MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CAMBOLASSE BRIDGE
ROUTE 2 OVER CAMBOLASSE STREAM
LINCOLN, MAINE

STATE OF MAINE
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Penobscot County
PIN 16712.00

Soils Report No. 2010-23
Bridge No. 2170

August 12, 2010

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GEOTECHNICAL DESIGN SUMMARY

This report provides geotechnical recommendations for the replacement of Cambolasse Bridge over the Cambolasse Stream in Lincoln, Maine. The proposed replacement bridge will be an 8.5-foot high by 14-foot wide concrete box culvert. Staged construction will be used to construct the new box culvert. The bridge will be widened to 34 feet rail to rail width with 11-foot travel lanes and 6-foot shoulders, as well as accommodation for guardrail. No significant horizontal or vertical alignment changes are planned. 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, service and extreme limit states and load combinations in accordance with the AASHTO LRFD Bridge Design Specifications, 5th Edition, 2010 (herein referred to as LRFD). 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 soil envelope bedding and 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 Headwall Design - Culvert headwalls should consider all relevant LRFD strength and service limit states and load combinations and be designed to resist and/or absorb lateral earth loads, vehicular loads, creep, and temperature and shrinkage deformations of the concrete box culverts.

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 – The factored bearing resistance at the strength limit state for a box culvert on compacted fill or native glacial till should not exceed 6.5 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 6.0 ksf may be used when analyzing box bottom slabs for the service limit state. In no instance shall the bearing stress exceed the nominal resistance of concrete, which may be taken as $0.3f'c$.

Settlement – Total and post-construction settlements of the prepared culvert subgrade consisting of compacted fill or native soil will be negligible since no grade changes are proposed.

Scour Protection – The box culverts will be fitted with concrete headwalls and inlet and outlet seepage cutoff walls below the culvert, all to provide scour protection. We recommend that the bridge approach slopes be armored with a 3-foot thick layer of riprap up and down the alignment beside the culvert openings. The 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. Riprap shall meet the requirements of 703.26, Plain and Hand Laid Riprap, of Special Provision 703, Aggregates. The riprap slope protection should 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.

Frost Protection – If used, foundations placed on granular soils shall be founded a minimum of 4.5 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 –

Excavation

- Construction of the new concrete box culvert will require staged construction and soil excavation. Earth support systems may be required.
- Protect the excavated subgrade from exposure to water and unnecessary construction traffic. Remove and replace water-softened, disturbed, or rutted subgrade soil with compacted gravel borrow.

Dewatering

- Control groundwater and surface water infiltration to permit construction in-the-dry.
- 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.

Reuse of Excavated Soil and Bedrock

- Do not use excavated existing subbase aggregate or approach fill soil for pavement structure construction or to re-base shoulders. Excavated subbase sand and gravel or granular fill may be used as fill below subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

Embankment Fill Areas

- Bench existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes.

Erosion Control

- 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 replace Cambolasse Bridge carrying Route 2 over Cambolasse Stream in the Town of Lincoln, Penobscot County, Maine. We show the project location on Sheet 1, Site Location Map, appended to this report. We conducted subsurface investigations at the bridge 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 bridge foundations.

The existing structure built in 1934 consisted of reinforced concrete cantilever abutments and wing walls. Structural problems were identified in the 2006 bridge inspection including ½ inch wide cracks and undermining of the abutments resulting in a Condition Rating of 3 – Serious. It was also noted that the center (oldest) section of the structural slab has settled 1.5 inches and shifted north 2 inches. The bridge had a sufficiency rating of 24 in 2008.

MaineDOT is proposing an 8.5-foot high by 14-foot wide, concrete box culvert to replace the existing bridge. The new box culvert will be on the same horizontal and vertical alignment and will have a rail-to-rail width of approximately 34 feet. Current plans include 11-foot travel lanes and 6-foot shoulders, as well as accommodation for guardrail, construction of concrete culvert headwalls and toe walls, and armoring the embankments with riprap.

2.0 GEOLOGIC SETTING

The Maine Geologic Survey (MGS) “Surficial Geology of Winn Quadrangle, Maine, Open-file No. 81-28” (1981) indicates that surficial soils in the vicinity of Cambolasse Bridge consist of glacial stream, glaciomarine, stream alluvium and glacial till soil unit contacts. The predominant soil units at the site based on our subsurface explorations are glaciomarine which may consist of silt, clay and sands, and glacial till which consists of heterogeneous mixtures of sand, silt, clay and stones.

According to the “Bedrock Geologic Map of Maine” MGS (1985), the bedrock at the Cambolasse Bridge site consists of Silurian, undifferentiated and interbedded pelites and sandstones, some of the Allsbury Formation and in part unnamed. No bedrock cores were collected during the explorations for this project since the proposed replacement structure is a concrete box culvert.

3.0 SUBSURFACE INVESTIGATION

We investigated subsurface conditions at the site by drilling two test borings, BB-LCS-101 and BB-LCS-102, conducted by the MaineDOT drill crew on April 14 and 15, 2010. The borings were terminated with casing and roller cone refusal on apparent bedrock. The boring locations and soil profile are shown on Sheet 2, Boring Location and Interpretive Subsurface Profile. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on Sheet 3, 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 consultant geotechnical engineer logged the subsurface conditions encountered on the field logs. The field crew tied down the boring locations by taping distances to adjacent site features.

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 0.84 to the raw field N-values obtained with the MaineDOT 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 eight (8) standard grain size analyses with natural water contents tests, two with hydrometer analysis, and two 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 bridge site is situated in an area of numerous widely variable glaciated sediment units. The bridge itself is situated at the end of short fill extensions built into the Cambolasse Stream flood plain. The approach embankment soil behind the existing bridge abutments is predominantly granular fill overlying approximately 5 to 7.5 feet of glaciomarine clay-silt which is underlain by approximately 6.4 to 8.3 feet of glacial till. The glacial till overlies apparent bedrock at the boring locations. We present a profile depicting the generalized soil stratigraphy at the bridge site on Sheet 2, Boring Location Plan and Interpretive Subsurface Profile, provided at the end of this report. A summary description of the subsurface conditions follows.

5.1 Granular Fill

We encountered granular fill to a depth ranging between approximately 10.0 and 10.5 feet below ground surface (bgs). The granular fill consists of fine to coarse sand, with some gravel to gravelly and trace to some silt. Drill attitude also indicated the presence of cobbles and boulders at various levels in the fill. The SPT N_{60}-values in the granular fill ranged from 7 to 69 blows per foot (bpf) indicating that the unit is loose to very dense in consistency.

The granular fill samples had water contents ranging between approximately 3 and 15 percent. Grain size analyses conducted on selected samples of the fill soils indicate that the soils are classified as A-1-a and A-2-4 by the AASHTO Classification System and SM and SW-SM under the Unified Soil Classification System.

5.2 Glaciomarine Silt and Clay

We encountered glaciomarine silt and clay beneath the approach fills. This soil unit consists of silty clay or clayey silt with trace fine sand, trace or no gravel, and occasional shell fragments. The fine grained soil unit is over-consolidated and moderately plastic. The thickness of the glaciomarine sediments encountered ranged between approximately 5.0 and 7.5 feet. SPT N_{60}-values ranged from 8 to 38 bpf, indicating those deposits are medium stiff to hard in consistency. The tested samples had identical liquid limits of 41 and plasticity indices of 19, and natural water contents of 34 and 38 percent. Grain size analyses indicate that the soils are classified as A-7-6 and CL by the AASHTO and Unified Soil Classification Systems, respectively. Below the glaciomarine silt and clay we encountered the glacial till soil unit.

5.3 Glacial Till

The glacial till found in the borings generally comprised of fine to coarse sand with some gravel to gravelly, or fine to coarse sandy gravel, all with little to some silt. The thickness of this soil unit ranged between approximately 6.4 and 8.3 feet. SPT N_{60}-values ranged from 27 to 38 bpf, indicating the till deposit is medium dense to dense in consistency. We observed the glacial till unit over apparent bedrock in each of the borings.

