} = \begin{pmatrix} 45.9 \\ 49.8 \\ 53.8 \\ 57.7 \end{pmatrix} \cdot in^2$$

STEEL AREA FOR 5/8" PILES

LRFD Code specifies Canadian Geotechnical Society Method 1985 for resistance determination of end bearing piles on bedrock. (LRFD Table 10.5.5.2.3-1)
Use Canadian Foundation Manual 4th Edition 2006 Section 18.6.3.3.

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

Spacing of discontinuities: $c := 24 \cdot in$ Assumed based on rock core

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

Footing width, b:

$b := dia_{steel}$ $b = \begin{pmatrix} 24 \\ 26 \\ 28 \\ 30 \end{pmatrix} \cdot in$

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}} \qquad K_{sp} = \begin{pmatrix} 0.3392 \\ 0.3327 \\ 0.3271 \\ 0.3222 \end{pmatrix}$$

K_{sp} includes a factor of safety of 3

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

Diameter of socket, B_s: $B_s := 0 \cdot ft$

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

$q_{aA} := Q_{uc} \cdot K_{sp} \cdot d_f$ $q_{aA} = \begin{pmatrix} 1465 \\ 1437 \\ 1413 \\ 1392 \end{pmatrix} \cdot ksf$

Nominal Geotechnical Tip Resistance, R_p:

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

$R_{pA0.5} := \overrightarrow{\left(3q_{aA} \cdot A_{0.5}\right)}$ $R_{pA0.5} = \begin{pmatrix} 1127 \\ 1199 \\ 1272 \\ 1344 \end{pmatrix} \cdot kip$ for 1/2" walls

$R_{pA0.625} := \overrightarrow{\left(3q_{aA} \cdot A_{0.625}\right)}$ $R_{pA0.625} = \begin{pmatrix} 1401 \\ 1492 \\ 1582 \\ 1673 \end{pmatrix} \cdot kip$ for 5/8" walls

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

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

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

$R_{f0.5} := \phi_{stat} \cdot R_{pA0.5}$ $R_{f0.5} = \begin{pmatrix} 507 \\ 540 \\ 572 \\ 605 \end{pmatrix} \cdot kip$ Strength Limit State for 1/2" walls

$R_{f0.625} := \phi_{stat} \cdot R_{pA0.625}$ $R_{f0.625} = \begin{pmatrix} 631 \\ 671 \\ 712 \\ 753 \end{pmatrix} \cdot kip$ Strength Limit State for 5/8" walls

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

$\phi := 1.0$

$R_{fse0.5} := \phi \cdot R_{pA0.5}$ $R_{fse0.5} = \begin{pmatrix} 1127 \\ 1199 \\ 1272 \\ 1344 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 1/2" walls

$R_{fse0.625} := \phi \cdot R_{pA0.625}$ $R_{fse0.625} = \begin{pmatrix} 1401 \\ 1492 \\ 1582 \\ 1673 \end{pmatrix} \cdot kip$ Service/Extreme Limit States for 5/8" walls

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

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

$f_y := 45 \cdot ksi$ yield strength of steel

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

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 40.5 \cdot ksi$ driving stresses in pile cannot exceed 40 ksi

Compute Resistance that can be achieved in a drivability analysis:

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

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

$\phi_{dyn} := 0.65$

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

$\phi_{dyn.reduced} := 0.65 \cdot 0.8$ $\phi_{dyn.reduced} = 0.52$

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
24-in Dia. pile with 1/2-in wall thickness

Pile Size = 24"D x 1/2"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
850.0	38.16	5.11	11.4	7.83	40.00
875.0	38.56	5.21	12.8	7.87	40.13
900.0	38.93	5.29	14.1	7.90	40.28
905.0	39.00	5.30	14.3	7.90	40.34
910.0	39.08	5.32	14.6	7.91	40.35
916.0	39.13	5.33	15.1	7.91	40.28
920.0	39.18	5.34	15.3	7.92	40.33
925.0	39.24	5.36	15.6	7.93	40.36
950.0	39.55	5.43	17.3	7.95	40.45
975.0	39.80	5.55	19.4	7.96	40.55

Limit blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_24x0.5\_factored} := 916 \cdot kip \cdot \phi_{dyn.reduced}$

$R_{dr\_24x0.5\_factored} = 476 \cdot kip$

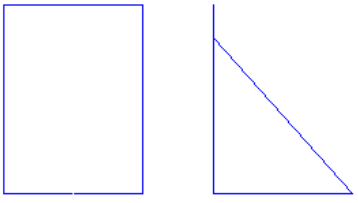
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_24x0.5\_servext} := 916 \cdot kip$

DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	115.00	ft
Pile Penetration	95.00	ft
Pile Top Area	36.90	in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the third fuel setting to install:
26-in Dia. pile with 1/2-in wall thickness

Pile Size = 26"D x 1/2"W

State of Maine Dept. Of Transportation 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
910.0	37.59	4.84	12.8	7.89	39.22
920.0	37.71	4.88	13.3	7.90	39.19
930.0	37.87	4.94	13.7	7.91	39.28
940.0	38.00	4.99	14.2	7.92	39.39
950.0	38.13	5.04	14.7	7.93	39.38
955.0	38.19	5.06	15.0	7.94	39.36
970.0	38.39	5.14	15.9	7.95	39.46
980.0	38.50	5.17	16.6	7.96	39.46
990.0	38.63	5.22	17.1	7.97	39.56
1000.0	38.71	5.25	17.9	7.98	39.56

Limit to blow count to 15 blows per inch

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	40.10 in2

Strength Limit State:

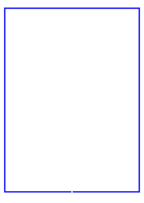
$$R_{dr\_26x0.5\_factored} := 955 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_26x0.5\_factored} = 497 \cdot kip$$

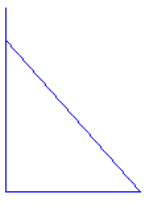
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_26x0.5\_servext} := 955 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
28-in Dia. pile with 1/2-in wall thickness

Pile Size = 28"D x 1/2"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
950.0	39.25	4.83	10.0	8.82	45.51
960.0	39.44	4.88	10.3	8.85	45.57
970.0	39.41	4.95	10.8	8.78	45.28
980.0	39.52	5.00	11.2	8.79	45.27
990.0	39.63	5.03	11.6	8.80	45.24
995.0	39.75	5.04	11.7	8.81	45.28
1000.0	39.82	5.08	11.9	8.81	45.34
1010.0	39.92	5.11	12.3	8.82	45.33
1020.0	40.04	5.16	12.6	8.83	45.45
1030.0	40.20	5.20	13.0	8.84	45.44

Limit driving stress to 40 ksi

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	43.20 in2

Strength Limit State:

