MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the New Construction of:

BEAVER BRIDGE
ROUTE 180 OVER UNNAMED TRIBUTARY TO THE UNION RIVER
ELLSWORTH, MAINE

Prepared by:

Michael J. Moreau, P.E.
Geotechnical Design Engineer

Reviewed by:

Laura Krusinski, P.E.
Senior Geotechnical Engineer

Hancock County
PIN 10063.10

Soils Report No. 2011-07
Bridge No. 6437

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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for the new construction of Beaver Bridge over an unnamed tributary to the Union River in Ellsworth, Maine. The proposed structure will be a 6-foot high by 10-foot wide concrete box culvert with one foot of stream bed soil placed in the bottom. The new box culvert will be constructed as part of a Route 180 re-alignment project. The structure will be comprised of a 32-foot rail to rail width with 11-foot travel lanes, 5-foot shoulders and accommodation for guardrail. The design and construction recommendations below are discussed in greater detail in Section 7.0 Foundation Considerations and Recommendations.

Box Culvert Design and Construction – The concrete box culvert will be supplier-designed and the design shall consider all relevant strength and service limit state load combinations in accordance with the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 (herein referred to as LRFD). 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 culvert will be constructed in general conformance with the MaineDOT Bridge Design Guide (BDG) Section 8, Buried Structures, and Special Provision 534, Precast Structural Concrete Arches, Box Culverts. A copy of the special provision is presented in Appendix D, Special Provision. The box culvert designer may assume Soil Type 4 (BDG Section 3.6.1) for backfill soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

The box culvert will be bedded on a two foot thick layer of ¾-inch crushed stone reinforced with geogrid and wrapped in geotextile fabric. The culvert soil envelope backfill shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill with a maximum particle size of 4.0 inches. Bedding and/or backfill should be placed in lifts 6 to 8 inches thick loose measure and compacted to manufacturer's specifications, but in no case shall the bedding and/or backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

Culvert Headwalls – We recommend integral concrete headwalls to prevent crushed stone slope protection from dropping or eroding into the waterway. Culvert headwalls larger than the nominal 1-foot by 1-foot dimension should consider all relevant LRFD strength and service limit state load combinations and be designed to resist and/or absorb lateral earth loads, a live load surcharge of 250 psf, other vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culverts. Footings for any headwall constructed independently of the box culvert shall be placed no less than 2 feet below the maximum anticipated depth of scour.

Culvert headwall sections that are fixed to the box culverts to resist movement should be designed for earth pressure 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, equal to 0.31. This assumes level backslope. The earth pressure coefficient may change if backslope conditions are different.

Box Culvert Bearing Resistance – For this project, the service limit state controls. In our analysis, we determined that a factored bearing resistance of 2.0 ksf should be used to control settlement when analyzing box bottom slabs. In no instance shall the bearing stress exceed the nominal resistance of concrete, which may be taken as $0.3f'c$.

Settlement – Approximately ten feet of embankment fill will be placed over the native glaciomarine silt to construct the approach embankment. We estimate that 1 to 2 inches of consolidation settlement will occur in the silt soil as the new embankments are constructed. The settlement estimate assumes that the clay-silt subgrade soil is not weakened by construction activity.

Because this settlement may impact pavement performance, we recommend constructing the fill embankment as early as possible in the project. We also recommend delaying the paving operation over this section for a minimum of 30 days after top of gravel grade is achieved. This will allow time for consolidation settlement to occur. We anticipate that post-construction settlement will be negligible.

Embankment Stability – We have analyzed embankment stability at the box culvert site. Our analysis indicated acceptable safety factors for the planned embankment configuration.

Scour Protection – Inlet and outlet seepage cutoff walls below the culvert will be provided for scour protection. The inlet and outlet cutoff walls should extend below the maximum depth of scour. We recommend that the bridge approach slopes be armored with a 3-foot thick layer of plain riprap adjacent to the culvert openings. The plain riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot thick layer of cushion material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill. Plain riprap shall meet the requirements of 703.26, Plain and Hand Laid Riprap, of Special Provision 703, Aggregates. The riprap slopes should also be constructed in accordance with Special Provision 610, Stone Fill, Riprap, Stone Blanket, and Stone Ditch Protection and be no steeper than a maximum 1.75:1 (H:V) extending from the edge of roadway down to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

Frost Protection – If used, foundations placed on fine-grained soils shall be founded a minimum of 4.0 feet below finish exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on soil and not those founded on bedrock.

Seismic Design Considerations – Since the buried structure does not cross active faults, no seismic analysis is required.

Construction Considerations –

<u>Excavation</u>

- Construction of the new concrete box culvert will require soil excavation. Earth support systems may be required.
- Protect the excavated subgrade from exposure to water and unnecessary construction traffic. **It is imperative that the contractor minimize aggressive excavation action or**

equipment movement over the clay-silt soil. This will disturb and/or soften the subgrade soil and may create embankment stability problems or result in excessive settlement. Remove and replace water-softened, disturbed, or rutted subgrade soil with compacted gravel borrow.

<u>Dewatering</u>

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- Cofferdams, temporary ditches, French drains, pumping from sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment may be needed to divert groundwater if significant seepage is encountered during excavation.

<u>Reuse of Excavated Soil and Bedrock</u>

- <u>Do not</u> use excavated marine clay-silt or silty sand soils for fill anywhere beneath the pavement structure or dressing slopes. Use these soils to dress slopes only below the bottom elevation of the shoulder subbase gravel.
- Marine clay-silt and silty sand may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. It may be necessary to spread out and dry portions of these soils that are excessively moist.

<u>Erosion Control</u>

- Use MaineDOT Best Management Practices February 2008 to minimize erosion of fine-grained soils found on the project site.

1.0 INTRODUCTION

The Maine Department of Transportation (MaineDOT) plans to construct a new box culvert over an unnamed tributary to the Union River at approximate station 1042+75 along the new Route 180 alignment in the Town of Ellsworth, Hancock County, Maine. We show the project location on Sheet 1, Site Location Map, appended to this report. The new structure will be 6 feet high and 10 feet wide and will have a rail-to-rail width of 32 feet. Current plans include 11-foot travel lanes, 5-foot shoulders and accommodation for guardrail, construction of integral concrete culvert headwalls and toe walls, and armoring the embankments with riprap.

We conducted subsurface investigations at the site to develop geotechnical recommendations for the structure replacement. This report summarizes our findings, discusses our evaluation of the subsurface conditions and presents our geotechnical recommendations for design and construction of the new box culvert.

2.0 GEOLOGIC SETTING

The ground surface rises north and south of the project site which lies at the base of an ancient erosion down-cut channel. The channel is an unnamed tributary to the Union River roughly one-half mile downstream from the project site. The Maine Geologic Survey (MGS) "Surficial Geology of Ellsworth Quadrangle, Maine, Open-File No. 82-3" (1982) indicates that surficial soils in the vicinity of Beaver Bridge consists primarily of glacial-marine deposits with a nearby glacial till soil unit contact. The predominant native soil units at the site based on our subsurface explorations are glaciomarine which consist of silt, clay and sands, and glacial till.

According to the "Bedrock Geologic Map of Maine" MGS (1985), the bedrock at the Beaver Bridge site consists of Ordovician-Precambrian age interbedded pelite and sandstone of the Ellsworth Formation. Locally we have identified the bedrock as chlorite schist.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions at the site by drilling two test borings, BB-EBB-101 and BB-EBB-102. Northern Test Borings of Gorham, Maine, conducted the borings on August 25 and 26, 2010. We show the boring locations and soil profile on Sheet 2, Boring Location Plan and Sheet 3, Interpretive Soil Cross Section. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on Sheet 4, Boring Logs, and in Appendix A, Boring Logs, provided at the end of this report.

The MaineDOT geotechnical team member selected the boring locations and drilling methods, designated the type and depth of sampling techniques, and identified field and laboratory testing requirements. A MaineDOT New England Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered on the field logs and staked the boring locations. The boring locations were later picked up by MaineDOT survey.

We used solid stem auger and cased wash boring techniques to conduct the borings. Soil samples were obtained, where possible, at 5-foot intervals using Standard Penetration Test (SPT) methods. The standard penetration resistances, or N-values, discussed in this report are corrected for average hammer energy transfer. We compute the corrected or, N_{60}-values, by applying an average hammer energy transfer factor of 0.713 to the raw field N-values obtained with the Northern Test Boring drill rig.

4.0 LABORATORY TESTING

We conducted a laboratory soil testing program on selected samples recovered from the test borings to evaluate soil classification, material reuse, and subgrade soil properties. Laboratory testing consisted of five standard grain size analyses with natural water contents tests, three with hydrometer analysis, and three Atterberg limits tests. We present results of laboratory testing in Appendix B, Laboratory Test Data. The AASHTO and Unified Soil Classification System (USCS) soil classifications and water content data are also presented on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

Regional surficial geology maps show that the site is situated in an area of predominantly glacial-marine clay-silts and sands. We encountered native glaciomarine soils over glacial till in the borings and we terminated the borings with ten-foot long bedrock cores. We present a profile depicting the generalized soil stratigraphy at the site on Sheet 3, Interpretive Soil Cross Section, provided at the end of this report. A summary description of the subsurface conditions follows.

5.1 Glaciomarine Silt

We observed 9.0 feet and 3.1 feet of native glaciomarine silt soil overlying glacial till in BB-EBB-101 and BB-EBB-102, respectively. The glaciomarine unit consists of clayey silt with trace fine sand, or silt with some clay and trace fine to coarse sand and trace gravel, or silt with some clay and trace fine to medium sand. SPT N_{60}-values in the glaciomarine silts ranged between approximately 5 and 13 bpf, indicating that the silt is medium stiff to stiff and also over-consolidated in consistency.

The tested silt samples had liquid limits ranging between approximately 29 and 33 and plasticity indices ranging between approximately 10 and 12. Natural water contents of the tested glaciomarine silt samples ranged between approximately 21 and 27 percent. Grain size analyses indicate that the silt soils are classified as A-4 and A-6 by the AASHTO classification system and CL by the Unified Soil Classification System.

5.2 Glacial Till

The glacial till found in the borings generally comprised of silty fine to coarse sand with little gravel, or silt with some gravel and some fine to coarse sand. The till unit typically has occasional cobbles. The thickness of this soil unit ranged between approximately 6.6 to 11.7 feet. SPT N_{60}-values ranged from 19 to 49 bpf, indicating the till deposit is medium dense to dense in consistency. We observed the glacial till unit over bedrock in each of the borings.

The glacial till samples had water contents of approximately 11 percent. Grain size analyses conducted on selected samples of the till soils indicate that the soils are classified as A-4 by the AASHTO Classification System and SM under the Unified Soil Classification System.

5.3 Bedrock

We encountered bedrock at approximate depths ranging between 15.2 and 15.6 feet below ground surface (bgs). Regionally, the bedrock is mapped by MGS as interbedded pelite and sandstone of the Ellsworth Formation.

We visually identified the local bedrock cores at all the boring locations as a grey, fine-grained, chlorite schist that is moderately hard, fresh to slightly weathered, highly foliated, with very close to close open to tight joints. The bedrock contains fractures that are oriented from horizontal to near vertical and have minor silt in-filling and iron-staining. The bedrock also contains occasional quartz intrusions. We determined that the rock quality designation (RQD) of the bedrock ranged from 38 to 72 percent which correlates to a poor to fair rock mass quality.

