

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

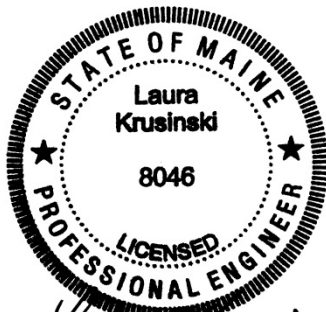
GEOTECHNICAL DESIGN REPORT

For the Replacement of:

**FARROW LAKE STREAM BRIDGE
STATE ROUTE 6 OVER DEADMAN STREAM
TOPSFIELD, MAINE**

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Washington County
WIN 21706.00

Soils Report No. 2018-41
Bridge No. 5378

STP-2170(600)
November 5, 2018

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1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present subsurface information and provide geotechnical design recommendations for the replacement of Farrow Lake Stream Bridge which carries State Route 6 over Deadman Stream in Topsfield, Maine. This report presents the subsurface information obtained at the site during the subsurface investigation, geotechnical design parameters, and construction recommendations for the new box culvert.

The existing structure was constructed in 1950 and consists of a single span reinforced concrete slab on mass concrete abutments. The existing bridge has a span of 10 feet and has a curb-to-curb deck width of 25.6 feet. The superstructure has heavy efflorescence emanating from the slab with multiple cracks and spalls. The substructure also has efflorescence emanating from several large full height cracks on both abutments. According to the 2016 Maine Department of Transportation (MaineDOT) Bridge Inspection Report, the superstructure, substructure, and deck are all rated as in poor condition. The existing bridge has a Federal Highway Administrations Sufficiency Rating of 39.7.

The proposed replacement structure is a 56-foot long precast concrete box culvert with a 15-foot span and 6-foot rise. The box culvert shall have 1-foot tall precast headwalls and inlet/outlet toe walls extending one foot below calculated scour depth. The upstream and downstream ends of the culvert will be slope-tapered 1.75H:1V (horizontal:vertical). The box culvert invert will be embedded 2 feet into the streambed and 2 feet of special fill will be placed inside the bottom of the culvert to create a natural streambed. The box shall be placed on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill bearing on compacted native soils or bedrock.

The new box culvert will be located on nearly the same horizontal alignment as the existing bridge. The finished grade over the proposed precast box culvert will increase approximately 1.75 feet over the existing grade. A temporary bridge will allow traffic to be maintained during construction of the new box culvert and provide one 11-foot wide lane of alternating two-way traffic on State Route 6.

2.0 GEOLOGIC SETTING

The existing structure carries State Route 6 over Farrow Lake (Deadman) Stream as shown on Sheet 1 – Location Map.

The Maine Geological Survey (MGS) Surficial Geology Map of the Fredericton 1° x 2° Quadrangle, Maine, Open-file No. 87-13 (1987), indicates the surficial soils in the vicinity of the bridge project consist of glacial till, bedrock with thin drift, and frequent bedrock outcrops nearby. Glacial till is a heterogeneous mixture of sand, silt, clay, and stones. These soils generally overly bedrock, but may overlie, or include, sand and gravel.

According to the MGS Bedrock Geology Map of the Danforth, Scraggly Lake, Forest, Waite, Vanceboro, and Kellyland 15' Quadrangles, Maine, Open-file No. 90-42 (1990), the project site is near a contact of the Flume Ridge Formation and the undivided Bottle Lake Complex. The

Flume Ridge Formation contains quartz-feldspar wacke, siltstone, and slate. The Bottle Lake Complex is comprised of intrusive granitic plutons.

3.0 SUBSURFACE INVESTIGATION

Two test borings explored subsurface conditions at the project location. Boring BB-TFLS-101 was drilled west of the existing structure. Borings BB-TFLS-102 was drilled east of the existing structure. The test boring locations are shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile.

The MaineDOT Drill Crew drilled the test borings on May 10, 2017. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

Borings were performed by using a combination of solid stem auger, cased wash boring, and rock coring techniques. Soil samples were typically obtained in at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration are recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. The MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated per ASTM D4633 “Standard Test Method for Energy Measurement for Dynamic Penetrometers” in April 2017. All N-values discussed in this report are corrected values computed by applying an average energy transfer of 0.854 to the raw field N-values. This hammer efficiency factor (0.854) and both the raw field N-value and corrected N-value (N_{60}) are shown on the boring logs.

Bedrock was cored in borings BB-TFLS-101 and BB-TFLS-102 using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core calculated. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector logged the subsurface conditions encountered. The MaineDOT geotechnical engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, reviewed boring logs and identified field-testing requirements. The borings were located in the field using taped measurements at the completion of the drilling program.

4.0 LABORATORY TESTING

A laboratory testing program was conducted on selected soil samples recovered from the test borings to assist in soil classification, evaluation of engineering properties of the soils, and geologic assessment of the project site. Laboratory testing consisted of two standard grain size analyses with natural water content. The results of soil tests are included as Appendix B – Laboratory Test Results. Moisture content information and other soil test results are also shown on the boring logs provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.0 SUBSURFACE CONDITIONS

Subsurface conditions encountered in the test borings generally consisted of fill material underlain

by metamorphic bedrock. The boring logs are provided in Appendix A – Boring Logs and on Sheet 3 – Boring Logs. A generalized subsurface profile is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile. The following paragraphs summarize the subsurface conditions encountered:

5.1 Fill Material

A fill unit was encountered in the test borings. The thickness was approximately 8.8 to 9.9 feet. The unit generally consisted of:

- Brown, damp, gravelly fine to coarse sand, little silt; and
- Brown to grey-brown, wet, sandy gravel, little to trace silt.

Corrected SPT N-values in the fill unit ranged from 28 to 142 blows per foot (bpf), indicating the fill is medium dense to very dense in consistency. Two grain size analyses of the fill material resulted in the soil being classified as A-1-a under the AASHTO Soil Classification System and GW and GW-GM under the Unified Soil Classification System (USCS). The natural water content of the samples tested ranged from approximately 12 to 13 percent.

5.2 Bedrock

Bedrock was encountered and cored in borings BB-TFLS-101 and BB-TFLS-102. Table 1 summarizes approximate depths to the bedrock core, corresponding approximate top of the bedrock core run, and RQD.

Boring	Station	Offset (feet)	Approximate Depth to Core (feet bgs)	Approximate Elevation of Top of Core (feet)	RQD (Core Run, %)
BB-TFLS-101	3+82.8	8.1 Rt	9.4	427.2	R1, 33
					R2, 27
BB-TFLS-102	4+13.7	8.3 Lt	10.4	426.5	R1, 50
					R2, 85

Table 1 – Summary of Approximate Bedrock Core Depths, Elevations, and RQD

The bedrock recovered from BB-TFLS-101 is identified as grey, fine grained, metasiltstone, hard, fresh, moderate to low angle joints spaced very close to close and occasionally healed with calcite. The upper two feet of bedrock recovered from BB-TFLS-102 is identified as grey, fine grained, schist, hard, fresh, steep to vertical foliation with breaks spaced very close to close. The remainder of bedrock recovered from BB-TFLS-102 is identified as grey, fine grained metasiltstone, hard, fresh, steep relic bedding with cross-cutting joints spaced close to moderately close and open. Detailed bedrock descriptions and the RQD are provided on the boring logs in Appendix A – Boring Logs and on Sheet 3 – Boring Logs.

5.3 Groundwater

Groundwater depths measured in the test borings ranged from 2.2 to 4.2 feet bgs. The measurements were recorded after completion of the test borings. Note that water was introduced into the borehole during drilling operations. Groundwater levels will fluctuate with seasonal changes, precipitation, runoff, river levels, and construction activities.

6.0 FOUNDATION ALTERNATIVES

Two replacement structure alternatives with distinct foundations were considered during preliminary design. One alternative was a precast concrete arch or frame structure placed on spread footings on bedrock. The other alternative was a precast concrete box with an integral bottom mat founded on soil.

The risk of encountering variable bedrock and the need to perform rock excavation was considered during the alternative selection process. Because the base slab of the box structure will be higher than the bottom of existing structure's footings, it is unlikely that rock excavation is necessary to install the precast box. Therefore, the precast concrete box alternative was selected because the box option is faster and more economical to construct.

7.0 GEOTECHNICAL DESIGN CONSIDERATIONS AND RECOMMENDATIONS

7.1 Precast Concrete Box Culvert Design and Construction

The proposed replacement structure will consist of a 56-foot-long precast concrete box culvert with slope tapered inlet and outlet walls. The box culvert will have 1-foot tall precast headwalls. To prevent undermining, the box culvert will have 2-foot tall inlet and outlet toe walls and riprap aprons. The bottom slab of the box culvert will be embedded approximately 2 feet into the streambed and 2 feet of engineered streambed material will be placed inside the culvert to create a natural streambed. The riprap apron should be embedded 6 inches into the streambed and covered with the engineered streambed material to provide continuity of the natural streambed.

Precast concrete box culverts are typically supplier-designed and are detailed on the contract plans with only basic layout and required hydraulic opening. The manufacturer selected by the Contractor is responsible for the design of the structure including determination of wall thickness, haunch thickness, and reinforcement. The design shall be designed in accordance with MaineDOT Standard Specification 534 – Precast Structural Concrete, MaineDOT Bridge Design Guide (BDG) Section 8 – Buried Structures, and American Association of State Highway and Transportation Officials Load Resistance and Factor Design Bridge Design Specifications, 8th Edition, 2017 with 2018 interims (LRFD).

The loading specified for the design of the box shall be Modified HL-93 Strength I in which the HS-20 design truck wheel loads are increased by a factor of 1.25. The precast concrete box culvert shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Article 3.4.1 and LRFD Section 12. The design should use Soil Type 4 as presented in the MaineDOT BDG Section 3.6 to calculate earth loads and earth pressures from the

soil envelope. The backfill properties are as follows: $\phi=32^\circ$, $\gamma = 125$ pcf.

