

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

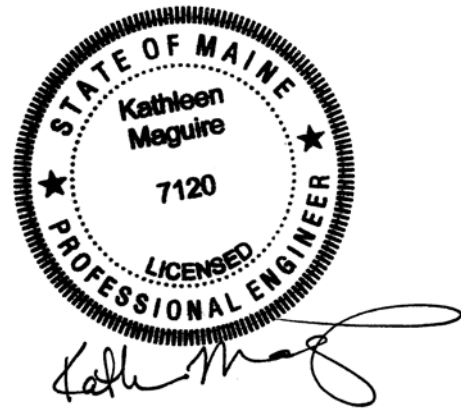
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**HUTCHINS BRIDGE
OVER GOFFS MILL BROOK
ARUNDEL, MAINE**

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of Hutchins Bridge which carries Log Cabin Road over Goffs Mill Brook in Arundel, Maine. Hutchins Bridge was built in 1947 and consists of a single span, buried, steel plate arch on concrete footings. The proposed replacement structure will be a single precast concrete box culvert with an 18 foot span and a 9 foot rise with 1 foot of streambed soil placed in the bottom. The proposed structure will have an overall length of approximately 108 feet on a 35 degree skew to the roadway alignment. The following design recommendations are discussed in detail in this Section 7 of this report.

Precast Concrete Box Culvert Design and Construction - The precast concrete box culvert shall be designed by the Manufacturer in accordance with Special Provision 534 and AASHTO LRFD Bridge Design Specifications 5th Edition 2010 (LRFD) specifications. The loading specified for the structure should be Modified HL-93 Strength 1 in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culverts shall be designed for all relevant strength and service limit states and load combinations. The box culverts shall be constructed with concrete inlet and outlet toe walls. The box culvert shall be bedded on a 2.0 foot thick layer of 3/4-inch crushed stone reinforced with geogrid and wrapped in geotextile fabric.

Precast Concrete Box Culvert Headwall Design - Concrete headwalls should be specified to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. A minimum 1 foot by 1 foot concrete headwall is recommended. Precast concrete box culvert headwalls that are any larger than the nominal 1 foot by 1 foot shall be designed for all relevant strength and service limit states and load combinations. The headwalls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culvert. The headwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}).

Bearing Resistance - The factored bearing resistance at the strength limit state for the box culvert on compacted fill shall not exceed 4 ksf, however, the service limit state will control. A factored bearing resistance of 3 ksf shall be used to control settlement when analyzing the service limit state. In no instance shall the bearing stress exceed the nominal resistance of the structural concrete which may be taken as $0.3f'_c$.

Settlement - Settlement along the new alignment at the location of the replacement structure is anticipated to be minimal. The installation of the proposed box culvert will result in a net unloading of the site soils at the structure location. Placement of fill soils at the location of the existing structure is not anticipated to exceed the past loading condition of the site soils. No settlement issues are anticipated at the site.

Scour and Riprap - The proposed box culvert will have a mitered inlet and a square ended outlet. The culvert should be fitted with integral concrete headwalls with nominal dimensions of 1 foot by 1 foot to prevent riprap slope protection from dropping or eroding into the waterway. Inlet and outlet cutoff walls to prevent undermining are required. Inlet and outlet cutoff walls shall extend

below the maximum depth of scour. The slopes shall be armored with a 3-foot thick layer of plain riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation. The riprap slopes should also be constructed in accordance with Special Provision 610 and be no steeper than 1.75H:1V.

Frost Protection - Any foundation placed on granular subgrade soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection.

Seismic Design Considerations - Seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

Construction Considerations - Construction of the proposed precast concrete box culverts will require soil excavation. Earth support systems will be required. The fill and native soils at the site will be susceptible to disturbance and rutting as a result of exposure to water and construction traffic. All subgrade surfaces should be protected from any unnecessary construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace disturbed areas with compacted gravel borrow. Any cobbles or boulder encountered in excess of 6 inches shall be removed and replaced with compacted gravel borrow.

The Contractor shall control groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of Hutchins Bridge which carries Log Cabin Road over Goffs Mill Brook, in Arundel, Maine. A subsurface investigation has been completed at the site. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site during the subsurface investigation, foundation design recommendations and geotechnical design parameters for the bridge replacement.

Hutchins Bridge was built in 1947 and consists of a 14 foot, single span, buried steel plate arch with a 7 foot rise on concrete footings. The structure has a natural stream bottom. The existing bridge has not undergone any rehabilitation other than wearing surface replacement and minor guardrail repair over the years. The 2010 Maine Department of Transportation (MaineDOT) Bridge Maintenance inspection report for the bridge indicates the structure has “excessive damage” (rating of 3) with heavy scaling at the top of the footings. The bottom of the steel arch at the spring line is in poor condition with holes, heavy scaling and loss of entire ribs. The bridge has a Bridge Sufficiency Rating of 48.7.

The MaineDOT Bridge Program is currently proposing a replacement structure consisting of a precast concrete box culvert with an 18 foot span and a 9 foot rise. In order to align the proposed culvert with the direction of stream flow the structure will be placed along a new alignment located approximately 54 feet west of the existing structure. The proposed structure will have a 35 degree skew to the roadway alignment and will be approximately 108 feet long. Staged construction with one lane of traffic controlled by flaggers will be utilized in the replacement of the structure.

2.0 GEOLOGIC SETTING

Hutchins Bridge is located on Log Cabin Road in Arundel, Maine and crosses Goffs Mill Brook approximately 1.8 miles east of US Route 1 as shown on Sheet 1 - Location Map presented 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 not completely covered by marine sediments. The unit generally is deposited in areas where the topography is gently sloping except where dissected by modern streams and commonly has a branching network of steep-walled stream gullies. These soils were generally deposited as glacial sediments that accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine.

According to the Bedrock Geologic Map of Maine, Maine Geologic Survey, 1985, and the Bedrock Geology of the Kittery Quadrangle, Maine and New Hampshire, Maine Geological Survey, Open-File No. 08-78 2008, the site is underlain by carboniferous, muscovite-biotite granite. The formation is the Biddeford Pluton.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by drilling seven (7) test borings. Test borings BB-AGMB-101 and BB-AGMB-102 were drilled on either side of the proposed new structure alignment west of the existing structure. Test borings BB-AGMB-103, BB-AGMB-103A, BB-AGMB-103B, BB-AGMB-104 and BB-AGMB-105 were drilled in the area of the existing structure in case the replacement structure is ultimately placed there. The borings were drilled between March 15 and April 21, 2010 using the MaineDOT drill rig. The boring locations are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. 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 solid stem auger and cased wash boring techniques. Soil samples were typically obtained 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 sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with a Central Mine Equipment (CME) automatic hammer to drive the split spoon. The hammer was calibrated in March of 2010 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. The hammer efficiency factor, 0.84, and both the raw field N-value and the corrected N-value are shown on the boring logs. Undisturbed tube samples were obtained in the soft soil deposits where possible. In-situ vane shear tests were made where possible in the soft soil deposits to measure the shear strength of the strata. The bedrock was cored in borings BB-AGMB-101, BB-AGMB-102, BB-AGMB-103, BB-AGMB-104 and BB-AGMB-105 using an NQ-2 inch core barrel and the Rock Quality Designation (RQD) of the core was calculated for the NQ cores.

The MaineDOT Geotechnical Team member selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed field logs for accuracy and identified field and laboratory testing requirements. The MaineDOT Geotechnical Team Member or a New England Transportation Technical Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The borings were located in the field by taping to site features after completion of the drilling program.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of eight (8) standard grain size analyses with natural water content, eleven (11) grain size analyses with hydrometer and natural water content, four (4) Atterberg Limits tests, one (1) 1-D consolidation and one (1) standard tube opening with laboratory vanes. The results of soil laboratory tests are included as Appendix B - Laboratory Test Results at the end of this report. Laboratory test information is also shown on the boring logs provided in Appendix A – Boring Logs and on Sheets 3 and 4 – Boring Logs at the end of this report.

5.0 SUBSURFACE CONDITIONS

A total of seven (7) borings were conducted at the site in order to investigate two possible locations for the replacement structure. The Bridge Program has chosen to locate the bridge along a new alignment west of the existing bridge. Therefore, the subsurface conditions discussed in this section will only utilize the borings conducted at that new location (BB-AGMB-101 and BB-AGMB-102) as these are the most relevant borings to the replacement. Boring logs for these borings and the additional borings conducted at the existing structure are provided in Appendix A – Boring Logs at the end of this report. In the event that the bridge location changes during final design, the additional borings drilled will be reviewed and an Addendum to this report will be published.

Subsurface conditions encountered at the new alignment test borings generally consisted of sand fill overlying sand, silt and clay overlying sand and gravel all underlain by bedrock. An interpretive subsurface profile depicting the generalized soil stratigraphy across the site is shown on Sheet 2 – Boring Location Plane and Interpretive Subsurface Profile found at the end of this report. A brief summary description of the strata encountered follows:

5.1 Fill

A layer of granular fill was encountered both borings. The encountered layer ranged from approximately 3.5 feet to 4.5 feet thick. The deposit generally consisted of brown, moist, fine to coarse sand, some gravel, and little silt, and brown, moist, gravelly fine to coarse sand, little silt and occasional cobbles. Two corrected SPT N-values in the fill unit were 14 and 27 blows per foot (bpf), indicating a soil that is medium dense in consistency.

Grain size analyses were conducted on the two (2) samples from the fill unit. Grain size analyses resulted in the soil being classified as an A-1-b under the AASHTO Soil Classification System and an SM under the Unified Soil Classification System. Measured natural water contents of samples tested ranged from approximately 6 to 7 percent.

5.2 Interbedded Sand, Silt and Clay

Layers of interbedded silty sand, silty clay, clayey silt and sandy silt were encountered below the fill unit. The thickness of the layer ranged from approximately 13.3 to 25.0 feet thick. The layer consisted of:

- Brown, wet, fine to medium sandy silt, trace clay;
- Brown, wet, silty, fine to medium sand, trace coarse sand, little clay, trace gravel;
- Dark brown, moist, sandy silt, trace clay, trace gravel, trace organics;
- Dark brown, wet, silty sand, trace clay, trace gravel, little organics, and wood.
- Grey, wet, silty clay, trace fine sand with black staining,
- Grey, wet, clayey silt, trace fine sand with black staining, and
- Grey, wet, silt, little clay, little fine to medium sand.

Corrected SPT N-values of the silty sand samples ranged from weight of hammer to 7 bpf indicating that the silty sand is very loose to loose in consistency. Vane shear testing conducted in the clay and silt samples showed measured undrained shear strengths ranging from approximately 402 to 580 psf while the remolded shear strength ranged from approximately 9 to 134 psf indicating that the clay and silt are soft to medium stiff in consistency. Based on the ratio of peak to remolded shear strengths from the vane shear tests, the clay and silt was determined to have sensitivity ranging from approximately 4.3 to 65.5 and is classified as sensitive to extra quick. Water contents from samples obtained within the layer range from approximately 25% to 66%. Grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-4, A-7-6 or A-6 by the AASHTO Classification System and a ML, CL, or CL-ML by the Unified Soil Classification System.

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

Sample No.	Soil Type	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index
BB-AGMB-101 4D	Silty Clay	46.2	43	23	20	1.16
BB-AGMB-101 1U	Clayey Silt	43.8	34	21	13	1.75
BB-AGMB-101 5D	Silt	43.4	38	20	18	1.30

Table 1 – Atterberg Limits Test Results

Interpretation of these results indicates that the upper portion of the layer is normally consolidated while the lower layers are on the verge of being a viscous liquid if disturbed. For all of these samples 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”.

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

Within this unit, an isolated boulder was encountered in boring BB-AGMB-102 at a depth of approximately 14.2 feet below ground surface (bgs).

5.3 Glacial Till

Glacial till was encountered underlying the interbedded silty sand, silty clay, clayey silt and sandy silt in both borings. The encountered thickness of the deposit was approximately 3.9 to 6.4 feet at the boring locations. The glacial till generally consisted of brown, wet, gravel, some silt, little clay, trace sand and gravelly fine to coarse sand, some silt. Two corrected SPT N-values in the glacial till ranged from 43 to greater than 50 bpf indicating that the deposit is dense to very dense in consistency.

A grain size analysis was conducted on one (1) sample from the glacial till unit. Grain size analysis resulted in the soil being classified as an A-4 under the AASHTO Soil Classification System and GC-GM under the Unified Soil Classification System. The measured natural water content of the sample tested was approximately 11 percent.

5.4 Bedrock

The bedrock at the site is identified as:

- Greenish, fine grained, meta-sandstone, changing to dark grey, meta-siltstone, with two joint sets at 35 to 60 degrees and 0 to 15 degrees;
- Dark grey, very fine grained, meta-siltstone, with small calcite crystals and iron staining; and
- Purplish to greenish, meta-siltstone, with quartzite and calcite, open joints dipping at 25 to 60 degrees.

The RQD of the bedrock was determined to range from 20 to 60 percent, correlating to a rock mass quality of very poor to fair.

Table 2 summarizes approximate top of bedrock elevations at the exploration locations.

Boring	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock Surface (feet)
BB-AGMB-101	32.4	22.9
BB-AGMB-102	24.2	31.2

Table 2 - Approximate Depth to Bedrock and Elevation of Bedrock Surface at Exploration Locations

5.5 Groundwater

The measures groundwater level in borings ranged from 9.5 feet bgs in boring BB-AGMB-102 to 14.0 feet bgs in boring BB-AGMB-101. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Groundwater levels will fluctuate with seasonal changes, runoff and adjacent construction activities.

6.0 FOUNDATION ALTERNATIVES

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

- Replacement with a precast concrete box culvert at the existing location and
- Replacement with a precast concrete box culvert at a new location approximately 54 feet west of the existing structure.

The Preliminary Design Report (PDR) prepared by the Bridge Program recommends placing the replacement structure at a new location approximately 54 feet west of the existing structure in order to improve hydraulic conditions and better align the channel flow at the site and to accommodate staged construction. This location will also reduce localized flooding.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for design of an 18 foot wide by 9 foot high precast concrete box culvert replacement structure. A natural stream bottom will be specified for the culvert.

7.1 Precast Concrete Box Culvert Design and Construction

Precast concrete box culverts are typically detailed on the contract plans with only the basic layout and require hydraulic opening so that the Contractor may choose the appropriate structure. The manufacturer is responsible for the design of the structure including determination of the wall thickness, haunch thickness and reinforcement in accordance with Special Provision 534 Precast Concrete Arches, Box Culverts, which is included in Appendix D of this report. The loading specified for the structure should be Modified HL-93 Strength 1 in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The designer should use Soil Type 4 as presented in the Maine DOT Bridge Design Guide (BDG) Section 3.6 to design earth loads from the soil envelope. The backfill properties are as follows: $\phi = 32$ degrees, $\gamma = 125$ pcf.

The precast concrete box shall include accommodations for toe walls at both the inlet and outlet ends to prevent undermining per MaineDOT BDG Section 8.3.1. The cutoff walls should extend below the maximum depth of scour.

The precast concrete box culverts shall be supplier-designed in accordance with AASHTO LRFD Bridge Design Specifications 5th Edition 2010 (LRFD) specifications. The precast concrete box culverts shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The precast concrete box culverts shall be constructed in conformance with MaineDOT BDG Section 8 and Special Provision 534.

The box culvert shall be bedded on a 2.0 foot thick layer of 3/4-inch crushed stone reinforced with geogrid and wrapped in geotextile fabric. The soil envelop and backfill shall consist of Standard Specification 703.19 - Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The crushed stone bedding should be placed in 12 inch maximum thick lifts and compacted with a minimum of four passes of a large walk behind compactor. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Precast Concrete Box Culvert Headwall Design

Concrete headwalls should be included in the culvert design to retain riprap slopes and prevent riprap from dropping or eroding into the waterway. A nominal 1 foot by 1 foot concrete headwall is recommended.