The glacial till samples had water contents ranging between approximately 8 and 16 percent. Grain size analyses conducted on selected samples of the till soils indicate that the soils are classified as A-1-b by the AASHTO Classification System and SM under the Unified Soil Classification System.

5.4 Bedrock

We encountered apparent bedrock at approximate depths ranging from 23.3 to 24.4 feet bgs. Locally, the bedrock is mapped as undifferentiated and interbedded pelites and sandstones, some of the Allsbury Formation and in part unnamed. We did not collect bedrock cores during the explorations for this project since the proposed replacement structure is a concrete box culvert. The table below summarizes the top of apparent bedrock elevations at the boring locations:

Substructure	Boring	Station	Depth to Apparent Bedrock (feet bgs)	Elev. of Apparent Bedrock Surface (feet)
Abutment No. 1	BB-LCS-101	2+83.7, 11.1 RT	23.3	154.9
Abutment No. 2	BB-LCS-102	3+17.3, 17.5 LT	24.4	153.6

Bedrock Depth and Elevation at the Boring Locations

5.5 Groundwater

We observed the groundwater level at an approximate depth of greater than 7.0 feet bgs in boring BB-LCS-101 and 7.0 feet in BB-LCS-102. 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 replacement designs: 1) 14-foot wide by 8.5-foot high concrete box culvert; and 2) 22-foot wide by 8.5-foot high concrete box culvert. The project team selected alternate No. 1, 14-foot wide by 8.5-foot high concrete box culvert, for the replacement structure. The following section presents geotechnical design recommendations for the concrete box culvert alternate.

7.0 FOUNDATION CONSIDERATIONS AND RECOMMENDATIONS

The design team has selected a concrete box culvert to replace the structure at the Lincoln site. The proposed replacement structure will consist of a 14-foot wide by 8.5-foot high concrete box culvert. The new culvert will be on the same horizontal and vertical alignment as the existing bridge and the new structure will have a rail-to-rail width of approximately 34 feet. The base of the bottom slab will be buried approximately three 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 HL-93 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 culverts will be supplier-designed for all relevant strength, service and extreme 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 soil envelope bedding and backfill shall consist of Standard Specification 703.19, Granular Borrow, Material for Underwater Backfill, except that the maximum particle size shall be limited to 4 inches. We recommend a bedding layer 12 inches thick. 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 backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density.

7.2 Culvert Headwall Design

Culvert headwalls are essentially retaining walls and should be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1, and 11.5.5 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 replacement, 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: ≥ 1 feet
5	5.0	2.0
10	3.5	2.0
≥ 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.

7.3 Box Culvert Bearing Resistance

The factored bearing resistance at the strength limit state for the box culvert on compacted fill should not exceed 6.5 ksf. Based on presumptive bearing resistance values, a factored bearing resistance of 6 ksf may be used when analyzing box bottom slabs for the service limit state as allowed in LRFD C10.6.2.6.1. In no instance shall the bearing stress exceed the nominal resistance of the structure concrete, which may be taken as 0.3 $f'c$.

7.4 Settlement

We have evaluated the potential settlement at the Lincoln site. MaineDOT currently does not plan horizontal or vertical alignment changes. The proposed concrete box culvert also increases the opening size and box culvert structure loads will be less than the existing bridge structure loads. Consequently, we estimate that total and post-construction settlements of the prepared culvert subgrade consisting of compacted fill or native glaciomarine soil will be negligible since no grade changes are proposed and fill and structure loads are reduced.

7.5 Scour Protection

The box culvert will be fitted with concrete headwalls and inlet and outlet section seepage cutoff walls below the culvert, all 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 slope protection should 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.

7.6 Frost Protection

We have evaluated the potential frost depth at the Lincoln 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 1880 F-degree days. Considering site soils and natural water contents, this correlates to a frost depth of 4.7 feet at this site. We also considered Modberg frost depth projections. The results of the Modberg frost depth model indicate a potential frost depth of 4.1 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 4.5 feet below finished exterior grade for frost protection.

7.7 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.

7.8 Construction Considerations

7.8.1 Excavation

Construction of the new concrete box culvert will require soil excavation. Earth support systems may be required. The fill and native glaciomarine and glacial till soils at the site will be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. 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 and replace with compacted gravel borrow. If the subgrade soil contains cobbles or boulders, we recommend that the contractor remove any cobbles and boulders larger than 6 inches in diameter. After excavating to the subgrade level, the contractor should proof-roll the surface to identify weak soil areas.

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).

7.8.2 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 glacial till 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 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.

7.8.3 Reuse of Excavated Soil

The project plans call for excavation of the existing approach areas to achieve planned grades. In the process, the contractor will excavate both the existing subbase gravel, and subgrade fill soils. We do not recommend using the excavated subbase aggregate to re-base the bridge approaches. Excavated subbase and subgrade sand and gravel may be used as fill below the roadway subgrade elevation in fill embankment areas provided all other requirements of MaineDOT Standard Specification Sections 203 and 703 are met.

We do not recommend using any glacial till soil excavation as fill beneath the pavement structure. This soil 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 the glacial till 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.

7.8.4 Embankment Fill Areas

The current project plans require construction of fill extensions along the bridge approaches. The plans indicate that the side slopes will be constructed to 1.75:1 (H:V) grades or flatter and will be armored with riprap. We recommend benching the existing fill slope soils in accordance with MaineDOT Standard Specification 203.09, Preparation of Embankment Area, where new fill slope extensions are constructed over existing slopes in preparation for construction of the riprap layer.

7.8.5 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 replacement of the Cambolasse Bridge over Cambolasse Stream in Lincoln, 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.