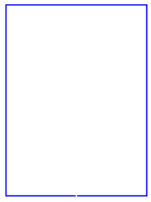
$$R_{dr\_28x0.5\_factored} := 1020 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_28x0.5\_factored} = 530 \cdot kip$$

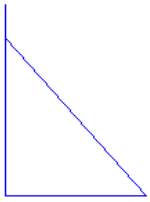
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_28x0.5\_servext} := 1020 \cdot kip$$

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
30-in Dia. pile with 1/2-in wall thickness

Pile Size = 30"D x 1/2"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1030.0	38.83	4.72	11.8	8.82	44.45
1040.0	38.96	4.75	12.2	8.83	44.49
1050.0	39.10	4.76	12.5	8.85	44.53
1060.0	39.25	4.79	12.9	8.85	44.60
1070.0	39.38	4.81	13.2	8.86	44.65
1180.0	40.54	4.75	18.8	8.96	45.03
1090.0	39.60	4.83	14.0	8.88	44.76
1100.0	39.70	4.84	14.5	8.89	44.69
1110.0	39.82	4.84	15.0	8.90	44.74
1120.0	39.95	4.83	15.4	8.91	44.80

Limit to blow count to 15 blows per inch

Strength Limit State:

$$R_{dr\_30x0.5\_factored} := 1110 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_30x0.5\_factored} = 577 \cdot kip$$

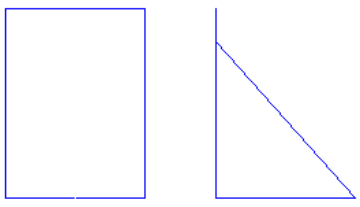
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_30x0.5\_servext} := 1110 \cdot kip$$

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	46.30 in2

Pile Model — Skin Friction Distribution



Res. Shaft = 10 % (Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
24-in Dia. pile with 5/8-in wall thickness

Pile Size = 24"D x 5/8"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1050.0	39.27	4.83	12.7	8.85	44.67
1060.0	39.42	4.85	13.0	8.85	44.75
1070.0	39.51	4.87	13.5	8.86	44.67
1080.0	39.62	4.89	13.9	8.87	44.73
1090.0	39.77	4.89	14.3	8.88	44.80
1100.0	39.89	4.90	14.7	8.89	44.86
1106.0	39.92	4.91	15.0	8.89	44.85
1110.0	39.95	4.91	15.3	8.90	44.80
1120.0	40.09	4.91	15.7	8.91	44.87
1130.0	40.20	4.90	16.2	8.92	44.93

Limit to blow count to 15 blows per inch

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	45.90 in2

Strength Limit State:

$$R_{dr\_24x0.625\_factored} := 1106 \cdot kip \cdot \phi_{dyn.reduced}$$

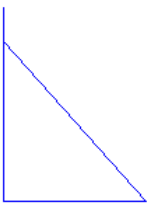
$$R_{dr\_24x0.625\_factored} = 575 \cdot kip$$

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

$$R_{dr\_24x0.625\_servext} := 1106 \cdot kip$$

Pile Model

Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
26-in Dia. pile with 5/8-in wall thickness

Pile Size = 26"D x 5/8"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1080.0	38.14	4.27	12.3	8.87	43.78
1090.0	38.21	4.27	12.7	8.88	43.75
1100.0	38.39	4.26	13.0	8.89	43.86
1110.0	38.48	4.25	13.4	8.90	43.86
1120.0	38.59	4.23	13.8	8.91	43.86
1130.0	38.69	4.22	14.2	8.91	43.85
1140.0	38.81	4.20	14.6	8.92	43.84
1150.0	38.92	4.18	15.0	8.93	43.96
1160.0	39.03	4.18	15.5	8.94	43.95
1170.0	39.17	4.17	15.8	8.95	44.08

Limit to blow count to 15 blows per inch

DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	49.80 in2

Strength Limit State:

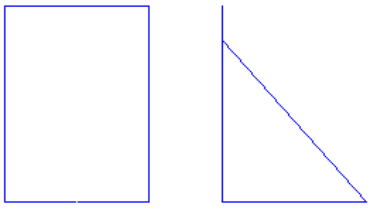
$$R_{dr\_26x0.625\_factored} := 1150 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_26x0.625\_factored} = 598 \cdot kip$$

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

$$R_{dr\_26x0.625\_servext} := 1150 \cdot kip$$

Pile Model — Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
28-in Dia. pile with 5/8-in wall thickness

Pile Size = 28"D x 5/8"W

State of Maine Dept. Of Transportation
Auburn CNR Crossing Bridge Pipe Pile

18-Sep-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
1140.0	37.33	3.68	13.1	8.92	42.95
1150.0	37.48	3.68	13.4	8.93	42.98
1160.0	37.60	3.67	13.8	8.95	43.03
1170.0	37.72	3.67	14.2	8.95	43.08
1180.0	37.84	3.66	14.5	8.96	43.14
1191.0	37.95	3.65	15.0	8.97	43.16
1200.0	38.04	3.66	15.4	8.98	43.12
1210.0	38.17	3.67	15.7	8.99	43.30
1220.0	38.29	3.70	16.1	9.00	43.36
1230.0	38.38	3.71	16.7	9.01	43.30

Limit to blow count to 15 blows per inch

Strength Limit State:

$R_{dr\_28x0.625\_factored} := 1191 \cdot kip \cdot \phi_{dyn.reduced}$

$R_{dr\_28x0.625\_factored} = 619 \cdot kip$

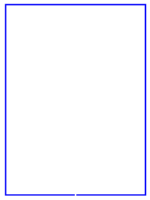
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr\_28x0.625\_servext} := 1190 \cdot kip$

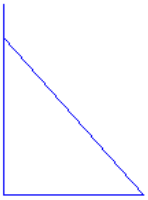
DELMAG D 36-32

Efficiency	0.800
Helmet	3.20 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.040 in
Skin Damping	0.200 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	115.00 ft
Pile Penetration	95.00 ft
Pile Top Area	53.80 in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Assume Contractor will use a Delmag D 36-32 hammer on the highest fuel setting to install:
30-in Dia. pile with 5/8-in wall thickness

Pile Size = 30"D x 5/8"W

State of Maine Dept. Of Transportation — 18-Sep-2008
Auburn CNR Crossing Bridge Pipe Pile — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
1160.0	36.27	3.27	12.6	8.95	42.25
1170.0	36.35	3.27	13.0	8.96	42.25
1180.0	36.46	3.28	13.3	8.96	42.25
1190.0	36.60	3.29	13.7	8.97	42.26
1200.0	36.69	3.31	14.0	8.98	42.27
1210.0	37.00	3.32	14.1	9.08	42.82
1220.0	36.97	3.34	14.6	9.01	42.49
1230.0	37.07	3.37	15.0	9.01	42.51
1240.0	37.38	3.36	15.1	9.11	42.94
1250.0	37.28	3.38	15.8	9.03	42.51