The table below summarizes the top of bedrock elevations at the boring locations:

Boring	Station	Depth to Bedrock (feet bgs)	Elev. of Apparent Bedrock Surface (feet)
BB-EBB-101	1042+16.2, 14.5 RT	15.6	99.7
BB-EBB-102	1042+87.7, 17.2 RT	15.2	100.8

Bedrock Depth and Elevation at the Boring Locations

5.4 Groundwater

We observed groundwater levels ranging between approximately 2.5 and 4.0 feet bgs in boring BB-EBB-101 and BB-EBB-102, respectively. However, the groundwater level will fluctuate with seasonal changes, runoff, and adjacent construction activities.

For a more detailed description of the subsurface conditions, please refer to Appendix A, Boring Logs, attached to this report.

6.0 FOUNDATION ALTERNATIVES

The project team considered two alternate designs: 1) construct a 6-foot diameter reinforced concrete pipe and 2) 6 foot high by 10 foot wide concrete box culvert on top of a base of crushed stone wrapped in geotextile due to poor soil conditions with one foot of streambed soil placed in the bottom of the culvert. The project team selected the box culvert alternate. The following section presents geotechnical design recommendations for the concrete box culvert alternate.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The proposed structure will consist of a 6-foot high by 10-foot wide concrete box culvert filled with one foot of streambed soil. The new structure will have a rail-to-rail width of 32 feet. Current plans include 11-foot travel lanes, 5-foot shoulders, accommodation for guardrail, construction of integral concrete culvert headwalls, toe walls, and armoring the embankments with riprap. The base of the bottom slab will be buried approximately 2.0 feet. The design methodology used in the following evaluation is referenced from the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010. See Appendix C, Calculations, for supporting documentation for the design parameters discussed below.

7.1 Box Culvert Design and Construction

Precast concrete boxes are typically detailed on the contract plans with only the basic layout and required hydraulic opening so that the contractor may choose among available proprietary products. The manufacturer is responsible for the design of the structure in accordance with Special Provision 534, Precast Structural Concrete Arches, Box Culverts, in Appendix D which includes determination of the wall thickness, haunch thickness and reinforcement. 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 Section 3.6, Earth Loads, of the BDG to design earth loads from the soil envelope. The Soil Type properties are as follows: ϕ = 32 degrees, γ = 125 pcf.

The concrete box culvert will be supplier-designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1, and LRFD Section 12. The culverts will be constructed in general conformance with BDG Section 8, Buried Structures, and Special Provision 534, Precast Structural Concrete Arches, Box Culverts.

The box culvert will be bedded on a two foot thick layer of ¾-inch crushed stone reinforced with geogrid and wrapped in geotextile fabric. The soil envelope and backfill shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill with a maximum particle size of 4.0 inches. The crushed stone bedding should be placed in 12-inch thick maximum lifts and compacted with a minimum of four passes of a large walk-behind compactor. The granular borrow backfill should be placed in lifts 6 to 8 inches thick loose measure and compacted to manufacturer's specifications, but in no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Culvert Headwalls

We recommend integral concrete headwalls with nominal 1-foot by 1-foot dimensions to prevent crushed stone slope protection from dropping or eroding into the waterway. Culvert headwalls larger than the nominal 1-foot by 1-foot dimension are essentially retaining walls sharing a continuous base slab and should be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, and 11.5.5 and 11.6. 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 wall shall also 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 the table below. For this culvert, the live load surcharge is 250 psf which is equivalent to two feet of soil.

Retaining 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 feet
5	5.0	2.0
10	3.5	2.0
$\geq$ 20	2.0	2.0

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, equal to 0.31. This assumes level backslope. The earth pressure coefficient may change if backslope conditions are different.

Footings for any headwall or wingwall constructed independently of the box culvert should be placed no less than 2 feet below the maximum anticipated depth of scour.

7.3 Box Culvert Bearing Resistance

In our analysis we determined the factored bearing resistance at the strength limit state for the box culvert on compacted fill should not exceed 6.0 ksf. However, when analyzing box bottom slabs for the service limit state as allowed in LRFD C10.6.2.6.1., we determined that a factored bearing resistance of 2 ksf should be used to control settlement based on presumptive bearing resistance values. Thus in this case, the service limit state bearing resistance controls. In no instance shall the bearing stress exceed the nominal resistance of the structure concrete, which may be taken as 0.3 $f'c$.

7.4 Settlement

We have evaluated the potential for settlement at the Ellsworth box culvert site. Roughly ten feet of embankment fill soil will be placed over the native glaciomarine silt soil to construct the box culvert approach embankment at approximate STA 1042+75. The new embankment fill loads and densification of the fill materials during construction will result in ground surface settlement and consolidation of the underlying soils. We estimate that 1 to 2 inches of consolidation settlement will occur in the native glaciomarine silt layer as the new approach embankments are constructed. This settlement estimate also assumes that the contractor exercises careful construction practices that minimize or prevent disturbance of the clay-silt subgrade soil. More settlement may result if the subgrade soil is disturbed and weakened by construction activity.

Because this settlement may impact pavement performance, we recommend constructing the fill embankment as early as possible in the project. We also recommend delaying the paving operation over this section for a minimum of 30 days after top of gravel grade is achieved. This will allow time for consolidation settlement to occur. We anticipate that post-construction settlement will be negligible since the glaciomarine silts are over-consolidated.

7.5 Embankment Stability

Plans require construction of a 10 foot high fill embankment over approximately 10 feet of glaciomarine clay-silt. We conducted slope stability analyses to determine the Factor of Safety (FS) against embankment slope failure considering site conditions and the proposed embankment configuration. Our analyses resulted in a FS of approximately 1.8 which is acceptable.

7.6 Scour Protection

The box culvert will be fitted with integral concrete headwalls to prevent crushed stone slope protection from dropping or eroding into the waterway, and inlet and outlet section seepage cutoff walls below the culvert to provide scour protection per BDG 8.3.1. We recommend that the bridge approach slopes be armored with a 3-foot thick layer of riprap adjacent to the culvert openings. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot thick layer of bedding material conforming to Standard Specification 703.19, Granular Borrow for Underwater Backfill and as shown in Standard Detail 610(02). Riprap shall meet the requirements of 703.26, Plain and Hand Laid Riprap, of Special Provision 703, Aggregates. The riprap slopes should also be constructed in accordance with Special Provision 610, Stone Fill, Riprap, Stone Blanket, and Stone Ditch Protection and be constructed no steeper than a maximum 1.75:1 (H:V) extending from the edge of roadway down to the existing ground surface. The toe of riprap sections shall be constructed 1 foot below the streambed elevation.

7.7 Frost Protection

We have evaluated the potential frost depth at the Ellsworth site. Based on State of Maine frost depth maps, MaineDOT Bridge Design Guide (BDG) Figure 5-1, the site has a design-freezing index of approximately 1350 F-degree days. Considering site soils and natural water contents, this correlates to a frost depth of 4.0 feet at this site. We also considered frost depth projections computed by Modberg software developed by the US Army Cold Regions

Research and Engineering Laboratory. The results of the Modberg frost depth model indicate a potential frost depth of 2.8 feet. Consequently, if spread footings are used, we recommend that any spread footing or leveling pads constructed at the site be founded a minimum of 4.0 feet below finished exterior grade for frost protection.

7.8 Seismic Design Considerations

In accordance with LRFD Article 12.6.1, Loading, earthquake loading should only be considered where buried structures cross active faults. Since there are no known active faults in Maine, no seismic analysis is required.

7.9 Construction Considerations

7.9.1 Excavation

Construction of the new concrete box culvert will require soil excavation. Earth support systems may be required. The native glaciomarine soils at the site will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. **It is imperative that the contractor minimize aggressive excavation action or equipment movement over the clay-silt soil. This will disturb and/or soften the subgrade soil and may create embankment stability problems or result in excessive settlement.** We recommend that the contractor protect any subgrade from exposure to water and any unnecessary construction traffic. If disturbance and rutting occur, we recommend that the contractor remove and replace the disturbed materials with compacted gravel borrow.

If encountered, unsuitable soils should also be excavated from the subgrade to a depth of one foot and replaced with compacted granular borrow. Granular borrow should conform to MaineDOT Standard Specification 703.19, Granular Borrow. The granular borrow should be compacted to 92 percent of the Modified Proctor maximum dry density (AASHTO T-180).

7.9.2 Dewatering

The native soils within the project area are both poorly drained and moderately to highly frost susceptible. In some locations, these soil units may be saturated and significant water seepage may be encountered during excavation. The groundwater may be trapped in layers and lenses of coarse-grained soil overlying or between glaciomarine sediments. We anticipate that this seepage will be temporary but there may be localized sloughing and near-surface instability of some soil slopes.

The contractor should control groundwater and surface water infiltration to permit construction in-the-dry. We recommend that the contractor use cofferdams, temporary ditches, sumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater if significant seepage is encountered during construction. We also recommend using French drains daylighted to nearby ditches if significant seepage is encountered in the subgrade along the construction areas. If the amount of seepage is significant, we anticipate that pumping from sumps will likely be needed to control the water.

7.9.3 Reuse of Excavated Soil

We do not recommend using any clay-silt soil excavation as fill beneath the pavement structure. Excavated clay-silt soils may be used as common borrow in accordance with MaineDOT Standard Specification Sections 203 and 703. Contractors should expect that, prior to placement and compaction, it may be necessary to spread out and dry portions of these soils that are excessively moist. This soil may also be used for dressing slopes, but only below the bottom elevation of the shoulder subbase gravel.

7.9.4 Erosion Control Recommendations

The fine-grained soils along the project are susceptible to erosion. We recommend using appropriate erosion control measures during construction as described in the MaineDOT Best Management Practices February 2008 guidelines to minimize erosion of the fine-grained soils at the site.

8.0 CLOSURE

This report has been prepared for use by the MaineDOT Bridge Program for specific application to the construction of Beaver Bridge over an unnamed tributary to Union River at approximate STA 1042+75 of the new Route 180 alignment in Ellsworth, Maine. We have prepared the report in accordance with generally accepted soil 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 completed at discrete locations on the project 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 recommend that we be provided the opportunity for a general review of the final design drawings and specifications in order that we may verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

REFERENCES

AASHTO, (2010), LRFD Bridge Design Specifications, Fifth, AASHTO, Washington, D.C.