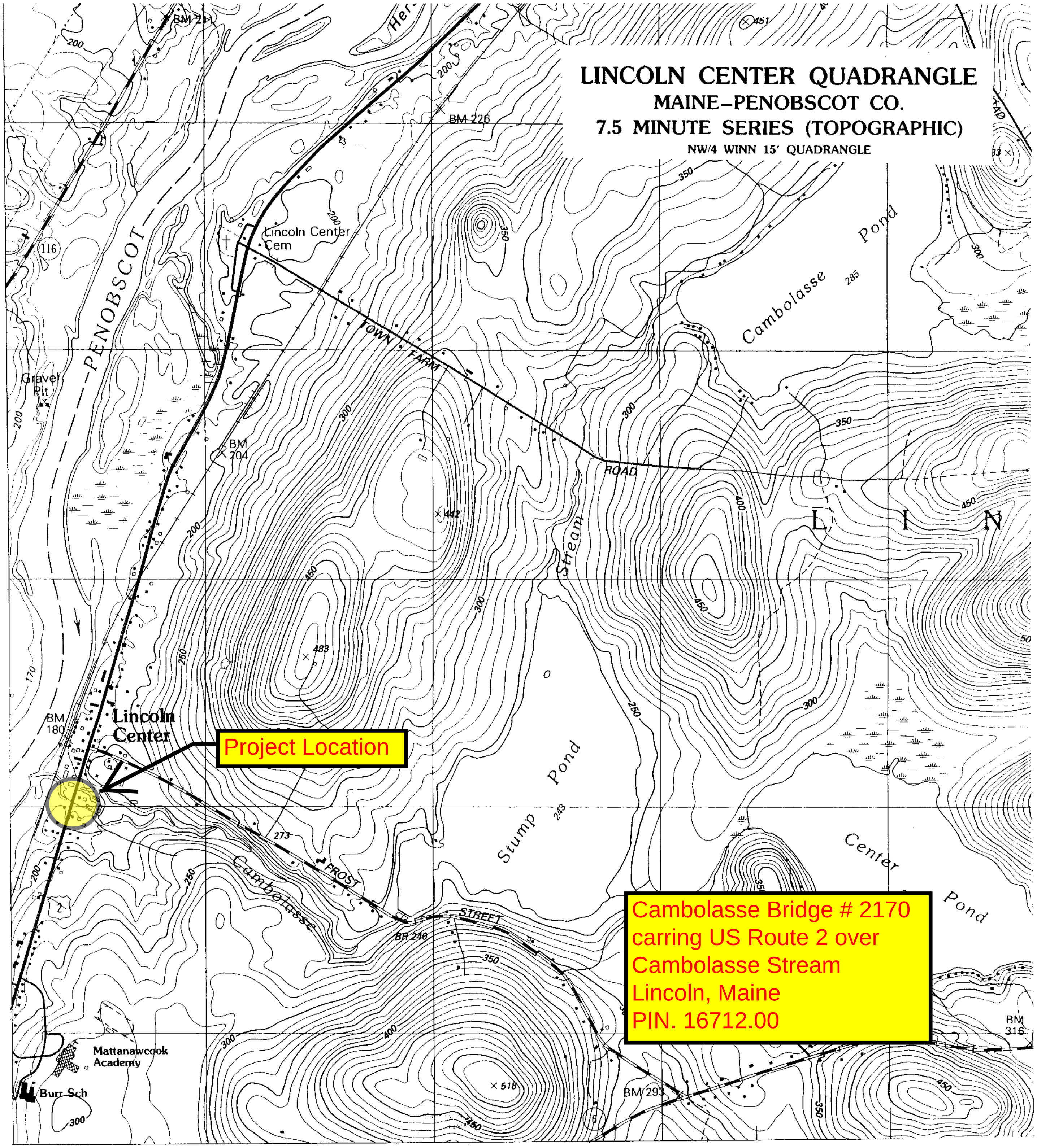
REFERENCES

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.

Sheets


LINCOLN CENTER QUADRANGLE
MAINE–PENOBSCOT CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)
NW/4 WINN 15′ QUADRANGLE
Project Location
Cambolasse Bridge # 2170
carring US Route 2 over
Cambolasse Stream
Lincoln, Maine
PIN. 16712.00
PENOBSCOT
Lincoln Center
Lincoln Center Cem
TOWN FARM ROAD
Cambolasse Pond
Stump Pond
Center Pond
Stream
Cambolasse
FROST STREET
Gravel Pit
Mattanawcook Academy
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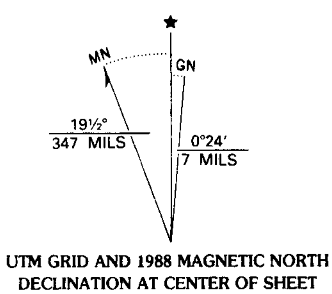
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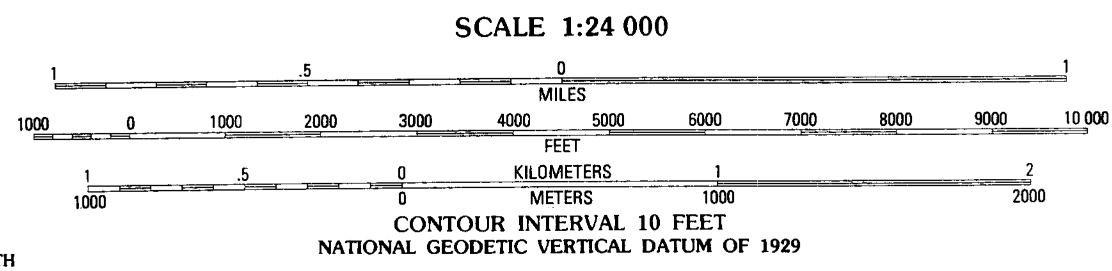
DECLINATION AT CENTER OF SHEET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METERS
CONTOUR INTERVAL 10 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

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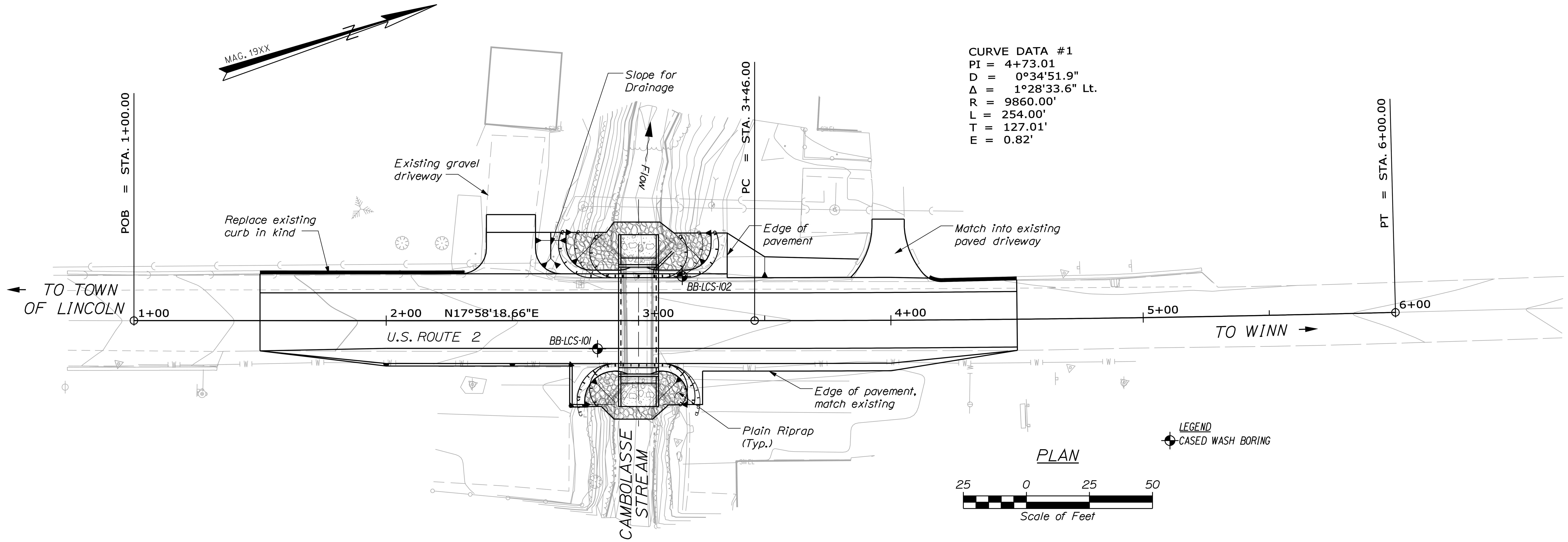