Limit to blow count to 15 blows per inch

Strength Limit State:

$$R_{dr\_30x0.625\_factored} := 1230 \cdot kip \cdot \phi_{dyn.reduced}$$

$$R_{dr\_30x0.625\_factored} = 640 \cdot kip$$

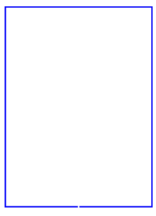
Service and Extreme Limit States: $\phi := 1.0$

$$R_{dr\_30x0.625\_servext} := 1230 \cdot kip$$

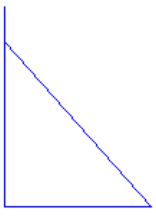
DELMAG D 36-32

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.200	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	115.00	ft
Pile Penetration	95.00	ft
Pile Top Area	57.70	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Abutment and Wingwall Passive and Active Earth Pressure:

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

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

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

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

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

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

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

$K_p = 6.89$

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

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

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

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

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:

$\phi := 32 \cdot deg$

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

Settlement Analyses:

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

The roadway will be widened by 8 feet behind both abutments with a maximum fill height of 4.3 feet. Look at Abutment No.1 at Station 4+52 with 4.3 ft of fill. Simplified soil profile based on BB-ACNR-101:

Finished Grade
Elevation 237.0 ft

Proposed Fill - Look at 4.3 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Elevation 232.7 ft

Existing Fill/Native sand - fine to coarse sand

$H_1 := 17.3 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{sand1} := 20$

Elevation 215.4 ft

Silt - Upper crust

Groundwater Elevation 215.4 ft

$H_2 := 16.7 \cdot ft$ $\gamma_{ucsilt} := 115 \cdot pcf$ $N_{ucsilt} := 10$

$\gamma_w := 62.4pcf$

Elevation 198.7 ft

Silt - Su=550 psf (medium stiff)

$H_3 := 9.0 \cdot ft$ $\gamma_{silt} := 115 \cdot pcf$ $C_{c\_silt1} := 0.1148$ $C_{r\_silt1} := 0.0103$ $e_{os1} := 0.81$

Elevation 189.7 ft

Clayey Silt - Su = 660 (medium stiff)

$H_4 := 26.5 \cdot ft$ $\gamma_{clayeysilt} := 115 \cdot pcf$ $C_{c\_clayeysilt1} := 0.3449$ $C_{r\_clayeysilt1} := 0.0299$ $e_{ocs1} := 1.07$

Elevation 163.2 ft

Silt - Su = 620 (medium stiff)

$H_5 := 7 \cdot ft$ $\gamma_{silt} := 115 \cdot pcf$ $C_{c\_silt2} := 0.1536$ $e_{os2} := 0.85$

Elevation 156.2 ft

Clayey Silt - Su =1300 (stiff)

$H_6 := 11.5 \cdot ft$ $\gamma_{clayeysilt} := 115 \cdot pcf$ $C_{c\_clayeysilt2} := 0.1841$ $e_{ocs2} := 0.96$

Elevation 144.7 ft

Sand - fine to coarse sand, very dense

$H_7 := 7.0 \cdot ft$ $\gamma_{sand} := 125 \cdot pcf$ $N_{sand2} := 50$

Top of Bedrock
Elevation 137.7 ft

Bedrock

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name: CNR Crossing Client Auburn Project Number: 15600.00
Project Manager : JWentworth Date: 09/18/08 Computed by: km

Embank. slope a = 8.00(ft)
Embank. width b = 31.00(ft)
p load/unit area = 537.50(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 18.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	537.50
2.00	536.71
4.00	531.72
6.00	520.33
8.00	502.79
10.00	480.76
12.00	456.24
14.00	430.92
16.00	405.97
18.00	382.12
20.00	359.74
22.00	338.98
24.00	319.87
26.00	302.35
28.00	286.30
30.00	271.61
32.00	258.15
34.00	245.82
36.00	234.48
38.00	224.06
40.00	214.44
42.00	205.56
44.00	197.33
46.00	189.70
48.00	182.61
50.00	176.00
52.00	169.82
54.00	164.05
56.00	158.64
58.00	153.57
60.00	148.79
62.00	144.30
64.00	140.06
66.00	136.05
68.00	132.26
70.00	128.67
72.00	125.27
74.00	122.04
76.00	118.96
78.00	116.04
80.00	113.25
82.00	110.59
84.00	108.05
86.00	105.62
88.00	103.30
90.00	101.07
92.00	98.94
94.00	96.90

at 8.7 ft $\Delta\sigma_{zsand1} := 495.08 \cdot psf$

at 25.7 ft $\Delta\sigma_{zucsilt} := 313.74 \cdot psf$

at 38.2 ft $\Delta\sigma_{zsilt1} := 223.10 \cdot psf$

at 56.3 ft $\Delta\sigma_{zclayeysilt1} := 158.08 \cdot psf$

at 73.0 ft $\Delta\sigma_{zsilt2} := 123.66 \cdot psf$

at 82.3 ft $\Delta\sigma_{zclayeysilt2} := 110.21 \cdot psf$

at 91.5 ft $\Delta\sigma_{zsand2} := 100.54 \cdot psf$

Existing Fill/Sand

Determine corrected SPT value N':

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

$\sigma_{1o} := \frac{H_1}{2} \cdot (\gamma_{sand})$ $\sigma_{1o} = 1081.25 \cdot psf$ at mid-point

SPT N-value (bpf) $N_{sand1} := 20$

AT P_o = 1080 psf N'/N_{sand} = r1 = 1.25 $r1 := 1.25$

Corrected Blow Count $N' := r1 \cdot N_{sand1}$ $N' = 25$

From Figure 13 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C1 := 80$

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

$\Delta\sigma_{zsand1} = 495.08 \cdot psf$

Upper Crust Silt

Determine corrected SPT value N':

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

$\sigma_{2o} := \left[\frac{H_2}{2} \cdot (\gamma_{ucsilt} - \gamma_w)\right] + H_1 \cdot (\gamma_{sand})$ $\sigma_{2o} = 2601.71 \cdot psf$ at mid-point

SPT N-value (bpf) $N_{ucsilt} := 10$

AT P_o = 2600 psf N'/N_{fill} = r1 = 0.87 $r1 := 0.87$

Corrected Blow Count $N' := r1 \cdot N_{ucsilt}$ $N' = 9$

From Figure 13 using the "Inorganic silt" curve

Bearing Capacity Index: $C2 := 27$

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

$\Delta\sigma_{zucsilt} = 313.74 \cdot psf$

Silt

Average values from lab data: $e_{os1} = 0.81$ $C_{r\_silt1} := 0.0103$

$\sigma_{3o} := H_1 \cdot (\gamma_{sand}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + \frac{H_3}{2}(\gamma_{silt} - \gamma_w)$ $\sigma_{3o} = 3277.62 \cdot psf$ at mid-point