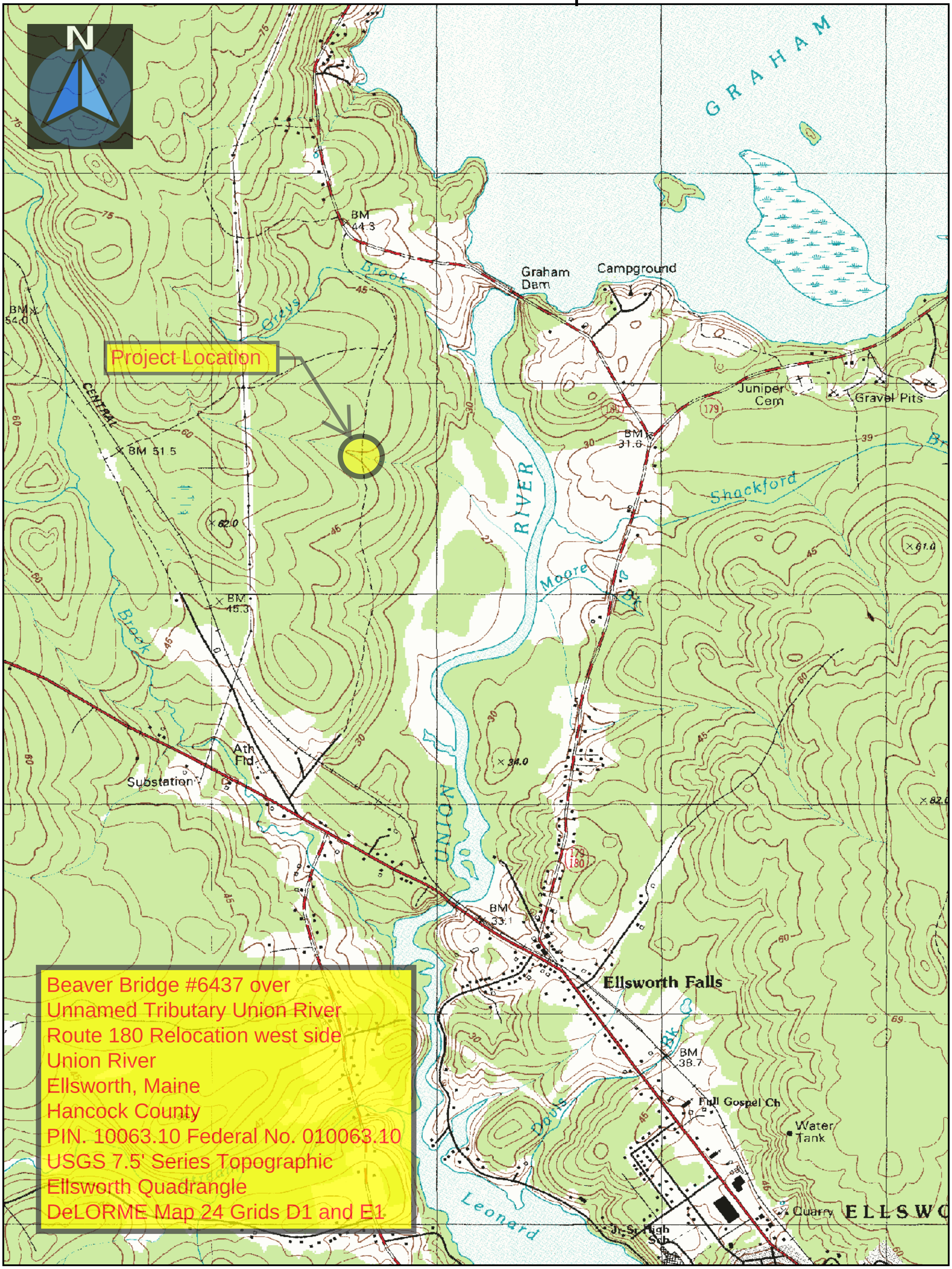
Bowles, Joseph E. (1996), Foundation Analysis and Design, Fifth Edition, McGraw-Hill, New York, NY.

MaineDOT, (2003), Bridge Design Guide, MaineDOT Bridge Program, Augusta, ME with various Interims.

Winterkorn, H.F., and Fang, H. (1975), Foundation Engineering Handbook, Van Nostrand Reinhold Company, Inc., New York, NY.

Sheets

Location Map



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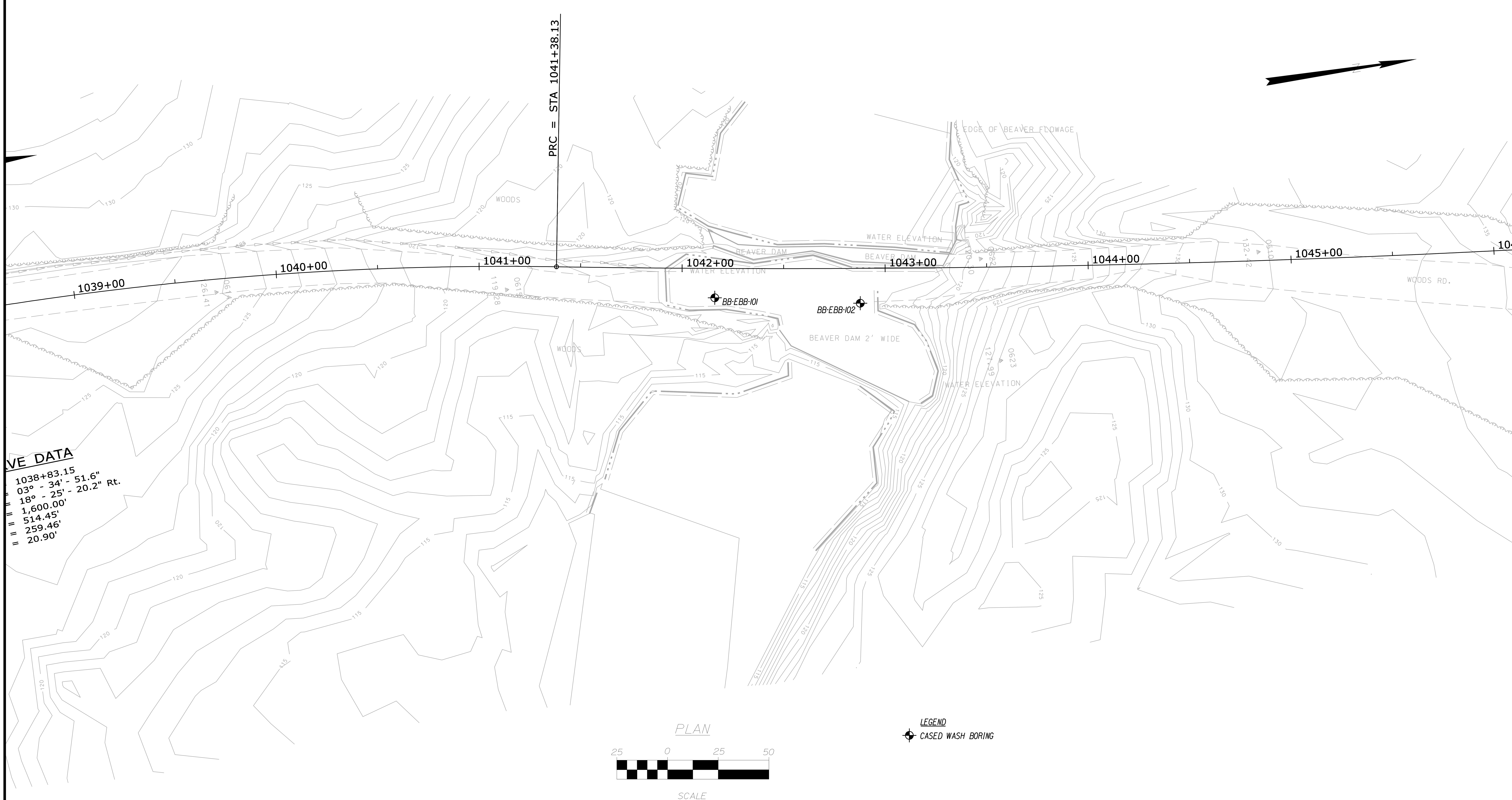


STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PROJECT NO. 10063.10

PIN 10063.10

HIGHWAY PLANS

BRIDGE NO. 6437

PROJ. MANAGER	ERNIE MARTIN	BY	DATE
DESIGN-DETAILED	M.MOREAU	T.WHITE	JAN 2011
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

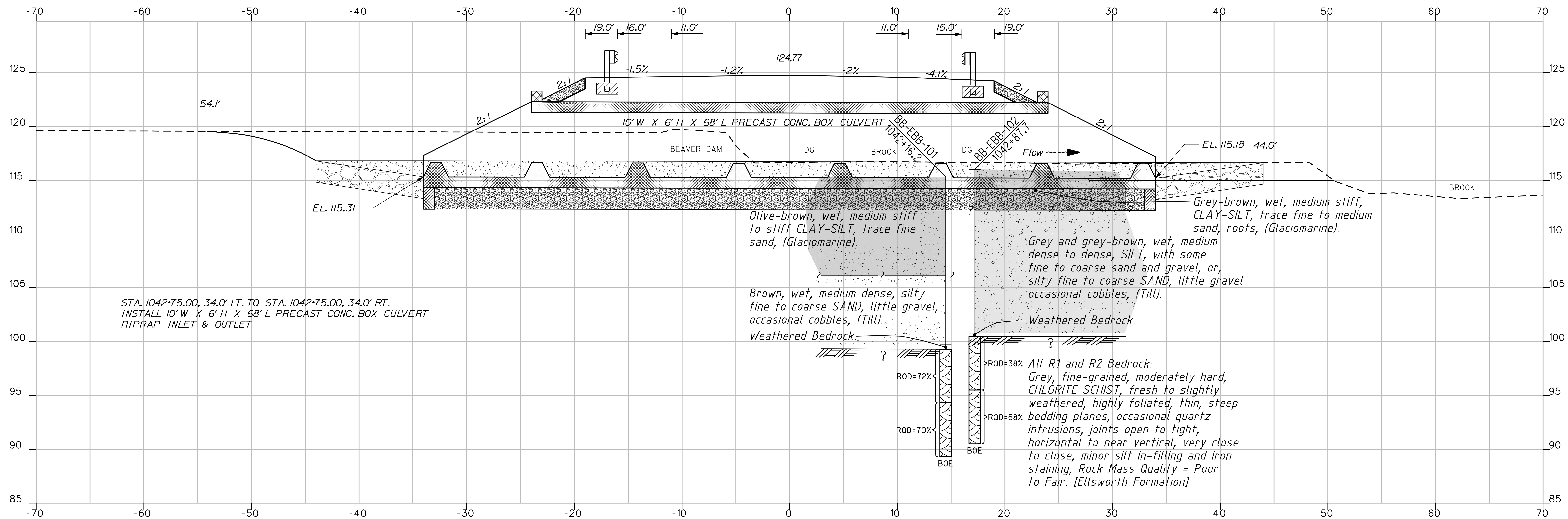
BEAVER BRIDGE
UNNAMED TRIBUTARY UNION RIVER
ELLSWORTH HANCOCK COUNTY

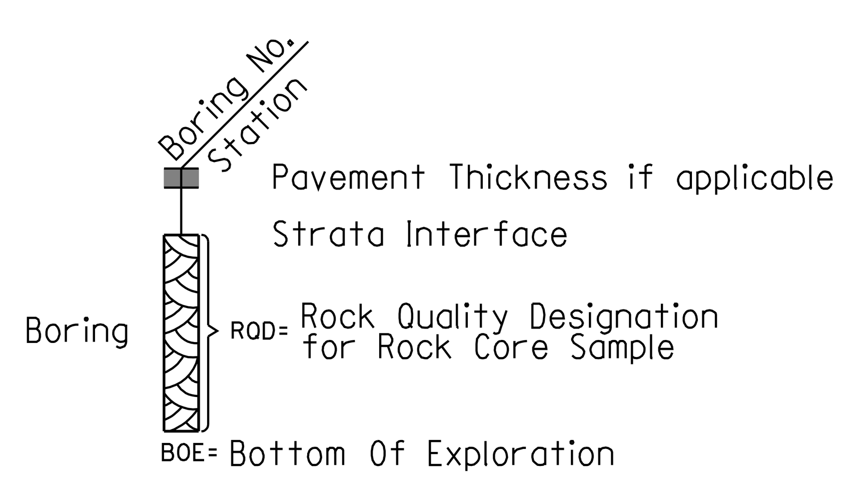
BORING LOCATION PLAN

SHEET NUMBER

2

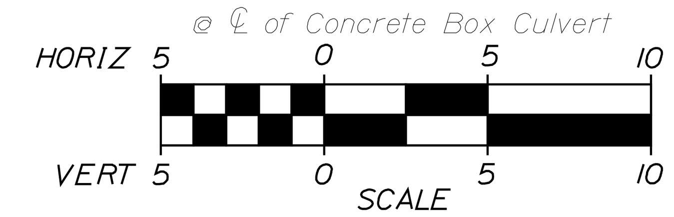
OF 4





1042+75.00

@ ℄ of Concrete Box Culvert



Notes: This generalized interpretive soil cross section 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.