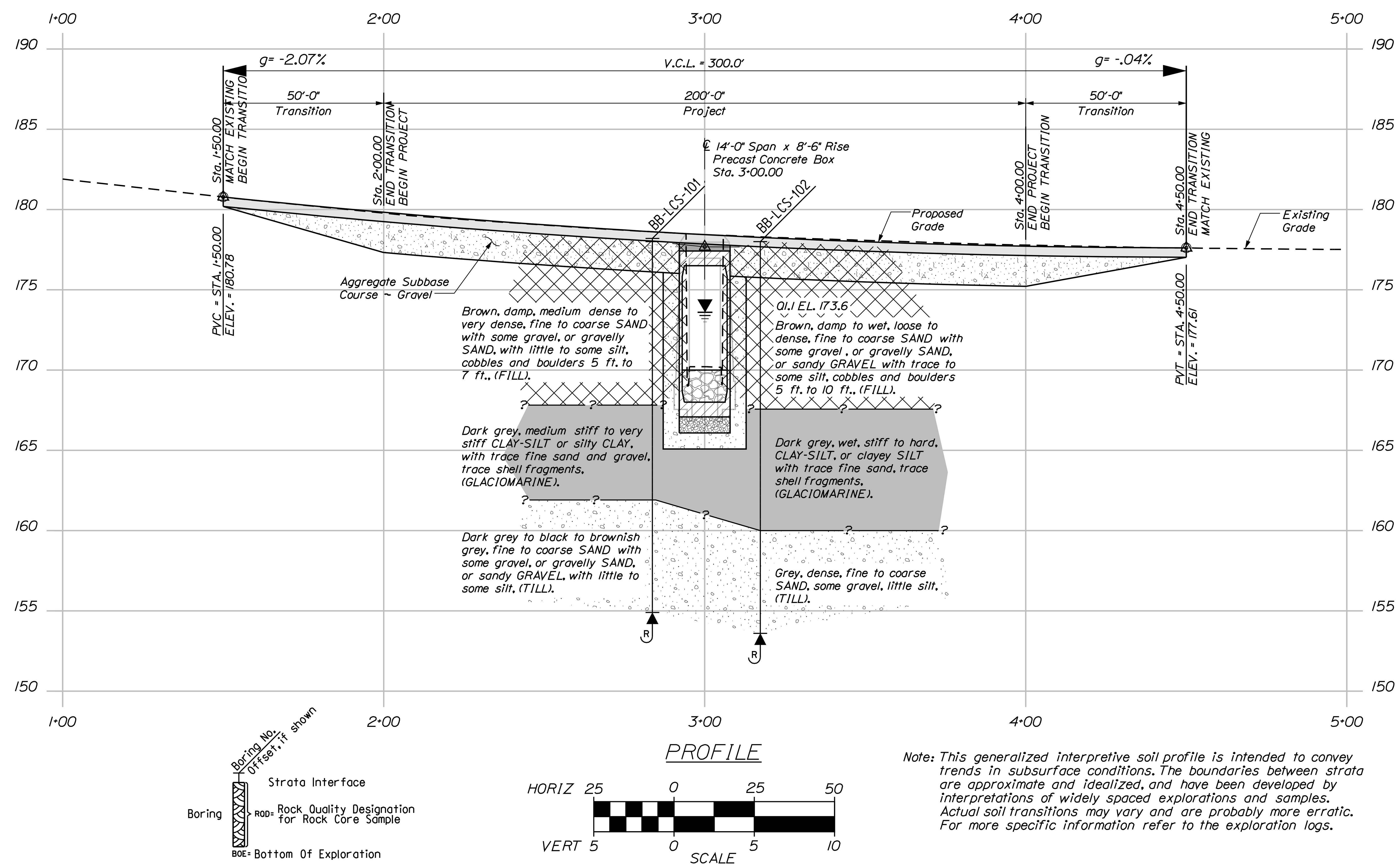

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably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

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRIDGE NO. 2170

PIN 16712.00

BRIDGE PLANS

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

SIGNATURE

P.E. NUMBER

DATE

CAMBOLASSE BRIDGE
CAMBOLASSE STREAM
LINCOLN PENOBSCOT COUNTY

BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\MSTA\005_BORING LOGS1.dgn   Division: GEOTECH   Username: terry.white   Date:9/9/2010

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cambolasse Bridge #2170 carring US Route 2 over Cambolasse Stream	Location: Lincoln, Maine	Boring No.: BB-LCS-101	PIN: 16712.00

Driller:	MaineDOT	Elevation (ft.)	178.2	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald (IVS)	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/14/10; 10:20-14:45	Drilling Method:	Cased Wash Boring	Core Barrel:	NX
Boring Location:	2+83.7, 11.1 Rt.	Casing ID/OD:	NW	Water Level*:	>7.0' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0								178.00		PAVEMENT. ——0.20	
	1D	24/15	1.00 - 3.00	24/30/19/22	49	69				Brown, damp medium dense to very dense, gravelly fine to coarse SAND, little silt, (Fill).	G#237481 A-1-a, SW-SM WC=2.8%
	2D/A	24/11	3.00 - 5.00	12/6/9/9	15	21				(2D) 3.0-4.8' bgs. Drill attitude indicates cobbles and boulders to approximately 7 ft.	G#237482 A-1-a, SW-SM WC=3.4%
5								173.40 173.20		——4.80 (2D/A) 4.8-5.0' bgs. Brown, damp, medium dense, fine SAND, some gravel, little silt, trace medium to coarse sand, (Reworked FILL).	(3D) G#237483 A-2-4, SM WC=15.1%
	3D	21.2/8	5.00 - 6.77	13/6/17/23(2.75)	23	32				——6.00 (3D) Brown, damp, dense, fine to coarse SAND, some gravel and silt, (Reworked Fill).	
10								168.20		——10.00 Dark grey, wet, very stiff, SILT and CLAY, low to moderate plasticity, behaves highly over consolidated.	
	4D	24/8	10.00 - 12.00	11/7/6/7	13	18					
	5D	24/20	12.50 - 14.50	4/3/3/2	6	8		165.70		——12.50 Dark grey, wet, medium stiff, silty CLAY, trace fine sand and gravel, moderate plasticity with occasional shell fragments and black organic staining, behaves highly over consolidated.	G#237484 A-7-6, CL WC=34.0% LL=41 PL=22 PI=19
15	MV/6D	24/12	15.00 - 17.00	12/13/10/12	23	32		163.20		——15.00 Failed vane attempt, unable to push vane. Dark grey, wet, dense, fine to coarse SAND, some gravel and silt, gravel is angular, (Till).	G#237485 A-1-b, SM WC=9.8%
	7D	24/10	17.50 - 19.50	10/13/13/13	26	36				Black-dark grey, wet, dense, fine to coarse sandy GRAVEL, little silt, gravel appears to be crushed rock, (Till).	
20	8D	24/11	20.00 - 22.00	11/9/10/7	19	27	24			Brownish-grey, wet, medium dense, gravelly fine to coarse SAND, little silt, gravel is subrounded to angular, (Till).	
							36				
							32				
								154.90		——23.30 Bottom of Exploration at 23.30 feet below ground surface. CASING AND ROLLER CONE REFUSAL, presumably on bedrock	
25											

Remarks:

Strong odor of petroleum hydrocarbons in upper 5 ft of material.
Drilling behavior suggests boney material to approximately 7 ft. Three attempts with auger refusal between 5 and 7 ft. In third hole, drove HW casing to 5 ft, roller cone to 5 ft, NX core at 5 ft, dropped through boulder at 7 ft, N roller cone to 10 ft, telescope NW casing to 10 ft, washed out N roller cone to 10 ft.
Streambed approximately 10 ft below top of boring elevation on highway.

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-LCS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cambolasse Bridge #2170 carring US Route 2 over Cambolasse Stream	Location: Lincoln, Maine	Boring No.: BB-LCS-102	PIN: 16712.00

Driller:	MaineDOT	Elevation (ft.)	178.0	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	Be Schonewald (IVS)	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	4/15/10; 07:50-10:20	Drilling Method:	Cased Wash Boring	Core Barrel:	N/A
Boring Location:	3+17.3, 17.5 Lt.	Casing ID/OD:	NW	Water Level*:	7.0' bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0	1D	24/17	1.00 - 3.00	18/17/13/10	30	42				Brown, damp, dense, gravelly fine to coarse SAND, trace silt, (Fill).	G#237486 A-1-a, SW-SM WC=3.2%
	2D	24/15	3.00 - 5.00	5/4/4/7	8	11		175.00		——3.00 Brown, damp, medium dense to loose, fine to coarse SAND, some gravel, little to some silt, (Fill).	
5	3D	24/6	5.00 - 7.00	4/3/2/6	5	7				Drill attitude indicates cobbles and boulders between approximately 5 ft and 10 ft.	
	4D	24/12	7.70 - 9.70	9/13/19/27	32	45		170.30		——7.70 Brown, wet, dense, fine to coarse sandy GRAVEL, little to some silt, (Fill). Drill attitude indicates cobbles and boulders between approximately 5 ft and 10 ft.	
10	5D	24/15	10.50 - 12.50	8/10/17/16	27	38		167.50		——10.50 Dark grey, wet, hard, SILT and CLAY, low to moderate plasticity, behaves highly overconsolidated.	
	6D	24/17	12.50 - 14.50	5/5/4/6	9	13	39			Dark grey, wet, stiff, SILT and CLAY, low to moderate plasticity, behaves highly overconsolidated.	
							39				
							39				
15	MV/7D	24/24	15.00 - 17.00	WOH/4/3/4	7	10	41	163.00		——15.00 Failed vane attempt, could not push. Dark grey, wet, stiff, clayey SILT, trace fine sand with few shell fragments, moderate plasticity, behaves slightly overconsolidated.	G#237487 A-7-6, CL WC=37.6% LL=41 PL=22 PI=19
							47				
	MV/8D	24/6	17.50 - 19.50	9/10/10/11	20	28	47	160.00		Failed vane attempt, could not push. Dark grey, wet, very stiff CLAY and SILT, moderate plasticity with fine to coarse sand on tip of sample. ——18.00	
							61				
							72				
20	9D	24/7	20.00 - 22.00	11/11/16/12	27	38	58			Grey, wet, dense, fine to coarse SAND, some gravel, little silt, (Till).	G#237488 A-1-b, SM WC=9.2%
							70				
							62				
							102				
								153.60		——24.40 Bottom of Exploration at 24.40 feet below ground surface. CASING AND ROLLER CONE REFUSAL, presumably on bedrock.	
25											

Remarks:

Strong odor of petroleum hydrocarbons in upper 3 ft of material.
Drilling behavior suggests boney material to approximately 5 to 10 ft., cobbles and boulders.