The box culvert will be bedded on a 1-foot-thick leveling layer of Granular Borrow – Material for Underwater Backfill conforming to Standard Specification 703.19. 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 inches. The granular borrow backfill should be placed in lifts of 6 to 8 inches thick loose measure and compacted to the manufacturer's specifications. In no case shall the backfill soil be compacted less than 92 percent of the AASHTO T-180 maximum dry density. The precast concrete box culvert shall be installed in conformance with MaineDOT BDG Section 8 and MaineDOT Standard Specification Section 534.

7.1.1 Precast Concrete Box Culvert Headwalls

Concrete headwalls will be included in the culvert design to retain crushed stone slope protection and prevent stones from dropping or eroding into the waterway. Nominal 1-foot thick by 1-foot high concrete headwalls are recommended.

7.1.2 Precast Concrete Inlet and Outlet Walls

The precast concrete box culvert's outlet walls will have a 1.75H:1V (horizontal:vertical) slope taper. The left and right outlet walls will share the same precast base slab. The sloped outlet walls are essentially retaining walls and shall be designed for all relevant strength and service limit states and load combinations specified in LRFD Articles 3.4.1, 11.5.5 and 11.6. The outlet walls shall be designed to resist lateral earth pressures, vehicular loads, creep and temperature and shrinkage deformations of the concrete box culvert. The outlet walls shall be designed considering a live load surcharge equal to a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet per LRFD Article 3.11.6.4.

Outlet walls shall be fixed to the box culvert and be designed to resist movement using an at-rest earth pressure coefficient, K_o , of 0.47 assuming a level backslope. Wingwalls sections that are independent of the box culvert should be designed using the Rankine active earth pressure coefficient, K_a , of 0.46 assuming a 2H:1V backslope. The active earth pressure coefficient will change if the backslope conditions are different. See Appendix C – Calculations for supporting documentation.

7.1.3 Concrete Inlet and Outlet Toe Walls

Toe walls shall extend below the bottom slab connecting the left and right walls at the inlet and outlet of the box culvert to prevent undermining per MaineDOT BDG Section 8.3.1. The inlet and outlet toe walls should extend a minimum of 1 foot below the maximum depth of scour. Casting the toe walls directly on bedrock may be required.

7.1.4 Bearing Resistance

The precast concrete box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill with the bottom of box excavation ranging from approximately 427.5 to 427.0 feet. The coarse-grained fill and reworked soils at this elevation are expected to

be medium to very dense in consistency. These soils are characterized as having adequate bearing resistance.

For a precast concrete box culvert with a base width of 17 feet, the factored bearing stress at the strength limit state shall not exceed the calculated factored bearing resistance of 11 kips per square foot (ksf). To control settlement, the factored bearing stress at the service limit state shall not exceed a bearing resistance of 5 ksf. The service limit state bearing resistance may govern the design. In no instance shall bearing stress exceed the nominal structural resistance of the structural concrete which may be taken as $0.3f'_c$. See Appendix C – Calculations for supporting calculations.

7.2 Settlement

The 9 to 10-foot-thick fill unit encountered at the site is medium dense to very dense in consistency. The proposed roadway grade is 1.75 feet above the existing grade. The coarse-grained fill soils are cohesionless and will undergo elastic, immediate, compression in response to the increase in vertical overburden pressure due to the proposed grade raise. Any settlement due to the grade raise is anticipated to be small and will occur relatively quickly during construction.

Any bedrock encountered at the foundation elevation should be excavated to a minimum of 1 foot below the foundation elevation and replaced with Granular Borrow – Material for Underwater Backfill. Any loose or soft soils encountered at the foundation elevation for the precast box culvert should be excavated in its entirety and replaced with Granular Borrow – Material for Underwater Backfill.

7.3 Frost Protection

Foundations placed on the native soils should be designed with an appropriate embedment for frost protection. According to MaineDOT BDG Figure 5-1, Maine Design Freezing Index Map, Topsfield has a design freezing index (DFI) of approximately 1850 F-degree days. A water content of 15% was used for coarse-grained soils. These components correlate to a frost depth of 7.0 feet. A similar analysis was performed using Modberg software by the US Army Cold Regions Research and Engineering Laboratory (CRREL). For the Modberg analysis, Madison, Maine lies near the 1850 F-degree isoline of Topsfield. Madison has an air DFI from the Modberg database of approximately 1847 F-degree days. A water content of 15% was assumed. These components correlate to a frost depth of approximately 7.3 feet.

Based on the MaineDOT BDG methodology it is recommended that foundations bearing on coarse-grained soils be designed with an embedment of approximately 7.0 feet for frost protection. See Appendix C – Calculations for supporting calculations.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

7.4 Scour and Riprap

The box culvert shall be constructed with integral concrete headwalls and inlet and outlet walls to retain stone slopes and prevent stone slope protection from dropping or eroding into the waterway. Inlet and outlet toe walls shall be provided that extend a minimum of 1-foot below the maximum depth of scour. Inlet and outlet toe walls shall also be protected with riprap aprons.

Where required, slopes shall be armored with a 3-foot thick layer of riprap conforming to MaineDOT Standard Specification 703.26 - Plain and Hand Laid Riprap. The riprap shall be underlain by a Class 1 erosion control geotextile and a 1-foot layer of bedding material conforming to MaineDOT Standard Specification 703.19 Granular Borrow Material for Underwater Backfill. The toe of the riprap sections shall be constructed 1-foot below the streambed elevation. The riprap slopes shall be constructed no steeper than 1.75H:1V extending from the edge of the roadway down to the existing ground surface. Riprap aprons will be installed at both ends of the culvert.

7.5 Seismic Design Considerations

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

7.6 Construction Considerations

The soil envelope and backfill for the box culvert shall consist of Standard Specification 703.19 – Granular Borrow Material for Underwater Backfill with a maximum particle size of 4 inches. The granular borrow backfill should be placed in lifts of 6- to 8-inches-thick loose measure and compacted to the manufacturer's specifications. To minimize future settlement, the envelope and backfill soil shall be compacted to no less than 92 percent of the AASHTO T-180 maximum dry density.

The proposed box culvert will be bedded on a 1-foot-thick layer of Granular Borrow – Material for Underwater Backfill, conforming to Standard Specification 703.19. Based on the soils encountered in the borings, medium dense to very dense, coarse-grained material will be present at the bearing elevations.

The Contractor shall minimize disturbance to the subgrade surface and protect the subgrade surface from any unnecessary construction traffic. Any cobbles, boulders, or bedrock encountered at the bearing elevation shall be removed and replaced with compacted Granular Borrow – Material for Underwater Backfill.

Earthwork and excavations may result in the exposure of silt or other soft soils. These soils may be susceptible to disturbance and rutting as a result of exposure to water or construction traffic. If disturbance and rutting occur, the Contractor shall remove and replace the disturbed materials with compacted Granular Borrow – Material for Underwater Backfill.

Soils may become saturated and water seepage may be encountered during construction and in

excavations. There may be localized sloughing and instability in some excavations and cut slopes. The Contractor should control groundwater and surface water infiltration using temporary ditches, sump pumps, granular drainage blankets, stone ditch protection, or hand-laid riprap with geotextile underlayment to divert groundwater and surface water.

8.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of Farrow Lake Stream Bridge in Topsfield, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

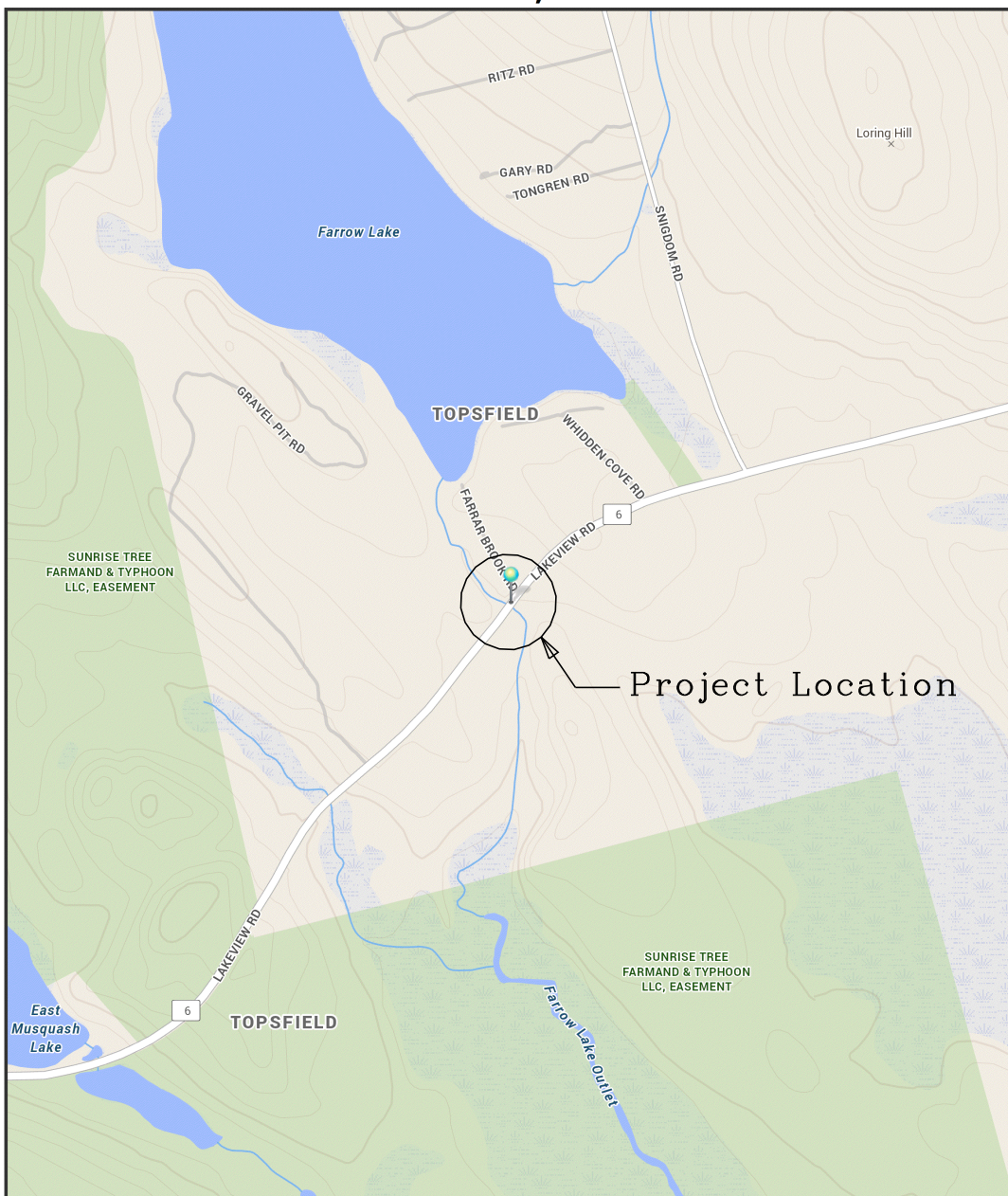
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon limited subsurface investigations at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is recommended that the geotechnical engineer be provided the opportunity for a review of the design and specifications so that the earthwork and foundation recommendations and construction considerations in the report are properly interpreted and implemented in the design and specifications.