Larger precast or cast-in-place concrete box culvert headwalls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The head walls shall be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culvert. The headwalls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) taken from Table 3 below:

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

Table 3 – Equivalent Height of Soil for Vehicular Loading on Retaining Walls

Culvert headwall sections that are fixed to the box culverts to resist movement should be designed using an at-rest earth pressure coefficient, K_o , of 0.5. Headwall sections that are independent of

the box culvert should be designed using the Rankine active earth pressure coefficient, K_a , of 0.31 assuming a level back slope. The active earth pressure coefficient may change if the back slope conditions are different. See Appendix C - Calculations for supporting documentation.

Footings for any headwall or wingwall constructed independently of the box culvert should be placed no less than 2.0 feet below the maximum anticipated scour depth.

7.3 Bearing Resistance

The factored bearing resistance for at the strength limit state for the box culvert on compacted fill shall not exceed 4 ksf, however, the service limit state bearing resistance will govern. A factored bearing resistance of 3 ksf shall be used to control settlement when analyzing the service limit state as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the structural concrete which may be taken as $0.3f'_c$. See Appendix C - Calculations for supporting documentation.

7.4 Settlement

Settlement along the new alignment at the location of the replacement structure is anticipated to be minimal. The installation of the proposed box culvert will result in a net unloading of the site soils at the structure location. Placement of fill soils at the location of the existing structure is not anticipated to exceed the past loading condition of the site soils. No settlement issues are anticipated at the site.

7.5 Scour and Riprap

The proposed box culvert will have a mitered inlet and a square ended outlet. The culvert should be fitted with integral concrete headwalls with nominal dimensions of 1 foot by 1 foot to prevent riprap slope protection from dropping or eroding into the waterway. Inlet and outlet cutoff walls to prevent undermining are required per BDG Section 8.3.1. Inlet and outlet cutoff walls shall extend below the maximum depth of scour. The slopes shall be armored with a 3-foot thick layer of plain riprap conforming to MaineDOT 703.26 Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1 foot below the streambed elevation. The riprap slopes should also be constructed in accordance with Special Provision 610 – Stone Fill, Riprap, Stone Blanket and Stone Ditch Protection.

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 1195 F-degree days. In a granular soil with a water content of approximately 10%,

this correlates to a frost depth of approximately 5.0 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 5.0 feet below finished exterior grade for frost protection. See Appendix C- Calculations at the end of this report for supporting documentation.

7.7 Seismic Design Considerations

In conformance with LRFD Article 3.10.1, seismic analysis is not required for buried structures, except where they cross active faults. There are no known active faults in Maine; therefore seismic analysis is not required.

7.8 Construction Considerations

Construction of the proposed precast concrete box culverts will require soil excavation. Earth support systems will be required. The fill and native soils at the site will be susceptible to disturbance and rutting as a result of exposure to water and construction traffic. All subgrade surfaces should be protected from any unnecessary construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace disturbed areas with compacted gravel borrow. Any cobbles or boulder encountered in excess of 6 inches shall be removed and replaced with compacted gravel borrow.

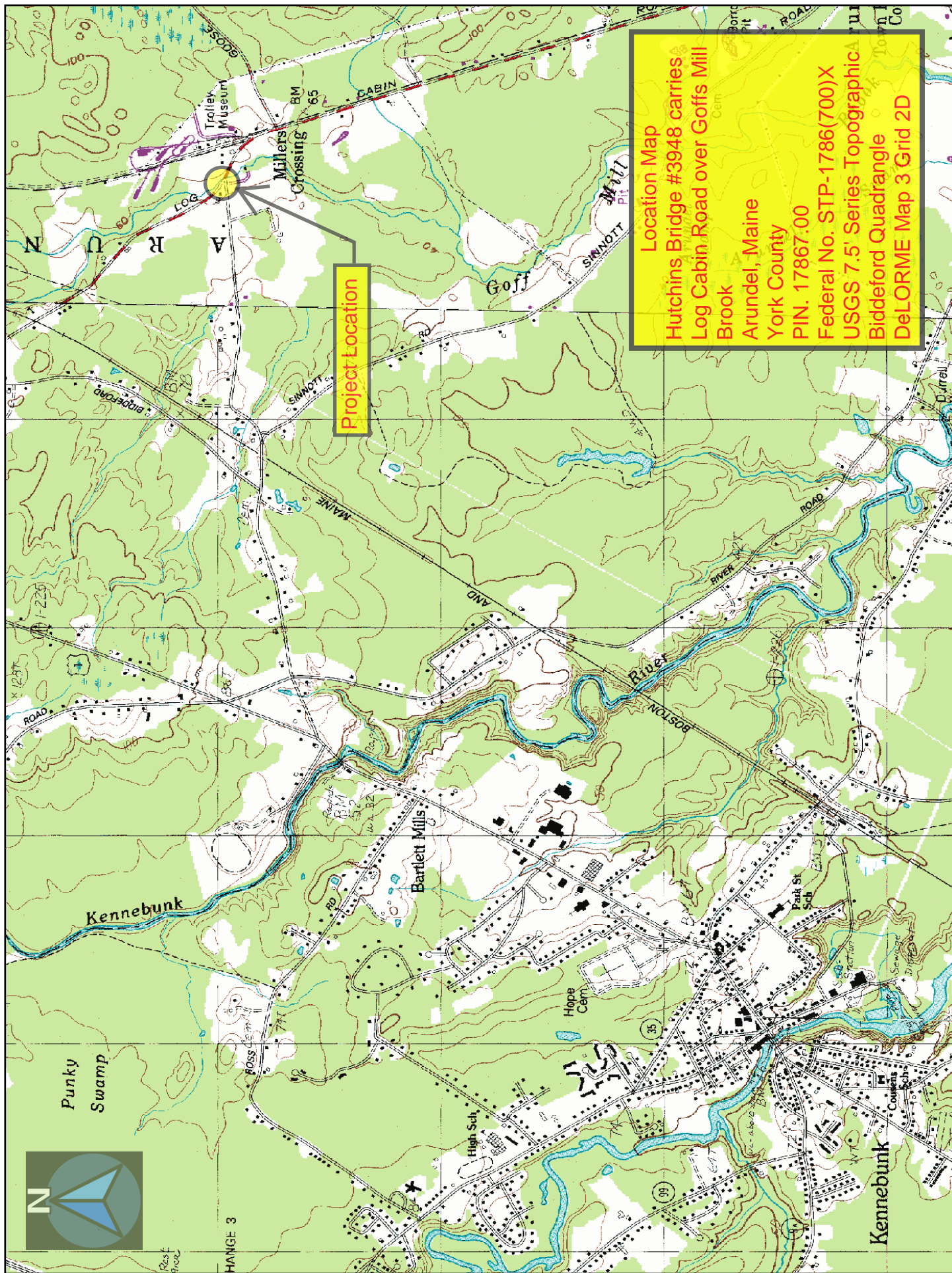
The Contractor shall control groundwater and surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

8.0 CLOSURE

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

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

Sheets



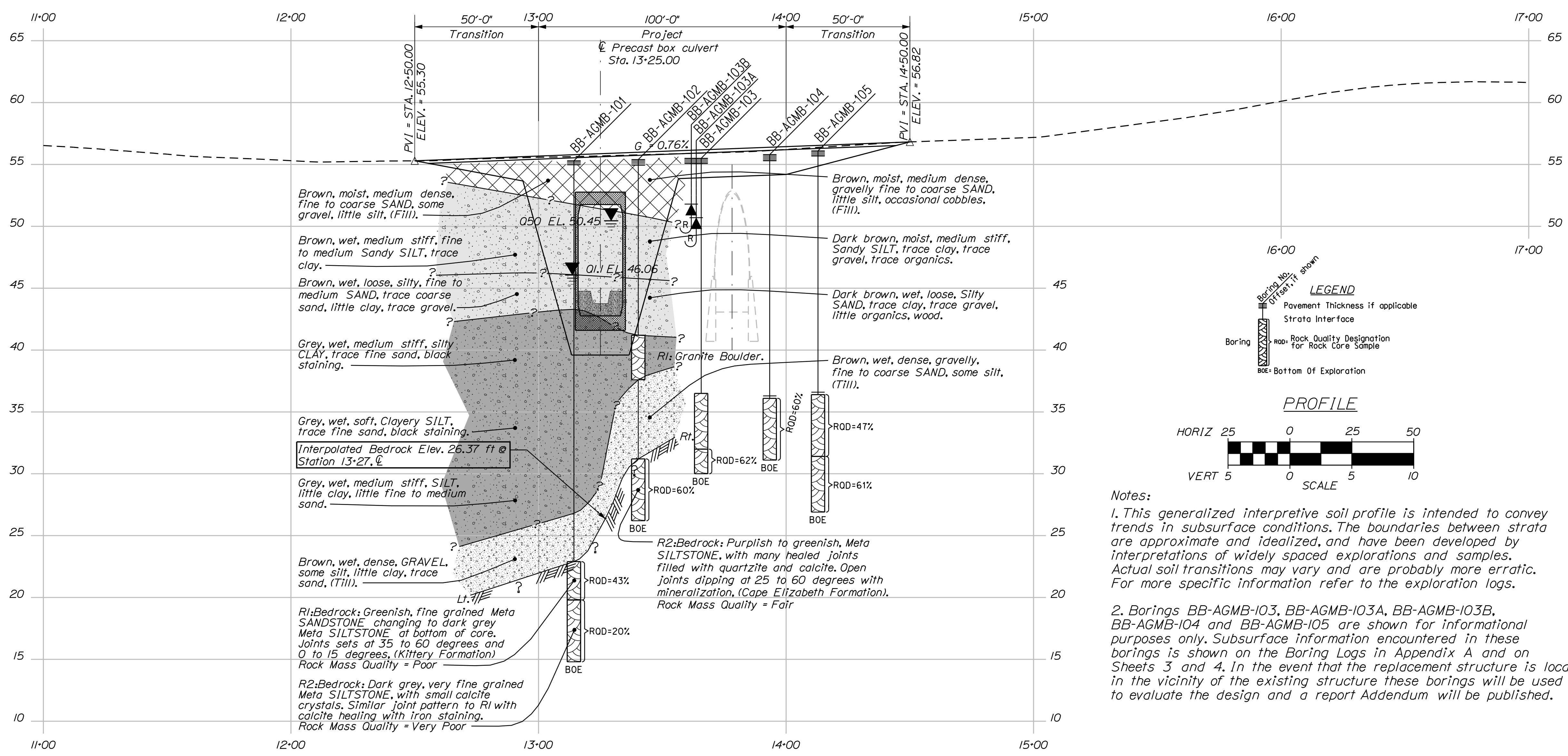
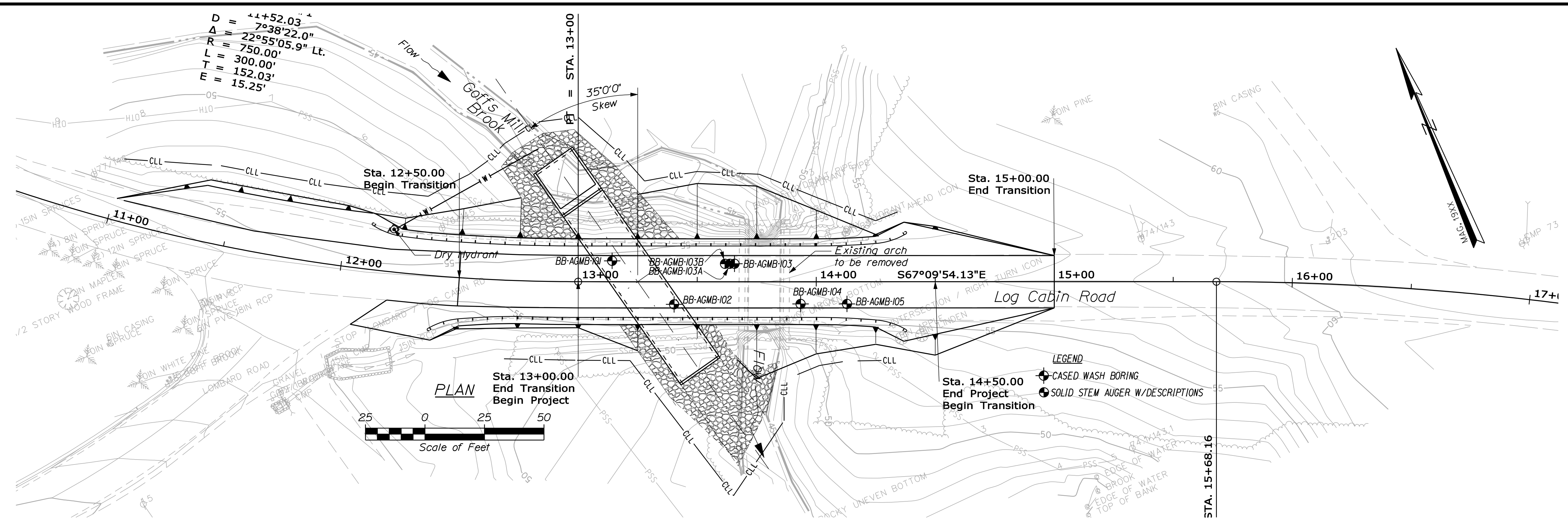
Map Scale 1:24000

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STATE OF MAINE		DEPARTMENT OF TRANSPORTATION	
HUTCHINS BRIDGE		STP-1786(700)X	
GOFFS MILL BROOK		BRIDGE NO. 3948	
YORK COUNTY		PIN 17867.00	
ARUNDEL		BRIDGE PLANS	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE		SHEET NUMBER	
		2	
		OF 4	