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-LCS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRIDGE PLANS
PIN 16712.00
BRIDGE NO. 2170

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

SIGNATURE
P.E. NUMBER
DATE

CAMBOLASSE BRIDGE
CAMBOLASSE STREAM
LINCOLN   PENOBSCOT COUNTY

BORING LOGS

SHEET NUMBER
3
OF 3

<u>Appendix A</u>

Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Cambolasse Bridge #2170 carring US Route 2 over Cambolasse Stream **Location:** Lincoln, Maine	**Boring No.:** BB-LCS-101 **PIN:** 16712.00

Driller:	MaineDOT	**Elevation (ft.)**	178.2	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald (IVS)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/14/10; 10:20-14:45	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NX
Boring Location:	2+83.7, 11.1 Rt.	**Casing ID/OD:**	NW	**Water Level*:**	>7.0' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0								178.00		PAVEMENT. ——0.20	
	1D	24/15	1.00 - 3.00	24/30/19/22	49	69				Brown, damp medium dense to very dense, gravelly fine to coarse SAND, little silt, (Fill).	G#237481 A-1-a, SW-SM WC=2.8%
	2D/A	24/11	3.00 - 5.00	12/6/9/9	15	21				(2D) 3.0-4.8' bgs. Drill attitude indicates cobbles and boulders to approximately 7 ft.	G#237482 A-1-a, SW-SM WC=3.4%
								173.40		——4.80	
5	3D	21.2/8	5.00 - 6.77	13/6/17/23(2.75)	23	32		173.20		(2D/A) 4.8-5.0' bgs. Brown, damp, medium dense, fine SAND, some gravel, little silt, trace medium to coarse sand, (Reworked FILL). ——5.00 (3D) Brown, damp, dense, fine to coarse SAND, some gravel and silt, (Reworked Fill).	(3D) G#237483 A-2-4, SM WC=15.1%
10								168.20		——10.00	
	4D	24/8	10.00 - 12.00	11/7/6/7	13	18				Dark grey, wet, very stiff, SILT and CLAY, low to moderate plasticity, behaves highly over consolidated.	
	5D	24/20	12.50 - 14.50	4/3/3/2	6	8		165.70		——12.50 Dark grey, wet, medium stiff, silty CLAY, trace fine sand and gravel, moderate plasticity with occasional shell fragments and black organic staining, behaves highly over consolidated.	G#237484 A-7-6, CL WC=34.0% LL=41 PL=22 PI=19
15	MV/6D	24/12	15.00 - 17.00	12/13/10/12	23	32		163.20		——15.00 Failed vane attempt, unable to push vane. Dark grey, wet, dense, fine to coarse SAND, some gravel and silt, gravel is angular, (Till).	G#237485 A-1-b, SM WC=9.8%
	7D	24/10	17.50 - 19.50	10/13/13/13	26	36				Black-dark grey, wet, dense, fine to coarse sandy GRAVEL, little silt, gravel appears to be crushed rock, (Till).	
20	8D	24/11	20.00 - 22.00	11/9/10/7	19	27	24			Brownish-grey, wet, medium dense, gravelly fine to coarse SAND, little silt, gravel is subrounded to angular, (Till).	
							36				
							32				
								154.90		——23.30 **Bottom of Exploration at 23.30 feet below ground surface.** CASING AND ROLLER CONE REFUSAL, presumably on bedrock	
25											

Remarks:

Strong odor of petroleum hydrocarbons in upper 5 ft of material.
Drilling behavior suggests boney material to approximately 7 ft. Three attempts with auger refusal between 5 and 7 ft. In third hole, drove HW casing to 5 ft, roller cone to 5 ft, NX core at 5 ft, dropped through boulder at 7 ft, N roller cone to 10 ft, telescope NW casing to 10 ft, washed out N roller cone to 10 ft.
Streambed approximately 10 ft below top of boring elevation on highway.

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-LCS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Cambolasse Bridge #2170 carring US Route 2 over Cambolasse Stream
Location: Lincoln, Maine

Boring No.: BB-LCS-102
PIN: 16712.00

Driller:	MaineDOT	**Elevation (ft.)**	178.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	Be Schonewald (IVS)	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	4/15/10; 07:50-10:20	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	N/A
Boring Location:	3+17.3, 17.5 Lt.	**Casing ID/OD:**	NW	**Water Level*:**	7.0' bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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.
	1D	24/17	1.00 - 3.00	18/17/13/10	30	42				Brown, damp, dense, gravelly fine to coarse SAND, trace silt, (Fill).	G#237486 A-1-a, SW-SM WC=3.2%
								175.00		3.00	
	2D	24/15	3.00 - 5.00	5/4/4/7	8	11				Brown, damp, medium dense to loose, fine to coarse SAND, some gravel, little to some silt, (Fill).	
5	3D	24/6	5.00 - 7.00	4/3/2/6	5	7				Drill attitude indicates cobbles and boulders between approximately 5 ft and 10 ft.	
	4D	24/12	7.70 - 9.70	9/13/19/27	32	45		170.30		7.70	
										Brown, wet, dense, fine to coarse sandy GRAVEL, little to some silt, (Fill).	
										Drill attitude indicates cobbles and boulders between approximeately 5 ft and 10 ft.	
10	5D	24/15	10.50 - 12.50	8/10/17/16	27	38		167.50		10.50	
										Dark grey, wet, hard, SILT and CLAY, low to moderate plasticity, behaves highly overconsolidated.	
	6D	24/17	12.50 - 14.50	5/5/4/6	9	13	39			Dark grey, wet, stiff, SILT and CLAY, low to moderate plasticity, behaves highly overconsolidated.	
							39				
							39				
15								163.00		15.00	
	MV/7D	24/24	15.00 - 17.00	WOH/4/3/4	7	10	41			Failed vane attempt, could not push. Dark grey, wet, stiff, clayey SILT, trace fine sand with few shell fragments, moderate plasticity, behaves slightly overconsolidated.	G#237487 A-7-6, CL WC=37.6% LL=41 PL=22 PI=19
							47				
	MV/8D	24/6	17.50 - 19.50	9/10/10/11	20	28	47			Failed vane attempt, could not push.	
							61	160.00		Dark grey, wet, very stiff CLAY and SILT, moderate plasticity with fine to coarse sand on tip of sample.	
							72			18.00	
20	9D	24/7	20.00 - 22.00	11/11/16/12	27	38	58			Grey, wet, dense, fine to coarse SAND, some gravel, little silt, (Till).	G#237488 A-1-b, SM WC=9.2%
							70				
							62				
							102				
								153.60		24.40	
25										**Bottom of Exploration at 24.40 feet below ground surface.** CASING AND ROLLER CONE REFUSAL, presumably on bedrock.	

Remarks:

Strong odor of petroleum hydrocarbons in upper 3 ft of material.
Drilling behavior suggests boney material to approximately 5 to 10 ft., cobbles and boulders.