Sheets



TOPSFIELD, MAINE

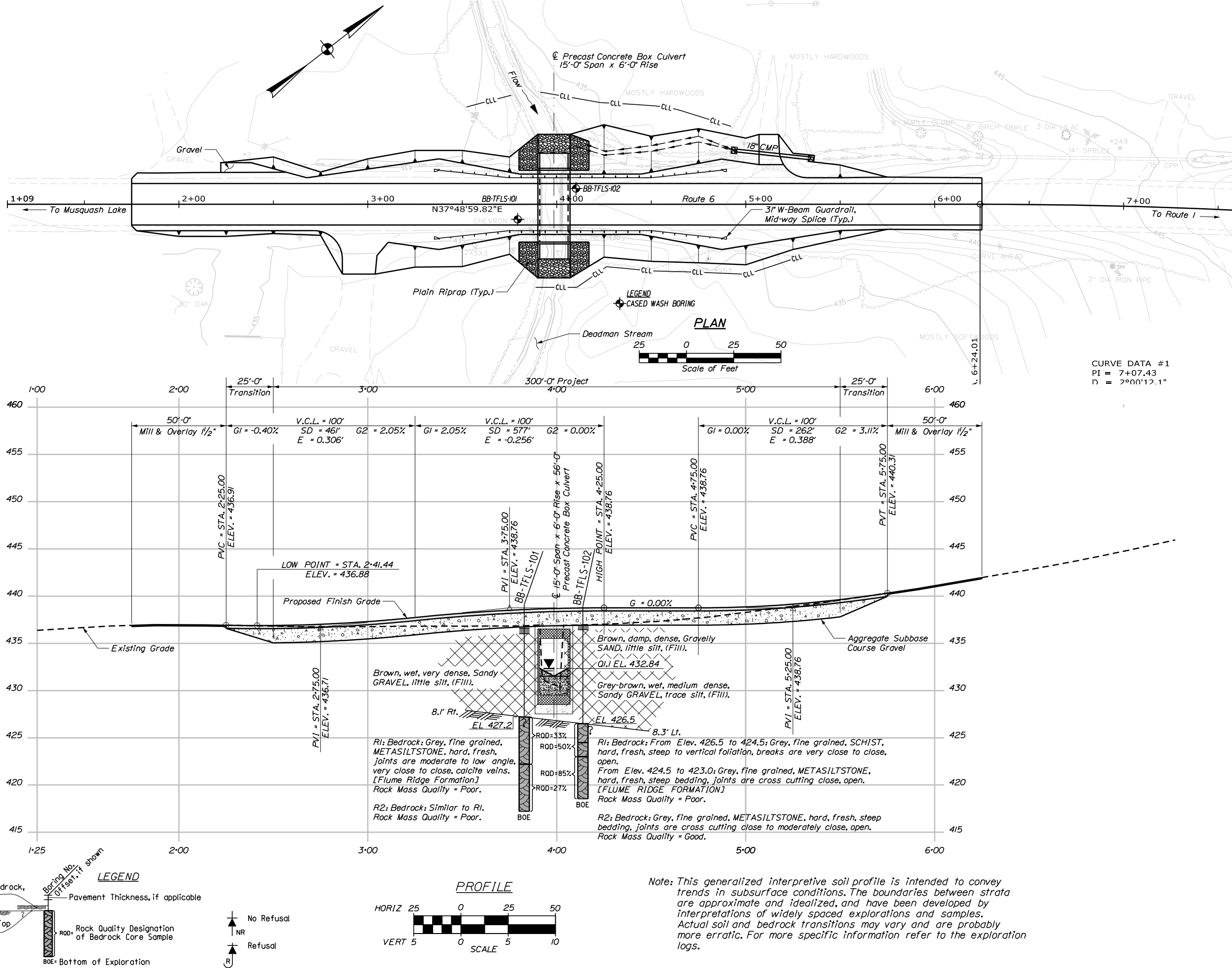


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0.25 Miles
1 inch = 0.28 miles

Date: 1/22/2018
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<div>SHEET NUMBER</div> <div>1</div> <div>OF 3</div>	<div>FARROW LAKE STREAM BRIDGE</div> <div>DEADMAN STREAM</div> <div>TOPSFIELD WASHINGTON COUNTY</div>	<div>STATE OF MAINE</div> <div>DEPARTMENT OF TRANSPORTATION</div>	
		<div>STP-2170(600)</div>	
	<div>LOCATION MAP</div>	<div>WIN</div> <div>BRIDGE NO. 5378 21706.00 BRIDGE PLANS</div>	



PROJ. MANAGER	DATE	BY	SIGNATURE
CHECKED-REVIEWED			
DESIGNED-DETAILED			
DESIGNED-DETAILED			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

FARROW LAKE STREAM BRIDGE	WASHINGTON COUNTY
DEADMAN STREAM	
TOPSFIELD	
BORING LOCATION PLAN & INTERPRETIVE SUBSURFACE PROFILE	

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

Project:Farrow Lake Stream Bridge #5378
carries Route 6 over Farrow Lake
Location: Topsfield, Maine

Boring No.: BB-TFLS-101

WIN: 21706.00

Driller: MaineDOT

Elevation (ft.): 436.6

Auger ID/OD: 5" Solid Stem

Operator: Travis/James/Rick

Datum: NAVD88

Sampler: Standard Split Spoon

Logged By: B. Wilder

Rig Type: CME 45C

Hammer Wt./Fall: 140#/30"

Date Start/Finish: 5/10/2017: 08:00-11:00

Drilling Method: Cased Wash Boring

Core Barrel: N0-2"

Boring Location: 3+82.8, 8.1 ft Rt.

Casing ID/OD: NW-3"

Water Level*: 2.2 ft bgs.

Hammer Efficiency Factor: 0.854

Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
V = Field Vane Shear Test
NW = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WDI = Weight of 140lb. Hammer
WDR/C = Weight of Rods or Casing
NDIP = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
S_u(lab) = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N = uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
N₆₀ = (Hammer Efficiency Factor/60%)N-uncorrected

T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
G = Grain Size Analysis
C = Consolidation Test

Sample Information										Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength or Rod (s))	N-uncorrected	N ₆₀	Casing	Blows					
0								SSA	436.0		7" HMA.	G#303872 A-1-a, GM WC=12.0%	
											(Visual description from auger flight) Brown, damp, Gravelly fine to coarse SAND, little silt, (Fill).		
5	1D	18/13	5.00 - 6.50	29/47/53	100	142					Brown, wet, very dense, Sandy GRAVEL, little silt, (Fill).		
10	R1	60/53	9.40 - 14.40	ROD = 33%				N0-2	427.2		Set in NW Casing.		
											Top of Bedrock at Elev. 427.2 ft. R1: Bedrock: Grey, fine grained, METASILTSTONE, hard, fresh, joints are moderate to low angle, very close to close, calcite veins. [Flume Ridge Formation] Rock Mass Quality = Poor. R1: Core Times (min:sec) 9.4-10.4 ft (2:17) 10.4-11.4 ft (2:28) 11.4-12.4 ft (3:13) 12.4-13.4 ft (3:04) 13.4-14.4 ft (3:00) 88% Recovery		
15	R2	60/57	14.40 - 19.40	ROD = 27%							R2: Bedrock: Similar to R1. Rock Mass Quality = Poor. R2: Core Times (min:sec) 14.4-15.4 ft (3:18) 15.4-16.4 ft (3:29) 16.4-17.4 ft (3:06) 17.4-18.4 ft (3:08) 18.4-19.4 ft (3:36) 95% Recovery		
20									417.2		Bottom of Exploration at 19.4 feet below ground surface.		
25													
30													

Remarks:

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

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

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Boring No.: BB-TFLS-101

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

Project:Farrow Lake Stream Bridge #5378
carries Route 6 over Farrow Lake
Location: Topsfield, Maine

Boring No.: BB-TFLS-102

WIN: 21706.00

Driller: MaineDOT

Elevation (ft.): 436.9

Auger ID/OD: 5" Solid Stem

Operator: Travis/James/Rick

Datum: NAVD88

Sampler: Standard Split Spoon

Logged By: B. Wilder

Rig Type: CME 45C

Hammer Wt./Fall: 140#/30"

Date Start/Finish: 5/10/2017: 11:30-13:00

Drilling Method: Cased Wash Boring

Core Barrel: N0-2"

Boring Location: 4+13.7, 8.3 ft Lt.

Casing ID/OD: NW-3"

Water Level*: 4.2 ft bgs.