PROJ. MANAGER	DATE	BY	DATE
K. MAGUIRE	MAR 2011	T. WHITE	
CHECKED-REVIEWED			
DESIGN DETAILED			
DESIGN REVIEWED			
DESIGN DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM				TERMS DESCRIBING DENSITY/CONSISTENCY																													
MAJOR DIVISIONS		GROUP SYMBOLS		TYPICAL NAMES																													
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	<p>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.</p> <p style="text-align: center;">Modified Burmister System</p> <table border="0"> <tr> <td style="text-align: center;"><u>Descriptive Term</u></td> <td style="text-align: center;"><u>Portion of Total</u></td> </tr> <tr> <td style="text-align: center;">trace</td> <td style="text-align: center;">0% - 10%</td> </tr> <tr> <td style="text-align: center;">little</td> <td style="text-align: center;">11% - 20%</td> </tr> <tr> <td style="text-align: center;">some</td> <td style="text-align: center;">21% - 35%</td> </tr> <tr> <td style="text-align: center;">adjective (e.g. sandy, clayey)</td> <td style="text-align: center;">36% - 50%</td> </tr> </table> <table border="0"> <tr> <td style="text-align: center;"><u>Density of Cohesionless Soils</u></td> <td style="text-align: center;"><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <tr> <td style="text-align: center;">Very loose</td> <td style="text-align: center;">0 - 4</td> </tr> <tr> <td style="text-align: center;">Loose</td> <td style="text-align: center;">5 - 10</td> </tr> <tr> <td style="text-align: center;">Medium Dense</td> <td style="text-align: center;">11 - 30</td> </tr> <tr> <td style="text-align: center;">Dense</td> <td style="text-align: center;">31 - 50</td> </tr> <tr> <td style="text-align: center;">Very Dense</td> <td style="text-align: center;">> 50</td> </tr> </table>	<u>Descriptive Term</u>	<u>Portion of Total</u>	trace	0% - 10%	little	11% - 20%	some	21% - 35%	adjective (e.g. sandy, clayey)	36% - 50%	<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50						
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Dense	31 - 50																																
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		GP	Poorly-graded gravels, gravel sand mixtures, little or no 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	SW	Well-graded sands, gravelly sands, little or no fines																													
			SP	Poorly-graded sands, gravelly sand, little or no fines.																													
			SM	Silty sands, sand-silt 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.	<p>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.</p> <table border="0"> <tr> <td style="text-align: center;"><u>Consistency of Cohesive soils</u></td> <td style="text-align: center;"><u>SPT N-Value blows per foot</u></td> <td style="text-align: center;"><u>Approximate Undrained Shear Strength (psf)</u></td> <td style="text-align: center;"><u>Field Guidelines</u></td> </tr> <tr> <td style="text-align: center;">Very Soft</td> <td style="text-align: center;">WOH, WOR, WOP, <2</td> <td style="text-align: center;">0 - 250</td> <td style="text-align: center;">Fist easily Penetrates</td> </tr> <tr> <td style="text-align: center;">Soft</td> <td style="text-align: center;">2 - 4</td> <td style="text-align: center;">250 - 500</td> <td style="text-align: center;">Thumb easily penetrates</td> </tr> <tr> <td style="text-align: center;">Medium Stiff</td> <td style="text-align: center;">5 - 8</td> <td style="text-align: center;">500 - 1000</td> <td style="text-align: center;">Thumb penetrates with moderate effort</td> </tr> <tr> <td style="text-align: center;">Stiff</td> <td style="text-align: center;">9 - 15</td> <td style="text-align: center;">1000 - 2000</td> <td style="text-align: center;">Indented by thumb with great effort</td> </tr> <tr> <td style="text-align: center;">Very Stiff</td> <td style="text-align: center;">16 - 30</td> <td style="text-align: center;">2000 - 4000</td> <td style="text-align: center;">Indented by thumb nail</td> </tr> <tr> <td style="text-align: center;">Hard</td> <td style="text-align: center;">>30</td> <td style="text-align: center;">over 4000</td> <td style="text-align: center;">Indented by thumb nail with difficulty</td> </tr> </table>	<u>Consistency of Cohesive soils</u>	<u>SPT N-Value blows per foot</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>	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 thumb nail	Hard	>30	over 4000	Indented by thumb nail with difficulty
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Hard	>30	over 4000	Indented by thumb nail with difficulty																														
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.																														
		OL	Organic silts and organic silty clays of low plasticity.																														
	SILTS AND CLAYS (liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.																														
			CH	Inorganic clays of high plasticity, fat clays.																													
			OH	Organic clays of medium to high plasticity, organic silts																													
	HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.																														
<p>Desired Soil Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Moisture (dry, damp, moist, wet, saturated)</p> <p>Density/Consistency (from above right hand side)</p> <p>Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)</p> <p>Gradation (well-graded, poorly-graded, uniform, etc.)</p> <p>Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)</p> <p>Structure (layering, fractures, cracks, etc.)</p> <p>Bonding (well, moderately, loosely, etc., if applicable)</p> <p>Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)</p> <p>Geologic Origin (till, marine clay, alluvium, etc.)</p> <p>Unified Soil Classification Designation</p> <p>Groundwater level</p>				<p>Rock Quality Designation (RQD):</p> <p>RQD = $\frac{\text{sum of the lengths of intact pieces of core}^* > 100 \text{ mm}}{\text{length of core advance}}$</p> <p style="text-align: center;">*Minimum NQ rock core (1.88 in. OD of core)</p> <p style="text-align: center;">Correlation of RQD to Rock Mass Quality</p> <table border="0"> <tr> <td style="text-align: center;"><u>Rock Mass Quality</u></td> <td style="text-align: center;"><u>RQD</u></td> </tr> <tr> <td style="text-align: center;">Very Poor</td> <td style="text-align: center;"><25%</td> </tr> <tr> <td style="text-align: center;">Poor</td> <td style="text-align: center;">26% - 50%</td> </tr> <tr> <td style="text-align: center;">Fair</td> <td style="text-align: center;">51% - 75%</td> </tr> <tr> <td style="text-align: center;">Good</td> <td style="text-align: center;">76% - 90%</td> </tr> <tr> <td style="text-align: center;">Excellent</td> <td style="text-align: center;">91% - 100%</td> </tr> </table> <p>Desired Rock Observations: (in this order)</p> <p>Color (Munsell color chart)</p> <p>Texture (aphanitic, fine-grained, etc.)</p> <p>Lithology (igneous, sedimentary, metamorphic, etc.)</p> <p>Hardness (very hard, hard, mod. hard, etc.)</p> <p>Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)</p> <p>Geologic discontinuities/jointing:</p> <ul style="list-style-type: none"> -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.) <p>Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)</p> <p>RQD and correlation to rock mass quality (very poor, poor, etc.)</p> <p>ref: AASHTO Standard Specification for Highway Bridges</p> <p>17th Ed. Table 4.4.8.1.2A</p> <p>Recovery</p>		<u>Rock Mass Quality</u>	<u>RQD</u>	Very Poor	<25%	Poor	26% - 50%	Fair	51% - 75%	Good	76% - 90%	Excellent	91% - 100%																
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Driller: MaineDOT	Elevation (ft.): 55.4	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 130#/30"
Date Start/Finish: 4/21/11; 08:00-14:30	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 13+40.3, 9.5 Rt.	Casing ID/OD: HW & NW	Water Level*: 9.5 ft bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
 MD = Unsuccessful Split Spoon Sample attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf) LL = Liquid Limit
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw field SPT N-value PL = Plastic Limit
 MU = Unsuccessful Thin Wall Tube Sample attempt WOH = weight of 140lb. hammer Hammer Efficiency Factor = Annual Calibration Value PI = Plasticity Index
 V = Insitu Vane Shear Test, PP = Pocket Penetrometer N₆₀ = SPT N-uncorrected corrected for hammer efficiency G = Grain Size Analysis
 MV = Unsuccessful Insitu Vane Shear Test attempt WO1P = Weight of one person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Sample Information										Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)					
25							NQ-2			26.20	Top of Bedrock at Elev. 31.2 ft. R2: Bedrock: Purplish to greenish, Meta SILTSTONE, with many healed joints filled with quartzite and calcite. Open joints dipping at 25 to 60 degrees with mineralization. (Cape Elizabeth Formation.) Rock Mass Quality = Fair. R2: Core Times (min:sec) 24.2-25.2 ft (4:00) 25.2-26.2 ft (4:35) 26.2-27.2 ft (5:10) 27.2-28.2 ft (4:00) 28.2-29.2 ft (4:00) 97% Recovery		
							↙				29.20	Bottom of Exploration at 29.20 feet below ground surface.	
30													
35													
40													
45													
50													

Remarks:
Rig broke down and bent casing, no second bedrock core run.

Driller: MaineDOT	Elevation (ft.): 55.5	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 130#/30"
Date Start/Finish: 3/22/11; 12:30-16:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 13+65.7, 7.4 Lt.	Casing ID/OD: NW	Water Level*: 12.0 ft bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25									30.00		fieldspar, traces of amphibole and garnet. Iron staining at top of core. With two joint sets: one at 60 to 75 degrees and one at 0 to 5 degrees. Rock Mass Quality = Fair. R2: Core Times (min:sec) 23.5-24.5 ft (2:45) 24.5-25.5 ft (3:30) 71% Recovery Core Blocked, burned bit up.	
30												
35												
40												
45												
50												

Remarks:

Bottom of Exploration at 25.50 feet below ground surface.

Driller: MaineDOT	Elevation (ft.): 55.5	Auger ID/OD: 5" Dia.
Operator: Giguere/Giles	Datum: NAVD88	Sampler: N/A
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: N/A
Date Start/Finish: 3/22/11-3/22/11	Drilling Method: Solid Stem Auger	Core Barrel: N/A
Boring Location: 13+63.7, 7.4 Lt.	Casing ID/OD: N/A	Water Level*: None Observed

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

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0								SSA	55.08		5" Pavement	
											Brown, moist, medium dense, fine to coarse SAND, some gravel, trace silt.	
5									50.70		Bottom of Exploration at 4.80 feet below ground surface. BOULDER REFUSAL	
10												
15												
20												
25												

Remarks:

Driller: MaineDOT	Elevation (ft.): 55.8	Auger ID/OD: 5" Solid Stem
Operator: Giguere/Giles	Datum: NAVD88	Sampler: Standard Split Spoon
Logged By: B. Wilder	Rig Type: CME 45C	Hammer Wt./Fall: 130#/30"
Date Start/Finish: 3/21/11; 09:30-14:00	Drilling Method: Cased Wash Boring	Core Barrel: NQ-2"
Boring Location: 13+93.4, 9.3 Rt.	Casing ID/OD: NW	Water Level*: 11.0 ft bgs.

Hammer Efficiency Factor: 0.84 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Insitu Field Vane Shear Strength (psf) S_{u(lab)} = Lab Vane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger T_v = Pocket Torvane Shear Strength (psf) WC = water content, percent
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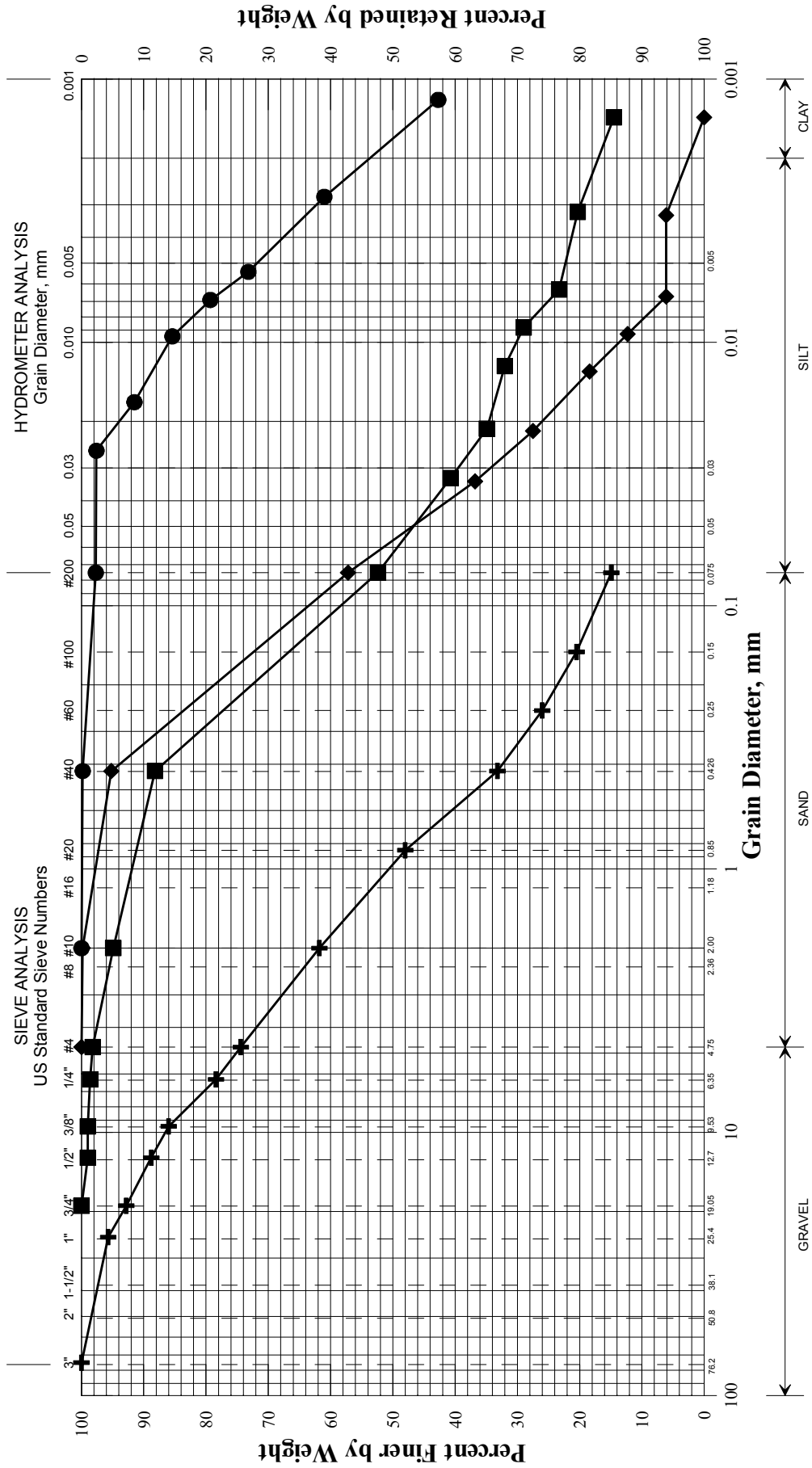
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0									55.30	6" Pavement		
										Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).		
									52.80			
5	1D	24/15	5.00 - 7.00	5/11/5/4	16	22				Brown, wet, medium dense, fine to coarse SAND, some silt, some gravel.	G#236400 A-2-4, SM WC=14.3%	
									48.30			
10	2D	24/14	10.00 - 12.00	4/5/25/20	30	42	5			Grey, wet, dense, fine to coarse SAND, some silt, little gravel.	G#262419 A-2-4, SM WC=30.5%	
									41.80			
15	3D	24/14	15.00 - 17.00	15/8/11/11	19	27	41			Brown, wet, medium dense, fine to coarse SAND, some gravel, some silt, (Till).	G#262420 A-1-b, SM WC=13.8%	
20	R1	60/60	19.70 - 24.70	RQD = 60%					36.30	a105 blows for 0.5 ft.		
									36.10	Top of Bedrock at Elev. 36.3 ft. Roller Coned ahead to 19.7 ft bgs.		
										R1:Bedrock: Greenish grey, medium grained GRANITE with quartz, feldspar, traces of amphibole and garnet. Iron staining at top of core. With two joint sets: one at 60 to 75 degrees and one at 0 to 5 degrees. Rock Mass Quality = Fair. R1:Core Times (min:sec) 19.7-20.7 ft (2:05) 20.7-21.7 ft (2:15) 21.7-22.7 ft (3:10)		
25									31.10			

Remarks:

Appendix B

Laboratory Test Results

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

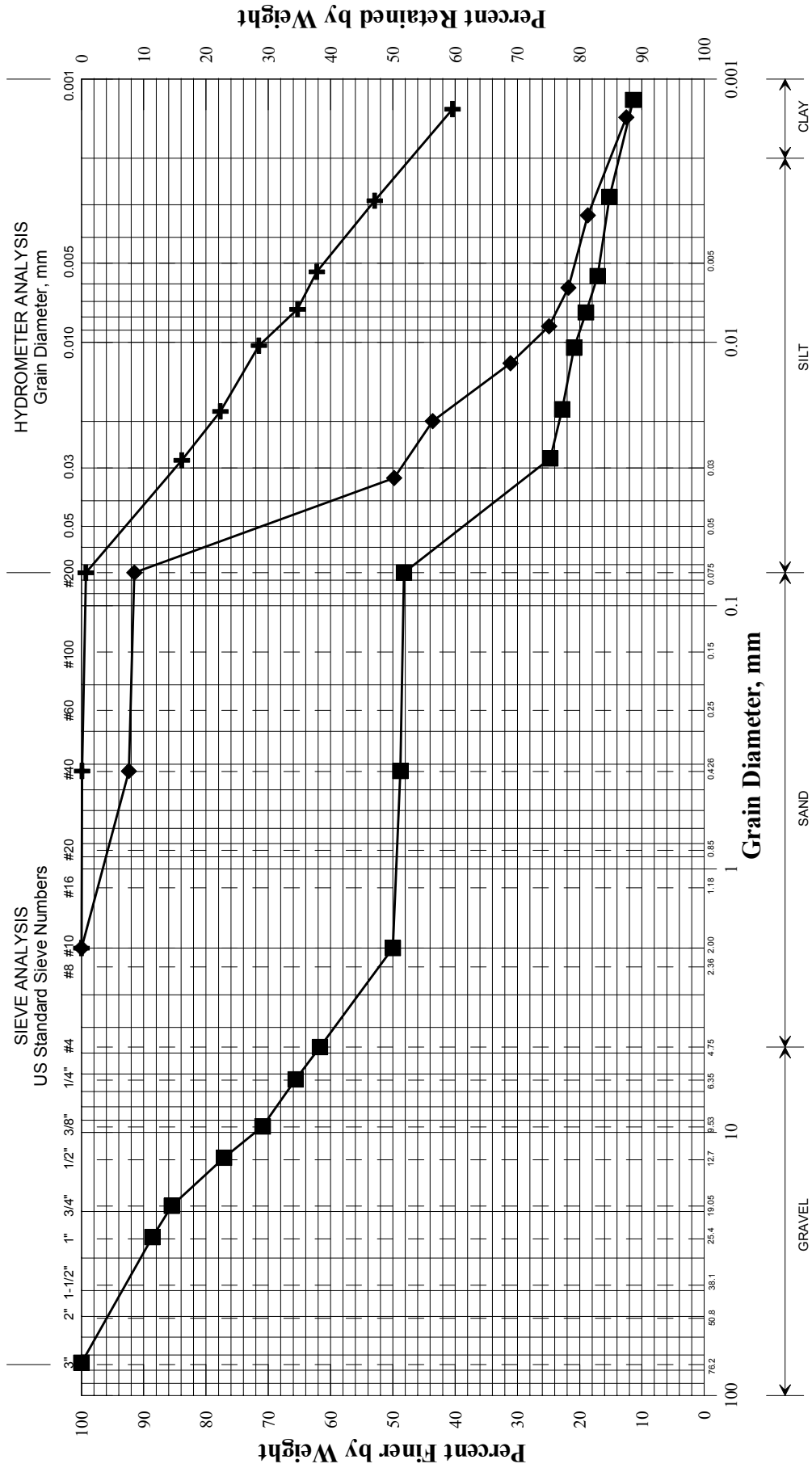


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AGMB-101/1D	8.8 LT	1.0-3.0	SAND, some gravel, little silt.	7.0			
◆	BB-AGMB-101/2D	8.8 LT	5.0-7.0	Sandy SILT, trace clay.	36.4			
■	BB-AGMB-101/3D	8.8 LT	10.0-12.0	Silty SAND, little clay, trace gravel.	27.1	43	23	20
●	BB-AGMB-101/4D	8.8 LT	15.0-17.0	Silty CLAY, trace sand.	46.2	43	23	20
×								

PIN	017867.00
Town	Arundel
Reported by/Date	WHITE, TERRY A 5/20/2011

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

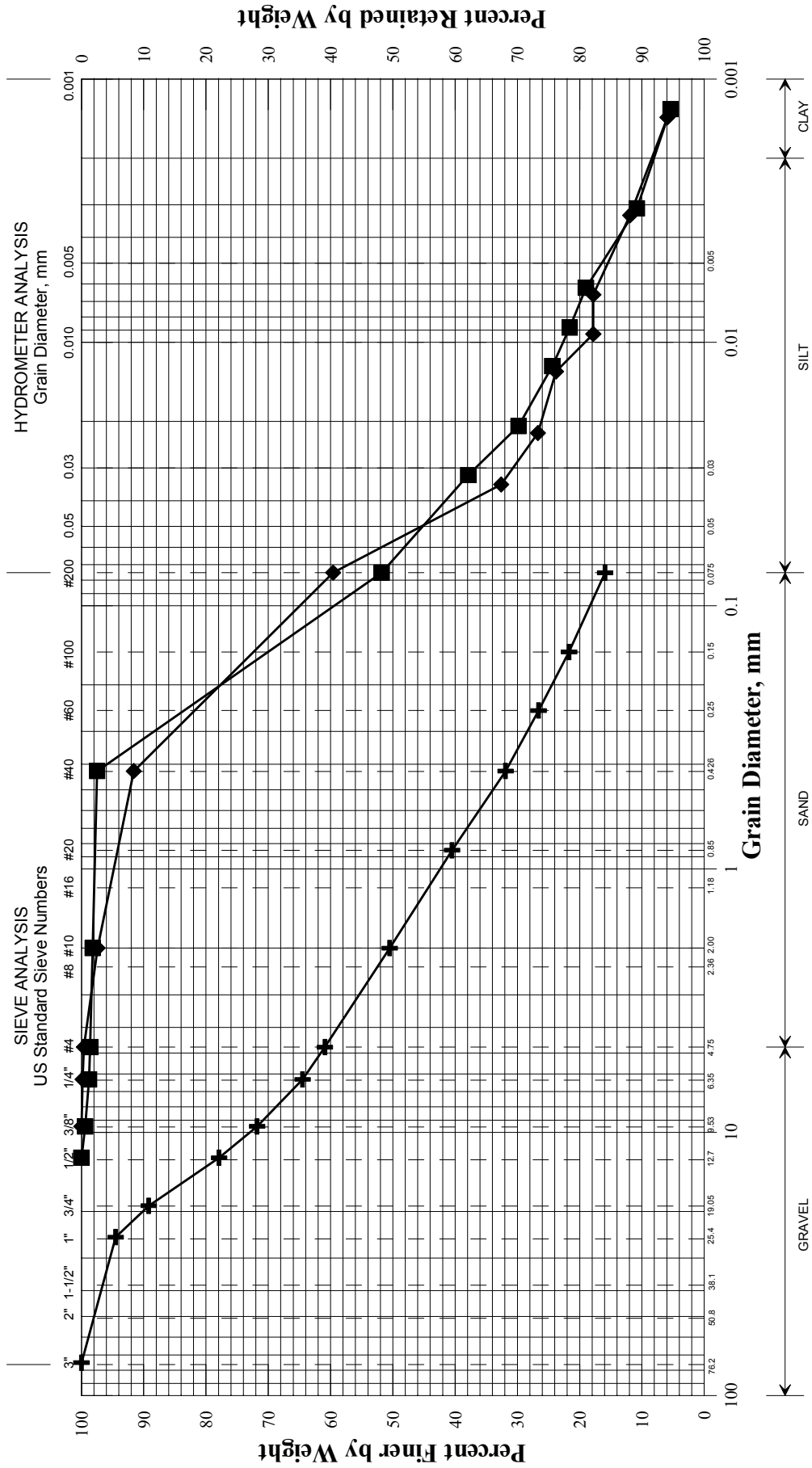


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AGMB-101/1U	13+14.3	8.8 LT	20.0-22.0	43.8	34	21	13
◆	BB-AGMB-101/5D	13+14.3	8.8 LT	25.0-27.0	43.4	38	20	18
■	BB-AGMB-101/6D	13+14.3	8.8 LT	30.0-31.4	10.6			
●								
▲								
×								

017867.00	PIN
Arundel	Town
WHITE, TERRY A	Reported by/Date
5/20/2011	

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

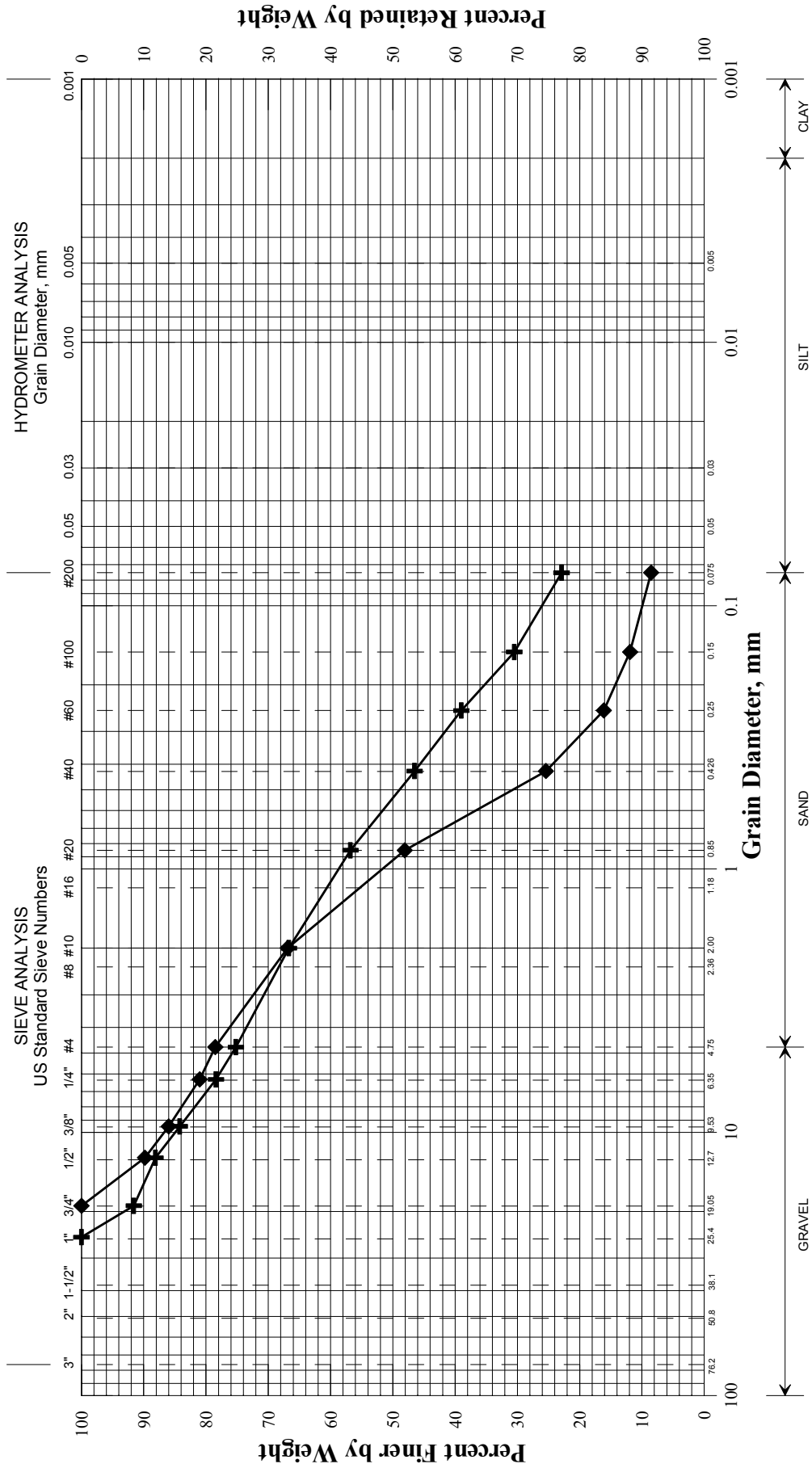


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AGMB-102/1D	13+40.3	9.5 RT	1.5-3.5	Gravelly SAND, little silt.	6.0		
◆	BB-AGMB-102/2D	13+40.3	9.5 RT	5.0-7.0	Sandy SILT, trace clay, trace gravel.	25.4		
■	BB-AGMB-102/3D	13+40.3	9.5 RT	10.0-12.0	Silty SAND, trace clay, trace gravel.	65.5		
●								
▲								
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PIN	017867.00
Town	Arundel
Reported by/Date	WHITE, TERRY A 6/6/2011

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

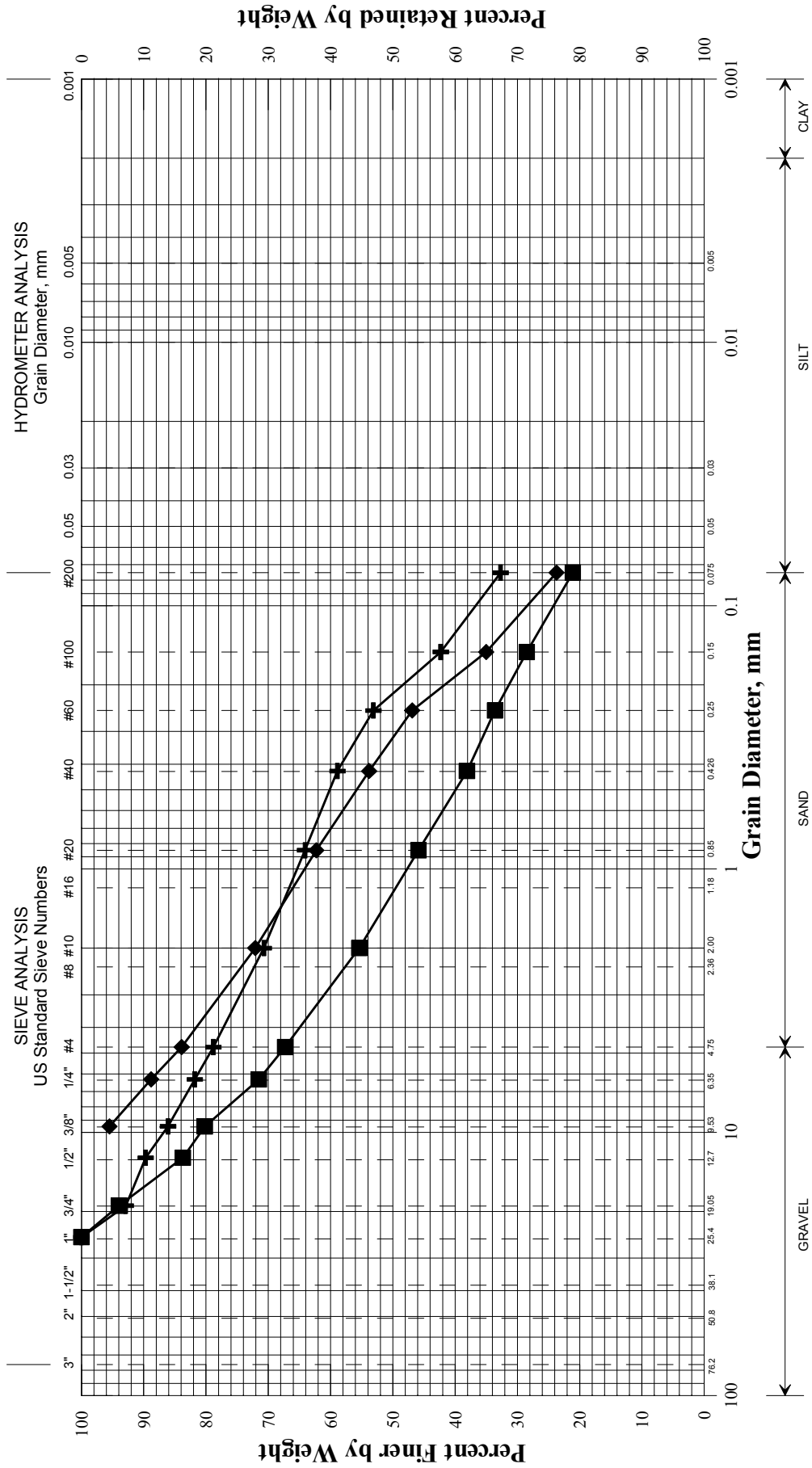


UNIFIED CLASSIFICATION

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AGMB-103/1D	13+65.7	7.4 LT	5.0-7.0	SAND, some gravel, some silt.	8.7		
◆	BB-AGMB-103/2D	13+65.7	7.4 LT	10.0-12.0	SAND, some gravel, trace silt.	19.5		
■								
●								
▲								
×								