Bedrock and soil strata are not connected from BB-EBB-101 to BB-EBB-102 due to the 70 plus feet distance between the two borings.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PROJECT NO. 10063.10

BRIDGE NO. 6437 PIN 10063.10 HIGHWAY PLANS

PROJ. MANAGER	ERNIE MARTIN	BY	DATE
DESIGN-DETAILED	M.MOREAU	T.WHITE	JAN 2011
CHECKED-REVIEWED	-----	-----	-----
DESIGN2-DETAILED2	-----	-----	-----
DESIGN3-DETAILED3	-----	-----	-----
REVISIONS 1	-----		-----
REVISIONS 2	-----		-----
REVISIONS 3	-----		-----
REVISIONS 4	-----		-----
FIELD CHANGES	-----		-----

SIGNATURE

P.E. NUMBER

DATE

BEAVER BRIDGE
UNNAMED TRIBUTARY UNION RIVER
ELLSWORTH HANCOCK COUNTY

INTERPRETIVE SUBSURFACE CROSS SECTION

SHEET NUMBER

3

OF 4

Filename: ...\msta\004_BORING LOGS EBB1.dgn   Division: GEOTECH   Username: terry.white   Date:4/7/2011

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union		Location: Ellsworth, Maine		Boring No.:	BB-EBB-101
				PIN:	10063.10
Driller:	Northern Test Boring	Elevation (ft.)	115.3	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Ryan	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/26/10; 07:00-11:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1042+16.2, 14.5 Rt.	Casing ID/OD:	HW	Water Level*:	2.5 ft bgs.
Hammer Efficiency Factor: 0.713		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/20	0.00 - 2.00	2/4/7/9	11	13	SSA		Olive-brown, wet, stiff, CLAY-SILT, trace fine sand, (Glaciomarine).	G#237545 A-6, CL WC=20.5% LL=33 PL=21 PI=12
								112.30	3.00	
5	2D	24/22	5.00 - 7.00	2/2/5/17	7	8			Olive-brown, wet, medium stiff, CLAY-SILT, trace fine to coarse sand, trace gravel, (Glaciomarine).	G#237546 A-4, CL WC=26.5% LL=29 PL=19 PI=10
								106.30	9.00	
10	3D	24/24	10.00 - 12.00	9/8/11/20	19	23	15		Brown, wet, medium dense, silty, fine to coarse SAND, little gravel, occasional cobbles, (Till).	G#237547 A-4, SM WC=11.4%
							40			
							46			
							54		Roller Coned ahead to 15.0 ft bgs.	
							60			
15	MD	6/0	15.00 - 15.50	50(6")	---		RC	99.70	Failed sample attempt. 15.60	
	R1	60/60	16.00 - 21.00	RQD = 72%			NQ-2	99.30	Roller Coned ahead to 16.0 ft bgs through weathered bedrock. 16.00	
									Top of Intact Bedrock at Elev. 99.3 ft.	
									R1 and R2 Bedrock: Grey, fine-grained moderately hard, CHLORITE SCHIST, fresh to slightly weathered, highly foliated, thin steep bedding planes, occasional minor quartz intrusions with one large quartzite vein at 23.8' to 25.0' bgs, joints occur from horizontal to near vertical, open to tight, very close to close, with minor silt in-filling and iron staining. Rock Mass Qality is Fair. [Ellsworth Formation]	
20									R1:Core Times (min:sec) 16.0-17.0 ft (5:00) 17.0-18.0 ft (5:20) 18.0-19.0 ft (5:00) 19.0-20.0 ft (5:10) 20.0-21.0 ft (5:30) 100% Recovery	
	R2	60/57	21.00 - 26.00	RQD = 70%					R2:Core Times (min:sec) 21.0-22.0 ft (5:00) 22.0-23.0 ft (4:00) 23.0-24.0 ft (4:30) 24.0-25.0 ft (4:20) 25.0-26.0 ft (4:30) 95% Recovery	
25								89.30	26.00	
									Bottom of Exploration at 26.00 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:

Auto Hammer #185

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

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

Page 1 of 1

Boring No.: BB-EBB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union		Location: Ellsworth, Maine		Boring No.:	BB-EBB-102
				PIN:	10063.10
Driller:	Northern Test Boring	Elevation (ft.)	116.0	Auger ID/OD:	5" Solid Stem
Operator:	Nick/Ryan	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	Diedrich D-50 Track	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	8/25/10; 12:30-16:30	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	1042+87.7, 17.2 Rt.	Casing ID/OD:	HW	Water Level*:	4.0 ft bgs.
Hammer Efficiency Factor: 0.713		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/16	0.00 - 2.00	2/2/2/2	4	5	SSA	115.60	TOPSOIL, (Grass and Roots). 0.40	
									Grey-brown, wet, medium stiff, CLAY-SILT, trace fine to medium sand, roots, (Glaciomarine).	G#237548 A-6, CL WC=23.1% LL=30 PL=18 PI=12
								112.50	3.50	
5	2D	24/18	5.00 - 7.00	6/9/7/12	16	19			Grey-brown, wet, medium dense, SILT, some fine to coarse sand, some gravel, occasional cobbles, (Till).	G#237549 A-4, SM WC=11.0%
									Cobble from 8.5-8.8 ft bgs.	
10								106.00	10.00	
	3D	24/20	10.00 - 12.00	14/17/24/26	41	49	30		Grey, wet, dense, silty fine to coarse SAND, little gravel, occasional cobbles, (Till). Roller Coned ahead to 15.0 ft bgs.	
							56			
							52			
							40			
							84			
15	MD	1.2/0	15.00 - 15.10	25(1.2")	---		NQ-2	100.80	Failed sample attempt. 15.20	
	R1	60/48	15.50 - 20.50	RQD = 38%				100.50	Roller Coned ahead to 15.5 ft bgs through weathered bedrock. 15.50	
									Top of Intact Bedrock at Elev. 100.5 ft.	
									R1 and R2 Bedrock: Grey, fine-grained moderately hard, CHLORITE SCHIST, fresh to slightly weathered, highly foliated, thin steep bedding planes, occasional minor quartz intrusions, joints occur from horizontal to near vertical, open to tight, very close to close, with minor silt in-filling and iron staining. Rock Mass Qality is Poor to Fair. [Ellsworth Formation]	
20	R2	60/52	20.50 - 25.50	RQD = 58%					R1:Core Times (min:sec) 15.5-16.5 ft (5:00) 16.5-17.5 ft (5:00) 17.5-18.5 ft (5:20) 18.5-19.5 ft (5:15) 19.5-20.5 ft (5:20) 80% Recovery	
									R2:Core Times (min:sec) 20.5-21.5 ft (4:30) 21.5-22.5 ft (4:40) 22.5-23.5 ft (5:00) 23.5-24.5 ft (5:00) 24.5-25.5 ft (4:50) 87% Recovery	
25								90.50	25.50	
									Bottom of Exploration at 25.50 feet below ground surface.	
30										
35										
40										
45										
50										

Remarks:

Auto Hammer #185

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

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

Page 1 of 1

Boring No.: BB-EBB-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
PROJECT NO. 10063.10
BRIDGE NO. 6437 | PIN 10063.10 | HIGHWAY PLANS

PROJ. MANAGER	ERNIE MARTIN	BY	DATE
DESIGN-DETAILED	M.MOREAU	T.WHITE	JAN 2011
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

BEAVER BRIDGE
UNNAMED TRIBUTARY UNION RIVER
ELLSWORTH HANCOCK COUNTY

BORING LOGS

SHEET NUMBER
4
OF 4

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

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

Desired Soil Observations: (in this order)

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

Desired Rock Observations: (in this order)

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

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

Sample Container Labeling Requirements:

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

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union River
Location: Ellsworth, Maine

Boring No.: BB-EBB-101
PIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	115.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/26/10; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1042+16.2, 14.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	2.5 ft bgs.
Hammer Efficiency Factor:	0.713	**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	1D	24/20	0.00 - 2.00	2/4/7/9	11	13	SSA			Olive-brown, wet, stiff, CLAY-SILT, trace fine sand, (Glaciomarine).	G#237545 A-6, CL WC=20.5% LL=33 PL=21 PI=12
								112.30		3.00	
5	2D	24/22	5.00 - 7.00	2/2/5/17	7	8				Olive-brown, wet, medium stiff, CLAY-SILT, trace fine to coarse sand, trace gravel, (Glaciomarine).	G#237546 A-4, CL WC=26.5% LL=29 PL=19 PI=10
								106.30		9.00	
10	3D	24/24	10.00 - 12.00	9/8/11/20	19	23	15			Brown, wet, medium dense, silty, fine to coarse SAND, little gravel, occasional cobbles, (Till).	G#237547 A-4, SM WC=11.4%
							40				
							46				
							54			Roller Coned ahead to 15.0 ft bgs.	
							60				
15	MD	6/0	15.00 - 15.50	50(6")	---		RC	99.70		Failed sample attempt. 15.60	
	R1	60/60	16.00 - 21.00	RQD = 72%			NQ-2	99.30		Roller Coned ahead to 16.0 ft bgs through weathered bedrock. 16.00	
										Top of Intact Bedrock at Elev. 99.3 ft.	
										R1 and R2 Bedrock: Grey, fine-grained moderately hard, CHLORITE SCHIST, fresh to slightly weathered, highly foliated, thin steep bedding planes, occasional minor quartz intrusions with one large quartzite vein at 23.8' to 25.0' bgs, joints occur from horizontal to near vertical, open to tight, very close to close, with minor silt in-filling and iron staining. Rock Mass Qality is Fair. [Ellsworth Formation]	
20										R1:Core Times (min:sec) 16.0-17.0 ft (5:00) 17.0-18.0 ft (5:20) 18.0-19.0 ft (5:00) 19.0-20.0 ft (5:10) 20.0-21.0 ft (5:30) 100% Recovery	
	R2	60/57	21.00 - 26.00	RQD = 70%						R2:Core Times (min:sec) 21.0-22.0 ft (5:00)	
25											

Remarks:

Auto Hammer #185

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:	Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union River			**Boring No.:**	BB-EBB-101
Location:	Ellsworth, Maine			**PIN:**	10063.10
Driller:	Northern Test Boring	**Elevation (ft.)**	115.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/26/10; 07:00-11:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1042+16.2, 14.5 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	2.5 ft bgs.
Hammer Efficiency Factor:	0.713	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								89.30		22.0-23.0 ft (4:00) 23.0-24.0 ft (4:30) 24.0-25.0 ft (4:20) 25.0-26.0 ft (4:30) 95% Recovery	
										26.00	
										Bottom of Exploration at 26.00 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #185

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

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

Boring No.: BB-EBB-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union River
Location: Ellsworth, Maine

Boring No.: BB-EBB-102
PIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	116.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/25/10; 12:30-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1042+87.7, 17.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.
Hammer Efficiency Factor:	0.713	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:

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

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	24/16	0.00 - 2.00	2/2/2/2	4	5	SSA	115.60		TOPSOIL, (Grass and Roots). 0.40	G#237548 A-6, CL WC=23.1% LL=30 PL=18 PI=12
										Grey-brown, wet, medium stiff, CLAY-SILT, trace fine to medium sand, roots, (Glaciomarine).	
								112.50		3.50	
5	2D	24/18	5.00 - 7.00	6/9/7/12	16	19				Grey-brown, wet, medium dense, SILT, some fine to coarse sand, some gravel, occasional cobbles, (Till).	G#237549 A-4, SM WC=11.0%
										Cobble from 8.5-8.8 ft bgs.	
10	3D	24/20	10.00 - 12.00	14/17/24/26	41	49	30	106.00		10.00 Grey, wet, dense, silty fine to coarse SAND, little gravel, occasional cobbles, (Till). Roller Coned ahead to 15.0 ft bgs.	
							56				
							52				
							40				
							84				
15	MD	1.2/0	15.00 - 15.10	25(1.2")	---		NQ-2	100.80		Failed sample attempt. 15.20	
	R1	60/48	15.50 - 20.50	RQD = 38%				100.50		Roller Coned ahead to 15.5 ft bgs through weathered bedrock. 15.50	
										Top of Intact Bedrock at Elev. 100.5 ft.	
										R1 and R2 Bedrock: Grey, fine-grained moderately hard, CHLORITE SCHIST, fresh to slightly weathered, highly foliated, thin steep bedding planes, occasional minor quartz intrusions, joints occur from horizontal to near vertical, open to tight, very close to close, with minor silt in-filling and iron staining. Rock Mass Qality is Poor to Fair. [Ellsworth Formation]	
20	R2	60/52	20.50 - 25.50	RQD = 58%						R1:Core Times (min:sec) 15.5-16.5 ft (5:00) 16.5-17.5 ft (5:00) 17.5-18.5 ft (5:20) 18.5-19.5 ft (5:15) 19.5-20.5 ft (5:20) 80% Recovery	
25										R2:Core Times (min:sec) 20.5-21.5 ft (4:30)	

Remarks:

Auto Hammer #185

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: Beaver Bridge #6437 carries Route 180 over Unnamed Tributary Union River
Location: Ellsworth, Maine

Boring No.: BB-EBB-102
PIN: 10063.10

Driller:	Northern Test Boring	**Elevation (ft.)**	116.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Nick/Ryan	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	Diedrich D-50 Track	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	8/25/10; 12:30-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	1042+87.7, 17.2 Rt.	**Casing ID/OD:**	HW	**Water Level*:**	4.0 ft bgs.

Hammer Efficiency Factor: 0.713 **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								90.50		21.5-22.5 ft (4:40) 22.5-23.5 ft (5:00) 23.5-24.5 ft (5:00) 24.5-25.5 ft (4:50) 87% Recovery	
										25.50	
										Bottom of Exploration at 25.50 feet below ground surface.	
30											
35											
40											
45											
50											

Remarks:

Auto Hammer #185

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

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

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Ellsworth   Project Number: 10063.10

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-EBB-101, 1D	1042+16.2	14.5 Rt.	0.0-2.0	237545	1	20.5	33	12	CL	A-6	III
BB-EBB-101, 2D	1042+16.2	14.5 Rt.	5.0-7.0	237546	1	26.5	29	10	CL	A-4	IV
BB-EBB-101, 3D	1042+16.2	14.5 Rt.	10.0-12.0	237547	1	11.4			SM	A-4	III
BB-EBB-102, 1D	1042+87.7	17.2 Rt.	0.4-2.0	237548	1	23.1	30	12	CL	A-6	III
BB-EBB-102, 2D	1042+87.7	17.2 Rt.	5.0-7.0	237549	1	11.0			SM	A-4	III

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

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

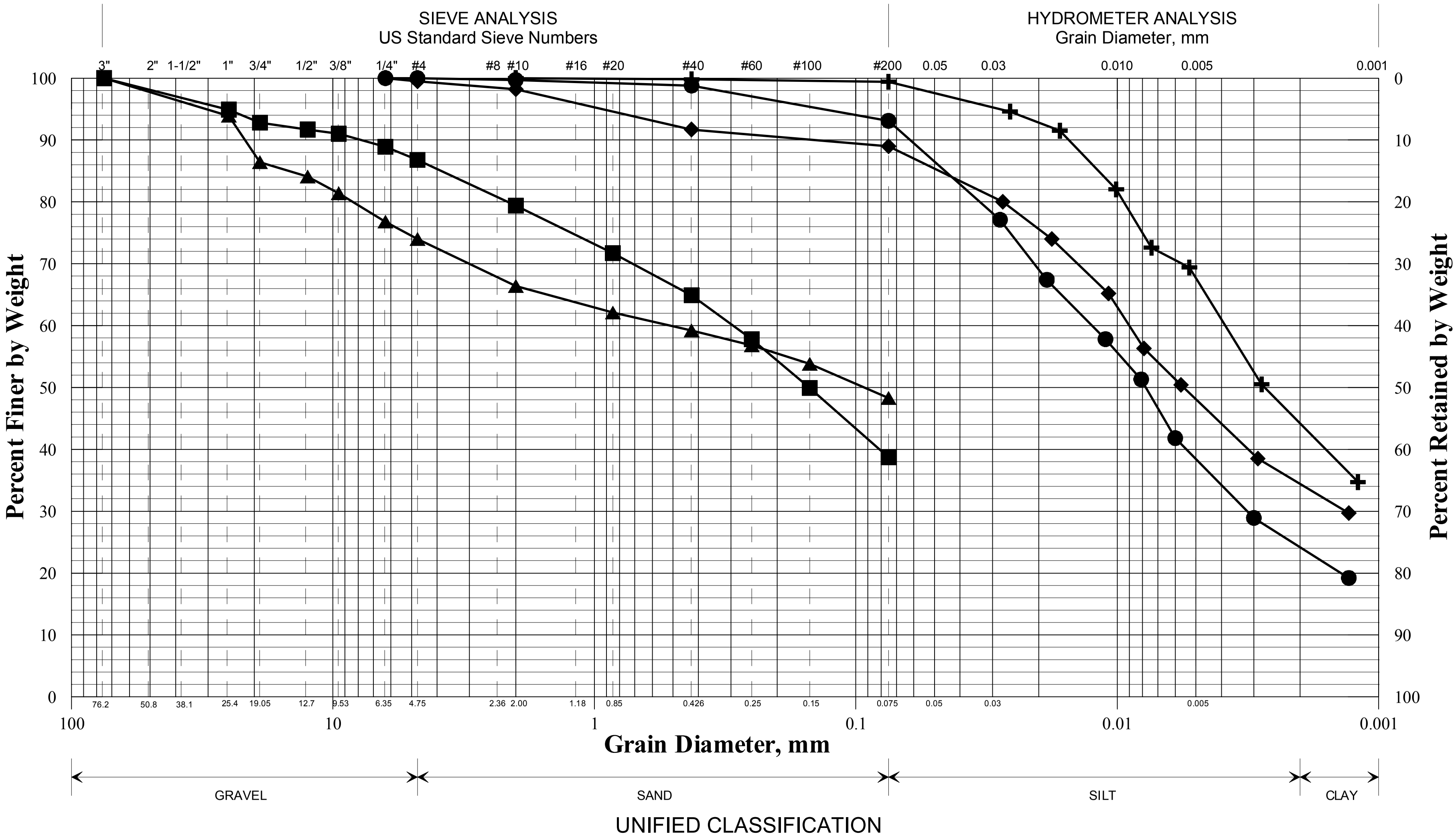
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

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

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

State of Maine Department of Transportation

GRAIN SIZE DISTRIBUTION CURVE



	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-EBB-101/1D	1042+16.2	14.5 RT	0.0-2.0	Clayey SILT, trace sand.	20.5	33	21	12
◆	BB-EBB-101/2D	1042+16.2	14.5 RT	5.0-7.0	SILT, some clay, trace sand, trace gravel.	26.5	29	19	10
■	BB-EBB-101/3D	1042+16.2	14.5 RT	10.0-12.0	Silty SAND, little gravel.	11.4			
●	BB-EBB-102/1D	1042+87.7	17.2 RT	0.0-2.0	SILT, some clay, trace sand.	23.1	30	18	12
▲	BB-EBB-102/2D	1042+87.7	17.2 RT	5.0-7.0	SILT, some gravel, some sand.	11.0			
✖									

PIN
010063.10
Town
Ellsworth
Reported by/Date
WHITE, TERRY A 11/10/2010

SHEET 1

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237545	**BB-EBB-101/1D**	**GEOTECHNICAL (DISTURBED)**	**8/26/2010**	**9/17/2010**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **1042+16.2** Offset, ft: **14.5 RT** Dbfg, ft: **0.0-2.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	
No. 10 [2.00 mm]	**100.0**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.8**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**99.4**
[0.0257 mm]	**94.6**
[0.0166 mm]	**91.5**
[0.0101 mm]	**82.0**
[0.0074 mm]	**72.6**
[0.0053 mm]	**69.4**
[0.0028 mm]	**50.5**
[0.0012 mm]	**34.7**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**33**
Plastic Limit (T 90), %	**21**
Plasticity Index (T 90), %	**12**
Specific Gravity, Corrected to 20°C (T 100)	**2.70**
Loss on Ignition (T 267) — Loss, %	
Loss on Ignition (T 267) — H2O, %	
Water Content (T 265), %	**20.5**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