Hammer Efficiency Factor: 0.854

Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
V = Field Vane Shear Test
NW = Unsuccessful Field Vane Shear Test Attempt

R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WDI = Weight of 140lb. Hammer
WDR/C = Weight of Rods or Casing
NDIP = Weight of One Person

S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
S_u(lab) = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N = uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency
N₆₀ = (Hammer Efficiency Factor/60%)N-uncorrected

T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
G = Grain Size Analysis
C = Consolidation Test

Sample Information										Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength or Rod (s))	N-uncorrected	N ₆₀	Casing	Blows					
0								SSA	436.4		6" HMA.	G#303873 A-1-a, GW-GM WC=13.1%	
	1D	24/12	1.00 - 3.00	9/10/23/19	33	47					Brown, damp, dense, Gravelly SAND, little silt, (Fill).		
5	2D	24/14	5.00 - 7.00	5/11/9/8	20	28		8	433.4		Grey-brown, wet, medium dense, Sandy GRAVEL, trace silt, (Fill).		
								43					
								18					
								56					
10	MD R1	4.8/0 42/42	10.00 - 10.40 - 10.40 - 13.90	36(4.8") ROD = 50%	---			N0-2	426.5		Failed sample attempt.		
											Top of Bedrock at Elev. 426.5 ft. R1: Bedrock: From Elev. 426.5 to 424.5: Grey, fine grained, SCHIST, hard, fresh, steep to vertical foliation, breaks are very close to close, open. From Elev. 424.5 to 423: Grey, fine grained, METASILTSTONE, hard, fresh, steep bedding, joints are cross cutting close, open. [FLUME RIDGE FORMATION] Rock Mass Quality = Poor. R1: Core Times (min:sec) 10.4-11.4 ft (2:43) 11.4-12.4 ft (3:08) 12.4-13.4 ft (6:06) 13.4-13.9 ft (4:00) Core Blocked 100% Recovery		
15	R2	54/54	13.90 - 18.40	ROD = 85%							R2: Bedrock: Grey, fine grained, METASILTSTONE, hard, fresh, steep bedding, joints are cross cutting close to moderately close, open. Rock Mass Quality = Good. R2: Core Times not given. 100% Recovery		
20									418.5		Bottom of Exploration at 18.4 feet below ground surface.		
25													
30													

Remarks:

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

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

Page 1 of 1

Boring No.: BB-TFLS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2170(600)
WIN 21706.00
BRIDGE NO. 5378
BRIDGE PLANS

FARROW LAKE STREAM BRIDGE
DEADMAN STREAM
TOPSFIELD WASHINGTON COUNTY
BORING LOGS

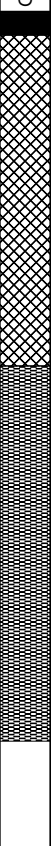
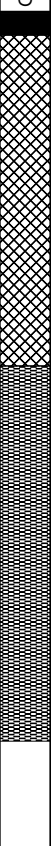
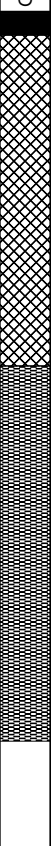
SHEET NUMBER
3
OF 3

PROJ. MANAGER	BY	DATE	SIGNATURE	P.E. NUMBER	DATE
DESIGN-DETAILED					
CHECKED-REVIEWED					
DESIGN-DETAILED	T. WHITE	OCT 2018			
DESIGN-DETAILED	B. SLAVEN				
REVISIONS 1					
REVISIONS 2					
REVISIONS 3					
REVISIONS 4					
FIELD CHANGES					

Appendix A

Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM					MODIFIED BURMISTER 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	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.	<u>Descriptive Term</u>		<u>Portion of Total (%)</u>	
		(little or no fines)	GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.	trace	0 - 10		
					little	11 - 20		
					some	21 - 35		
					adjective (e.g. sandy, clayey)	36 - 50		
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.	TERMS DESCRIBING DENSITY/CONSISTENCY			
		GC	Clayey gravels, gravel-sand-clay mixtures.					
		CLEAN SANDS	SW	Well-graded sands, gravelly sands, little or no fines	<u>Density of Cohesionless Soils</u>		<u>Standard Penetration Resistance N-Value (blows per foot)</u>	
		(little or no fines)	SP	Poorly-graded sands, gravelly sand, little or no fines.	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 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.	<u>Consistency of Cohesive soils</u>		<u>SPT N-Value (blows per foot)</u>	<u>Approximate Undrained Shear Strength (psf)</u>	<u>Field Guidelines</u>
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	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	
	SILTS AND CLAYS (liquid limit greater than 50)	OL	Organic silts and organic silty clays of low plasticity.	Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail	
				Hard	>30	over 4000	Indented by thumbnail with difficulty	
		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	Rock Quality Designation (RQD):				
		CH	Inorganic clays of high plasticity, fat clays.	RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$				
		OH	Organic clays of medium to high plasticity, organic silts.	*Minimum NQ rock core (1.88 in. OD of core)				
HIGHLY ORGANIC SOILS	Pt	Peat and other highly organic soils.	Correlation of RQD to Rock Mass Quality					
			<u>Rock Mass Quality</u>		<u>RQD (%)</u>			
Desired Soil Observations (in this order, if applicable): Color (Munsell color chart) Moisture (dry, damp, moist, wet) Density/Consistency (from above right hand side) Texture (fine, medium, coarse, etc.) Name (sand, silt, 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.,) Cementation (weak, moderate, or strong) Geologic Origin (till, marine clay, alluvium, etc.) Groundwater level					Very Poor	≤25		
					Poor	26 - 50		
					Fair	51 - 75		
					Good	76 - 90		
					Excellent	91 - 100		
					Desired Rock Observations (in this order, if applicable): Color (Munsell color chart) Texture (aphanitic, fine-grained, etc.) Rock Type (granite, schist, sandstone, 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 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.) -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet) -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: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))			
					Sample Container Labeling Requirements:			
					WIN	Blow Counts		
					Bridge Name / Town	Sample Recovery		
					Boring Number	Date		
Sample Number	Personnel Initials							
Sample Depth								
Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information								

<div>Maine Department of Transportation</div> <div>Soil/Rock Exploration Log</div> <div>US CUSTOMARY UNITS</div>				<div>Project: Farrow Lake Stream Bridge #5378 carries</div> <div>Route 6 over Farrow Lake Stream</div> <div>Location: Topsfield, Maine</div>		<div>Boring No.: BB-TFLS-101</div> <div>WIN: 21706.00</div>																																																																																																																																																																																																																																																																																																																																																										
Driller: MaineDOT		Elevation (ft.): 436.6		Auger ID/OD: 5" Solid Stem																																																																																																																																																																																																																																																																																																																																																												
Operator: Travis/James/Rick		Datum: NAVD88		Sampler: Standard Split Spoon																																																																																																																																																																																																																																																																																																																																																												
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"																																																																																																																																																																																																																																																																																																																																																												
Date Start/Finish: 5/10/2017; 08:00-11:00		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"																																																																																																																																																																																																																																																																																																																																																												
Boring Location: 3+82.8, 8.1 ft Rt.		Casing ID/OD: NW-3"		Water Level*: 2.2 ft bgs.																																																																																																																																																																																																																																																																																																																																																												
Hammer Efficiency Factor: 0.854		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>																																																																																																																																																																																																																																																																																																																																																														
<div>Definitions:</div> <div>D = Split Spoon Sample</div> <div>MD = Unsuccessful Split Spoon Sample Attempt</div> <div>U = Thin Wall Tube Sample</div> <div>MU = Unsuccessful Thin Wall Tube Sample Attempt</div> <div>V = Field Vane Shear Test, PP = Pocket Penetrometer</div> <div>MV = Unsuccessful Field Vane Shear Test Attempt</div> <div>R = Rock Core Sample</div> <div>SSA = Solid Stem Auger</div> <div>HSA = Hollow Stem Auger</div> <div>RC = Roller Cone</div> <div>WOH = Weight of 140lb. Hammer</div> <div>WOR/C = Weight of Rods or Casing</div> <div>WO1P = Weight of One Person</div> <div>S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)</div> <div>S_{u(lab)} = Lab Vane Undrained Shear Strength (psf)</div> <div>q_p = Unconfined Compressive Strength (ksf)</div> <div>N-uncorrected = Raw Field SPT N-value</div> <div>Hammer Efficiency Factor = Rig Specific Annual Calibration Value</div> <div>N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency</div> <div>N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected</div> <div>T_v = Pocket Torvane Shear Strength (psf)</div> <div>WC = Water Content, percent</div> <div>LL = Liquid Limit</div> <div>PL = Plastic Limit</div> <div>PI = Plasticity Index</div> <div>G = Grain Size Analysis</div> <div>C = Consolidation Test</div>																																																																																																																																																																																																																																																																																																																																																																
<table><tr><th colspan="8">Sample Information</th><th rowspan="2">Elevation (ft.)</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Laboratory Testing Results/ AASHTO and Unified Class.</th></tr><tr><th>Depth (ft.)</th><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N₆₀</th><th>Casing Blows</th></tr><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>SSA</td><td>436.0</td><td rowspan="10"></td><td>7" HMA.</td><td rowspan="10">G#303872 A-1-a, GM WC=12.0%</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>(Visual description from auger flight) Brown, damp, Gravelly fine to coarse SAND, little silt, (Fill).</td></tr><tr><td>5</td><td>1D</td><td>18/13</td><td>5.00 - 6.50</td><td>29/47/53</td><td>100</td><td>142</td><td></td><td></td><td>Brown, wet, very dense, Sandy GRAVEL, little silt, (Fill).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>10</td><td>R1</td><td>60/53</td><td>9.40 - 14.40</td><td>RQD = 33%</td><td></td><td></td><td>NQ-2</td><td>427.2</td><td>Set in NW Casing.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Top of Bedrock at Elev. 427.2 ft.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>R1: Bedrock: Grey, fine grained, METASILTSTONE, hard, fresh, joints are moderate to low angle, very close to close, calcite veins. 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Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	0							SSA	436.0		7" HMA.	G#303872 A-1-a, GM WC=12.0%										(Visual description from auger flight) Brown, damp, Gravelly fine to coarse SAND, little silt, (Fill).	5	1D	18/13	5.00 - 6.50	29/47/53	100	142			Brown, wet, very dense, Sandy GRAVEL, little silt, (Fill).											10	R1	60/53	9.40 - 14.40	RQD = 33%			NQ-2	427.2	Set in NW Casing.										Top of Bedrock at Elev. 427.2 ft.										R1: Bedrock: Grey, fine grained, METASILTSTONE, hard, fresh, joints are moderate to low angle, very close to close, calcite veins. [Flume Ridge Formation]										Rock Mass Quality = Poor.										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* 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-TFLS-101																																																																																																																																																																																																																																																																																																																																																						