017867.00	PIN
Arundel	Town
WHITE, TERRY A	Reported by/Date
5/20/2011	

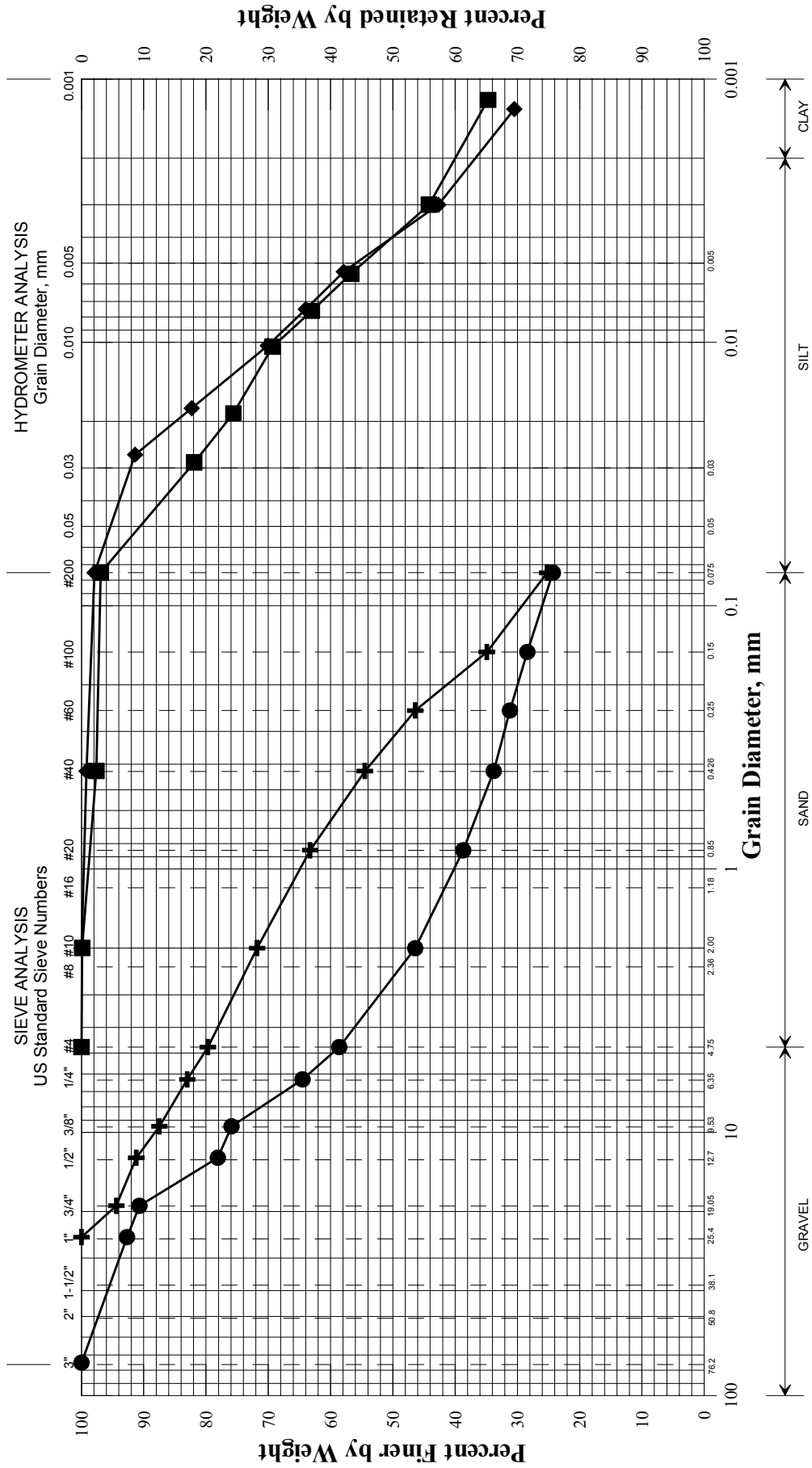
State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



017867.00	PIN
Arundel	Town
WHITE, TERRY A	Reported by/Date
	6/6/2011

Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
BB-AGMB-104/1D	13+93.4	9.3 RT	5.0-7.0	SAND, some silt, some gravel.	14.3			
BB-AGMB-104/2D	13+93.4	9.3 RT	10.0-12.0	SAND, some silt, little gravel.	30.5			
BB-AGMB-104/3D	13+93.4	9.3 RT	15.0-17.0	SAND, some gravel, some silt.	13.8			

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

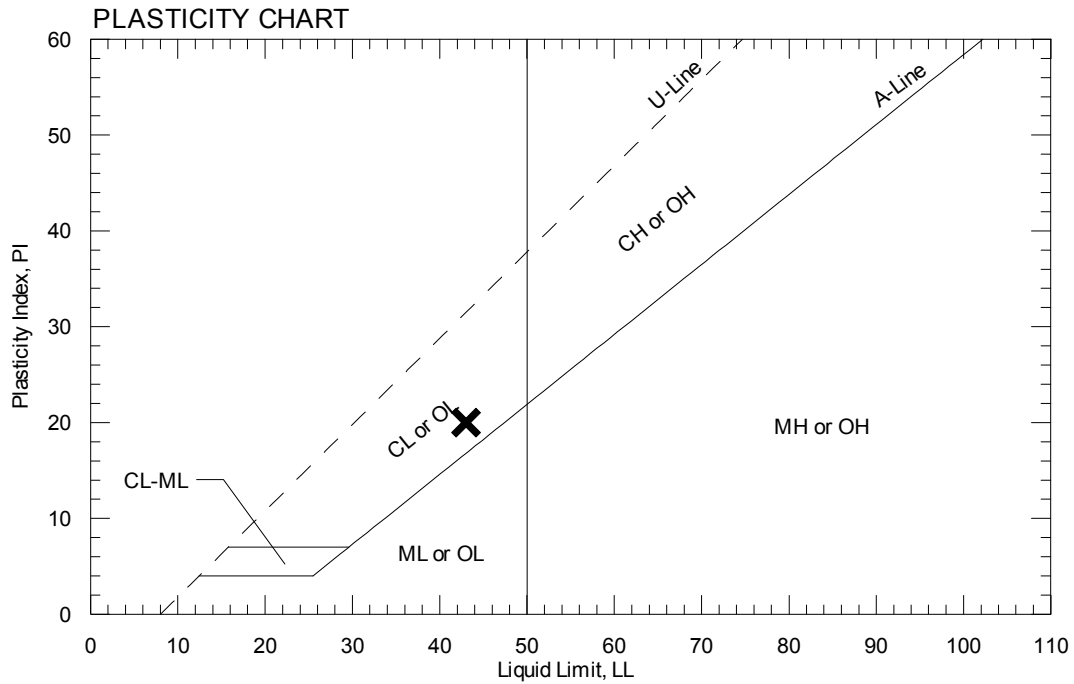
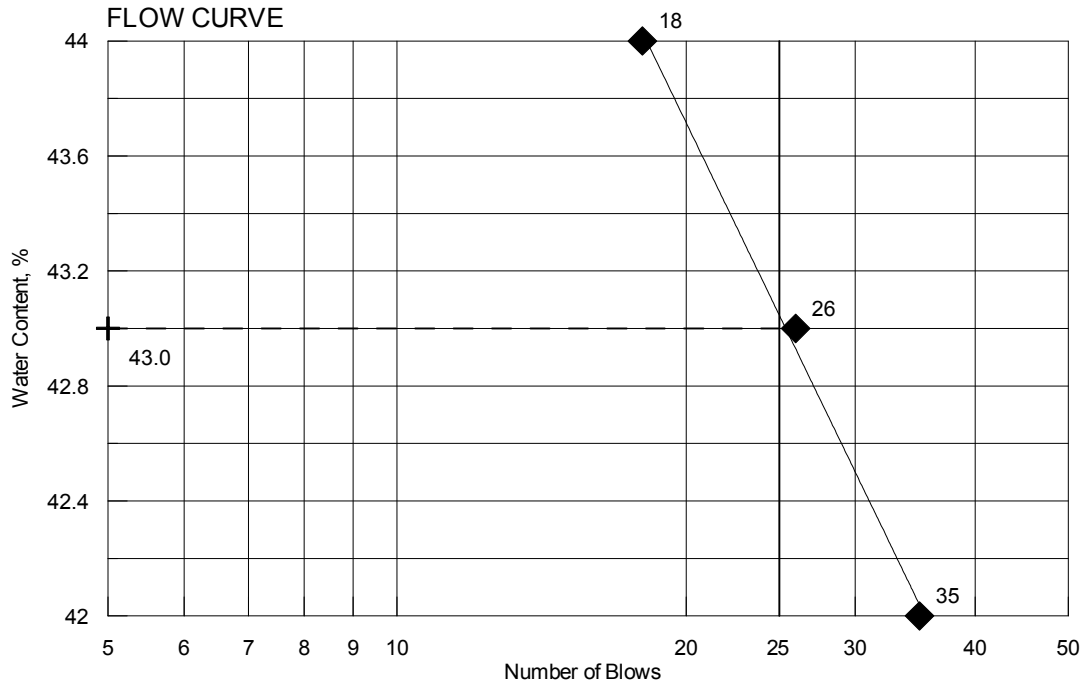


UNIFIED CLASSIFICATION

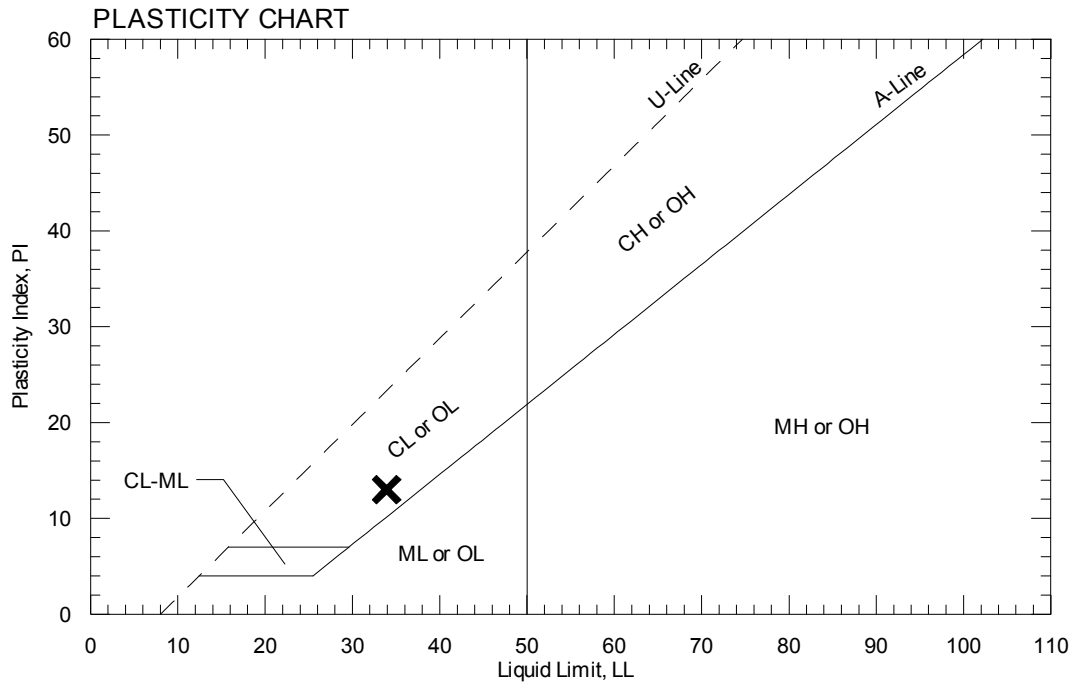
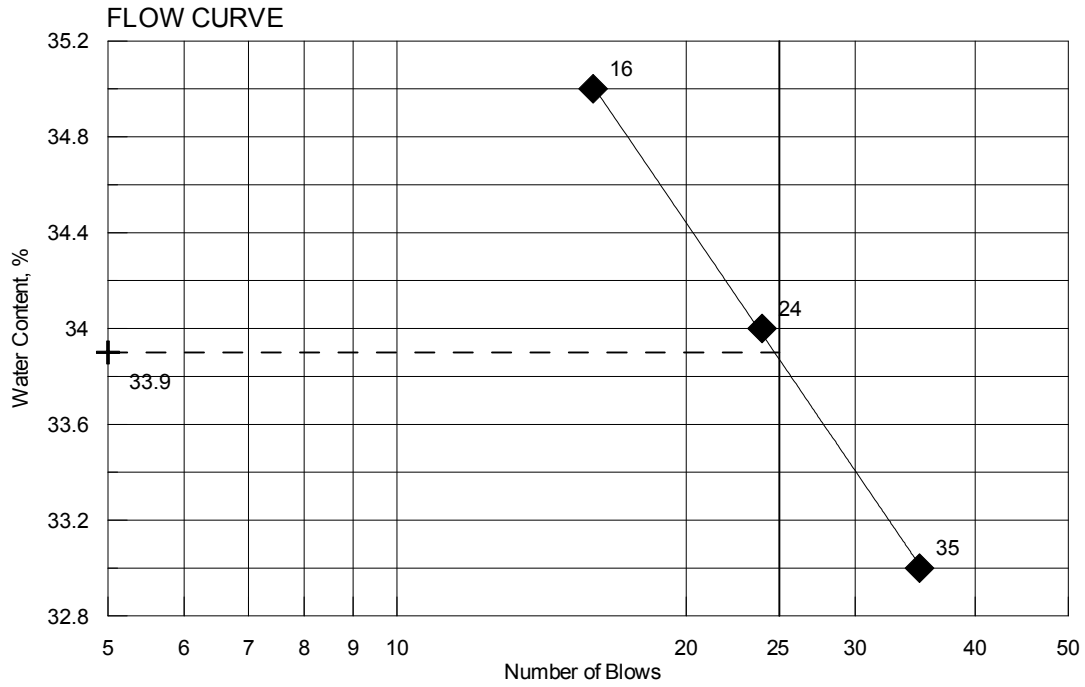
Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
+	BB-AGMB-105/1D	14+12.9	9.3 RT	1.5-3.5	SAND, some silt, little gravel.	10.1		
◆	BB-AGMB-105/2D	14+12.9	9.3 RT	5.0-7.0	Clayey SILT, trace sand.	25.3		
■	BB-AGMB-105/3D	14+12.9	9.3 RT	11.0-12.0	Clayey SILT, trace sand.	39.5	35	20
●	BB-AGMB-105/4D	14+12.9	9.3 RT	15.5-17.5	GRAVEL, some sand, some silt.	11.6		
▲								
×								

PIN	017867.00
Town	Arundel
Reported by/Date	WHITE, TERRY A 5/20/2011

TOWN	Arundel	Reference No.	236394
PIN	017867.00	Water Content, %	46.2
Sampled	3/15/2011	Plastic Limit	23
Boring No./Sample No.	BB-AGMB-101/4D	Liquid Limit	43
Station	13+14.3	Plasticity Index	20
Depth	15.0-17.0	Tested By	BBURR



TOWN	Arundel	Reference No.	236395
PIN	017867.00	Water Content, %	43.8
Sampled	3/15/2011	Plastic Limit	21
Boring No./Sample No.	BB-AGMB-101/1U	Liquid Limit	34
Station	13+14.3	Plasticity Index	13
Depth	20.0-22.0	Tested By	BBURR



CONSOLIDATION TEST DATA

Project: HUTCHINS BRIDGE
 Boring No.: BB-AGMB-101
 Sample No.: 1U
 Test No.: 236395

Location: ARUNDEL
 Tested By: G LIDSTONE
 Test Date: 4/5/11
 Sample Type: Shelby Tube