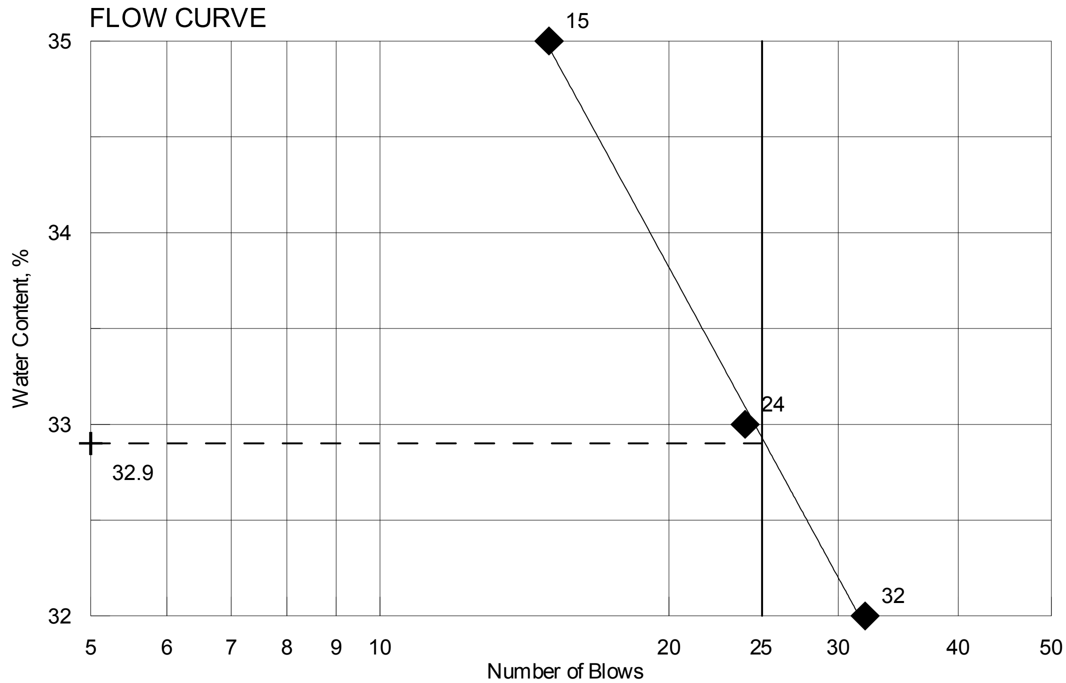
Comments:

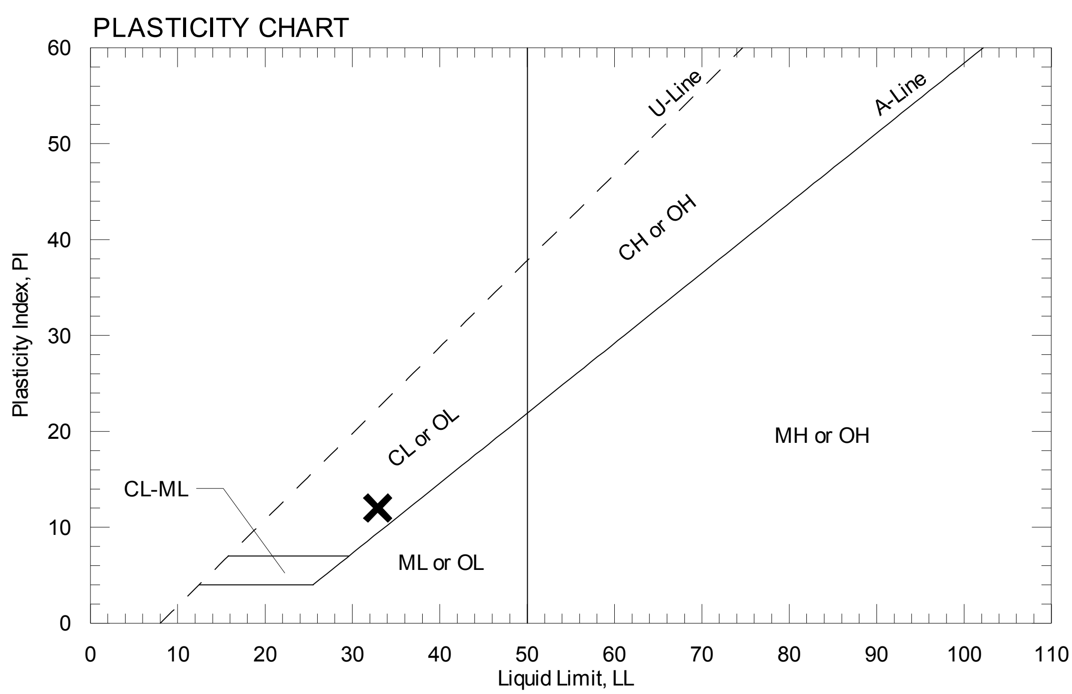
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **11/9/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237545
PIN	010063.10	Water Content, %	20.5
Sampled	8/26/2010	Plastic Limit	21
Boring No./Sample No.	BB-EBB-101/1D	Liquid Limit	33
Station	1042+16.2	Plasticity Index	12
Depth	0.0-2.0	Tested By	BBURR





Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237546	BB-EBB-101/2D	GEOTECHNICAL (DISTURBED)	8/26/2010	9/17/2010

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **1042+16.2** Offset, ft: **14.5 RT** Dbfg, ft: **5.0-7.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	**100.0**
No. 4 [4.75 mm]	**99.5**
No. 10 [2.00 mm]	**98.2**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**91.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**89.0**
[0.0274 mm]	**80.0**
[0.0178 mm]	**74.0**
[0.0108 mm]	**65.2**
[0.0079 mm]	**56.3**
[0.0057 mm]	**50.4**
[0.0029 mm]	**38.5**
[0.0013 mm]	**29.7**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Test	Result
Liquid Limit @ 25 blows (T 89), %	**29**
Plastic Limit (T 90), %	**19**
Plasticity Index (T 90), %	**10**
Specific Gravity, Corrected to 20°C (T 100)	**2.68**
Loss on Ignition (T 267) — Loss, % / H2O, %	
Water Content (T 265), %	**26.5**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

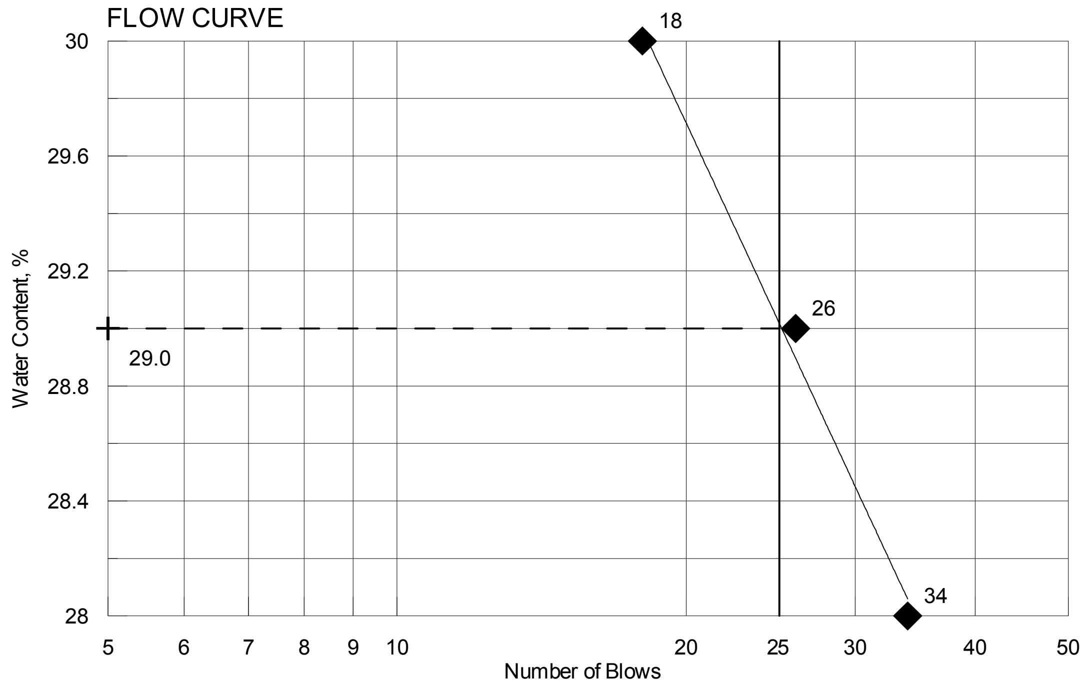
Comments:

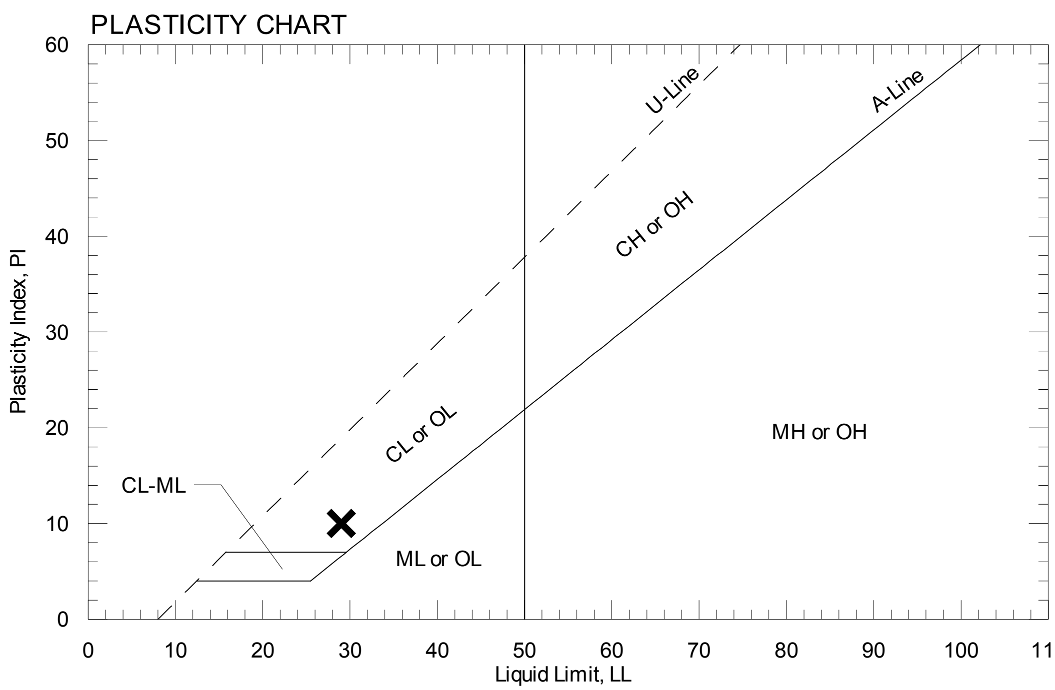
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/26/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237546
PIN	010063.10	Water Content, %	26.5
Sampled	8/26/2010	Plastic Limit	19
Boring No./Sample No.	BB-EBB-101/2D	Liquid Limit	29
Station	1042+16.2	Plasticity Index	10
Depth	5.0-7.0	Tested By	BBURR





Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
237548	**BB-EBB-102/1D**	**GEOTECHNICAL (DISTURBED)**	**8/25/2010**	**9/17/2010**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **1042+87.7** Offset, ft: **17.2 RT** Dbfg, ft: **0.0-2.0**

PIN: **010063.10** Town: **Ellsworth** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	**100.0**
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.7**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**98.8**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**93.1**
[0.0281 mm]	**77.1**
[0.0186 mm]	**67.4**
[0.0111 mm]	**57.8**
[0.0081 mm]	**51.3**
[0.0060 mm]	**41.8**
[0.0030 mm]	**28.9**
[0.0013 mm]	**19.2**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %	**30**
Plastic Limit (T 90), %	**18**
Plasticity Index (T 90), %	**12**
Specific Gravity, Corrected to 20°C (T 100)	**2.65**
Loss on Ignition (T 267) — Loss, %	
Loss on Ignition (T 267) — H2O, %	
Water Content (T 265), %	**23.1**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In. U. Shear tons/ft²	3 In. Remold tons/ft²	6 In. U. Shear tons/ft²	6 In. Remold tons/ft²	Water Content, %	Description of Material Sampled at the Various Tube Depths