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Farrow Lake Stream Bridge #5378 carries Route 6 over Farrow Lake Stream Location: Topsfield, Maine		Boring No.: BB-TFLS-102 WIN: 21706.00	
Driller: MaineDOT		Elevation (ft.) 436.9		Auger ID/OD: 5" Solid Stem			
Operator: Travis/James/Rick		Datum: NAVD88		Sampler: Standard Split Spoon			
Logged By: B. Wilder		Rig Type: CME 45C		Hammer Wt./Fall: 140#/30"			
Date Start/Finish: 5/10/2017; 11:30-13:00		Drilling Method: Cased Wash Boring		Core Barrel: NQ-2"			
Boring Location: 4+13.7, 8.3 ft Lt.		Casing ID/OD: NW-3"		Water Level*: 4.2 ft bgs.			
Hammer Efficiency Factor: 0.854		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field 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 = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected						T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test	

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
0							SSA	436.4		G#303873 A-1-a, GW-GM WC=13.1%		
	1D	24/12	1.00 - 3.00	9/10/23/19	33	47						
5	2D	24/14	5.00 - 7.00	5/11/9/8	20	28	8					
							43					
							18					
							56					
							262					
10	MD R1	4.8/0 42/42	10.00 - 10.40 10.40 - 13.90	36(4.8") RQD = 50%	---		NQ-2					
	R2	54/54	13.90 - 18.40	RQD = 85%								
15												
20												
25												
30												

Remarks:

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-TFLS-102

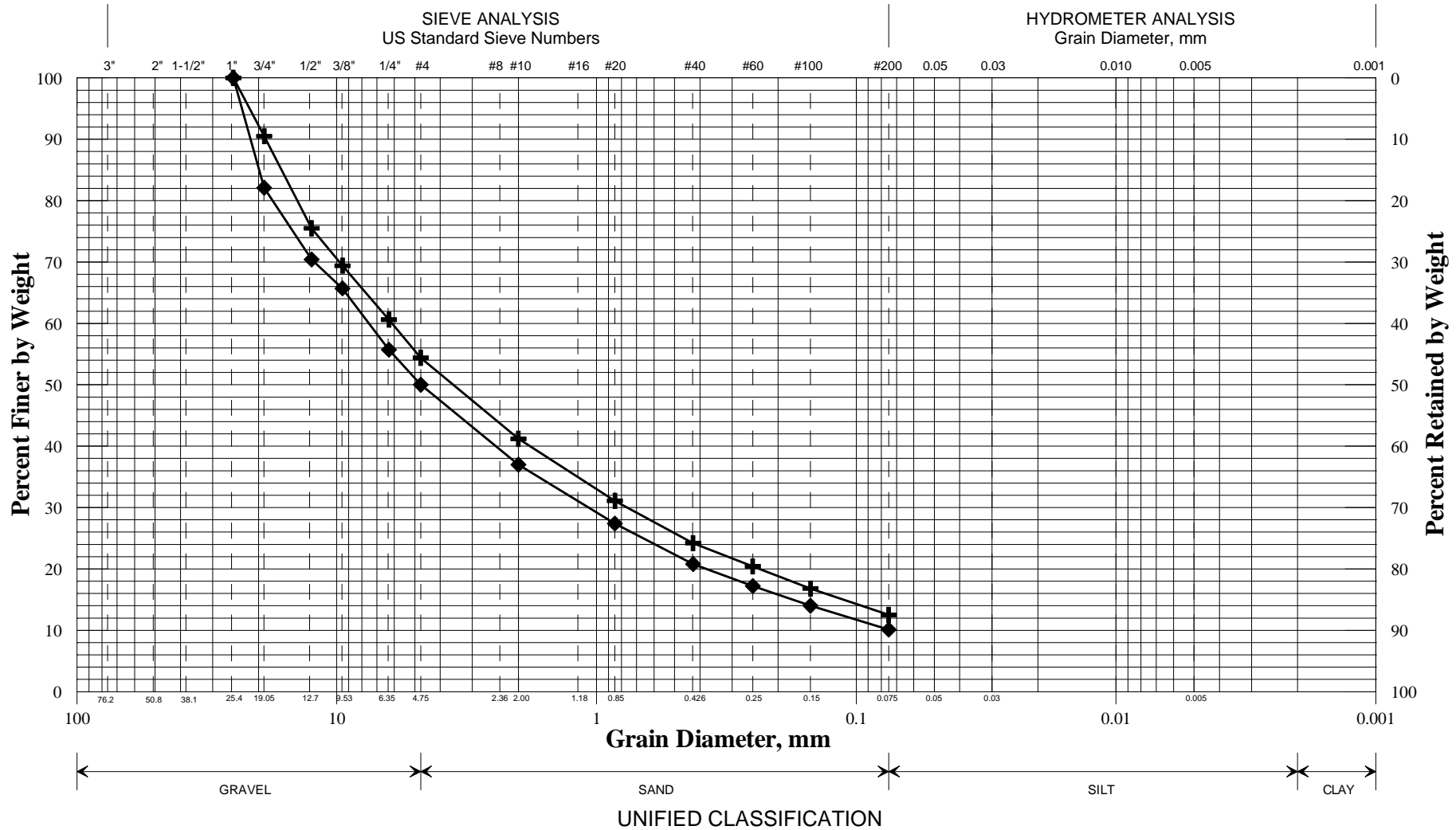
Appendix B

Laboratory Test Results

Work Number: 21706.00

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-TLFS-101/1D	3+82.8	8.1 RT	5.0-6.5	Sandy GRAVEL, little silt.	12.0			
◆	BB-TFLS-102/2D	4+13.7	8.3 LT	5.0-7.0	Sandy GRAVEL, trace silt.	13.1			
■									
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WIN	
021706.00	
Town	
Topsfield	
Reported by/Date	
WHITE, TERRY A	6/9/2017

Appendix C

Calculations

Earth Pressure

Soil Parameters:

Assume existing material removed and replaced with material with properties similar to Soil Type 4, MaineDOT BDG Section 3.6.1.

Unit weight	$\gamma := 125 \cdot \text{pcf}$
Internal friction angle	$\phi := 32 \cdot \text{deg}$
Cohesion	$c := 0 \cdot \text{psf}$

1. Outlet walls fixed to box - At-Rest Earth Pressure - Rankine Theory

$$K_o := 1 - \sin(\phi)$$

Fang, Foundation Engineering
Handbook 2nd ed. Pg. 224, Eq. 6.2

$$K_o = 0.47$$

Recommend: At-Rest Earth Pressure Coefficient, $K_o = 0.47$

2. Independent walls free to rotate - Active Earth Pressure - Rankine Theory

The earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of the soil on the inside of the vertical plane is considered as part of the wall weight. The failure sliding surface is not restricted by the top of the wall or back face of wall.

For cantilever walls with horizontal backslope:

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

MaineDOT BDG Pg. 3-7

$$K_{ar} = 0.31$$

For a sloped 2H:1V backfill

$$\beta = \text{Angle of fill slope to the horizontal} \quad \beta := 26.56 \cdot \text{deg}$$

$$K_{ar_slope} := \cos(\beta) \cdot \frac{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

MaineDOT BDG Pg. 3-7

$$K_{ar_slope} = 0.46$$

Pa is oriented at an angle of β to the vertical plane - See MaineDOT Bridge Design Guide Figure 3-3 attached.

Figure 3-2 Calculating β with Broken Backfill Surface

Rankine theory, as described in Section 3.6.5.2, may also be used for the design of yielding walls, for a simplified analysis (at the Structural Designer's option). The use of Rankine theory will result in a slightly more conservative design.

3.6.5.2 Rankine Theory

Rankine theory should be used for long-heeled cantilever walls. Refer to AASHTO LRFD Figure C3.11.5.3-1 (a) for the definition of a long heeled cantilever wall. For simplicity (at the Structural Designer's option), Rankine theory may also be used to compute lateral earth pressures on any yielding wall listed in 3.6.5.1 Coulomb Theory, although its use will result in a slightly more conservative design.

For these cases, interface friction between the wall backface and the backfill is not considered. Rankine earth pressure is applied to a plane extending vertically from the heel of the wall base, as shown in Figure 3-3.

For a horizontal backfill surface where $\beta = 0^\circ$, the value of the coefficient of active earth pressure (Rankine), K_a , may be taken as:

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

where:

ϕ = angle of internal soil friction (degrees), taken from Table 3-3.

β = angle of backfill to the horizontal (degrees), as shown in Figure 3-3.

For a sloped backfill surface where $\beta > 0^\circ$, the coefficient of active earth pressure (Rankine), K_a , may be taken as:

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

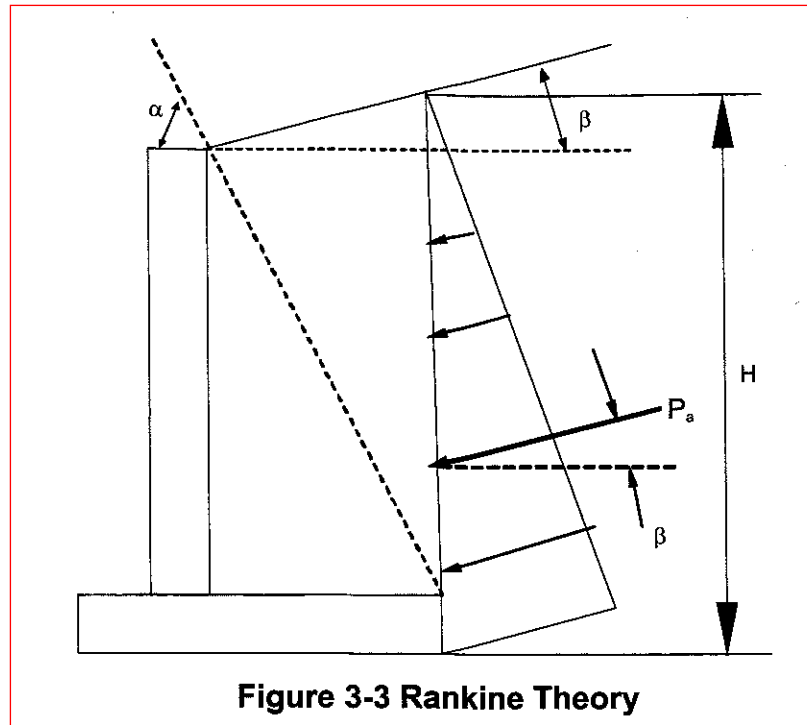


Figure 3-3 Rankine Theory

The resultant earth pressure force, P_a , is oriented at an angle, β , as shown in Figure 3-3. The resultant acts at a distance, $H/3$, from the base of the footing.