Project No.: 17867.00
 Checked By: km
 Depth: 20-22 FT
 Elevation: ---

Soil Description: Grey Clayey SILT

Remarks: OCR = 1.298; Cc = 0.675; C'c = 0.288; Cr = 0.034

Measured Specific Gravity: 2.77
 Initial Void Ratio: 1.35
 Final Void Ratio: 0.74

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

Initial Height: 1.00 in
 Specimen Diameter: 2.48 in

Container ID	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
	203	RING	RING	59
Wt. Container + Wet Soil, gm	204.41	401.53	380.66	167.58
Wt. Container + Dry Soil, gm	159.69	355.63	355.63	142.54
Wt. Container, gm	64.39	262.09	262.09	48.98
Wt. Dry Soil, gm	95.3	93.536	93.536	93.56
Water Content, %	46.93	49.08	26.76	26.76
Void Ratio	---	1.35	0.74	---
Degree of Saturation, %	---	100.82	100.00	---
Dry Unit Weight, pcf	---	73.611	99.254	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

CONSOLIDATION TEST DATA

Project: HUTCHINS BRIDGE
 Boring No.: BB-AGMB-101
 Sample No.: 1U
 Test No.: 236395

Location: ARUNDEL
 Tested By: G LIDSTONE
 Test Date: 4/5/11
 Sample Type: Shelby Tube

Project No.: 17867.00
 Checked By: km
 Depth: 20-22 FT
 Elevation: ---

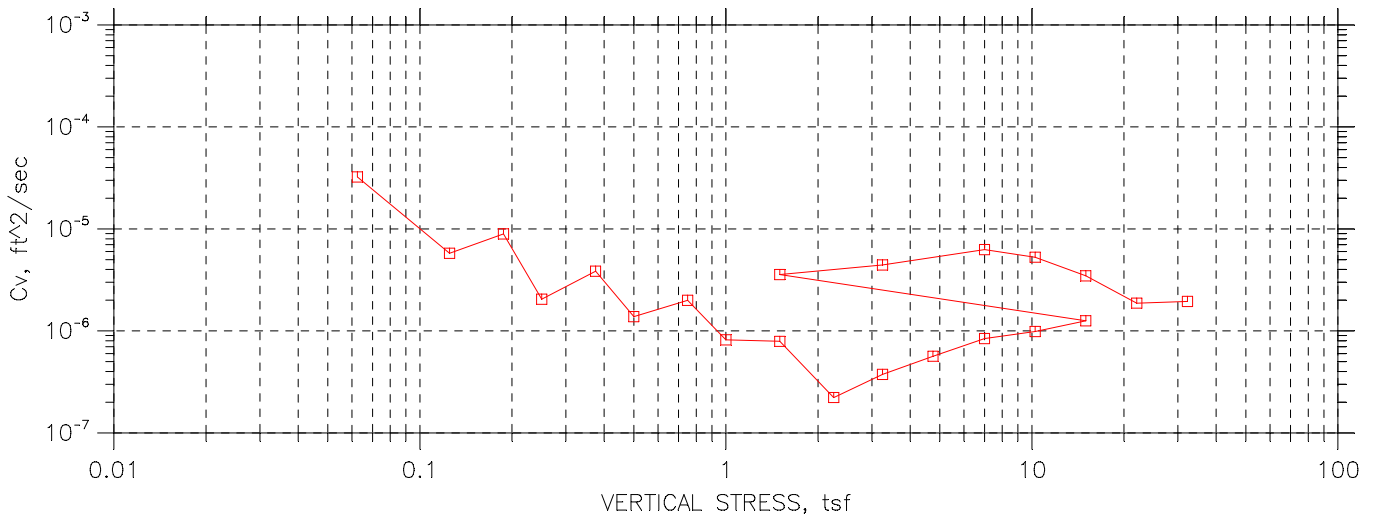
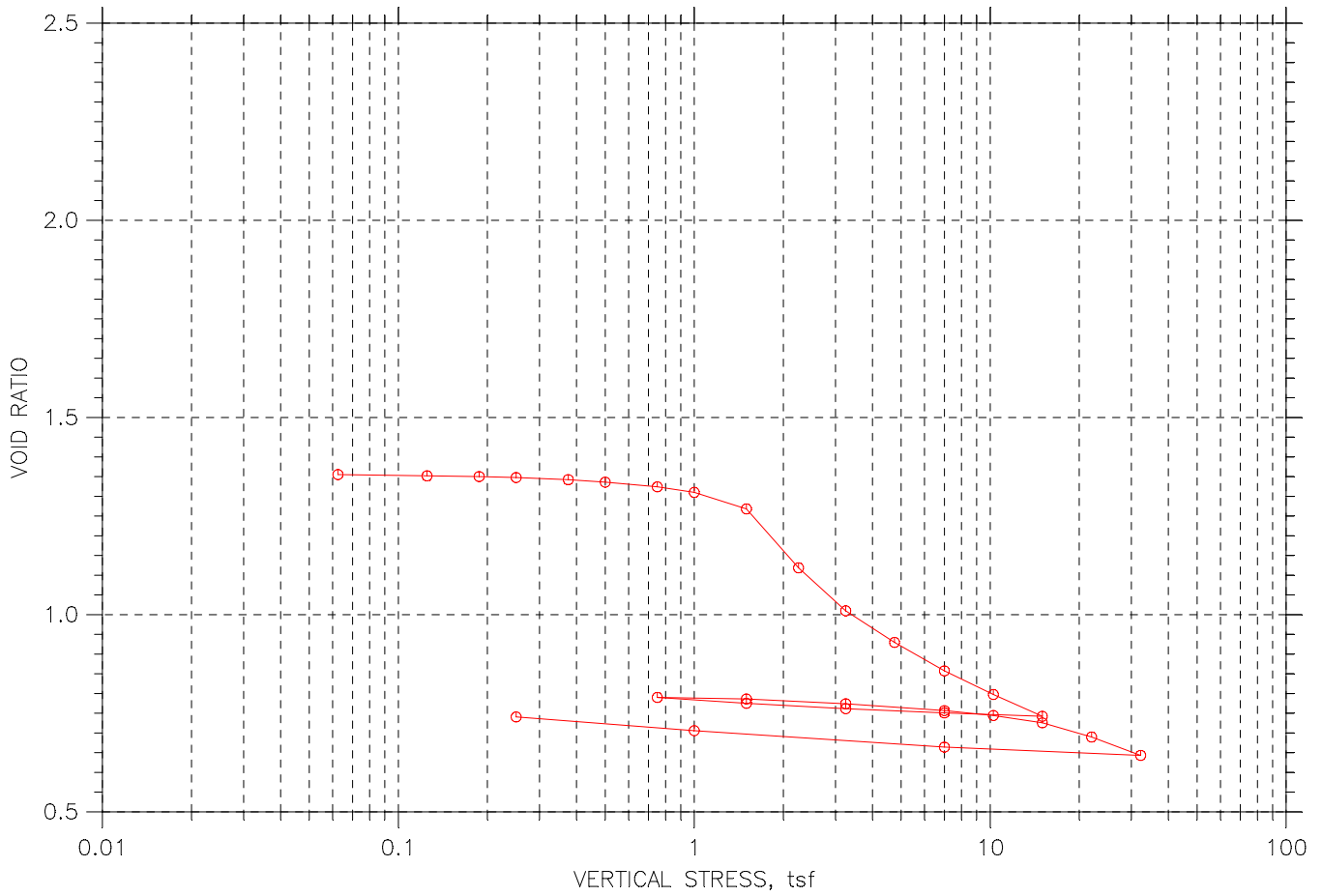
Soil Description: Grey Clayey SILT

Remarks: OCR = 1.298; Cc = 0.675; C'c = 0.288; Cr = 0.034

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	T50 Fitting		Coefficient of Consolidation		
					Sq.Rt. min	Log min	Sq.Rt. ft ² /sec	Log ft ² /sec	Ave. ft ² /sec
1	0.0625	-0.003496	1.355	-0.35	0.2	0.1	2.62e-005	4.21e-005	3.23e-005
2	0.125	-0.002104	1.352	-0.21	1.0	0.0	5.77e-006	0.00e+000	5.77e-006
3	0.188	-0.001364	1.350	-0.14	0.6	0.0	8.93e-006	0.00e+000	8.93e-006
4	0.25	-0.0002556	1.348	-0.03	4.7	0.9	1.21e-006	6.63e-006	2.04e-006
5	0.375	0.001964	1.342	0.20	1.5	1.4	3.70e-006	4.01e-006	3.85e-006
6	0.5	0.004657	1.336	0.47	5.0	3.2	1.14e-006	1.76e-006	1.38e-006
7	0.75	0.009758	1.324	0.98	3.4	2.2	1.65e-006	2.53e-006	2.00e-006
8	1	0.01572	1.310	1.57	6.8	0.0	8.17e-007	0.00e+000	8.17e-007
9	1.5	0.03352	1.268	3.36	6.8	0.0	7.90e-007	0.00e+000	7.90e-007
10	2.25	0.09713	1.119	9.73	21.8	22.8	2.27e-007	2.17e-007	2.22e-007
11	3.25	0.1435	1.010	14.38	13.2	10.3	3.34e-007	4.27e-007	3.75e-007
12	4.75	0.1775	0.930	17.78	7.0	7.2	5.75e-007	5.54e-007	5.64e-007
13	7	0.2082	0.857	20.86	4.6	4.2	8.08e-007	8.78e-007	8.42e-007
14	10.3	0.2337	0.798	23.41	3.5	3.5	9.99e-007	9.74e-007	9.87e-007
15	15	0.2572	0.742	25.77	2.3	2.8	1.40e-006	1.14e-006	1.26e-006
16	7	0.2535	0.751	25.40	0.0	0.0	8.56e-005	0.00e+000	8.56e-005
17	3.25	0.249	0.762	24.95	0.5	0.3	6.91e-006	1.16e-005	8.65e-006
18	1.5	0.2432	0.775	24.37	1.3	0.0	2.55e-006	0.00e+000	2.55e-006
19	0.75	0.2369	0.790	23.73	3.5	4.1	9.51e-007	8.08e-007	8.74e-007
20	1.5	0.2385	0.786	23.90	0.9	0.0	3.57e-006	0.00e+000	3.57e-006
21	3.25	0.2437	0.774	24.41	0.7	0.0	4.44e-006	0.00e+000	4.44e-006
22	7	0.2511	0.757	25.16	0.7	0.3	4.64e-006	9.67e-006	6.27e-006
23	10.3	0.2563	0.744	25.68	0.9	0.3	3.55e-006	1.03e-005	5.28e-006
24	15	0.2643	0.726	26.48	0.9	0.9	3.44e-006	3.49e-006	3.47e-006
25	22	0.2795	0.690	28.01	1.6	1.6	1.88e-006	1.87e-006	1.87e-006
26	32.3	0.2993	0.643	29.99	1.4	1.5	2.04e-006	1.86e-006	1.94e-006
27	7	0.2904	0.664	29.10	0.0	0.0	5.87e-005	2.32e-004	9.37e-005
28	1	0.2727	0.706	27.32	1.7	0.0	1.73e-006	0.00e+000	1.73e-006
29	0.25	0.2579	0.741	25.84	9.4	13.4	3.27e-007	2.28e-007	2.69e-007

CONSOLIDATION TEST DATA

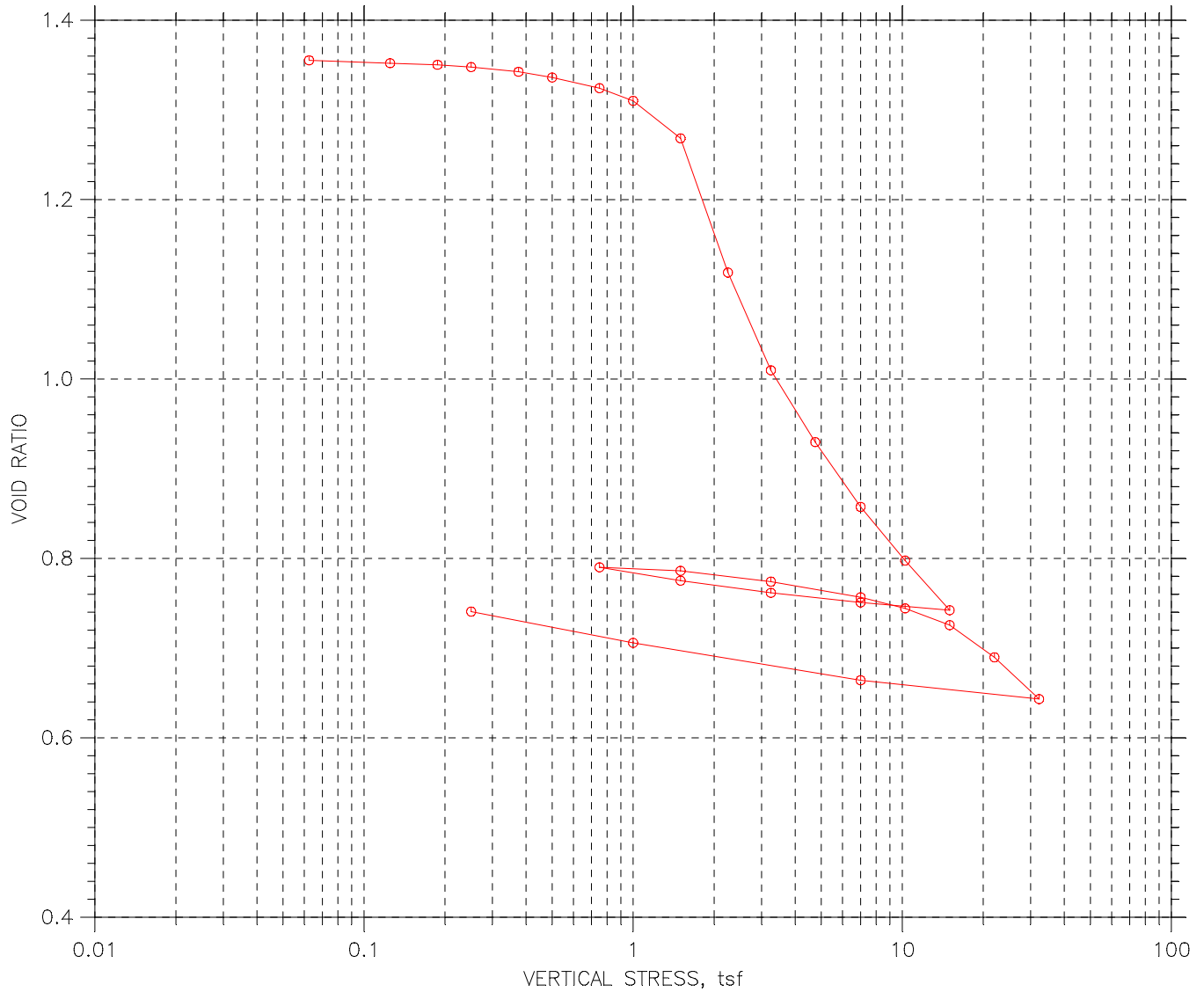
SUMMARY REPORT



Project: HUTCHINS BRIDGE	Location: ARUNDEL	Project No.: 17867.00
Boring No.: BB-AGMB-101	Tested By: G LIDSTONE	Checked By: km
Sample No.: 1U	Test Date: 4/5/11	Depth: 20-22 FT
Test No.: 236395	Sample Type: Shelby Tube	Elevation: ---
Description: Grey Clayey SILT		
Remarks: OCR = 1.298; C_c = 0.675; C'_c = 0.288; C_r = 0.034		