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UNIFIED SOIL CLASSIFICATION SYSTEM

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

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

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

TERMS DESCRIBING DENSITY/CONSISTENCY

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

Modified Burmister System

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

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

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

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

Rock Quality Designation (RQD):

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

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

Correlation of RQD to Rock Mass Quality

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

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

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

Sample Container Labeling Requirements:

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

Appendix B

Laboratory Test Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Lincoln Project Number: 16712.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-LCS-101, 1D	2+83.7	11.1 Rt.	1.0-3.0	237481	1	2.8			SW-SM	A-1-a	0
BB-LCS-101, 2D	2+83.7	11.1 Rt.	3.0-4.8	237482	1	3.4			SW-SM	A-1-a	0
BB-LCS-101, 3D	2+83.7	11.1 Rt.	5.0-6.7	237483	1	15.1			SM	A-2-4	II
BB-LCS-101, 5D	2+83.7	11.1 Rt.	12.5-14.5	237484	1	34.0	41	19	CL	A-7-6	III
BB-LCS-101, 6D	2+83.7	11.1 Rt.	15.0-17.0	237485	1	9.8			SM	A-1-b	II
BB-LCS-102, 1D	3+17.3	17.5 Lt.	1.0-3.0	237486	2	3.2			SW-SM	A-1-a	0
BB-LCS-102, 7D	3+17.3	17.5 Lt.	15.0-17.0	237487	2	37.6	41	19	CL	A-7-6	III
BB-LCS-102, 9D	3+17.3	17.5 Lt.	20.0-22.0	237488	2	9.2			SM	A-1-b	II

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

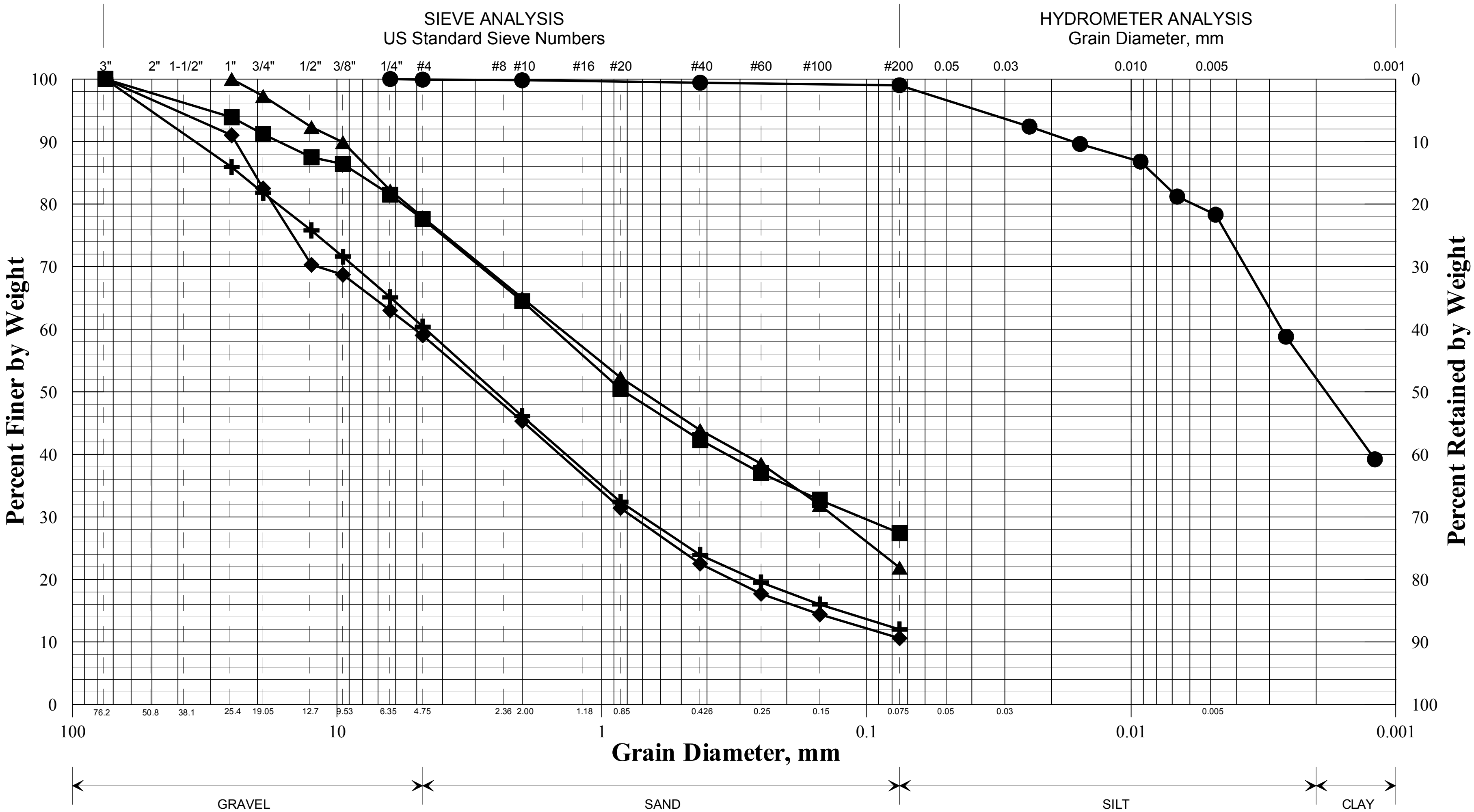
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

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

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

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

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



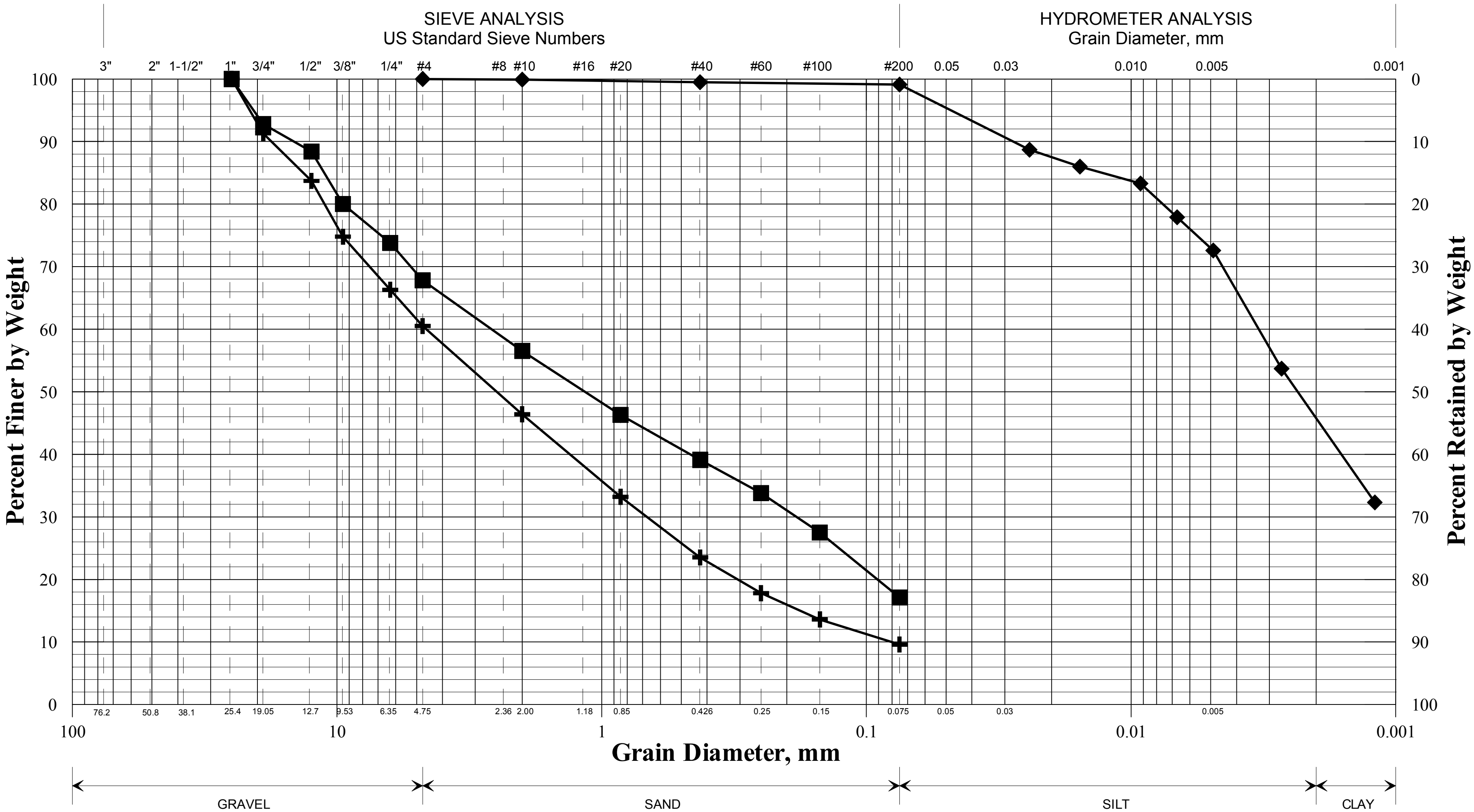
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LCS-101/1D	2+83.7	11.1 RT	1.0-3.0	Gravelly SAND, little silt.	2.8			
◆	BB-LCS-101/2D	2+83.7	11.1 RT	3.0-4.8	Gravelly SAND, little silt.	3.4			
■	BB-LCS-101/3D	2+83.7	11.1 RT	5.0-6.7	SAND, some silt, some gravel.	15.1			
●	BB-LCS-101/5D	2+83.7	11.1 RT	12.5-14.5	Silty CLAY, trace sand, trace gravel.	34.0	41	22	19
▲	BB-LCS-101/6D	2+83.7	11.1 RT	15.0-17.0	SAND, some gravel, some silt.	9.8			
✖									