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

$\Delta\sigma_{zsilt1} = 223.1 \cdot psf$

Clayey Silt

Average values from lab data:

$e_{ocs1} = 1.07$ $C_{r\_clayeysilt1} := 0.0299$

$\sigma_{4o} := H_1 \cdot (\gamma_{sand}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + H_3 \cdot (\gamma_{silt} - \gamma_w) + \frac{H_4}{2} \cdot (\gamma_{clayeysilt} - \gamma_w)$ $\sigma_{4o} = 4211.27 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclayeysilt1} = 158.08 \cdot psf$

Silt

Average values from lab data: $e_{os2} = 0.85$ $C_{c\_silt2} = 0.1536$

$\sigma_{5o} := H_1 \cdot (\gamma_{sand}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + H_3 \cdot (\gamma_{silt} - \gamma_w) + H_4(\gamma_{clayeysilt} - \gamma_w) + \frac{H_5}{2}(\gamma_{silt} - \gamma_w)$

$\sigma_{5o} = 5092.32 \cdot psf$ at mid-point

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

$\Delta\sigma_{zsilt2} = 123.66 \cdot psf$

Clayey Silt

Average values from lab data:

$e_{ocs2} = 0.96$ $C_{c\_clayeysilt2} = 0.1841$

$\sigma_{6o} := H_1 \cdot (\gamma_{sand}) + (H_2 + H_3 + H_4 + H_5) \cdot (115 \cdot pcf - \gamma_w) + \frac{H_6}{2} \cdot (\gamma_{clayeysilt} - \gamma_w)$

$\sigma_{6o} = 5578.87 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclayeysilt2} = 110.21 \cdot psf$

Sand

Determine corrected SPT value N':

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

$$\sigma_{7o} := H_1 \cdot (\gamma_{sand}) + (H_2 + H_3 + H_4 + H_5 + H_6) \cdot (115 \cdot pcf - \gamma_w) + \frac{H_7}{2} \cdot (\gamma_{sand} - \gamma_w)$$

$\sigma_{7o} = 6100.42 \cdot psf$ at mid-point

SPT N-value (bpf) $N_{sand2} := 50$

AT P_o = 6100 psf N'/N_{sand} = r1 = 0.65 $r1 := 0.65$

Corrected Blow Count $N' := r1 \cdot N_{sand2}$ $N' = 33$

From Figure 13 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C3 := 95$

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

$\Delta\sigma_{zsand2} = 100.54 \cdot psf$

Calculate Settlement:

Fill/Sand: $\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{zsand1}}{\sigma_{1o}}\right)$ $\Delta H_1 = 0.4249 \cdot in$

Upper Crust Silt: $\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{zucsilt}}{\sigma_{2o}}\right)$ $\Delta H_2 = 0.367 \cdot in$

Silt: $\Delta H_3 := H_3 \cdot \left(\frac{C_{r\_silt1}}{1 + e_{os1}}\right) \cdot \log\left(\frac{\sigma_{3o} + \Delta\sigma_{zsilt1}}{\sigma_{3o}}\right)$ $\Delta H_3 = 0.0176 \cdot in$

Clayey Silt: $\Delta H_4 := H_4 \cdot \left(\frac{C_{r\_clayeysilt1}}{1 + e_{ocs1}}\right) \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{zclayeysilt1}}{\sigma_{4o}}\right)$ $\Delta H_4 = 0.0735 \cdot in$

Silt: $\Delta H_5 := H_5 \cdot \left(\frac{C_{c\_silt2}}{1 + e_{os2}}\right) \cdot \log\left(\frac{\sigma_{5o} + \Delta\sigma_{zsilt2}}{\sigma_{5o}}\right)$ $\Delta H_5 = 0.0727 \cdot in$

Clayey Silt: $\Delta H_6 := H_6 \cdot \left(\frac{C_{c\_clayeysilt2}}{1 + e_{ocs2}}\right) \cdot \log\left(\frac{\sigma_{6o} + \Delta\sigma_{zclayeysilt2}}{\sigma_{6o}}\right)$ $\Delta H_6 = 0.1101 \cdot in$

Sand: $\Delta H_7 := H_7 \cdot \frac{1}{C3} \cdot \log\left(\frac{\sigma_{7o} + \Delta\sigma_{zsand2}}{\sigma_{7o}}\right)$ $\Delta H_7 = 0.0063 \cdot in$

Total Settlement = $\Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 + \Delta H_7 = 1.072 \cdot in$

Consolidation Settlement = $\Delta H_3 + \Delta H_4 + \Delta H_5 + \Delta H_6 = 0.2739 \cdot in$

CNR Crossing Bridge
Over Canadian National Railway
Auburn, Maine
PIN 15600.00

By: Kate Maguire
July 2008
Checked by: LK 10-1-2008

Check Clay settlement using Das in an Excel spreadsheet:

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Abutment No.1 Boring BB-ACNR-101													
2	Auburn (15600.00) embankment settlements for widened roadway @ Station 4+54													
3	4.3 ft of new fill overlying 17 ft of existing sand and 71 ft of silt and clay. Groundwater table at 17 ft bgs.													
4														
5	Unit Weight of clay	115	pcf				B1	23	ft					
6	Unit Weight of sand	125	pcf											
7	Unit Weight of water	62.4	pcf				B2	8	ft					
8							H	4.3						
9														
10	Depth	Ho	Po	ocr	Pmax		a1	a2	Dstress		settlement			
11	(ft)	(ft)	(psf)		(psf)		(rad)	(rad)	(psf)		(ft)			
12	17		2662.5	2.0	5325		-0.134904	-0.934288	498.5751			Silt -UC		
13	18	1	2715.1	2.0	5430.2		-0.13798	-0.90675	493.2302		0.000793	Cc	0.2	
14	19	1	2767.7	2.0	5535.4		-0.140593	-0.88035	487.6619		0.000771	Cr	0.022	
15	20	1	2820.3	2.0	5640.6		-0.142777	-0.855053	481.9017		0.00075	e	1.01	
16	21	1	2872.9	2.0	5745.8		-0.144565	-0.830821	475.9804		0.000729			
17	22	1	2925.5	2.0	5851		-0.145988	-0.807617	469.9273		0.000708			
18	23	1	2978.1	2.0	5956.2		-0.147078	-0.785398	463.77		0.000688			
19	24	1	3030.7	2.0	6061.4		-0.147866	-0.764125	457.5343		0.000668			
20	25	1	3083.3	2.0	6166.6		-0.148378	-0.743756	451.2442		0.000649			
21	26	1	3135.9	2.0	6271.8		-0.148644	-0.72425	444.9214		0.000631			
22	27	1	3188.5	2.0	6377		-0.148686	-0.705568	438.5858		0.000613			
23	28	1	3241.1	2.0	6482.2		-0.148531	-0.687671	432.2551		0.000595			
24	29	1	3293.7	2.0	6587.4		-0.148198	-0.670522	425.9452		0.000578			
25	30	1	3346.3	2.0	6692.6		-0.147707	-0.654083	419.6701		0.000562			
26	31	1	3398.9	2.0	6797.8		-0.147078	-0.63832	413.4422		0.000546			
27	32	1	3451.5	2.0	6903		-0.146327	-0.623199	407.2722		0.00053			
28	33	1	3504.1	2.0	7008.2		-0.145469	-0.608689	401.1693		0.000515			
29	34	1	3556.7	2.0	7113.4		-0.144518	-0.594759	395.1413		0.000501			
30	35	1	3609.3	1.8	6496.74		-0.143486	-0.58138	389.1951		0.001555	Silt	Cc	0.1148
31	36	1	3661.9	1.8	6591.42		-0.142385	-0.568525	383.336		0.001512		Cr	0.0633
32	37	1	3714.5	1.8	6686.1		-0.141225	-0.556166	377.5688		0.00147		e	0.81
33	38	1	3767.1	1.8	6780.78		-0.140014	-0.54428	371.8969		0.00143			
34	39	1	3819.7	1.8	6875.46		-0.138762	-0.532844	366.3234		0.001391			
35	40	1	3872.3	1.8	6970.14		-0.137476	-0.521834	360.8502		0.001353			
36	41	1	3924.9	1.8	7064.82		-0.136161	-0.511231	355.4791		0.001317			
37	42	1	3977.5	1.8	7159.5		-0.134825	-0.501013	350.2109		0.001282			
38	43	1	4030.1	1.8	7254.18		-0.133472	-0.491164	345.0461		0.001248			
39	44	1	4082.7	1.5	6124.05		-0.132106	-0.481664	339.985		0.003594	ClayeySilt	Cc	0.4097
40	45	1	4135.3	1.5	6202.95		-0.130733	-0.472497	335.027		0.003501		Cr	0.2059
41	46	1	4187.9	1.5	6281.85		-0.129355	-0.463648	330.1718		0.00341		e	0.99
42	47	1	4240.5	1.5	6360.75		-0.127976	-0.455101	325.4185		0.003322			
43	48	1	4293.1	1.5	6439.65		-0.126598	-0.446842	320.7659		0.003238			
44	49	1	4345.7	1.5	6518.55		-0.125224	-0.438859	316.2128		0.003156			
45	50	1	4398.3	1.5	6597.45		-0.123857	-0.431139	311.7578		0.003077			
46	51	1	4450.9	1.5	6676.35		-0.122498	-0.423669	307.3993		0.003001			
47	52	1	4503.5	1.5	6755.25		-0.121148	-0.416439	303.1357		0.002927			
48	53	1	4556.1	1.5	6834.15		-0.11981	-0.409437	298.9652		0.002856			
49	54	1	4608.7	1.5	6913.05		-0.118483	-0.402655	294.886		0.002787			
50	55	1	4661.3	1.5	6991.95		-0.117171	-0.396081	290.8962		0.00272			
51	56	1	4713.9	1.5	7070.85		-0.115872	-0.389709	286.9939		0.002656			
52	57	1	4766.5	1.5	7149.75		-0.114589	-0.383528	283.1773		0.002593			
53	58	1	4819.1	1.5	7228.65		-0.113322	-0.37753	279.4443		0.000304		Cc	0.28
54	59	1	4871.7	1.5	7307.55		-0.11207	-0.371709	275.793		0.000297		Cr	0.0268
55	60	1	4924.3	1.5	7386.45		-0.110835	-0.366057	272.2216		0.00029		e	1.16
56	61	1	4976.9	1.5	7465.35		-0.109617	-0.360566	268.7279		0.000283			
57	62	1	5029.5	1.5	7544.25		-0.108417	-0.355231	265.3102		0.000277			
58	63	1	5082.1	1.5	7623.15		-0.107233	-0.350045	261.9666		0.000271			
59	64	1	5134.7	1.5	7702.05		-0.106067	-0.345002	258.6951		0.000265			
60	65	1	5187.3	1.5	7780.95		-0.104919	-0.340097	255.494		0.000259			
61	66	1	5239.9	1.5	7859.85		-0.103788	-0.335324	252.3614		0.000253			
62	67	1	5292.5	1.5	7938.75		-0.102675	-0.330679	249.2956		0.000248			
63	68	1	5345.1	1.5	8017.65		-0.101579	-0.326156	246.2948		0.000243			
64	69	1	5397.7	1.5	8096.55		-0.100501	-0.321751	243.3573		0.000238			

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
65	70	1	5450.3	1.4	7630.42		-0.099439	-0.317459	240.4816		0.000833	Silt	Cc	0.1536
66	71	1	5502.9	1.4	7704.06		-0.098395	-0.313276	237.6658		0.000816		Cr	0.0822
67	72	1	5555.5	1.4	7777.7		-0.097368	-0.309199	234.9086		0.000799		e	0.85
68	73	1	5608.1	1.4	7851.34		-0.096357	-0.305223	232.2082		0.000783			
69	74	1	5660.7	1.4	7924.98		-0.095363	-0.301345	229.5633		0.000767			
70	75	1	5713.3	1.4	7998.62		-0.094386	-0.297562	226.9724		0.000752			
71	76	1	5765.9	1.4	8072.26		-0.093424	-0.293869	224.4339		0.000737			
72	77	1	5818.5	1.3	7564.05		-0.092479	-0.290265	221.9467		0.000265	Clayey Silt	Cc	0.36
73	78	1	5871.1	1.3	7632.43		-0.091549	-0.286745	219.5092		0.00026		Cr	0.032
74	79	1	5923.7	1.3	7700.81		-0.090634	-0.283308	217.1202		0.000255		e	0.96
75	80	1	5976.3	1.3	7769.19		-0.089734	-0.27995	214.7784		0.00025			
76	81	1	6028.9	1.3	7837.57		-0.08885	-0.276668	212.4826		0.000246			
77	82	1	6081.5	1.3	7905.95		-0.08798	-0.273461	210.2315		0.000241			
78	83	1	6134.1	1.3	7974.33		-0.087124	-0.270325	208.0241		0.000236			
79	84	1	6186.7	1.3	8042.71		-0.086283	-0.267259	205.8591		0.000232			
80	85	1	6239.3	1.3	8111.09		-0.085455	-0.26426	203.7356		0.000228		total	
81	86	1	6291.9	1.3	8179.47		-0.084641	-0.261326	201.6523		0.000224		settlement	
82	87	1	6344.5	1.3	8247.85		-0.083841	-0.258455	199.6083		0.00022		(in.)	
83	88	1	6397.1	1.3	8316.23		-0.083053	-0.255645	197.6025		0.000216	0.07781166	**0.9**	
84														
85	Reference: Principles of Foundation Engineering Fourth Edition Braja M. Das													
86		Section 4.6 Stress Increase Under an Embankment pg 233												

Both methods to calculate consolidation settlement result in approximately 1 inch of settlement - OK

CNR Crossing Bridge
Over Canadian National Railway
Auburn, Maine
PIN 15600.00

By: Kate Maguire
July 2008
Checked by: LK 10-1-2008

The roadway will be widened by 8 feet behind both abutments with a maximum fill height of 4.3 feet. Look at Station 6+00 as the critical section on the north side.