CONSOLIDATION TEST DATA

SUMMARY REPORT



				Before Test	After Test
Overburden Pressure: 1 tsf		Water Content, %		49.08	26.76
Preconsolidation Pressure: 1.3 tsf		Dry Unit Weight, pcf		73.61	99.25
Compression Index: 0.675		Saturation, %		100.82	100.00
Diameter: 2.485 in		Height: 0.9981 in		Void Ratio	1.35
LL: 34	PL: 21	PI: 13	GS: 2.77		0.74

	Project: HUTCHINS BRIDGE	Location: ARUNDEL	Project No.: 17867.00
	Boring No.: BB-AGMB-101	Tested By: G LIDSTONE	Checked By: km
	Sample No.: 1U	Test Date: 4/5/11	Depth: 20-22 FT
	Test No.: 236395	Sample Type: Shelby Tube	Elevation: ---
	Description: Grey Clayey SILT		
	Remarks: OCR = 1.298; Cc = 0.675; C'c = 0.288; Cr = 0.034		

236395

0-6"



236395

7'12-12



236395

12-18

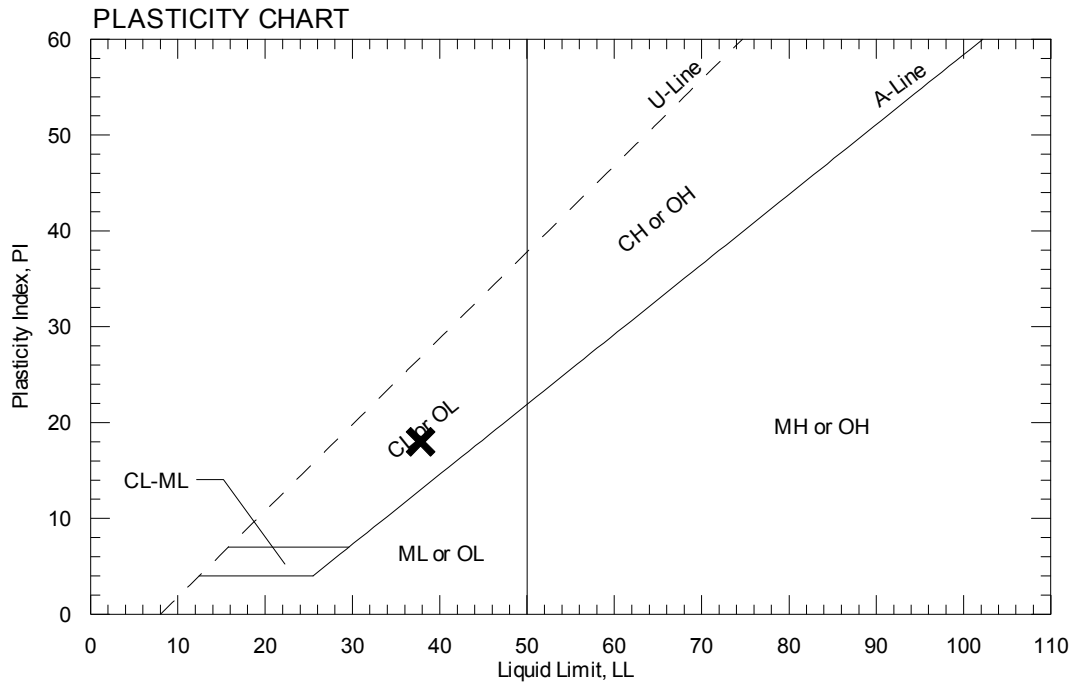
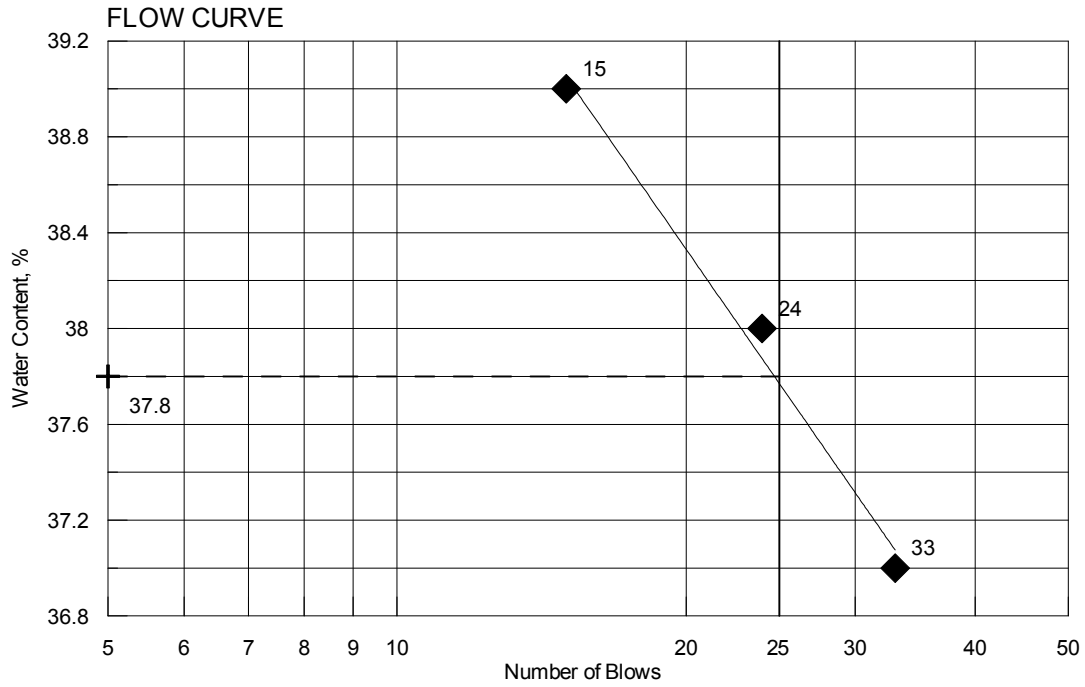


236395

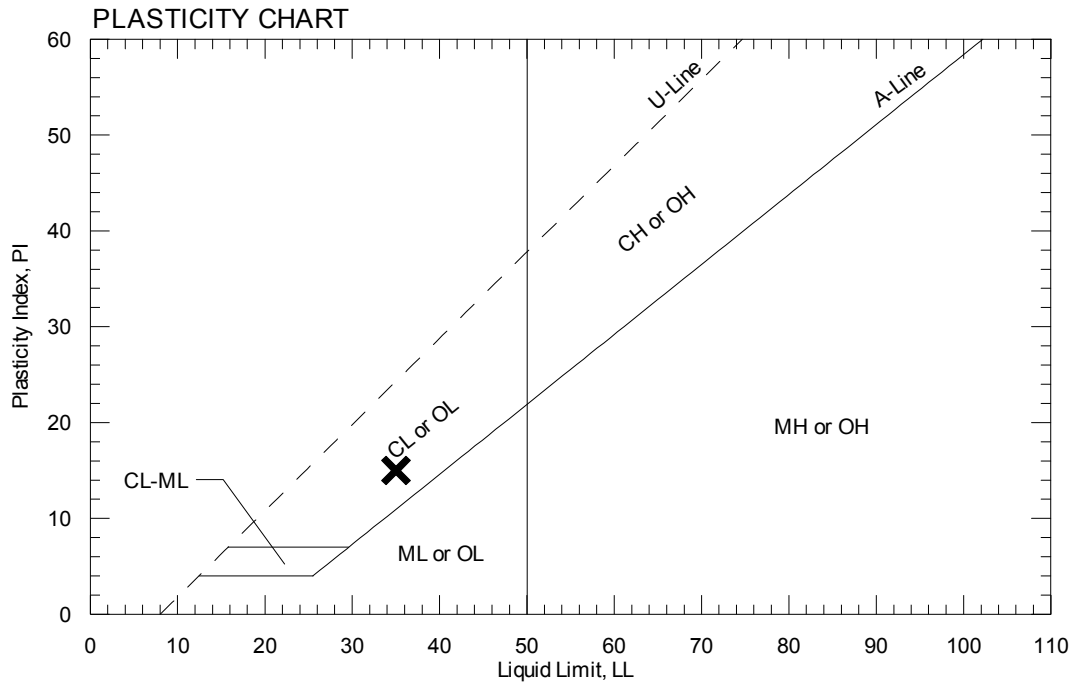
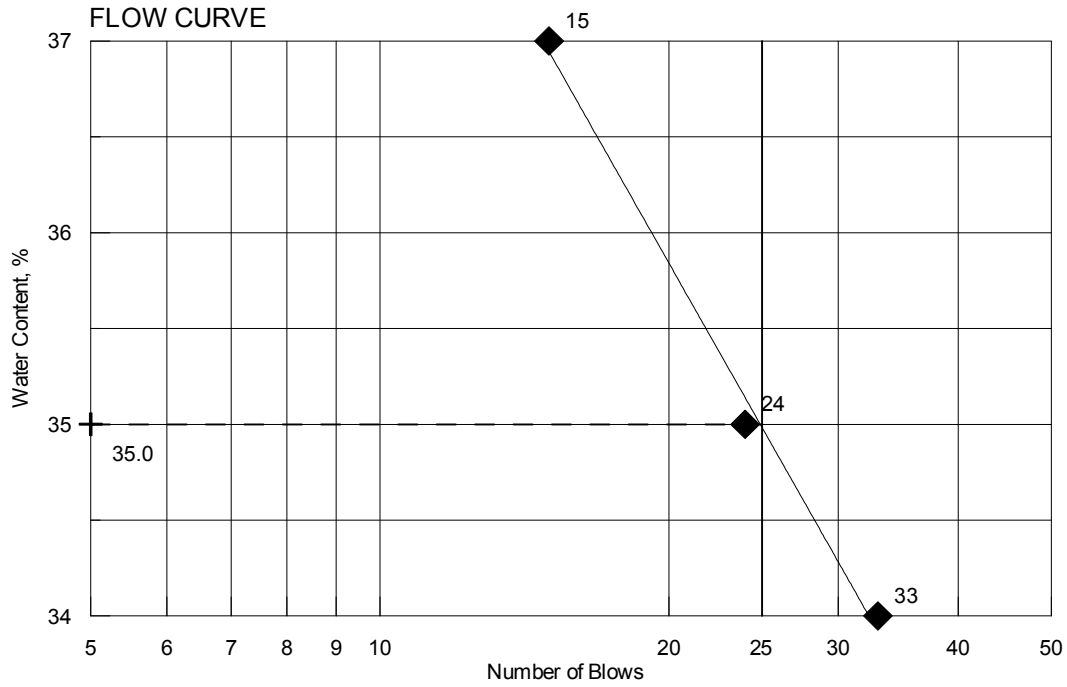
18-24



TOWN	Arundel	Reference No.	236396
PIN	017867.00	Water Content, %	43.4
Sampled	3/15/2011	Plastic Limit	20
Boring No./Sample No.	BB-AGMB-101/5D	Liquid Limit	38
Station	13+14.3	Plasticity Index	18
Depth	25.0-27.0	Tested By	BBURR



TOWN	Arundel	Reference No.	262423
PIN	017867.00	Water Content, %	39.5
Sampled	3/16/2011	Plastic Limit	20
Boring No./Sample No.	BB-AGMB-105/3D	Liquid Limit	35
Station	14+12.9	Plasticity Index	15
Depth	11.0-12.0	Tested By	BBURR



Appendix C

Calculations

LIQUIDITY INDEX (LI):

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

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

Sample	WC	LL	PL	PI	LI	
BB-AGMB-101/4D	46.2	43	23	20	1.16	Normally Consolidated
BB-AGMB-101/1U	43.8	34	21	13	1.75	Viscous Liquid
BB-AGMB-101/5D	43.4	38	20	18	1.30	Viscous Liquid

At-Rest and Active Earth Pressure:

At-Rest Lateral Earth Pressure
 from LRFD Article 3.11.5.2 pg 3-71

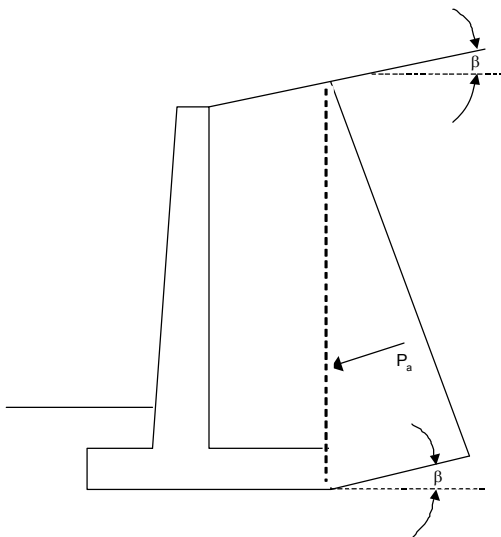
Effective friction angle of soil $\phi_f := 30 \cdot \text{deg}$

$$K_o := 1 - \sin(\phi_f) \quad K_o = 0.5$$

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

Soil Type 4 Properties from MaineDOT Bridge Design Guide (BDG)

- unit weight: $\gamma_{\text{type4}} := 125 \cdot \text{pcf}$
- Internal Friction Angle: $\phi_{\text{type4}} := 32 \cdot \text{deg}$
- Cohesion: $c_{\text{sand}} := 0 \cdot \text{psf}$



Generally use Rankine for long heeled cantilever walls where the failure surface is interrupted by the top of the wall system. The earth pressure is applied to a plane extending vertically up from the heel of the wall base and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or the backface of the wall.

For cantilever walls with sloped backfill surface:

β = Angel of fill slope to the horizontal

$\beta := 0 \cdot \text{deg}$ assume horizontal backfill surface

$$K_{a_rankine_slope} := \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_{type4})^2}} \quad K_{a_rankine_slope} = 0.31$$

Pa is oriented at an angle of β to the vertical plane.

Bearing Resistance - Native Granular Soils:

Part 1 - Service Limit State

Nominal and factored Bearing Resistance - box culvert on silty soils

Presumptive Bearing Resistance for Service Limit State ONLY

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

Type of Bearing Material: Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)

Consistency In Place: Based on N-values of 6 - medium stiff

Bearing Resistance: Ordinary Range (ksf) 2 to 6

Recommended Value of Use: 3 ksf

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

Recommended Value: $3 \cdot \text{ksf} = 1.5 \cdot \text{tsf}$

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

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

$$q_{factored_bc} := 1.5 \cdot \text{tsf} \quad \text{or} \quad q_{factored_bc} = 3 \cdot \text{ksf}$$

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

Part 2 - Strength Limit State

Nominal and factored Bearing Resistance - box culvert on silty soils

Reference: Foundation Engineering and Design by JE Bowles Fifth Edition

Assumptions:

1. The box culverts will be founded at ~ Elev 41.5
Approx. 14 ft below roadway surface $D_{\text{box}} := 2.0\text{-ft}$
2. Assumed parameters for soils: (Ref: Bowles 5th Ed Table 3-4)
Saturated unit weight: $\gamma_s := 115\text{-pcf}$
Dry unit weight: $\gamma_d := 110\text{-pcf}$
Internal friction angle: $\phi_{\text{ns}} := 26\text{-deg}$
Undrained shear strength: $c_{\text{ns}} := 400\text{-psf}$
3. Use Terzaghi strip equations as $L > B$
4. Effective stress analysis footing on ϕ -c soil (Bowles 5th Ed. Example 4-1 pg 231)

Depth the water table: $D_w := 0\text{-ft}$ Unit Weight of water: $\gamma_w := 62.4\text{-pcf}$

Effective stress at box bearing level:

$$q_{\text{eff}} := D_w \cdot \gamma_s + (D_{\text{box}} - D_w) \cdot (\gamma_s - \gamma_w) \quad q_{\text{eff}} = 0.105\text{-ksf}$$

Width: $B := 18\text{-ft}$

Terzaghi Shape factors from Table 4-1 For a strip footing: $s_c := 1.0$ $s_\gamma := 1.0$

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

For $\phi=26$ deg $N_c := 22.25$ $N_q := 11.8$ $N_\gamma := 8.0$

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

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

$$q_{\text{nominal}} = 9.2\text{-tsf}$$

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

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

$$q_{\text{factored}} = 4.1\text{-tsf} \quad B = 18\text{-ft}$$

At Strength Limit State:

Recommend a limiting factored bearing resistance of 4 ksf

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:
 Arundel, Maine
 DFI = 1100 degree-days

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

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1100 and wc =10%
 Frost Penetration = 69.8 inches

Frost_depth := 69.8in Frost_depth = 5.8·ft

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Portland

ModBerg Results ---

Project Location: Portland Wsfo Airport, Maine

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

Layer #	Type	t	w%	d	Cf	Cu	Kf	Ku	L
1	Coarse	59.6	10.0	120.0	26	32	1.7	1.5	1,728

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

 Total Depth of Frost Penetration = 4.97 ft = 59.6 in.