For situations with a broken backfill surface, the active earth pressure coefficient, K_a , may be determined using a β value adjusted per AASHTO LRFD Figures 3.11.5.8 -1 through 3, or substituted with β^* , as shown in Figure 3-2.

3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient

Values of the coefficient of passive lateral earth pressure, K_p , may be taken from Figures 3.11.5.4-1 and 2 in AASHTO LRFD or using Coulomb theory, as shown below:

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

where:

α = angle (degrees) of back of wall to the horizontal as shown in Figure 3-1.

ϕ = angle of internal soil friction (degrees), taken from Table 3-3.

Bearing Resistance

Objective:

Estimate the factored bearing resistance for a box culvert bearing on soil at the Service Limit State and Strength Limit State.

Given:

1. Limited lab data
2. Soil engineering properties based on correlations to SPT N-values

Assumptions:

1. The box culvert's embedment into the streambed is assumed as 2 foot.
2. The one foot thick layer of proposed Granular Borrow bedding material is ignored.
3. The proposed bearing elevation is approximately 248.0 feet.
4. Proposed precast concrete box has a span of 15 feet and a 17-foot-wide base.
5. The subsurface conditions present at the proposed bearing elevation in the borings are representative of the conditions for the entire site. Use N-value of 28 bpf to represent the consistency of the soils encountered at the box bearing elevation for bearing resistance calculations, based on BB-TFLS-102;2D.
6. The bottom of the box culvert will be submerged for the structure's design life.

1. Estimate the factored bearing resistance at the Service Limit State:

The use of presumptive values may be used when sufficient knowledge of geological conditions at or near the structure site exists. AASHTO LRFD Table C10.6.2.6.1-1 provides presumptive bearing resistances for spread footings when a settlement limited bearing resistance is appropriate. For more information see *NavFac DM 7.2, May 1983, Foundations and Earth Structures*, Table 1, p. 7.2-142.

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Medium dense	4-8	5

**Assume the granular fill is medium dense in consistency at the bearing elevation.
Recommend 5 ksf to limit settlement to 1.0 inch for Service Limit State Loads**

2. Estimate the factored bearing resistance at the Strength Limit State:

Assumed Foundation Width, Depth, and Water Surface

$$B := 17\text{ft}$$

$$D_f := 2.0\text{ft}$$

$$D_w := 0\text{ft}$$

$$\gamma_w := 62.4\text{pcf}$$

Foundation soils:

$$\gamma_{1d} := 112 \cdot \text{pcf}$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59: Table 3.2
Average of Loose silty sand (102 pcf) and Dense silty sand (121 pcf)

$$w_{\text{sat}} := .13$$

Moisture content of saturated sample BB-TFLS-102;2D

$$\gamma_{1\text{sat}} := \gamma_{1d} (1 + w_{\text{sat}})$$

Das, Principles of Geotechnical Eng. 7th Ed. p. 59:
Table 3.1 Unit weight relationships

$$\gamma_{1\text{sat}} = 127 \cdot \text{pcf}$$

$$N_{\text{design}} := 28$$

$$\phi := 35 \cdot \text{deg}$$

Lambe and Whitman, Soil Mechanics, 1969
Figure 11.14 N vs. phi

Cohesion $c := 0$

Nominal Bearing Resistance for Strength Limit States: Terzaghi Method - ϕ and c soil.

Shape Factors for strip footing

$$s_{\gamma} := 1.0$$

$$s_c := 1.0$$

Bowles 5th Ed., p. 220 Table 4-1

Meyerhof Bearing Capacity Factors - (Ref: Bowles Table 4-4, 5th Ed. pg 223) for Gravelly SAND $\phi = 35$ degrees.

$$N_c := 46.35$$

$$N_q := 33.6$$

$$N_{\gamma} := 37.8$$

Nominal Bearing Resistance per Terzaghi equation

$$q := D_f (\gamma_{1\text{sat}} - \gamma_w) \quad q = 128.32 \cdot \text{psf}$$

Das Principles of Foundation Engineering 7th Ed. p. 142:
Eq. 3.16 Water table modification

$$q_n := c \cdot N_c \cdot s_c + q \cdot N_q + 0.5 \cdot (\gamma_{1\text{sat}} - \gamma_w) \cdot B \cdot N_{\gamma} \cdot s_{\gamma}$$

Bowles Foundation Analysis and Design 5th Ed. p. 220:
Table 4-1 Bearing-capacity Equations

$$q_n = 24.9 \cdot \text{ksf}$$

Factored Bearing Resistance for strength limit states

Use a resistance factor per AASHTO LRFD Table 10.5.5.2.2-1

$$\phi_b := 0.45$$

$$q_r := q_n \cdot \phi_b$$

$$q_r = 11.2 \cdot \text{ksf}$$

for

$$B = 17 \cdot \text{ft}$$

Nominal Bearing Resistance for Strength Limit States

Reference: Munfakh, et al (2001) LRFD Article 10.6.3.1.2a

Total unit weight of the soil above the base slab/soil envelope

$$\gamma_{\text{above}} := 125 \cdot \text{pcf}$$

MainDOT Bridge Design Guide p. 3-3
Soil Type 4

Bearing Capacity Factors (Ref: LRFD Table 10.6.3.1.2a-1)

$$N_c := 46.1$$

$$N_q := 33.3$$

$$N_\gamma := 48.0$$

Shape Factors - per LRFD Table 10.6.3.1.2a-3

assume:

$$L := 40 \text{ ft}$$

Neglecting the 8 feet on each end of slope tapered wingwalls

$$s_\gamma := 1 - 0.4 \cdot \left(\frac{B}{L} \right)$$

$$s_q := 1 + \frac{B}{L} \cdot \tan(\phi)$$

$$s_\gamma = 0.83$$

$$s_q = 1.298$$

Groundwater Coefficients - LRFD Table 10.6.3.1.2a-2

The highest anticipated groundwater level should be used in design.

Assume groundwater, or stream elevation, will be above the invert of the structure for the entire design life.

Where the depth of water is less than the depth of the footing, all water coefficients are 0.5.

$$C_{wq} := .5$$

$$C_{w\gamma} := 0.5$$

$$c_1 := 0$$

Load Inclination factors

No knowledge of vertical and horizontal loads at this time. Use 1.0

$$i_c := 1.0$$

$$i_\gamma := 1.0$$

$$i_q := 1.0$$

Depth correction factors - only used when soils above the footing bearing elevation are as competent as the soils beneath the footing level. Otherwise 1.0

LRFD Table 10.6.3.1.2a-4

$$\frac{D_f}{B} = 0.118$$

Therefore :

$$d_q := 1.0$$

Terms

$$N_{cm} := N_c \cdot s_c \cdot i_c$$

$$N_{qm} := N_q \cdot s_q \cdot d_q \cdot i_q$$

$$N_{\gamma m} := N_{\gamma} \cdot s_{\gamma} \cdot i_{\gamma}$$

$$N_{cm} = 46.1$$

$$N_{\gamma m} = 39.84$$

$$N_{qm} = 43.21$$

Nominal Bearing Resistance (LRFD Eq 10.6.3.1.2a-1)

$$q_n := \left[c_1 \cdot N_{cm} + \gamma_{\text{above}} \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \gamma_{\text{sat}} \cdot \overrightarrow{(B \cdot N_{\gamma m})} \cdot C_{w\gamma} \right]$$

$$q_n = 26.8 \cdot \text{ksf}$$

Factored Bearing Resistance

$$\phi_b := 0.45$$

$$q_r := q_n \cdot \phi_b$$

$$q_r = 12.1 \cdot \text{ksf}$$

Recommend a factored bearing resistance of 12 ksf for precast box culvert base slabs that are 17 ft or greater on compacted granular fill.

3.4 Various Unit-Weight Relationships

In Sections 3.2 and 3.3, we derived the fundamental relationships for the moist unit weight, dry unit weight, and saturated unit weight of soil. Several other forms of relationships that can be obtained for γ , γ_d , and γ_{sat} are given in Table 3.1. Some typical values of void ratio, moisture content in a saturated condition, and dry unit weight for soils in a natural state are given in Table 3.2.