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-ACNR-104:

Finished Grade
Elevation 240.0 ft

Proposed Fill
Assume: 4.3 feet of fill
N = 25 bpf (medium dense)
γ = 125 pcf

Elevation 235.7 ft

Existing Fill: fine to coarse sand
12.6 feet thick
N = 20 bpf (medium dense)
γ = 125 pcf

$H_1 := 12.6 \cdot ft$

$\gamma_{fill} := 125 \cdot pcf$

$N_{fill} := 20$

Elevation 223.1 ft

Groundwater Elevation 223.1 ft

$\gamma_w := 62.4pcf$

Silt: Upper crust
15.4 feet thick
N = 16 bpf (medium dense)
γ = 115 pcf

$H_2 := 15.4 \cdot ft$

$\gamma_{ucsilt} := 115 \cdot pcf$

$N_{ucsilt} := 16$

Elevation 207.7 ft

Clayey Silt
10.0 feet thick
Su=600 psf (medium stiff)
γ = 115 pcf

$H_3 := 10.0 \cdot ft$

$\gamma_{clayeysilt} := 115 \cdot pcf$

$e_{ocs} := 1.12$

$C_{c\_clayeysilt} := 0.382$

$C_{r\_clayeysilt} := 0.0448$

Elevation 197.7 ft

Silty Clay
10.0 feet thick
Su = 1075 (medium stiff)
γ = 115 pcf

$H_4 := 10.0 \cdot ft$

$\gamma_{siltyclay} := 115 \cdot pcf$

$C_{c\_siltyclay} := 0.274$

$e_{osc} := 0.99$

$C_{r\_siltyclay} := 0.0419$

Elevation 187.7 ft

Silt
2.7 feet thick
Su = 500 (medium stiff)
γ = 115 pcf

$H_5 := 2.7 \cdot ft$

$\gamma_{silt} := 115 \cdot pcf$

$C_{c\_silt} := 0.2743$

$e_{os} := 1.09$

Top of Bedrock
Elevation 185.0 ft

Bedrock

LOADING ON AN INFINITE STRIP - VERTICAL EMBANKMENT LOADING

Project Name: CNR Crossing Client: Auburn Project Number: 15600.00
Project Manager : JWentworth Date: 09/22/08 Computed by: km

Embank. slope a = 8.00(ft)
Embank. width b = 31.00(ft)
p load/unit area = 537.50(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION

X = 18.00(ft)

Z (ft)	Vert. Δz (psf)
5.00	526.86
6.00	520.33
7.00	512.25
8.00	502.79
9.00	492.21
10.00	480.76
11.00	468.69
12.00	456.24
13.00	443.59
14.00	430.92
15.00	418.34
16.00	405.97
17.00	393.88
18.00	382.12
19.00	370.73
20.00	359.74
21.00	349.15
22.00	338.98
23.00	329.23
24.00	319.87
25.00	310.92
26.00	302.35
27.00	294.14
28.00	286.30
29.00	278.79
30.00	271.61
31.00	264.73
32.00	258.15
33.00	251.85
34.00	245.82
35.00	240.03
36.00	234.48
37.00	229.16
38.00	224.06
39.00	219.15
40.00	214.44
41.00	209.91
42.00	205.56
43.00	201.37
44.00	197.33
45.00	193.45
46.00	189.70
47.00	186.09
48.00	182.61
49.00	179.24
50.00	176.00
51.00	172.86
52.00	169.82
53.00	166.89
54.00	164.05

at 6.3 ft $\Delta\sigma_{zfill} := 517.91 \cdot psf$

at 20.3 ft $\Delta\sigma_{zucsilt} := 358.15 \cdot psf$

at 33.0 ft $\Delta\sigma_{zclayeysilt} := 251.85 \cdot psf$

at 43.0 ft $\Delta\sigma_{zsiltyclay} := 201.37 \cdot psf$

at 49.35 ft $\Delta\sigma_{zsilt} := 178.67 \cdot psf$

Existing Fill

Determine corrected SPT value N':

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

$\sigma_{1o} := \frac{H_1}{2} \cdot (\gamma_{fill})$ $\sigma_{1o} = 787.5 \cdot psf$ at mid-point

SPT N-value (bpf) $N_{fill} := 20$

AT P_o = 780 psf N'/N_{fill} = r1 = 1.1 $r1 := 1.1$

Corrected Blow Count $N' := r1 \cdot N_{fill}$ $N' = 22$

From Figure 13 using the "clean well graded fine to coarse sand" curve

Bearing Capacity Index: $C1 := 73$

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

$\Delta\sigma_{zfill} = 517.91 \cdot psf$

Upper Crust Silt

Determine corrected SPT value N':

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

$\sigma_{2o} := \left[\frac{H_2}{2} \cdot (\gamma_{ucsilt} - \gamma_w)\right] + H_1 \cdot (\gamma_{fill})$ $\sigma_{2o} = 1980.02 \cdot psf$ at mid-point

SPT N-value (bpf) $N_{ucsilt} := 16$

AT P_o = 1980 psf N'/N_{fill} = r1 = 0.95 $r1 := 0.95$

Corrected Blow Count $N' := r1 \cdot N_{ucsilt}$ $N' = 15$

From Figure 13 using the "Inorganic silt" curve

Bearing Capacity Index: $C2 := 35$

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

$\Delta\sigma_{zucsilt} = 358.15 \cdot psf$

Clayey Silt

Average values from lab data:

$e_{ocs} = 1.12$ $C_{r\_clayeysilt} = 0.0448$

$\sigma_{3o} := H_1 \cdot (\gamma_{fill}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + \frac{H_3}{2} \cdot (\gamma_{clayeysilt} - \gamma_w)$ $\sigma_{3o} = 2648.04 \cdot psf$ at mid-point

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

$\Delta\sigma_{zclayeysilt} = 251.85 \cdot psf$

Silty Clay

Average values from lab data: $e_{osc} = 0.99$ $C_{r\_siltyclay} = 0.0419$

$$\sigma_{4o} := H_1 \cdot (\gamma_{fill}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + H_3 \cdot (\gamma_{clayeysilt} - \gamma_w) + \frac{H_4}{2}(\gamma_{siltyclay} - \gamma_w)$$