Frost_depth_{modberg} := 59.6·in

Frost_depth_{modberg} = 4.967·ft

Use Frost Depth = 5.0 feet for design

CONSOLIDATION TEST RESULTS

BB-AGMB-101 Sample 1U

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

Determine in-situ overburden stress:

Sample depth = 20 to 22 ft below ground surface

Groundwater table at 14.0 ft below ground surface

Unit weight of water = 62.4pcf

Initial void ratio $e_0 := 1.35$

Clay is overlain by:

3.5 ft of sand fill at 125 pcf

8.5 ft of Silty Sand at 120 pcf

8.0 ft of Silty Clay/Clayey Silt at 115 pcf

$$\sigma'_{v0} := 3.5 \cdot \text{ft} \cdot 125 \cdot \text{pcf} + 8.5 \cdot \text{ft} \cdot 120 \cdot \text{pcf} + 2 \cdot \text{ft} \cdot 115 \cdot \text{pcf} + 6 \cdot \text{ft} \cdot (115 - 62.4) \cdot \text{pcf}$$

$$\sigma'_{v0} = 2003 \cdot \text{psf} \quad \text{or} \quad \sigma'_{v0} = 1.002 \cdot \text{tsf}$$

Maximum past pressure from consolidation curve Casagrande construction: $\sigma'_p := 1.3 \cdot \text{tsf}$

Determine OCR: $\text{OCR} := \frac{\sigma'_p}{\sigma'_{v0}}$ $\text{OCR} = 1.298$ *over consolidated*

Determine C_c :

from consolidation curve and lab results:

$$p_1 := 1.5 \cdot \text{tsf} \quad e_1 := 1.268 \quad p_2 := 4.75 \cdot \text{tsf} \quad e_2 := 0.930$$

$$C_c := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_c = 0.675$$

Determine C'_c :

from consolidation curve and lab results:

$$\varepsilon_1 := \frac{3.36}{100} \quad \varepsilon_2 := \frac{17.78}{100} \quad \text{strain is given in percent}$$

$$C'_c := \frac{\varepsilon_2 - \varepsilon_1}{\log\left(\frac{p_2}{p_1}\right)} \quad C'_c = 0.288 \quad \text{or:} \quad C'_c := \frac{C_c}{1 + e_0} \quad C'_c = 0.287$$

Determine C_r :

from consolidation curve and lab results:

$$p_1 := 0.75 \cdot \text{tsf} \quad e_1 := 0.79 \quad p_2 := 7 \cdot \text{tsf} \quad e_2 := 0.757$$

$$C_r := \frac{e_1 - e_2}{\log\left(\frac{p_2}{p_1}\right)} \quad C_r = 0.034$$

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 534
PRECAST STRUCTURAL CONCRETE
(Precast Structural Concrete Arches, Box Culverts, Frames)

534.10 Description The Contractor shall design, manufacture, furnish, and install elements, precast structural concrete structures, arches box culverts or three sided frames and associated wingwalls, headwalls, toe walls/cut off walls and appurtenances, in accordance with the contract documents.

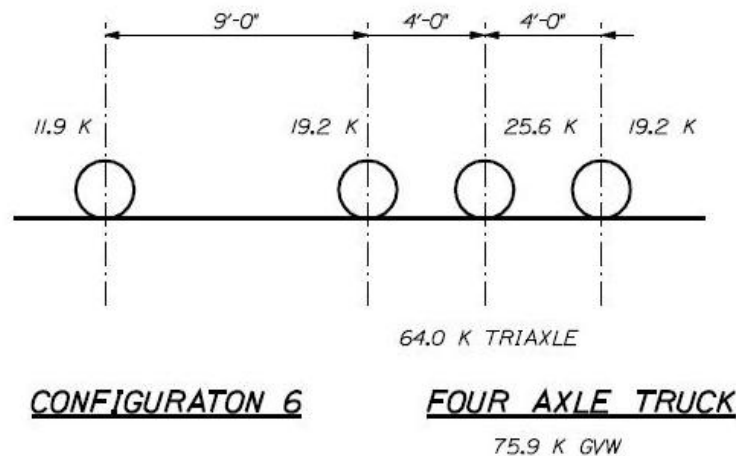
534.20 Materials Structural precast elements for the arch, box culvert or frame and associated precast elements shall meet the requirements of the following Subsection:

Structural Precast Concrete Units 712.061

Grout, concrete patching material, and geotextiles shall be one of the products listed on the Department's list of prequalified materials, unless otherwise approved by the Department.

Box culvert bedding and backfill material shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill, with the additional requirement that the maximum particle size be limited to 4 inches, or as shown on the Plans.

534.30 Design Requirements The Contractor shall design the precast structural concrete structure in accordance with the AASHTO LRFD Bridge Design Specifications, latest edition. The HL-93 live load specified in the AASHTO LRFD Bridge Design Specifications shall be used for all limit states except for Strength I. The live load used for the Strength I limit state shall be the Maine Modified live load which consists of the standard HL-93 Live Load with a 25% increase in the Design Truck. (Wheel loads based on the Design Truck shall be increased 25%). In addition, the following Maine legal truck (Configuration #6) shall also be checked to ensure that the rating factor is equal to or greater than 1.0 (see diagram immediately below).



Legal Load Configuration #6 is not a design load. Refer to Table 6A.4.5.4.2a-1 in the latest edition of the AASHTO Manual for Bridge Evaluation for the correct live load factor to use for a permit type of "Routine or Annual" with permit weight up to 100 kips.

The live load deflection check per AASHTO LRFD Bridge Design Specifications Section 2.5.2.6.2 for the top slab of box culverts with clear spans 15 feet or greater and cover depths of 4 feet or less is mandatory. The live load deflection check shall be documented in the design computations submittal.

Design calculations that consist of computer program generated output shall be supplemented with at least one hand calculation and graphic demonstrating the design methodology used. The hand calculation shall document, at a minimum, the Strength I load case flexural design check of the top slab positive moment reinforcing steel. Design calculations shall provide thorough documentation of the sources of equations used and material properties.

The design shall be load rated in accordance with the AASHTO Manual for Bridge Evaluation, latest edition, by the LRFR method. The design shall be load rated based on both the HL-93 live load and the HL-93 modified live load. The live load rating computations shall include a completed MaineDOT Summary of Rating Form based on the rating factors for the HL-93 live load only unless the rating factors based on the Legal Load Configuration #6 controls. The MaineDOT Summary of Rating Form may be accessed at the following MaineDOT web address: www.maine.gov/mdot/contractors/

A load rating shall be provided in the following situations:

1. All structures with clear spans between 10 feet and 15 feet with 4 feet or less of fill over the top of the structure.
2. All structures with clear spans between 15 feet and 20 feet with 8 feet or less of fill over the top of the structure.
3. All structures with clear spans of 20 feet or greater.

The Contractor shall submit design calculations, load rating, if applicable, and working/shop drawings for the precast structure to the Department for approval. A Registered Professional Engineer, licensed in accordance with State of Maine laws, shall sign and seal all design calculations and drawings. Drawings shall conform with Section 105.7 - Working Drawings.

The Contractor shall submit the following items for review by the Resident at least ten working days prior to production:

- A) The name and location of the manufacturer.
- B) Method of manufacture and material certificates.
- C) Description of method of handling, storing, transporting, and erecting the members.
- D) Design computations (bond and indexed)
- E) Load rating computations and completed load rating form (bond and indexed)
- F) Shop Drawings with the following minimum details:
 - 1) Fully dimensioned views showing the geometry of the members, including all projections, recesses, notches, openings, block outs, and keyways.

- 2) Details and bending schedules of reinforcing steel including the size, spacing, and location. Reinforcing provided under lifting devices shall be shown in detail.
- 3) Details and locations of all items to be embedded.
- 4) Total weight of each member.

534.40 Construction Requirements The applicable provisions of Subsection 535.10 - Forms and Casting Beds and Subsection 535.20 – Finishing Concrete and Repairing Defects shall be met.

Manufacture of Precast Units The internal dimensions shall not vary by more than 1 percent from the design dimensions or 1½ inches, whichever is less with the exception of the cross diagonal dimension which shall not vary by more than ½ inch from the design dimension. The haunch dimensions shall not vary by more than ¾ inch from the design dimension. The dimension of the legs shall not vary by more than ¼ inch from the dimension shown on the approved shop drawings.

The slab and wall thickness shall not be less than the design thickness by more than ¼ inch. A thickness greater than the design thickness shall not be cause for rejection.

Variations in laying lengths of two opposite surfaces shall not be more than ⅝ inch in any section, except where beveled ends for laying of curves are specified.

The under-run in length of any section shall not be more than ½ inch.

The cover of concrete over the outside circumferential reinforcement shall be 2 inches minimum. The concrete cover over the inside reinforcement shall be 1½ inches minimum. The clear distance of the end of circumferential wires shall not be less than 1 inch or more than 2 inches from the end of the sections. Reinforcement shall be single or multiple layers of welded wire fabric or a single layer of deformed billet steel bars.

Welded wire fabric shall meet the space requirements and contain sufficient longitudinal wires extending through the section to maintain the shape and position of the reinforcement. Longitudinal distribution reinforcement may be welded wire fabric or deformed steel bars which meet the spacing requirements. The ends of the longitudinal distribution reinforcement shall be not more than 3 inches from the ends of the sections.

The inside circumferential reinforcing steel for the haunch radii or fillet shall be bent to match the radii or fillets of the forms.

Tension splices in the reinforcement will not be permitted. For splices other than tension splices, the overlap shall be a minimum of 12 inches for welded wire fabric or deformed steel bars. The spacing center to center of the circumferential wires in a wire fabric sheet shall be not less than 2 inches or more than 4 inches. For the wire fabric, the spacing center to center of the longitudinal wires shall not be more than 8 inches. The spacing center to center of the longitudinal distribution steel for either line of reinforcing in the top slab shall be not more than 15 inches.

The members shall be free of fractures. The ends of the members shall be normal to the walls and centerline of the section, within the limits of variation provided, except where beveled ends are specified. The surfaces of the members shall be a smooth steel form or troweled surface finish, unless a form liner is specified. The ends and interior of the assembled structure shall make a continuous line of members with a smooth interior surface.

Defects which may cause rejection of precast units include the following:

- 1) Any discontinuity (crack or rock pocket etc.) of the concrete which could allow moisture to reach the reinforcing steel.
- 2) Rock pockets or honeycomb over 6 square inches in area or over 1 inch deep.
- 3) Edge or corner breakage exceeding 12 inches in length or 1 inch in depth.
- 4) Extensive fine hair cracks or checks.
- 5) Any other defect that clearly and substantially impacts the quality, durability, or maintainability of the structure as measured by accepted industry standards.

The manufacturer of the members shall sequentially number and shop fit each adjacent member to ensure that they fit together in the field. This fit up shall be witnessed by the QA inspector. Any non fitting members shall be corrected or replaced at no cost to the Department.

The Contractor shall store and transport members in a manner to prevent cracking or damage. The Contractor shall not place precast members in an upright position until a compressive strength of at least 4350 psi is attained.

Installation of Precast Units The Contractor shall not ship precast members until sufficient strength has been attained to withstand shipping, handling and erection stresses without cracking, deformation, or spalling. A minimum strength of 4350 psi shall be attained prior to shipping in all cases.

The Contractor shall set precast members on ½ inch neoprene pads during shipment to prevent damage to the section legs. The Contractor shall repair any damage to precast members resulting from shipping or handling by saw cutting a minimum of ½ inch deep around the perimeter of the damaged area and placing a polymer-modified cementitious patching material.

When footings are required, the Contractor shall install the precast members on concrete footings that have reached a compressive strength of at least 2900 psi. The Contractor shall construct the completed footing surface to the lines and grades shown on the plans. When checked with a 10 foot straightedge, the surface shall not vary more than ¼ inch in 10 feet. The footing keyway shall be filled with a non-shrink flowable cementitious grout with a design compressive strength of at least 5075 psi.

The Contractor shall fill holes that were cast in the units for handling, with either Portland cement mortar, or with precast plugs secured with Portland cement mortar or other approved adhesive. The Contractor shall completely fill the exterior face of joints between precast members with an approved material and cover with a minimum 12 inch wide joint wrap. The surface shall be free of dirt and deleterious materials before applying the filler material and

joint wrap. The Contractor shall install the external wrap in one continuous piece over each member joint, taking care to keep the joint wrap in place during backfilling. The Contractor shall seal the joints between the end unit and attached elements with a non-woven geotextile. The Contractor shall install and tighten the bolts fastening the connection plate(s) between the elements that are designed to be fastened together as designated by the manufacturer.

The Contractor shall place and compact the bedding material as shown on the plans prior to lifting and setting the box culvert sections. The Contractor shall backfill the structure in accordance with the manufacturer's instructions and the Contract Documents. The Contractor shall uniformly distribute backfill material in layers of not more than 8 inches in depth, loose measure, and thoroughly compact each layer using approved compactors before successive layers are placed. The Contractor shall compact the Granular Borrow bedding and backfill in accordance with Section 203.12 - Construction of Earth Embankment with Moisture and Density Control, except that the minimum required compaction shall be 92 percent of maximum density as determined by AASHTO T180, Method C or D. The Contractor shall place and compact backfill without disturbance or displacement of the structure, keeping the fill at approximately the same elevation on both sides of the structure. Whenever a compaction test fails, the Contractor shall not place additional backfill over the area until the lift is re-compacted and a passing test achieved.

The Contractor shall use hand-operated compactors within 5 feet of the precast structure as well as over the top until it is covered with at least 12 inches of backfill. The Contractor shall take adequate precautions to protect the top of the culvert from damage during backfilling and/or paving operations. Any damage to the top of the culvert shall be repaired or members replaced at no cost to the Department.

534.50 Method of Measurement The Department will measure Precast Structural Concrete Arch, Box Culvert or Frames for payment per Lump Sum each, complete in place and accepted.

534.60 Basis of Payment The Department will pay for the accepted quantity of Precast Structural Concrete Arch, Box Culvert or Frames at the Contract Lump Sum price, such payment being full compensation for all labor, equipment, materials, professional services, and incidentals for furnishing and installing the precast concrete elements and accessories. Falsework, reinforcing steel, jointing tape, grout, cast-in-place concrete fill or grout fill for anchorage of precast wings and/or other appurtenances is incidental to the Lump Sum pay item. Cast-in-place concrete, reinforcing steel in cast-in-place elements, and membrane waterproofing will be measured and paid for separately under the provided Contract pay items. Pay adjustments for quality level will not be made for precast concrete.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
534.70 Precast Structural Concrete Arch	Lump Sum
534.71 Precast Concrete Box Culvert	Lump Sum