Table 3.1 Various Forms of Relationships for γ , γ_d , and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	Relationship	Given	Relationship	Given	Relationship
w, G_s, e	$\frac{(1 + w)G_s\gamma_w}{1 + e}$	γ, w	$\frac{\gamma}{1 + w}$	G_s, e	$\frac{(G_s + e)\gamma_w}{1 + e}$
S, G_s, e	$\frac{(G_s + Se)\gamma_w}{1 + e}$	G_s, e	$\frac{G_s\gamma_w}{1 + e}$	G_s, n	$[(1 - n)G_s + n]\gamma_w$
w, G_s, S	$\frac{(1 + w)G_s\gamma_w}{1 + \frac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1 - n)$	G_s, w_{sat}	$\left(\frac{1 + w_{\text{sat}}}{1 + w_{\text{sat}}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1 - n)(1 + w)$	G_s, w, S	$\frac{G_s\gamma_w}{1 + \left(\frac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\frac{e}{w_{\text{sat}}}\right)\left(\frac{1 + w_{\text{sat}}}{1 + e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1 - n) + nS\gamma_w$	e, w, S	$\frac{eS\gamma_w}{(1 + e)w}$	n, w_{sat}	$n\left(\frac{1 + w_{\text{sat}}}{w_{\text{sat}}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{\text{sat}} - \frac{e\gamma_w}{1 + e}$	γ_d, e	$\gamma_d + \left(\frac{e}{1 + e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{\text{sat}} - n\gamma_w$	γ_d, n	$\gamma_d + n\gamma_w$
		γ_{sat}, G_s	$\frac{(\gamma_{\text{sat}} - \gamma_w)G_s}{(G_s - 1)}$	γ_d, S	$\left(1 - \frac{1}{G_s}\right)\gamma_d + \gamma_w$
				γ_d, w_{sat}	$\gamma_d(1 + w_{\text{sat}})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft ³	kN/m ³
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Design SPT
28

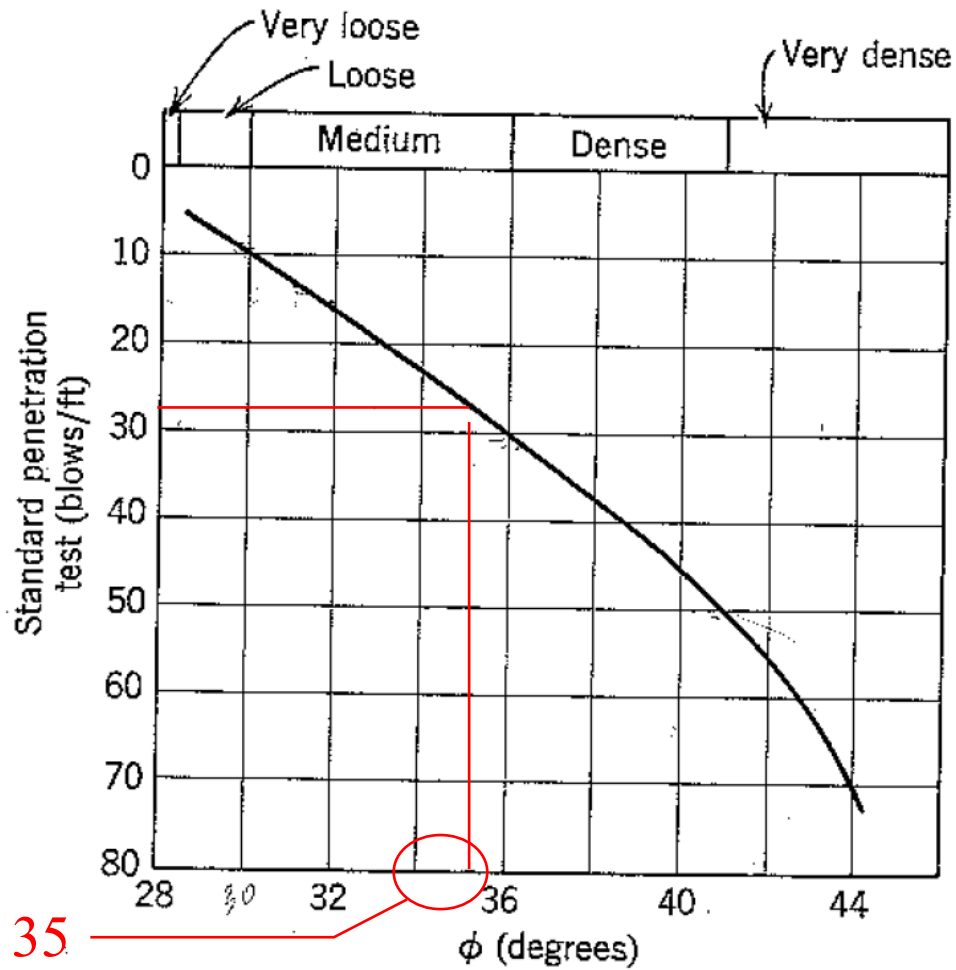


Fig. 11.14 Correlation between friction angle and penetration resistance (From Peck, Hanson, and Thornburn, 1953).

The Terzaghi Bearing-Capacity Equation

One of the early sets of bearing-capacity equations was proposed by Terzaghi (1943) as shown in Table 4-1. These equations are similar to Eq. (k) derived in the previous section, but Terzaghi used shape factors noted when the limitations of the equation were discussed. Terzaghi's equations were produced from a slightly modified bearing-capacity theory devel-

TABLE 4-1
Bearing-capacity equations by the several authors indicated

Terzaghi (1943). See Table 4-2 for typical values and for K_{py} values.

$$q_{ult} = cN_c s_c + \bar{q}N_q + 0.5\gamma B N_\gamma s_\gamma$$

$$N_q = \frac{a^2}{a \cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2) \tan \phi}$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = \frac{\tan \phi}{2} \left(\frac{K_{py}}{\cos^2 \phi} - 1 \right)$$

For: strip	round	square
$s_c = 1.0$	1.3	1.3
$s_\gamma = 1.0$	0.6	0.8

Meyerhof (1963).* See Table 4-3 for shape, depth, and inclination factors.

Vertical load: $q_{ult} = cN_c s_c d_c + \bar{q}N_q s_q d_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma$

Inclined load: $q_{ult} = cN_c d_c i_c + \bar{q}N_q d_q i_q + 0.5\gamma B' N_\gamma d_\gamma i_\gamma$

$$N_q = e^{\pi \tan \phi} \tan^2 \left(45 + \frac{\phi}{2} \right)$$

$$N_c = (N_q - 1) \cot \phi$$

$$N_\gamma = (N_q - 1) \tan (1.4\phi)$$

Hansen (1970).* See Table 4-5 for shape, depth, and other factors.

General:† $q_{ult} = cN_c s_c d_c i_c g_c b_c + \bar{q}N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma$

when $\phi = 0$

use $q_{ult} = 5.14s_u(1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q}$

$$N_q = \text{same as Meyerhof above}$$

$$N_c = \text{same as Meyerhof above}$$

$$N_\gamma = 1.5(N_q - 1) \tan \phi$$

Vesic (1973, 1975).* See Table 4-5 for shape, depth, and other factors.

Use Hansen's equations above.

$$N_q = \text{same as Meyerhof above}$$

$$N_c = \text{same as Meyerhof above}$$

$$N_\gamma = 2(N_q + 1) \tan \phi$$

*These methods require a trial process to obtain design base dimensions since width B and length L are needed to compute shape, depth, and influence factors.

†See Sec. 4-6 when $i_i < 1$.

3.4 Construction Loads

The construction live load to be used for constructibility checks is 50 psf applied over the entire deck area. Consideration should be given to slab placement sequence for calculation of maximum force effects.

3.5 Railroad Loads

Railroad bridges should be designed according to the latest American Railroad Engineering and Maintenance-of-Way Association specifications (AREMA, 2002), with the Cooper live loading as determined by the railroad company.

3.6 Earth Loads

3.6.1 General

Earth pressures considered for wall and substructure design must use the appropriate soil weight shown in Table 3-3.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, $\tan \delta$, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	36°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

N_γ values shows the following:

Bowles Foundation Analysis and Design 5th ed.

ϕ	Terzaghi* (1943)	Bolton and Lau (1993)	Kumbhojkar (1993)	Table 4-2 (this text)
34°	36	43.5	32	36
48	780	638	650.7	780.1

*See Terzaghi (1943), Fig. 38 and page 128.

Fortunately the N_γ term does not make a significant contribution to the computed bearing capacity, so any of the values from Tables 4-2 or 4-4 can be used (or perhaps an average).

Bolton and Lau (1993) produced new N_q and N_γ values for strip and circular footings for both smooth and rough ground interfacings. Their N_q values for either smooth or rough strips are little different from the Hansen values for rough strips. The N_q values for circular footings range to more than two times the strip values. The N_γ values for rough footings compare well with the Vesic values in Table 4-4. Since the Table 4-4 values have shape s_i and depth d_i factors to be applied, it appears that these "new" values offer little advantage and are certainly more difficult to compute (see comparison with Terzaghi values in preceding table).

Meyerhof's Bearing-Capacity Equation

Meyerhof (1951, 1963) proposed a bearing-capacity equation similar to that of Terzaghi but included a shape factor s_q with the depth term N_q . He also included depth factors d_i and

TABLE 4-4

Bearing-capacity factors for the Meyerhof, Hansen, and Vesic bearing-capacity equations

Note that N_c and N_q are the same for all three methods; subscripts identify author for N_γ

ϕ	N_c	N_q	$N_{\gamma(H)}$	$N_{\gamma(M)}$	$N_{\gamma(V)}$	N_q/N_c	$2 \tan \phi (1 - \sin \phi)^2$
0	5.14*	1.0	0.0	0.0	0.0	0.195	0.000
5	6.49	1.6	0.1	0.1	0.4	0.242	0.146
10	8.34	2.5	0.4	0.4	1.2	0.296	0.241
15	10.97	3.9	1.2	1.1	2.6	0.359	0.294
20	14.83	6.4	2.9	2.9	5.4	0.431	0.315
25	20.71	10.7	6.8	6.8	10.9	0.514	0.311
26	22.25	11.8	7.9	8.0	12.5	0.533	0.308
28	25.79	14.7	10.9	11.2	16.7	0.570	0.299
30	30.13	18.4	15.1	15.7	22.4	0.610	0.289
32	35.47	23.2	20.8	22.0	30.2	0.653	0.276
34	42.14	29.4	28.7	31.1	41.0	0.698	0.262
36	50.55	37.7	40.0	44.4	56.2	0.746	0.247
38	61.31	48.9	56.1	64.0	77.9	0.797	0.231
40	75.25	64.1	79.4	93.6	109.3	0.852	0.214
45	133.73	134.7	200.5	262.3	271.3	1.007	0.172
50	266.50	318.5	567.4	871.7	761.3	1.195	0.131

* = $\pi + 2$ as limit when $\phi \rightarrow 0^\circ$.