$\sigma_{4o} = 3174.04 \cdot psf$ at mid-point

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

$\Delta\sigma_{zsiltyclay} = 201.37 \cdot psf$

Silt

Average values from lab data: $e_{os} = 1.09$ $C_{c\_silt} = 0.2743$

$$\sigma_{5o} := H_1 \cdot (\gamma_{fill}) + H_2 \cdot (\gamma_{ucsilt} - \gamma_w) + H_3 \cdot (\gamma_{clayeysilt} - \gamma_w) + H_4(\gamma_{siltyclay} - \gamma_w) + \frac{H_5}{2}(\gamma_{silt} - \gamma_w)$$

$\sigma_{5o} = 3508.05 \cdot psf$ at mid-point

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

$\Delta\sigma_{zsilt} = 178.67 \cdot psf$

Calculate Settlement:

Fill: $\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{zfill}}{\sigma_{1o}}\right)$ $\Delta H_1 = 0.4546 \cdot in$

Upper Crust Silt: $\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{zucsilt}}{\sigma_{2o}}\right)$ $\Delta H_2 = 0.3813 \cdot in$

Clayey Silt: $\Delta H_3 := H_3 \cdot \left(\frac{C_{r\_clayeysilt}}{1 + e_{ocs}}\right) \cdot \log\left[\frac{\sigma_{3o} + (\Delta\sigma_{zclayeysilt})}{\sigma_{3o}}\right]$ $\Delta H_3 = 0.1001 \cdot in$

Silty Clay: $\Delta H_4 := H_4 \cdot \left(\frac{C_{r\_siltyclay}}{1 + e_{osc}}\right) \cdot \log\left(\frac{\sigma_{4o} + \Delta\sigma_{zsiltyclay}}{\sigma_{4o}}\right)$ $\Delta H_4 = 0.0675 \cdot in$

Silt: $\Delta H_5 := H_5 \cdot \left(\frac{C_{c\_silt}}{1 + e_{os}}\right) \cdot \log\left(\frac{\sigma_{5o} + \Delta\sigma_{zsilt}}{\sigma_{5o}}\right)$ $\Delta H_5 = 0.0917 \cdot in$

Total Settlement = $\Delta H_1 + \Delta H_2 + \Delta H_3 + \Delta H_4 + \Delta H_5 = 1.0952 \cdot in$

Consolidation Settlement = $\Delta H_3 + \Delta H_4 = 0.1676 \cdot in$

CNR Crossing Bridge
Over Canadian National Railway
Auburn, Maine
PIN 15600.00

By: Kate Maguire
July 2008
Checked by: LK 10-1-2008

Check Clay settlement using Das in an Excel spreadsheet:

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	Auburn (15600.00) embankment settlements for Station 6+00													
2	4.3 ft of new fill overlying 12.6 ft of existing fill and 38.1 ft of silt and clay. Groundwater at 12.6 ft bgs.													
3														
4	Unit weight of clay	115	pcf				B1	23	ft					
5	Unit Weight of sand	125	pcf											
6	Unit weight of water	62.4	pcf				B2	8	ft					
7							H	4.3						
8														
9	Depth	Ho	Po	ocr	Pmax		a1	a2	Dstress		settlement			
10	(ft)	(ft)	(psf)		(psf)		(rad)	(rad)	(psf)		(ft)			
11	13		1575	2.0	3150		-0.117372	-1.056345	517.0938					
12	14	1	1627.6	2.0	3255.2		-0.122594	-1.024007	512.9534		0.001302	Silt -UC		
13	15	1	1680.2	2.0	3360.4		-0.127241	-0.992894	508.4676		0.001257	Cc	0.2	
14	16	1	1732.8	2.0	3465.6		-0.131335	-0.962994	503.6648		0.001213	Cr	0.022	
15	17	1	1785.4	2.0	3570.8		-0.134904	-0.934288	498.5751		0.001171	e	1.01	
16	18	1	1838	2.0	3676		-0.13798	-0.90675	493.2302		0.00113			
17	19	1	1890.6	2.0	3781.2		-0.140593	-0.88035	487.6619		0.001091			
18	20	1	1943.2	2.0	3886.4		-0.142777	-0.855053	481.9017		0.001053			
19	21	1	1995.8	2.0	3991.6		-0.144565	-0.830821	475.9804		0.001017			
20	22	1	2048.4	2.0	4096.8		-0.145988	-0.807617	469.9273		0.000982			
21	23	1	2101	2.0	4202		-0.147078	-0.785398	463.77		0.000948			
22	24	1	2153.6	2.0	4307.2		-0.147866	-0.764125	457.5343		0.000916			
23	25	1	2206.2	2.0	4412.4		-0.148378	-0.743756	451.2442		0.000885			
24	26	1	2258.8	2.0	4517.6		-0.148644	-0.72425	444.9214		0.000855			
25	27	1	2311.4	2.0	4622.8		-0.148686	-0.705568	438.5858		0.000826			
26	28	1	2364	2.0	4728		-0.148531	-0.687671	432.2551		0.000798			
27	29	1	2416.6	2.0	4833.2		-0.148198	-0.670522	425.9452		0.000772			
28	30	1	2469.2	2.0	4938.4		-0.147707	-0.654083	419.6701		0.000746			
29	31	1	2521.8	2.0	5043.6		-0.147078	-0.63832	413.4422		0.000722			
30	32	1	2574.4	2.0	5148.8		-0.146327	-0.623199	407.2722		0.000698			
31	33	1	2627	1.4	3677.8		-0.145469	-0.608689	401.1693		0.001304	ClayeySilt	Cc	0.382
32	34	1	2679.6	1.4	3751.44		-0.144518	-0.594759	395.1413		0.001262		Cr	0.0448
33	35	1	2732.2	1.4	3825.08		-0.143486	-0.58138	389.1951		0.001222		e	1.12
34	36	1	2784.8	1.4	3898.72		-0.142385	-0.568525	383.336		0.001184			
35	37	1	2837.4	1.4	3972.36		-0.141225	-0.556166	377.5688		0.001147			
36	38	1	2890	1.4	4046		-0.140014	-0.54428	371.8969		0.001111			
37	39	1	2942.6	1.4	4119.64		-0.138762	-0.532844	366.3234		0.001077			
38	40	1	2995.2	1.4	4193.28		-0.137476	-0.521834	360.8502		0.001044			
39	41	1	3047.8	1.4	4266.92		-0.136161	-0.511231	355.4791		0.001012			
40	42	1	3100.4	1.4	4340.56		-0.134825	-0.501013	350.2109		0.000979	Silty Clay	Cc	0.274
41	43	1	3153	1.4	4414.2		-0.133472	-0.491164	345.0461		0.00095		Cr	0.0419
42	44	1	3205.6	1.4	4487.84		-0.132106	-0.481664	339.985		0.000922		e	0.99
43	45	1	3258.2	1.4	4561.48		-0.130733	-0.472497	335.027		0.000895			
44	46	1	3310.8	1.4	4635.12		-0.129355	-0.463648	330.1718		0.000869			
45	47	1	3363.4	1.4	4708.76		-0.127976	-0.455101	325.4185		0.000845			
46	48	1	3416	1.4	4782.4		-0.126598	-0.446842	320.7659		0.000821			
47	49	1	3468.6	1.3	4509.18		-0.125224	-0.438859	316.2128		0.000451	Silt	Cc	0.2743
48	50	1	3521.2	1.3	4577.56		-0.123857	-0.431139	311.7578		0.000439		Cr	0.0249
49	51	1	3573.8	1.3	4645.94		-0.122498	-0.423669	307.3993		0.000427		e	1.09
50														
51													total settlement (in.)	
52														
53												0.03634	0.4	
54	Reference: Principles of Foundation Engineering Fourth Edition Braja M. Das													
55		Section 4.6 Stress Increase Under an Embankment pg 233												