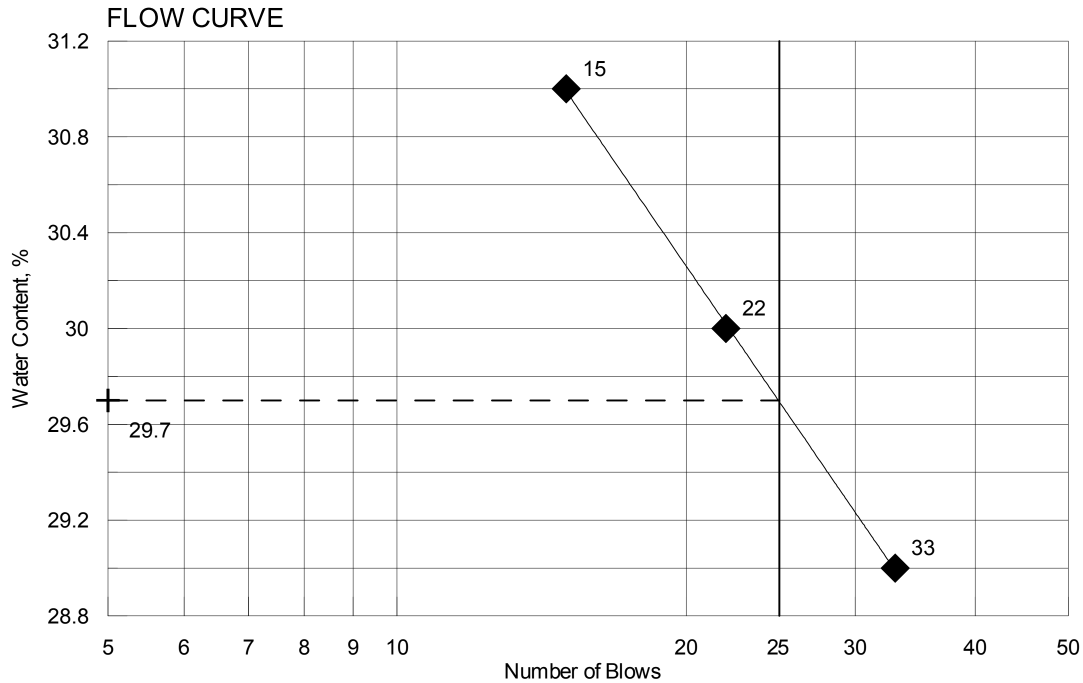
Comments:

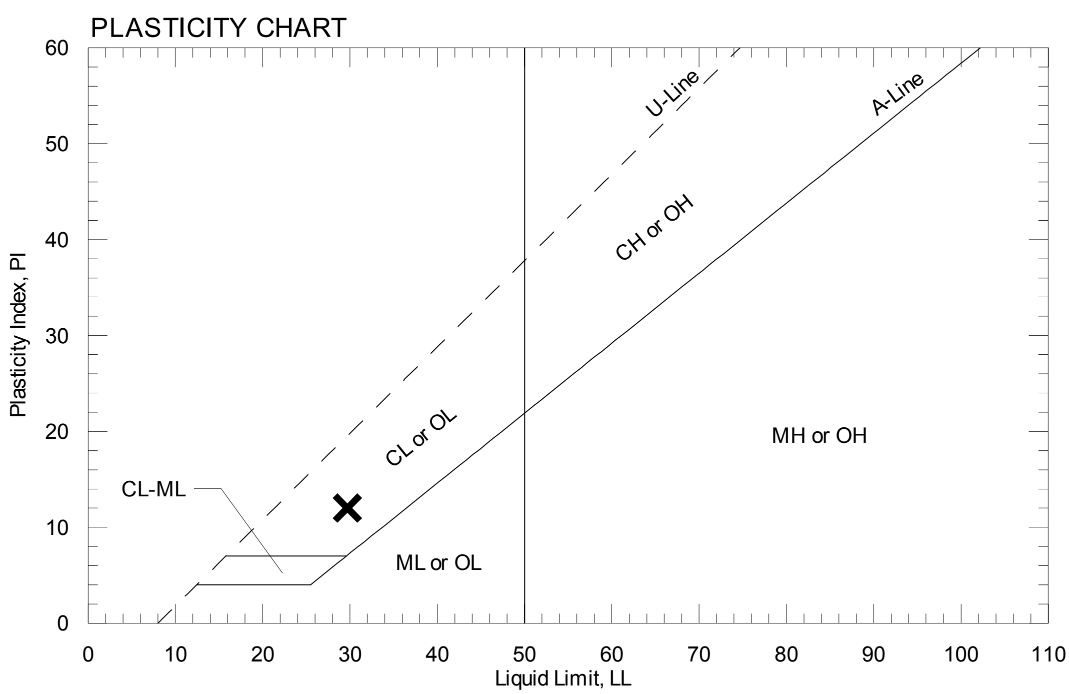
AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **11/3/2010**

Paper Copy: Lab File; Project File; Geotech File

TOWN	Ellsworth	Reference No.	237548
PIN	010063.10	Water Content, %	23.1
Sampled	8/25/2010	Plastic Limit	18
Boring No./Sample No.	BB-EBB-102/1D	Liquid Limit	30
Station	1042+87.7	Plasticity Index	12
Depth	0.0-2.0	Tested By	BBURR





Appendix C

Calculations

HEADWALL ACTIVE EARTH PRESSURE:

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

Either Rankine or Coulomb may be used for long-heeled cantilever walls where the failure surface is uninterrupted by the top of the wall stem. In general, use Rankine though.

Soil angle of internal friction: $\phi := 32deg$

Slope angle of backfill soil from horizontal: $\beta := 0deg$

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

$K_a = 0.31$

FROST PROTECTION

Method 1:

From the Maine Design Freezing Index Map:
DFI = 1350 degree-days
Site has Fine Grained Soils With Wn 20%

From the 2003 Bridge Design Guide Table 5-1:

$Frost\_depth := [0.5\cdot(48.5in - 46.6in) + 46.6in]$

$Frost\_depth = 47.55\cdot in$

$Frost\_depth = 3.96\cdot ft$

Method 2:

```
                        ------------------------
                        --- ModBerg Results ---
                        -----------------------

    Project Location: Ellsworth, Maine

    Air Design Freezing Index        =  1256 F-days
    N-Factor                         =  0.70
    Surface Design Freezing Index    =   879 F-days
    Mean Annual Temperature          =  44.6 deg F
    Design Length of Freezing Season =  126 days

    ---------------------------------------------------------
    Layer
    #:Type            t    w%    d    Cf   Cu   Kf   Ku     L
    ---------------------------------------------------------
    1-Fine         33.6 20.0  85.0   23   31   .7   .6  2,448
    ---------------------------------------------------------

    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 = 2.80 ft = 33.6 in.
    *********************************************************
```

Use 4.0 feet

BEARING RESISTANCE ON NATIVE SOILS:

Consider this for use with Box Culverts, Headwalls and Wingwalls.

SERVICE LIMIT STATE:

LRFD Table C10.6.2.6.1-1, Pg. 10-66 (Based on NAVFAC DM 7.2) - "Presumptive Bearing Resistances for Spread Footing Foundations at the Service Limit State"

Bearing Material	Consistency in Place	Bearing Resistance (kips per sq. foot)	Recommended Value
Inorganic Silt,	Very stiff to hard	4 to 8	4 ksf
Sandy or Clayey Silt,	Medium stiff to stiff	2 to 6	2 ksf
Varved Silt-Clay-Fine Sand	Soft	1 to 2	1 ksf

Recommend **2.0 ksf** to control settlements for **Service Limit State** analyses and for preliminary footing sizing.

STRENGTH LIMIT STATE:

Nominal and Factored Bearing Resistance for box culvert and retaining wall base slab on fill soils at the Strength Limit State:

Assumptions:

1. Box Culvert will be embedded 2.0 feet for streambed simulation.

$$D_f := 2.0ft$$

2. Assumed parameters for soils:
   Assume granular fill

   Moist unit weight: $\gamma_m := 105pcf$

   Saturated unit weight: $\gamma_{sat} := 110pcf$

   Soil angle of internal friction: $\phi_{ns} := 25$

   Undrained shear strength (cohesion): $c_{ns} := 500psf$

3. Use Terzaghi strip equations as L > B

Depth to Groundwater table based on boring data: $D_w := 0 \cdot ft$

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

Effective Stress at the footing bearing level: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.1 \cdot ksf$

Box Culvert Width: $B := 10ft$

Terzaghi Shape Factors from Table 4-1, p. 220
For strip footing:

$s_c := 1.0$

$s_\gamma := 1.0$

Meyerhof Bearing Capacity Factors For ϕ = 25 deg — Bowles 5th Ed. Table 4-4 pg. 223

$N_c := 20.71$ $N_q := 10.7$ $N_\gamma := 6.8$

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

$$q_{nom} := c_{ns} \cdot N_c \cdot s_c + q_{eff\_str} \cdot N_q + 0.5(\gamma_{sat} - \gamma_w) \cdot B \cdot N_\gamma \cdot s_\gamma$$

$q_{nom} = 13 \cdot ksf$

Resistance Factor from LRFD Table 10.5.5.2.2-1 pg. 10-32: $\phi_b := 0.45$

$q_{fac} := q_{nom} \cdot \phi_b$

$q_{fac} = 5.8 \cdot ksf$

The **Strength Limit State** Factored Bearing Resistance is **6.0 ksf** for the box culvert.

For this project settlement controls. Recommend 2.0 ksf Factored Bearing Resistance for box culvert design.

CONSOLIDATION SETTLEMENT, Approx STA 1042+75.00:

Estimate Consolidation Parameters From Liquid Limit Correlation
Note: Site Has OC Olive Presumpscot Clay-Silt

$LL_{AVG} := 31$

$C_c := 0.009(LL_{AVG} - 10)$ Sowers p. 157

$C_c = 0.19$

Estimate e_o Using Correlation: Gw = Se

$w := .24$

Avg Specific Gravity $G := 2.68$

Assume $S := 1.0$

$e_o := \frac{G \cdot w}{S}$ $e_o = 0.64$

Estimate Compression Ratio and Recompression Ratio

$C'_c := \frac{C_c}{(1 + e_o)}$ $C'_c = 0.12$ For Maine Clays C'_r Typically 10% of C'_c $(C'_r) := 0.012$

$\Delta H = C'_c H \log\left[(p_o + \Delta p)/p_o\right]$ Bowles p. 83

Break Up Clay-Silt Layer Into 3 Layers 2ft, 3ft and 4ft Thick

$p_{o1} := (105 - 62.4)\text{pcf} \cdot 1\text{ft}$ $p_{o1} = 42.6 \cdot \text{psf}$ $H_1 := 2\text{ft}$

$p_{o2} := (105 - 62.4)\text{pcf} \cdot 3.5\text{ft}$ $p_{o2} = 149.1 \cdot \text{psf}$ $H_2 := 3\text{ft}$