Slight differences in above table can be obtained using program BEARING.EXE on diskette depending on computer used and whether or not it has floating point.

Interpolate for $\phi=35$ $N_c=46.35$, $N_q=33.6$, $N_{\gamma}=37.8$

**AASHTO LRFD Bridge Design
Specification, 7th ed. 2014**

The foundation resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this Article.

10.5.5.2.2—Spread Footings

The resistance factors provided in Table 10.5.5.2.2-1 shall be used for strength limit state design of spread footings, with the exception of the deviations allowed for local practices and site specific considerations in Article 10.5.5.2.

Note that not all of the resistance factors provided in this Article have been derived using statistical data from which a specific β value can be estimated, since such data were not always available. In those cases, where data were not available, resistance factors were estimated through calibration by fitting to past allowable stress design safety factors, e.g., the AASHTO *Standard Specifications for Highway Bridges* (2002).

Additional discussion regarding the basis for the resistance factors for each foundation type and limit state is provided in Articles 10.5.5.2.2, 10.5.5.2.3, 10.5.5.2.4, and 10.5.5.2.5. Additional, more detailed information on the development of the resistance factors for foundations provided in this Article, and a comparison of those resistance factors to previous Allowable Stress Design practice, e.g., AASHTO (2002), is provided in Allen (2005).

Scour design for the design flood must satisfy the requirement that the factored foundation resistance after scour is greater than the factored load determined with the scoured soil removed. The resistance factors will be those used in the Strength Limit State, without scour.

C10.5.5.2.2

Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

Method/Soil/Condition			Resistance Factor
Bearing Resistance	ϕ_b	Theoretical method (Munfakh et al., 2001), in clay	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using CPT	0.50
		Theoretical method (Munfakh et al., 2001), in sand, using SPT	0.45
		Semi-empirical methods (Meyerhof, 1957), all soils	0.45
		Footings on rock	0.45
		Plate Load Test	0.55
Sliding	ϕ_τ	Precast concrete placed on sand	0.90
		Cast-in-Place Concrete on sand	0.80
		Cast-in-Place or precast Concrete on Clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50

The resistance factors in Table 10.5.5.2.2-1 were developed using both reliability theory and calibration by fitting to Allowable Stress Design (ASD). In general, ASD safety factors for footing bearing capacity range from 2.5 to 3.0, corresponding to a resistance factor of approximately 0.55 to 0.45, respectively, and for sliding, an ASD safety factor of 1.5, corresponding to a resistance factor of approximately 0.9. Calibration by fitting to ASD controlled the selection of the resistance factor in cases where statistical data were limited in quality or quantity.

$$i_y = \left[1 - \frac{H}{V + cBL \cot \phi_f} \right]^{(n+1)} \quad (10.6.3.1.2a-8)$$

$$n = [(2 + L/B)/(1 + L/B)] \cos^2 \theta + [(2 + B/L)/(1 + B/L)] \sin^2 \theta \quad (10.6.3.1.2a-9)$$

where:

B = footing width (ft)

L = footing length (ft)

H = unfactored horizontal load (kips)

V = unfactored vertical load (kips)

θ = projected direction of load in the plane of the footing, measured from the side of length L (degrees)

It should further be noted that the resistance factors provided in Article 10.5.5.2.2 were derived for vertical loads. The applicability of these resistance factors to design of footings resisting inclined load combinations is not currently known. The combination of the resistance factors and the load inclination factors may be overly conservative for footings with an embedment of approximately $D_f/B = 1$ or deeper because the load inclination factors were derived for footings without embedment.

In practice, therefore, for footings with modest embedment, consideration may be given to omission of the load inclination factors.

Figure C10.6.3.1.2a-1 shows the convention for determining the θ angle in Eq. 10.6.3.1.2a-9.

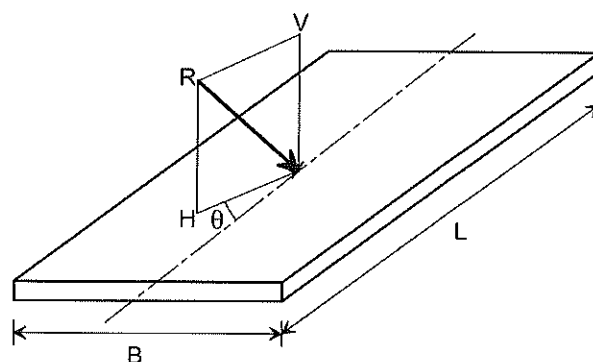


Figure C10.6.3.1.2a-1—Inclined Loading Conventions

Table 10.6.3.1.2a-1—Bearing Capacity Factors N_c (Prandtl, 1921), N_q (Reissner, 1924), and N_γ (Vesic, 1975)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	26.0
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	67.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8

Frost

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table, BDG Section 5.2.1.

From Design Freezing Index Map: **Topsfield, Maine**

DFI = 1850 degree-days.

Case 1 - coarse grained granular fill soils W=15% (assumed).

For DFI = 1800 $d1 := 82.8$

For DFI = 1900 $d2 := 84.6$

$$d := \text{in} \cdot \left(\frac{d2 - d1}{10} \cdot 2 + d1 \right)$$

Depth of Frost Penetration $d = 83 \cdot \text{in}$ $d = 6.9 \cdot \text{ft}$

Method 2 - ModBerg Software

Examine foundations placed on coarse grained fill soils

Madison lies along the same Maine Design Freezing Index contour - use Madison data from Modberg's freezing index database.

--- ModBerg Results ---

Project Location: Madison, Maine

Air Design Freezing Index = 1847 F-days

N-Factor = 0.80

Surface Design Freezing Index = 1478 F-days

Mean Annual Temperature = 42.4 deg F

Design Length of Freezing Season = 136 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	87.1	15.0	125.0	31	40	2.9	1.8	2,700

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).

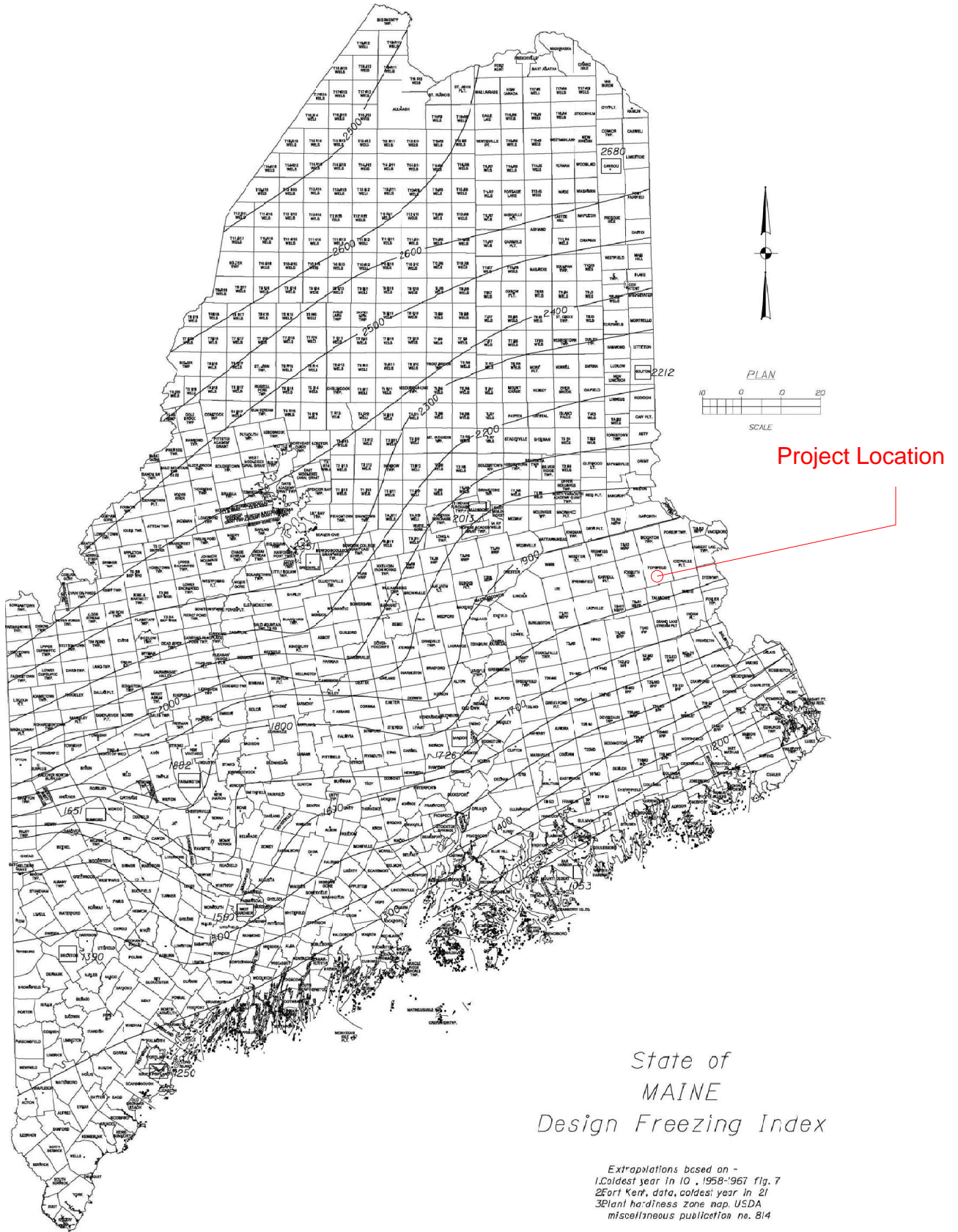
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).

L = Latent heat of fusion, in BTU / cubic f

Total Depth of Frost Penetration = 7.3 ft = 87.7 in.

Recommendation: 7.0 feet for design of foundations constructed on coarse grained soils

Figure 5-1 Maine Design Freezing Index Map



5.2 General

MaineDOT Bridge Design Guide

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Interpolate for 15% moisture: DFI 1800 = 82.8 in, DFI 1900 = 84.6 in