PIN
016712.00
Town
Lincoln
Reported by/Date
WHITE, TERRY A 6/14/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE

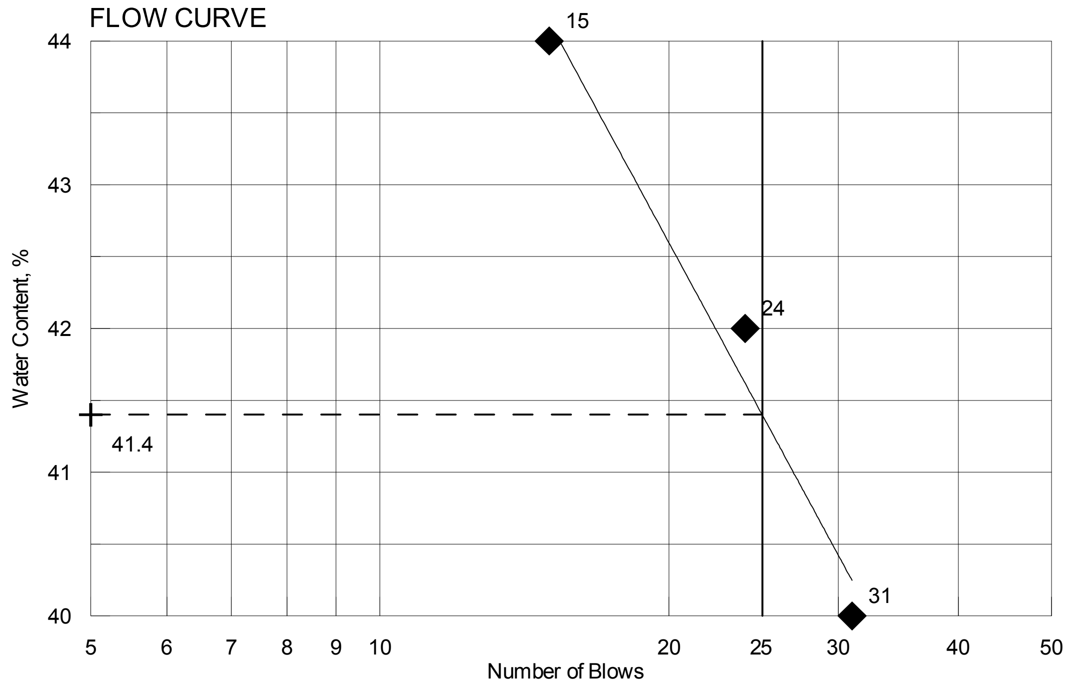


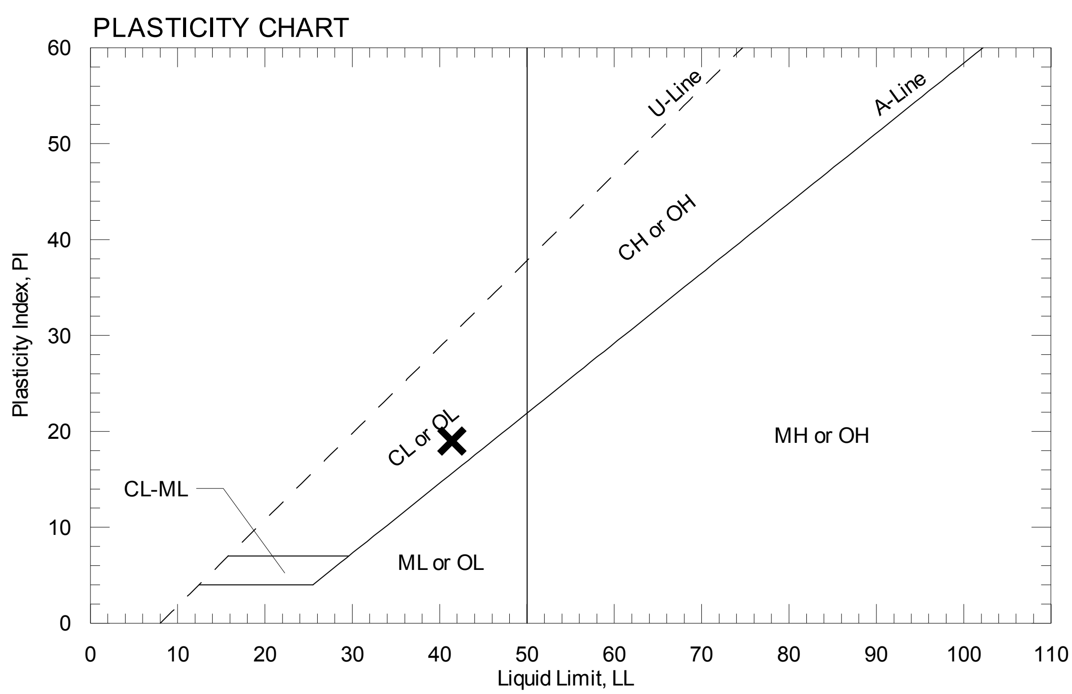
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-LCS-102/1D	3+17.3	17.5 LT	1.0-3.0	Gravelly SAND, trace silt.	3.2			
◆	BB-LCS-102/7D	3+17.3	17.5 LT	15.0-17.0	Clayey SILT, trace sand.	37.6	41	22	19
■	BB-LCS-102/9D	3+17.3	17.5 LT	20.0-22.0	SAND, some gravel, little silt.	9.2			
●									
▲									
✖									