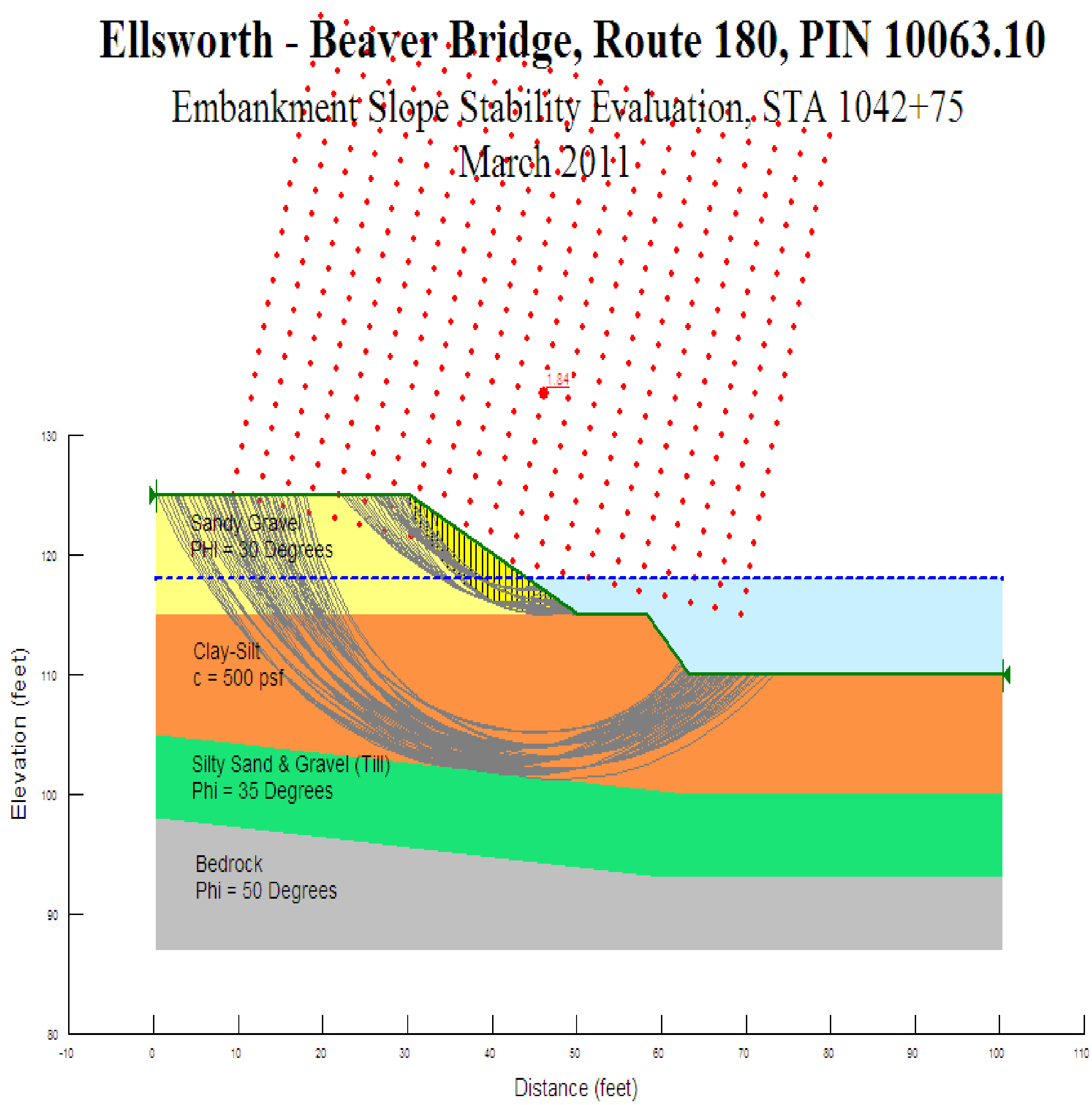
$p_{o3} := (105 - 62.4)\text{pcf} \cdot 7\text{ft}$ $p_{o3} = 298.2 \cdot \text{psf}$ $H_3 := 4\text{ft}$

$\Delta p := 125\text{pcf} \cdot 10\text{ft}$ $\Delta p = 1250 \cdot \text{psf}$

$$\Delta H := \left(C'_r \cdot H_1 \cdot \log\left(\frac{p_{o1} + \Delta p}{p_{o1}}\right) \cdot 12 \frac{\text{in}}{\text{ft}}\right) + \left(C'_r \cdot H_2 \cdot \log\left(\frac{p_{o2} + \Delta p}{p_{o2}}\right) \cdot 12 \frac{\text{in}}{\text{ft}}\right) + \left(C'_r \cdot H_3 \cdot \log\left(\frac{p_{o3} + \Delta p}{p_{o3}}\right) \cdot 12 \frac{\text{in}}{\text{ft}}\right)$$

$\Delta H = 1.26 \cdot \text{in}$

Say 1 To 2 InchesB/C OC Silt, Settlement Will Occur Largely During Embankment Construction



Ellsworth - Beaver Bridge, Route 180, PIN 10063.10
Embankment Slope Stability Evaluation, STA 1042+75
March 2011
1.84
Sandy Gravel
PHI = 30 Degrees
Clay-Silt
c = 500 psf
Silty Sand & Gravel (Till)
Phi = 35 Degrees
Bedrock
Phi = 50 Degrees
Elevation (feet)
Distance (feet)
130
120
110
100
90
80
-10
0
10
20
30
40
50
60
70
80
90
100
110

SLOPE/W

File Information

Title: Ellsworth - Beaver Bridge Embank STA 1042+75
Revision Number: 31
Last Edited By: Moreau, Michael
Date: 3/17/2011
Time: 3:43:45 PM
File Name: Ellsworth - Beaver Bridge Embank STA 1042+75.gsz
Directory: C:\MyFiles\Slope W\

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

SLOPE/W

Kind: SLOPE/W
Method: Bishop, Ordinary and Janbu
Settings
- Apply Phreatic Correction: Yes
- PWP Conditions Source: Piezometric Line
- Use Staged Rapid Drawdown: No

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Grid and Radius
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No
- Tension Crack
  - Tension Crack Option: (none)

FOS Distribution
- FOS Calculation Option: Constant

Advanced
- Number of Slices: 30

Optimization Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Sandy Gravel Fill

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Unit Wt. Above Water Table: 135 pcf
Cohesion: 75 psf
Phi: 30 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Clay-Silt

Model: Mohr-Coulomb
Unit Weight: 110 pcf
Unit Wt. Above Water Table: 90 pcf
Cohesion: 500 psf
Phi: 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Till

Model: Mohr-Coulomb
Unit Weight: 138 pcf
Cohesion: 0 psf
Phi: 35 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Bedrock

Model: Mohr-Coulomb
Unit Weight: 160 pcf
Cohesion: 0 psf
Phi: 50 °

Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Grid

Upper Left: (19.8683, 164.986) ft
Lower Left: (9.41353, 124.986) ft
Lower Right: (69.4883, 114.986) ft
Grid Horizontal Increment: 20
Grid Vertical Increment: 20
Left Projection Angle: 0 °
Right Projection Angle: 0 °

Slip Surface Radius

Upper Left Coordinate: (35, 120) ft
Upper Right Coordinate: (55, 115) ft
Lower Left Coordinate: (35, 100) ft
Lower Right Coordinate: (55, 105) ft
Number of Increments: 20
Left Projection: No
Left Projection Angle: 0 °
Right Projection: No
Right Projection Angle: 0 °

Slip Surface Limits

Left Coordinate: (0.26509, 125) ft
Right Coordinate: (100.3897, 110) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
	0.26509	118
	100.3897	118

Seismic Loads

Horz Seismic Load: 0

Vert Seismic Load: 0

Regions

	Material	Points	Area (ft²)
Region 1	Sandy Gravel Fill	1,2,3,9	400.4984
Region 2	Clay-Silt	9,3,4,5,7,8,6,10	1146.4268
Region 3	Till	10,6,8,13,12,11	708.3816
Region 4	Bedrock	11,12,13,15,14	750.93459

Points

	X (ft)	Y (ft)
Point 1	0.26509	125
Point 2	30.30247	125
Point 3	50.32739	115
Point 4	58.33736	115
Point 5	63.34359	110
Point 6	63.34359	100
Point 7	100.3897	110
Point 8	100.3897	100
Point 9	0.26509	115
Point 10	0.26509	105
Point 11	0.26509	98
Point 12	60.33986	93
Point 13	100.3897	93
Point 14	0.26509	87
Point 15	100.3897	87

Appendix D

Special Provision

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 wings, headwalls, toe walls/cut off walls and appurtenances, in accordance with the contract documents.

534.20 Materials Structural precast elements for the arch or box culvert 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, current edition. The live load 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 is 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%.

The live load deflection check per AASHTO LRFD Bridge Design Specifications Section 2.5.6.2 for the top slab of the 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.

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 as a minimum the Strength 1 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.

All design options 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. The MaineDOT Summary of Rating Form

maybe accessed at the following MaineDOT web address:
http://www.maine.gov/mdot/contractors/

Loading ratings shall be provided in the following situations:

1. All structures with clear spans between 10 feet and 15 feet with 3 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 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 (bound and indexed)
E) Load rating computations and completed load rating form (bound 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 mass (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 38 mm [1 ½ in], whichever is less. The haunch dimensions shall not vary by more than 19 mm [¾ in] from the design dimension. The dimension of the legs shall not vary by more than 6 mm [¼ in] 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 6 mm [¼ in]. 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 15 mm [⅝ in] 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 12 mm [½ in].

The cover of concrete over the outside circumferential reinforcement shall be 50 mm [2 in] minimum. The concrete cover over the inside reinforcement shall be 38 mm [1 ½ in] minimum. The clear distance of the end of circumferential wires shall not be less than 25 mm [1 in] or more than 50 mm [2 in] 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 billet steel bars which meet the spacing requirements. The ends of the longitudinal distribution reinforcement shall be not more than 75 mm [3 in] 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 300 mm [12 in] for welded wire fabric or billet steel bars. The spacing center to center of the circumferential wires in a wire fabric sheet shall be not less than 50 mm [2 in] or more than 100 mm [4 in]. For the wire fabric, the spacing center to center of the longitudinal wires shall not be more than 200 mm [8 in]. The spacing center to center of the longitudinal distribution steel for either line of reinforcing in the top slab shall be not more than 375 mm [15 in].

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 4000 mm² [6 in²] in area or over 25 mm [1 in] deep.
3) Edge or corner breakage exceeding 300 mm [12 in] in length or 25 mm [1 in] 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 30 MPa [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 (but in no case less than 30 MPa [4350 psi].

The Contractor shall set precast members on 12 mm [½ in] 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 12 mm [½ in] 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 20 MPa [2900 psi]. The Contractor shall construct the completed footing surface to the lines and grades shown on the plans. When checked with a 3 m [10 ft] straightedge, the surface shall not vary more than 6 mm [¼ in] in 3 meters [10 ft]. The footing keyway shall be filled with a non-shrink flowable cementitious grout with a design compressive strength of at least 35 MPa [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 300 mm [12 in] 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. Final assembly shall be approved by the manufacturer's representative prior to backfilling.

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 200 mm [8 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 wall units, 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 1.5 m [5 ft] of the precast structure as well as over the top until it is covered with at least 300 mm [12 in] 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.

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

<u>534.60 Basis of Payment</u> The Department will pay for the accepted quantity of Precast Structural Concrete Arch or Box Culvert 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:

Pay Item	Pay Unit
534.71 Precast Concrete Box Culvert	Lump Sum