Time Rate of Settlement:

Look at case of most fill: 4.3 feet of fill; 1.0 inches of settlement

Determine the time for 90% consolidation for primary settlement
Reference: FHWA Soils and Foundation Workshop Manual Second Edition page 179

At Station 4+52:

Thickness of the compressable layer = $H_{c452} := 70.7 \cdot ft$

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

$H_{cv452} := \frac{H_{c452}}{2}$ $H_{cv452} = 35.35\ ft$

Time factor from Table on page 179 $TF := 0.848$
At 90% primary consolidation

Coefficient of consolidation from lab data: $C_v := 1.6 \cdot 10^{-6} \cdot \frac{ft^2}{sec}$ $C_v = 0.1382 \cdot \frac{ft^2}{day}$

Time rate of settlement to achieve 90% Primary Settlement

$t_{90} := \frac{TF \cdot H_{cv452}^2}{C_v}$ $t_{90} = 7665.5084 \cdot day$ $year := 365 \cdot day$

$t_{90} = 21.0014 \cdot year$

At Station 6+00:

Thickness of the compressable layer = $H_{c600} := 38.1 \cdot ft$

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

$H_{cv600} := \frac{H_{c600}}{2}$ $H_{cv600} = 19.05\ ft$

Time factor from Table on page 179 $TF := 0.848$
At 90% primary consolidation

Coefficient of consolidation from lab data: $C_v := 1.7 \cdot 10^{-6} \cdot \frac{ft^2}{sec}$ $C_v = 0.1469 \cdot \frac{ft^2}{day}$

Time rate of settlement to achieve 90% Primary Settlement

$t_{90} := \frac{TF \cdot H_{cv600}^2}{C_v}$ $t_{90} = 2095.1887 \cdot day$ $year := 365 \cdot day$

$t_{90} = 5.7402 \cdot year$

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 $\gamma_t := 125 \cdot pcf$

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

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

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

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

Stress from overburden material. Overburden consists of a maximum of 4.3 feet of fill on 17 feet of existing fill material on 71 feet of silt and clay. Water table is at the bottom of the existing fill.

Additional Overburden Stress due to fill =

$\sigma_{v\_ob} := 4.3 \cdot ft \cdot \gamma_t \qquad \sigma_{v\_ob} = 537.5 \cdot psf$

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

At Station 4+53: Most consolidation settlement; thickest silt and clay

Total thickness of each stratum

$D_{efill} := 17.0 \cdot ft \qquad D_{siltandclay} := 71.0 \cdot ft$

$$\sigma'_{v\_fill} := \sigma_{v\_ob} + \frac{D_{efill}}{2} \cdot \gamma_t \qquad \sigma'_{v\_fill} = 1600 \cdot psf$$

$$\sigma'_{v\_siltandclay} := \sigma_{v\_ob} + D_{efill} \cdot \gamma_t + \frac{D_{siltandclay}}{2} \cdot \gamma'_{siltandclay} \qquad \sigma'_{v\_siltandclay} = 4529.8 \cdot psf$$

Pile parameters:

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

Pile depth:

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

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

Flange width:

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

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

Box perimeter:

$$P := 2 \cdot \left(d + B_f\right) \quad P = \begin{pmatrix} 47.65 \\ 56.39 \\ 57.05 \\ 58.19 \end{pmatrix} \cdot in$$

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

Magnitude of maximum downdrag, considered over entire clay thickness

$$Q_{dd} := \left(D_{siltandclay} \cdot \sigma'_{v\_siltandclay} \cdot \beta_{clay}\right) \cdot P \qquad Q_{dd} = \begin{pmatrix} 166.0208 \\ 196.4724 \\ 198.772 \\ 202.7439 \end{pmatrix} \cdot kip$$

If downdrag is considered over entire clay stratum, what is the factor of safety.
Ultimate capacity based on 50ksi steel and area of pile

Pile area:

$$A_{pile} := \begin{pmatrix} 15.5 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$$

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

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

$$FS := \frac{Q_{ult}}{Q_{app} + Q_{dd}} \qquad FS = \begin{pmatrix} 1.3692 \\ 1.7939 \\ 2.1795 \\ 2.8536 \end{pmatrix}$$

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

$$\sigma'_{v.cl.2\_3} := \sigma_{v\_ob} + D_{efill} \cdot \gamma' + \frac{D_{siltandclay} \cdot \frac{2}{3}}{2} \cdot \gamma'_{siltandclay} \qquad \sigma'_{v.cl.2\_3} = 2846.5667 \cdot psf$$

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

$$Q_{dd.2\_3} = \begin{pmatrix} 69.5526 \\ 82.31 \\ 83.2734 \\ 84.9374 \end{pmatrix} \cdot kip$$

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

$$FS := \frac{Q_{ult}}{Q_{app} + Q_{dd.2\_3}} \qquad FS = \begin{pmatrix} 1.6505 \\ 2.2185 \\ 2.7003 \\ 3.5468 \end{pmatrix}$$

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:
Auburn, Maine
DFI = 1400 degree-days

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

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

Frost_depth := 79.2in Frost_depth = 6.6 · ft

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Lewiston

ModBerg Results

Project Location: Lewiston, Maine

Air Design Freezing Index	=	1224 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	979 F-days
Mean Annual Temperature	=	46.4 deg F
Design Length of Freezing Season	=	118 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	63.4	10.0	125.0	28	34	2.0	1.6	1,800

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

****

Total Depth of Frost Penetration = 5.28 ft = 63.4 in.

****

Use Modberg Frost Depth = 5.3 feet for design

Seismic:

Auburn CNR Crossing Bridge PIN 15600.00
Date and Time: 9/9/2008 3:02:05 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04210
Zip Code Latitude = 44.097300
Zip Code Longitude = -070.240100
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.088	PGA - Site Class B
0.2	0.177	Ss - Site Class B
1.0	0.047	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04210
Zip Code Latitude = 44.097300
Zip Code Longitude = -070.240100
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 (sec)	Sa (g)	
0.0	0.221	As - Site Class E
0.2	0.442	SDs - Site Class E
1.0	0.163	SD1 - Site Class E