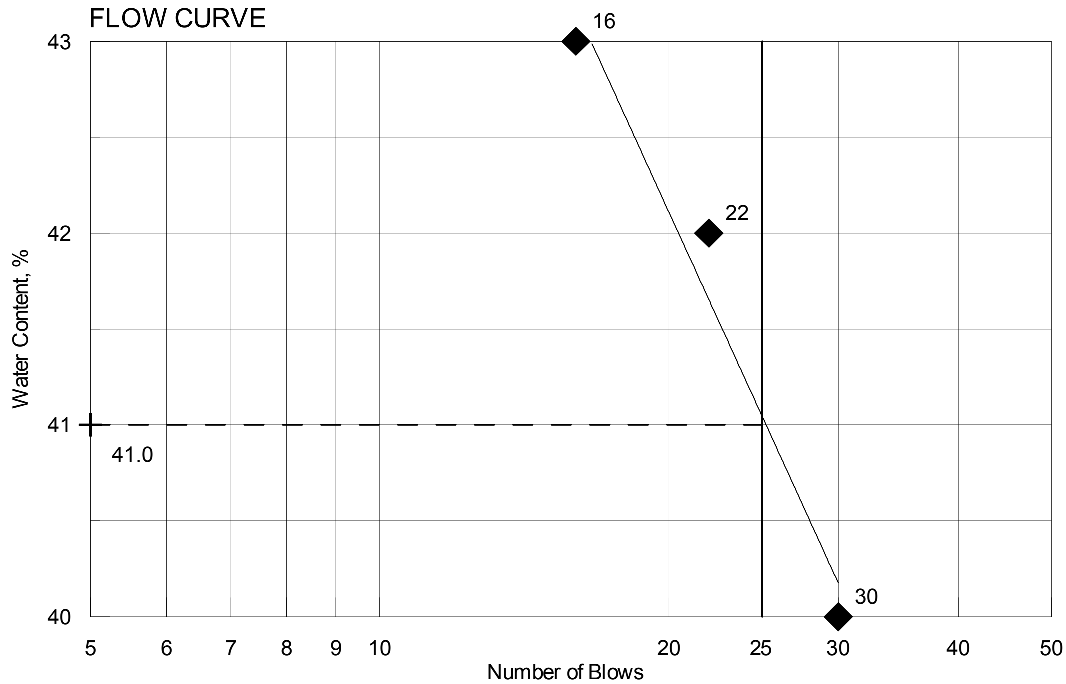
PIN
016712.00
Town
Lincoln
Reported by/Date
WHITE, TERRY A 6/14/2010

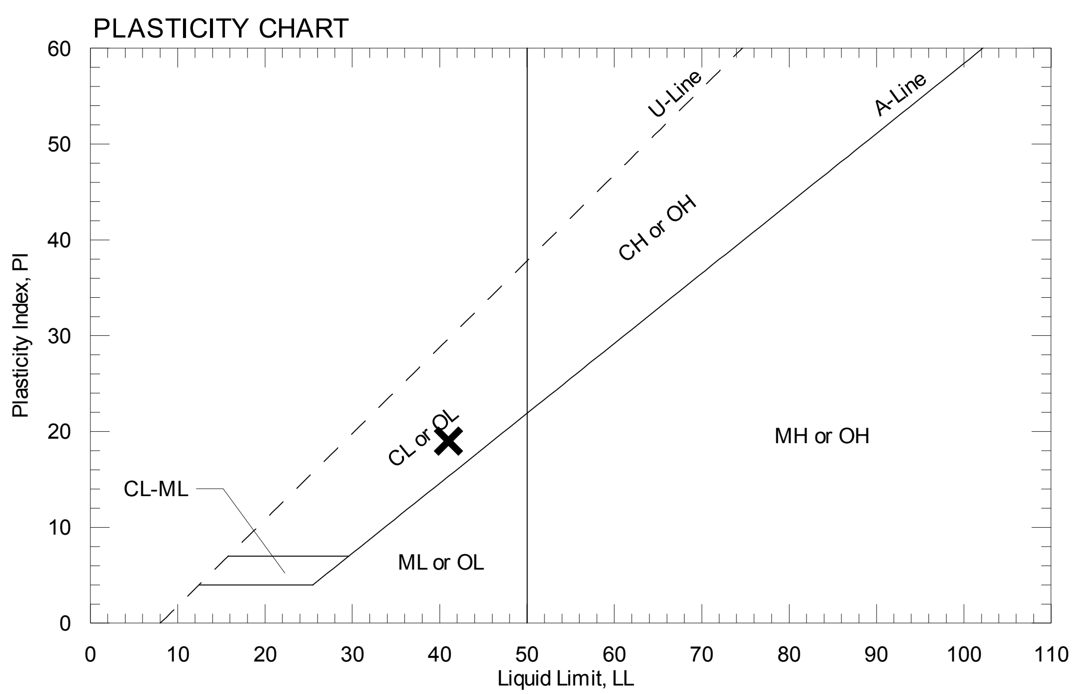
TOWN	Lincoln	Reference No.	237484
PIN	016712.00	Water Content, %	34
Sampled	4/14/2010	Plastic Limit	22
Boring No./Sample No.	BB-LCS-101/5D	Liquid Limit	41
Station	2+83.7	Plasticity Index	19
Depth	12.5-14.5	Tested By	KDRES





TOWN	Lincoln	Reference No.	237487
PIN	016712.00	Water Content, %	37.6
Sampled	4/15/2010	Plastic Limit	22
Boring No./Sample No.	BB-LCS-102/7D	Liquid Limit	41
Station	3+17.3	Plasticity Index	19
Depth	15.0-17.0	Tested By	KDRES





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 = 1880 degree-days
Site has Fine Grained Native Soils With Wn = 15% to 37%, Assume 20%

From the 2003 Bridge Design Guide Table 5-1:

$Frost\_depth := [0.8\cdot(56.7in - 55.1in) + 55.1in]$

$Frost\_depth = 56.38\cdot in$

$Frost\_depth = 4.7\cdot ft$

Method 2:

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

Project Location: Millinocket, Maine

Air Design Freezing Index         =  2048 F-days
N-Factor                          =  0.70
Surface Design Freezing Index     =  1434 F-days
Mean Annual Temperature           =  41.4 deg F
Design Length of Freezing Season  =  142 days

---------------------------------------------------------
Layer
#:Type            t    w%     d    Cf   Cu    Kf    Ku      L
---------------------------------------------------------
1-Asphalt        6.0    .1 140.0   28   28    .9    .9      0
2-Coarse        36.0   5.0 125.0   24   28   1.2   1.3    900
3-Fine           7.3  30.0 130.0   42   61   2.9   1.9  5,616
---------------------------------------------------------

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

*********************************************************
  Total Depth of Frost Penetration = 4.11 ft = 49.3 in.
*********************************************************
```

Use 4.5 feet

BEARING RESISTANCE ON COMPACTED FILL SOILS:

Consider this for use with Box Culverts and Headwalls.

SERVICE LIMIT STATE:

LRFD Table C10.6.2.6.1-1, (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)	Recommend Value
Coarse to Medium sand, little gravel	Very dense	8 to 12	8 ksf
	Medium dense to dense	4 to 8	6 ksf
	Loose	2 to 4	3 ksf

Recommend **6.0 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 on fill soils at the Strength Limit State:

Assumptions:

1. Box Culvert will be embedded 3.0 feet for frost protection.

   $D_f := 3.0ft$

2. Assumed parameters for soils:
   Assume granular fill

   Moist unit weight: $\gamma_m := 125pcf$

   Saturated unit weight: $\gamma_{sat} := 130pcf$

   Soil angle of internal friction: $\phi_{ns} := 32$

   Undrained shear strength (cohesion): $c_{ns} := 0psf$

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: $q_{eff\_str} := D_w \cdot \gamma_m + (D_f - D_w) \cdot (\gamma_{sat} - \gamma_w)$

$q_{eff\_str} = 0.2 \cdot ksf$

Box Culvert Width: $B := 14ft$

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 = 32 deg Bowles 5th Ed. Table 4-4 pg. 223

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

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

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

$$q_{nom} = 15.1 \cdot ksf$$

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

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

$$q_{fac} = 6.8 \cdot ksf$$

Recommend **Strength Limit State** Factored Bearing Resistance of **6.5 ksf** for the box culvert.

Appendix D

Special Provision

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

534.10 Description The Contractor shall design, manufacture, furnish, and install elements, precast structural concrete structures, arches, or box culverts and associated wings, headwalls, 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 Standard Specifications for Highway Bridges, current edition. The design live load shall be as follows: *modified HL-93 Strength I for LRFD method. *(modify HL-93 by increasing all wheel loads by a factor of 1.25)

The Contractor shall submit design calculations and 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. The Contractor shall submit a bridge rating on the Department's Standard Bridge Rating Summary Sheet with the design calculations. 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) 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 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.

<u>Installation of Precast Units</u> 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. Equipment in excess of 11 Mg [12 ton] shall not use the structure until a minimum of 600 mm [24 in] of backfill cover is in place and compacted.

<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, excavation, backfill material, 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